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The effect of small earth structures and channel improvements on the flooding of agricultural land in south-western Australia

J R. Davies

D J. McFarlane

R Ferdowsian

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The contents of this report were based on the best available information at the time of publication. It is based in part on various assumptions and predictions. Conditions may change over time and conclusions should be interpreted in the light of the latest information available.

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1. Abstract

A number of Soil Conservation District Advisory Committees in wheatbelt areas have identified flood control as a priority for cooperative action. There is a long history of flooding in the wheatbelt and this report reviews the results of a number of flood investigations. As earthworks have commonly been built to lessen or confine floodwaters, a review of the literature has also been made.

Following these reviews/ three case studies are described of the effect of earthworks on flooding in three areas; Cowcowing Creek, Beacon and West Nugadong. The case studies involve the use of a runoff routing model, which simulates the effects of the earthworks on floodwaters.

From the reviews and case studies, it is concluded that earthworks have a role to play in delaying or containing floodwaters in wheatbelt catchments, but the cost is high if large storms are to be controlled. The effectiveness of absorption banks in decreasing peak flows can be greatly improved if the banks are placed in that part of the catchment that peaks at the same time that the main stream peaks. Grade banks probably have little effect on the flood peaks of major storms, although the runoff may contain less soils and the location of the runoff will be better known. Levee banks can be effective in containing floodwaters in valley floors but they need to be comprehensive and well designed. Peak flows inside the levees will be increased which will increase scouring and problems at road crossings, and result in silt deposits in discharge areas. Other conservation measures may need to be adopted on catchments to lessen the volume of water that needs to be controlled by the levees. In the flat wheatbelt catchments, small raised road and rail embankments can result in significant ponding, which needs to be considered when designing flood control works on farm properties.

There are still a large number of poorly quantified relationships in the estimation of wheatbelt flooding and the effect of structures on the floods. There are also a number of poorly quantified side effects of flood control on water erosion, waterlogging and salinity, which need to be researched. In any flood investigation, the cost of the works need to be calculated and inadvertent effects anticipated where possible. With present low land prices in many wheatbelt catchments it may be best to accept large floods as an inevitable consequence of clearing and to concentrate on diverting the floods past important areas (e.g. towns) and to drain the flood waters away once they have formed.

2. Introduction

Agricultural valleys/ particularly those in the wheatbelt east of the Meckering Line (Mulcahy, 1967), commonly consist of chains of saline lakes and braided channels, bordered by floodplains 2 to 3 km wide. These old valley forms are susceptible to flooding and waterlogging and have been classified by Bettenay and Mulcahy (1972). Carder (1971) noted that, on reaching the valleys, natural drainage lines often become shallow, ill-defined and branch out. Traditionally the floodplain soils have been considered fertile by Western Australian standards. The soils have been managed with little or no regard for natural drainage patterns. Townsites, roads and railways were often sited within the valleys. Subsequent clearing of upslope catchment areas has resulted in periodic flooding damage to these capital investments.

Following changes to the Soil and Land Conservation Act in 1982, it became possible for groups of landholders to form Soil Conservation Districts to jointly tackle soil degradation in their area. Commonly, landholders in the wheatbelt identified flooding as a problem that was suited for collaborative action.

This report briefly outlines the flooding situation that exists, and reviews a number of attempts that have been made to overcome the problems caused by flooding in agricultural catchments in Western Australia. It then reviews the literature on the effects of small earth structures on flood runoffs to help determine what structural options there are for mitigating the flooding problem. Three case studies which use the runoff routing model RORB are then outlined; the effect of absorption banks and dams on flooding in the Cowcowing Creek Catchment, the effect of levees and a road crossing on flooding in the Beacon Catchment and the effect of drains and a road crossing on flooding in the West Nugadong Catchment. From the historical review, the literature review and the three studies, general conclusions and recommendations have been drawn.

3. Reports On Flooding In Agricultural Areas Of Western Australia

Local experience of flooding in agricultural areas is reviewed (in chronological order of their first documentation) to give an indication of how the flooding problem has been viewed and tackled since the 1960s. The location of the areas discussed in this report are shown on Figure 3.1.

3.1 Belka Valley

Most of the information detailed below comes from unpublished files held in the Water Authority of Western Australia (then called the Public Works Department). In 1963, flood waters over 2 km wide spread out over the lower parts of the Belka Valley, 30 km east of Bruce Rock. About 1000 km2 of the 1700 km2 catchment is at risk from flooding. Farmers in the area decided to construct a levee bank system to contain the flood waters within the main channel in the valley.

Initial advice from the Public Works Department was that the levees should be 120 to 160 m apart. In 1964 farmers in the valley constructed levees 20 to 60 m apart. The farmers considered the increased returns they received in the first year after construction were sufficient to pay for the works, despite problems of insufficient provision for the controlled entry of side waters and the levees not being continuous at road and rail intersections. Further levees were constructed after flooding in 1968.

In the late 1960s, further clearing took place in sandplain areas in the upper catchment to the east. Very heavy rains in the lower parts of the valley in February 1978 washed away 48 kilometres of the levees and caused widespread flooding worth $256,530 in the Bruce Rock Shire. After the flooding the Belka Valley Flood Relief and Conservation Planning Committee was formed. The Public Works Department was asked to survey, design and supervise the construction of a fully engineered levee system. After an investigation, it was decided that no levee system would be capable of coping with the 1978 flooding. Only a comprehensive soil conservation system on the whole catchment would reduce the flows sufficiently for levees to work. The design that was finally recommended was sufficient for the 1968 event. The levees were to be 6.3 m wide at the base and 1.4 m high with a 1.5 m wide top. A tender was let to construct the levees, but a number of landholders subsequently withdrew support and only a small proportion of the scheme was constructed (some by contractor and some by individual farmers' plant). Consequently the flood control scheme has not been very effective.

3.2 Lake Toolibin

Negus (1968) reported on flooding problems in the Lake Toolibin Catchment, which has a floodplain 1.6 km wide with grades of 1:1000. Channels on the plain were found to meander and change course due to silt deposits reducing channel capacities. Also the culvert capacity of some Shire roads was found to be inadequate.

It was proposed to reduce the flood peaks from the surrounding sloping lands by developing conservation farm plans for areas with moderate to high erosion hazards. It was thought that the adoption of contour banks and contour cropping would reduce peak runoff rates by as much as 25 per cent (no source was quoted for this figure). It was also proposed that a defined channel be constructed straight through the flood plain, which would enable flood waters to retain sufficient velocities to prevent silting (while not scouring the channel). Shallow channels on the outside of the levees were proposed to carry away local drainage waters.

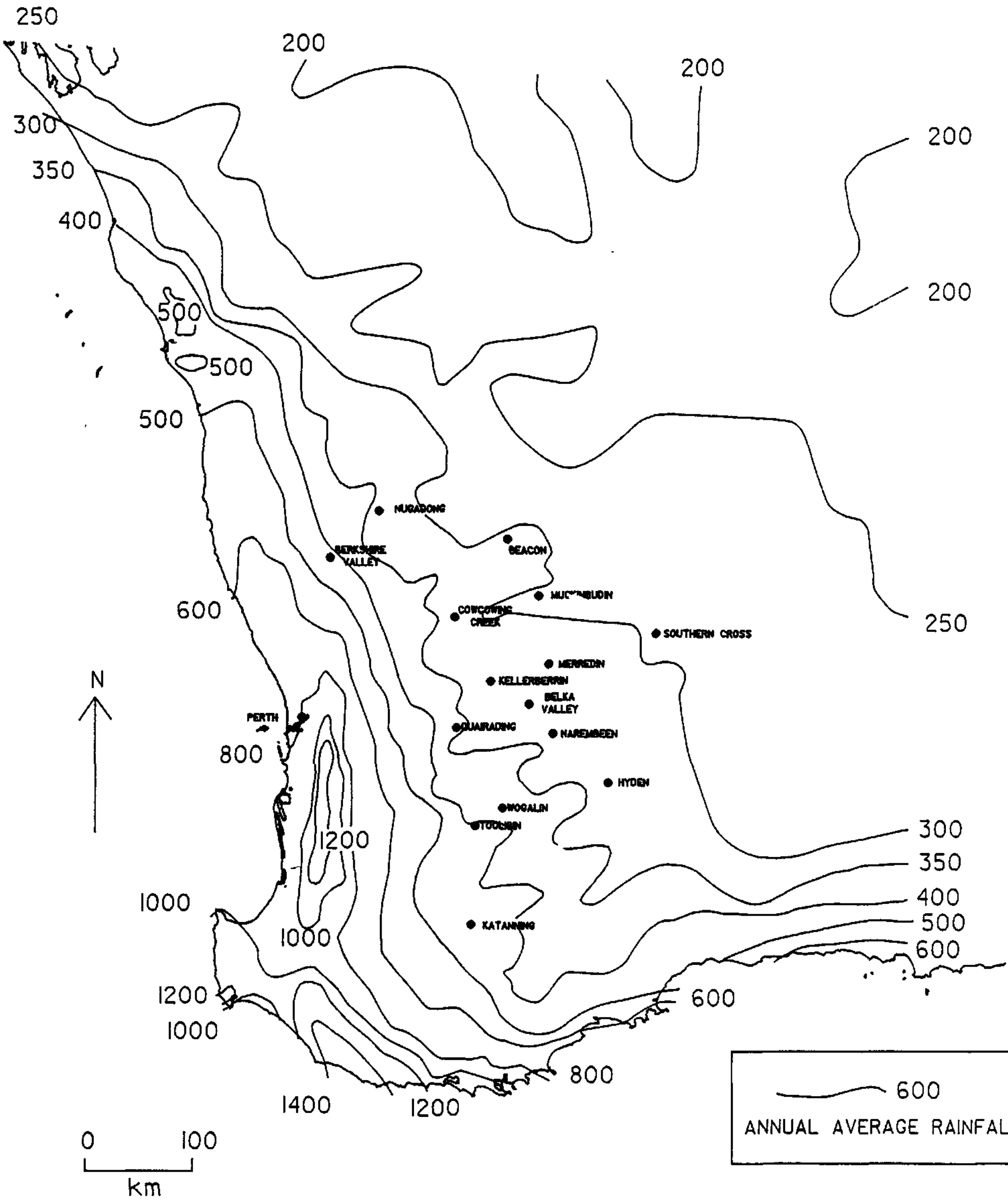


Figure 3.1 Location Of Sites Mentioned In This Report

Subsequent to these recommendations, some small levees were constructed on the floodplain but were found to be not effective in containing major floods.

3.3 Wogalin Creek

Negus (1970) reported on the performance of single- and double-sided levees, which had been installed in the Wogalin Creek Catchment (Wickepin/Kulin Shires) by farmers between 1964 and 1966. The double levees were 1 m high and 160 m apart and controlled flood waters on the lower parts of the catchment. The spacing between the levees was increased by 50 per cent at road crossings to reduce the depth and velocity of flow. The single levees were used where the natural creek channel was within 300 to 600 m of natural sloping land (which acts as a second levee).

While the drains performed well during floods, the following problems (and possible solutions) were noted:

• Where the single levees curved (due to following the natural creek line), alternating silting and scouring occurred. It was recommended that the levees be straightened to overcome this problem.

• There was a lack of continuity of the levees in parts of the catchment, due to some landholders not participating in the scheme.

• Inadequate culvert capacities at road crossings caused flooding.

• There was inadequate local drainage adjacent to the levees. Possible solutions included the construction of a channel outside the levee, or the addition of the local waters to the drain at "venturi" inlets or through flood gates.

Farmers who were part of the scheme reported advantages of increased crop yields and the ability to safely graze their sheep on the plains over winter. Even with the drains, water continued to pond in low spots on the flats and infiltration rates into the clay soils were slow, resulting in waterlogged areas. The adoption of soil conservation measures in the surrounding uplands could help alleviate the need to de-silt the drains every ten years by reducing sediment transport. While it was thought the measures would reduce flood peaks, they were thought to have little effect on runoff volumes during wet years. Well-managed pastures were thought to be able to reduce both runoff and silting.

The report acknowledged that flood control structures which eliminated the natural and temporary storage of flood waters on the plains could result in severe damage to other parts of the drainage system. This included the deposition of silt in major drainages such as the Avon River.

Carder (1971) recognized three approaches to flood mitigation in the broad wheatbelt valleys; flood training, catchment improvement and changing the land-use. Flood training was considered the most common approach although costly, liable to failure and with serious legal implications. Catchment improvements included contouring and pasture improvements on the upslope areas to reduce runoff and erosion. These were considered to be no sure solution, requiring upslope farmers to co-operate and involving the treatment of areas which are 10 to 20 times larger than the areas affected by the flooding. It was noted that the flats themselves contributed water and it was not proved to what extent contouring would reduce the severity and frequency of flooding, or to what extent improved pastures reduced runoff.

Carder considered that changing land use on the flats from cropping to pastures had not been given the attention it deserved as a possible solution to flooding problems. He recognized the change would require a change in attitudes, that pasture species were (then) limited/ that some farms would have little or no arable land, and that there would be an increased need for fences and sheep yards. The validity of each approach had to be considered for each situation.

3.4 Eastern wheatbelt

Following a wet January and February in 1978, widespread flooding occurred when thunderstorms occurred in late February in the area between Kellerberrin, Mukinbudin, Southern Cross and Hyden (Kratchler, 1980). At Moorine Rock a pond 3.7 km2 in area formed, cutting off the Great Eastern Highway. At Southern Cross, Lake Polaris filled, cutting the town into two parts and flooding houses. The Narembeen township was flooded on two occasions (further details later). Road and rail embankments throughout the area were damaged and there were widespread losses of fences. As mentioned previously, the storm washed away flood protection levees in the Belka Valley.

Kratchler concluded that extensive clearing in the 1960s had accentuated flooding, as more intense rainfall in 1966 and 1970 had not resulted in comparable flooding. He also noted the immature nature of the streams in wheatbelt valleys. The wet antecedent conditions in February 1978 would have reduced the initial and continuing losses from the storms.

3.5 Merredin

Storms in the 101 km2 catchment to the east of the Merredin townsite were responsible for a 55 m3/s flood peak passing through the town in 1978 and a 105 m^/s peak in 1979 (Bretnall, 1984). The capacity of the main drain through the town was only 25 m^/s, so the 1978 and 1979 floods caused extensive damage (e.g. $300,000 in 1979). To simulate the effect of constructing five retarding basins in the catchment, the runoff routing model RORB (Laurenson and Mein, 1985) was used. This model is described in some detail in Appendix A. The simulated basins comprised an earth embankment across a drainage line with a pipe outlet and an emergency spillway.

Parameters for the RORB model were derived by reproducing the 1979 storm runoff (a 100 year average recurrence interval event) from the rainfall that produced it. The storm rainfall was 70 mm, spread evenly over a two-hour period. The initial loss on the catchment was estimated as 15 mm, with a continuing loss of 6 mm/h. Thus the total rainfall excess was 43 mm.

Several options (Table 3.1) were tested for lessening the peak flow and therefore the peak level for the 1979 event. The effect of five retarding basins (with 3 or 4 m bank heights) had only a marginal effect, considering the town drain's capacity of only 25 m3/s. A 14.9 km^2 sub-catchment immediately east of the town (and below all of the basins) was responsible for much of the peak flow. Diverting runoff from a 5.8 km^2 part of this sub-catchment (Option 4) resulted in a substantial reduction in the peak runoff, but still exceeded the capacity of the main drain.

The effect of absorption banks with a storage capacity of 15 mm of runoff from the contributing catchment and located over the entire catchment was simulated by increasing the initial loss of the storm by 15 mm (i.e. assuming the banks were empty at the start of the storm). This resulted in a 34 per cent reduction in the flood peak, but the peak was still 275 per cent of the town drain's capacity. The banks that were subsequently constructed on the

Merredin Catchment had a storage capacity of about 28 mm but did not cover the whole catchment. Due to the off-site benefits, 80 per cent of the cost of the banks was funded by the Western Australian Government. All the retarding basin options were expensive (e.g. Option 4 was about $1.2 million in 1983 prices), and did not include maintenance costs and the loss of land from agricultural production due to periodic flooding.

Table 3.1. Simulated peak flows with different options (after Bretnall, 1984)

Option	No. of retarding basins	Bank height (m)	Other management	Peak runoff (m^3/s)
1.	0	0	none	105
2.	5	3	none	82
3.	5	4	none	81
4.	5	4	Diversion of 580 ha sub-catchment near town into a basin	47
5.	0	0	Absorption banks with 15 mm storage over whole catchment	69

Consideration was also given to upgrading the drain through the town. Lowering and lining the drain would increase the drain's capacity from 25 m^3/s to 45 to 50 m^3/s. However five bridges would need upgrading and the total cost would approach the cost of the methods for limiting the flood peak.

A bank was subsequently constructed along the northern town boundary to divert flow from the local catchment past the town. There has been no assessment of the works, as constructed, on flooding in Merredin.

3.6 Quairading

On June 2, 1983, 40.6 mm of rain fell in 110 minutes on a 4 km2 catchment located north west of the Quairading townsite. A drain through the town overtopped and flooded properties, town roads, the Quairading to York road and the railway line

(Swanson, 1983). The time of concentration for the catchment was estimated by Swanson (using the Bransby-Williams formula) to be 110 minutes, the same as the storm duration. Assuming a runoff coefficient of 0.6, the flood flow was estimated to be 15 m^3/s. The storm that caused the flooding had a 35 year average recurrence interval while the flood had a 35 to 40 year average recurrence interval.

Constructing a large capacity drain through the town was considered to be impractical by Swanson (1983), and a diversion channel capable of containing the 100 year average recurrence interval flood and costing $150,000 to $170,000 was proposed. It was also recommended that the capacity of railway and road culverts in the area be increased.

Subsequent to this investigation, the WISALTS organization proposed constructing 12.5 km of level banks in the catchment (costing $14,000) to mitigate the flooding (File 84SCPF46, W. Aust. Dept. Agric.). However landholders in the catchment were opposed to level banks on the grounds of loss of land and hindrance to cropping operations. A proposal for a combination of detention basins in the central drainage line (capacity 40,000 m^3) and some absorption banks (capacity 20,000 m^3) was proposed by the Department of Agriculture (cost $50,000). The likely effects of the flood detention structures and banks on flood levels in the town have not been investigated.

3.7 Katanning

A 13 km^2 catchment west of Katanning periodically contributes flood waters to the south west corner of the town. To mitigate the flooding, about 18 km of level and graded interceptor banks were constructed in the catchment (File 84SCPF22, W. Aust. Dept. Agric.). Due to the off-site benefits arising from the banks, 80 per cent of the costs were contributed by the Western Australian Government.

No estimates were made of the frequency of the flooding problem in the town, or of the effect the banks would have on the flooding. The banks in the catchment are double-push WISALTs banks which are not turned up at their ends. The capacity of these banks is only 40 to 70 per cent the capacity of absorption banks with a 0.5 m turn up (e.g. 3 to 3.5 m^3 storage/m bank length versus 5 to 8 m^3/m). The cost of constructing the banks ($1120/km) was 60 per cent higher than the estimated cost of constructing a single-push absorption bank. Surveying costs were $67/km as a backhoe was used to check the depth of the clay subsoil along the survey lines. There has been concern expressed that the banks will contribute additional recharge to the saline aquifer that underlies the Katanning townsite. Piped outlets have been constructed through some banks to lessen this recharge, to reduce waterlogging adjacent to the banks and to improve the storage efficiency of the bank channels. Other methods of reducing the flooding problem in the town (e.g. diverting the flood waters) have not been investigated.

3.8 Narembeen

The flooding potential of the Narembeen townsite and possible flood mitigation strategies were examined in a consultant's report to the Water Authority of Western Australia (Sinclair Knight and Partners, 1987). The catchment contributing to the flooding is 2810 km^2. The 1978 flood (also mentioned by Kratchler, 1980) was estimated to have a 50 year average recurrence interval.

Several methods of estimating the peak flow for different return periods were investigated for the main catchment and a smaller catchment to the north of the town. The methods included the Statistical Rational Method (Flavell, 1983), the Index Flood Method (Flavell, 1983), a regional flood estimation method using catchments adjacent to Narembeen, and catchment modelling using RORB. The RORB estimates were between the Index Flood Method and the Statistical Rational Method. RORB was considered to be the most reliable method of estimating flood on the main catchment. However for the smaller (6.3 km^2) catchment north of the townsite, the Statistical Rational Method was preferred as the database from which it was derived included catchments of similar size within the wheatbelt.

The report recommended that a levee bank be constructed to divert flood flows away from the town.

3.9 Conclusions

From the above reports, it is concluded that flooding of wheatbelt valleys has been a serious problem for over 20 years.

There have been a number of investigations of the use of levees and drains to contain or divert flood waters in the valleys. A major problem with the use of levees appears to be the cost of a fully engineered system and the need for the system to be comprehensive. Drains which divert flood waters around towns appear to have been more effective, possibly due to their construction on public land by a single authority.

The use of structures to retain flood waters higher in the catchment has been less investigated and less adopted. Section 5 examines this option in greater detail.

There has been no documented adoption of catchment improvement and of changed land-use to overcome flooding problems as suggested by Carder (1971).

4. Studies Of The Effect, Of Banks, Drains, Dams And Levees On Flood Runoff

While there is an extensive literature on the effect of large storage reservoirs on flood runoff, the literature on the effect of small structures is less extensive. This difference is partly due to the legal requirement for investigations when large structures are built for controlling flood waters, there being no such requirement for small structures.

Studies of the effect of soil and water conservation structures on flood runoff have taken several forms; a comparison of different treatments on the same catchment, a comparison of runoff before and after banking on a single catchment, paired catchment studies and modelling the effects of the structures. The literature on studies from different parts of the world are reviewed below.

4.1 U.S.A.

Baird (1929) measured the runoff from variable grade terraces (banks) installed at three vertical intervals (2, 3 and 4 foot), constructed on a hillslope of 5 per cent. The total amount of runoff increased as the vertical interval was decreased, but the amount of soil loss decreased slightly with decreasing vertical interval.

The American Society of Civil Engineers (ASCE) discussed the effects of conservation practices on runoff at their 1946 and 1947 meetings (ASCE, 1948). While it was concluded that the data were not sufficiently clear to justify definite conclusions, the following observations were made for monitored catchments in Missouri:

(i) On one soil type, terracing reduced the mean annual runoff by 30 per cent. However, for the nine most severe storms over an eight-year period, the average reduction in runoff was only 11 per cent, while for two storms there was more runoff from a terraced catchment.

(ii) Contour farming reduced mean annual runoff by 20 per cent. However the reduction in runoff averaged only 2 per cent for the nine most severe storms. For four severe storms, there was more runoff from a contour farmed catchment.

(iii) Peak runoff from a terraced catchment was less than for both a contour worked and a control catchment for all severe storms.

From a study in Wisconsin, it was found that over several years, the greatest runoff occurred from cultivated land, less from pasture, still less from a terraced (banked) and cultivated area and least from alfalfa. However for intense storms the situation was different with both the highest and lowest runoffs occurring on catchments with full canopy cover.

In the Middle West, more runoff occurred from contoured plots than from up-and-down slope plots during one intense storm. Similar runoff amounts were also reported from cultivated and pasture plots under intense rainfall and it was concluded that pasture was not effective in reducing flood runoff.

In the Southeast, complete ground cover reduced runoff during moderate storms, with little difference occurring during extreme storms. However the cover reduced erosion significantly, even on 11 per cent slopes.

On heavy black soils in Texas, conservation practices resulted in lower rates of runoff, with the effect being less during extreme storms. When the soils were fully wet, only those conservation practices that reduced the velocity of flow, or affected the amount of temporary storage, affected runoff rates. It was concluded that with wet soils, the amount of runoff was little affected by the practices.

Given the variability in the soils, catchment conditions, treatments and rainfalls, the reported inconsistent responses to conservation practices in the U.S.A. are understandable. However there is an indication that while grade banks may lessen mean annual runoff, during severe storms their effect may be negligible or even to increase runoff. In all cases, close-grown vegetation greatly reduced erosion, even during severe storms. Runoff from grassed areas during intense storms will therefore contain less sediment. Whether this would result in increased or decreased waterway and stream bank erosion is not clear.

There has been controversy in the U.S.A. as to whether a number of small flood detention techniques (e.g. contour ploughing, banks, dams and small detention ponds), located where runoff originates, will be more or less effective in mitigating floods than larger structures located on major drainage lines. The case for small detention structures on agricultural land was put by Peterson (1954). Peterson argued that large institutions preferred large structures, while a number of small on-farm structures would perform as well, if not better. Peterson's book is largely subjective and there is no quantification of whether the storages available in on-farm banks and drains are significant in comparison with storages in drainage lines, and whether the on-farm storages would significantly affect flood peaks.

The effect of spatial variability of soils, land-use or cover conditions, topography and rainfall on overland and channel flow was studied by Stanholz et al. (1981) using a finite element model of a catchment. They noted that the partial area concept was one of the first attempts to include heterogeneity into catchment models of runoff. There can be interactions between factors affecting runoff. For example, soil properties which affect infiltration, water storage, drainage and the hydraulics of surface flow can be related to topographic position.

Stanholz et al. (1981) found that the factors that were most sensitive in estimating runoff volumes were soil depth, antecedent soil moisture and vegetative cover. Factors which most affected runoff timing were Manning's roughness coefficient and flow element length (e.g. including small first order streams in the simulation significantly increased the lag time). The factor which was most sensitive to spatial variability in the catchment was rainfall. Thus the spatial distribution of storm rainfall within the catchment has a profound effect on the resultant storm runoff and this variability should be included in predictive models if possible.

The model was used to simulate the effect of detention structures located in different parts of the catchment. Structures had most effect when they intercepted channel flows rather than overland flows.

Absorption banks intercept overland flow whereas retaining basins and dams in drainage lines intercept channel flow. This finding of the model is not consistent with the argument of Peterson (1954) that small structures placed near the origin of the runoff will have an equal or greater effect on runoff than large structures on drainage lines.

4.2 India

Sastry and Narayana (1984) measured rainfall, runoff and soil loss from three agricultural and forested catchments for ten years before constructing earthen banks (bunds) around individual fields in one agricultural catchment and constructing brushwood check dams in a forested catchment. The measurements were then continued for nine years. The banks reduced peak runoff rates and runoff volumes dramatically. Runoff volumes were reduced to only 28 per cent of the volume from a grazed forested catchment and the improved soil moisture regime allowed a change in land use to be made from corn to rice and sugar cane. While the brushwood check dams in the grazed forested catchment reduced the volume of runoff, they had no appreciable effect on peak runoff.

This study highlights two aspects; the long term nature of catchment studies and the need to consider that a change in the hydrologic regime by earth structures may allow a change to a more profitable land use.

4.3 Australia

4.3.1 General

Bird (1980) detailed the geomorphological history of erosion along the Lang Lang River in Gippsland. Excavating and straightening of the river channel, in conjunction with levee banks, enlarged the capacity of the channel and reduced the danger of flooding. Sediment deposits in the lower reaches of the river were commercially mined. Early benefits of the mining were followed by problems of lateral scouring of bridge abutments and increased erosion, resulting in the need for an expensive weir to control flooding. Many of the side effects of the early flood control measures were not predicted.

Warner (1985) reviewed man's impact on Australian drainage systems and concluded that farm dams greatly reduced the runoff and sediment yields in some systems. He also found that there had been little study of the impact of soil conservation measures on Australian drainage channels. Improvements to one section of a drainage line (e.g. straightening or levee building) can result in erosion upstream (and in the improved section) and deposition downstream of the improvements. Warner concluded that the effect of piecemeal channel improvements needs to be carefully considered.

4.3.2 N.S.W.

Probably the best documented analyses of the effect of soil conservation structures on runoff and soil loss in Australia are those for the paired catchments at the Wagga Wagga Research Centre, Soil Conservation Service of New South Wales (Adamson, 1974, 1976; Ryan, 1986; Burch et al., 1986). The conservation treatments on one of the 7 ha catchments comprises gully filling, contour furrows at 1 m vertical intervals, improved pastures and regular fertilizer applications to the pastures. The control catchment has no conservation practices, is not fertilized and is stocked so as to maintain low pasture. This catchment is moderately sheet eroded and has an active gully.

Cumulative runoff from the treated catchment is only about 27 per cent of that for the untreated catchment while soil loss is about 1 per cent (Adamson, 1974). The pasture furrows restrict overland flow and increase depression storage by about 40 mm (Adamson, 1976). The hydrograph for the treated catchment is delayed and attenuated in comparison with the untreated catchment. Base flows are higher in the treated catchment, perhaps due to the greater infiltration in pasture furrows. As reported from the U.S.A. catchments, the effects of the conservation treatments are greatest in years

of below average to average rainfall. In above average years, annual runoff increases but soil losses do not increase proportionately. The greatest reduction in runoff occurs early in the storm (presumably due to the increased initial loss caused by the pasture furrows). In large storms the rate of runoff may exceed that of the untreated catchment.

A 592 ha catchment at Red Creek, south of Wagga, has been monitored since 1973 for rainfall, runoff and sediment load. In 1976, detention and permanent storage structures with a capacity of 6 mm of runoff were constructed on the major gullies (Ryan, 1981, 1982, 1986). Runoff volumes after installation were only 34 per cent of that before installation (reduced from 25 to 14 per cent of rainfall). Soil losses have been only 16 per cent of the pre-installation losses. Some of the differences are probably due to a decrease in annual rainfall (and erosivities) after the structures were installed.

Lang (1979) showed that there is an inverse curvilinear relationship between ground cover and storm runoff such that when ground cover declines below about 75 per cent, runoff amount and rate increase dramatically. The results were obtained from pasture dominated by perennial tussock grasses, and the dramatic increase in runoff occurred when the bare areas between the tussocks became connected. Lang and McCaffrey (1984) found a similar relationship between ground cover and soil loss.

4.3.3 Queensland

The Queensland Department of Primary Industries acknowledge that graded banks may increase flooding (by decreasing the time of concentration), but argue that the runoff from bank systems contains less sediment and that the location of the runoff can be predicted and planned for (Armstrong and Maschmedt, 1984). Relative to unchannelled overland flow, banks increase flow lengths and decrease the gradient along the channels, but they also decrease both the hydraulic radius (the ratio of the cross-sectional area to wetted perimeter) and the roughness coefficient.

4.3.4 Western Australia

Marsh used a rainfall simulator to investigate the effect of different conservation measures on runoff and soil loss (DRM Annual Report, 1982). Increasing the cropping frequency greatly increased runoff, even when direct drilling was used (Table 4.1). Soil loss was decreased by more frequent cropping using direct drilling due to the presence of extra stubble residue in the seed bed. Direct drilling was found to reduce runoff by about 12 per cent and soil loss by 44 per cent in comparison with conventional cultivation. Using harrows on contour worked land greatly reduced surface storages and resulted in a 57 per cent increase in runoff in comparison with a rough seedbed. Surprisingly, contour working only decreased runoff by 9 per cent relative to up and down hill working. Grasby and Marsh (File 2186 EX, W. Aust. Dept. Agric.) had earlier failed to find an effect of contour working on wheat yield after a number of trials.

The effect of contour banks on runoff has been assessed by Bligh (undated) by comparing the runoff from similar storms on the Berkshire Valley Catchment before and after banking. From a comparison of one intense storm before and after banking, Bligh concluded that banking increased the duration of runoff by about 50 per cent and delayed the time to peak by about 80 per cent. There was some indication that the banks also reduced flow rates.

Table 4.1 Effects of cultural treatments on overland flow and soil loss (Source: DRM Annual Report, 1982)

	Treatment	Change in overland flow	Change in soil loss
1.	Cropping frequency - conventional cultivation		
	(relative to 1 crop in every 3 years)		
	1 crop in every 2 years,	63% increase,	96% increase,
	2 crops in every 3 years	48% increase	15% increase
2.	Cropping frequency - direct drilling		
	(relative to 1 crop in every 2 years),		
	2 crops in every 3 years continuous cropping	17% increase, 76% increase	31% decrease, 56% decrease*
3.	Direct drilling		
	(relative to conventional cultivation)	12% decrease	44% decrease
4.	Use of harrows behind combine		
	Contour working Up and down hill working	57% increase, No change	33% increase, Decreased
5.	Contour working (relative to up and down hill working on, a 2% slope)	9% decrease	28% decrease

* Effect of extra stubble residue.

4.4 Conclusions

Various studies have shown that grade banks (terraces) reduce and delay runoff from small and moderate storms but may have no effect or even increase runoff from major storms. The runoff through a properly designed and maintained grade bank/waterway system is likely to contain less sediment than unchannelled overland flow, and the location of the runoff can be predicted and planned for.

Vegetative cover is also likely to decrease runoff from agricultural areas in small and moderate storms but have little effect (except on soil loss) in larger storms. Runoff with little sediment load from vegetated areas can cause severe erosion in cultivated paddocks (McFarlane and Ryder, 1987).

It is not clear whether structures which intercept channel flow are more effective in reducing flooding than structures which intercept overland flow, although modelling would indicate this to be the case. From a soil conservation viewpoint there are advantages in intercepting overland flow.

Soil conservation practices such as contour working without harrows and minimum tillage are also likely to reduce runoff in all but severe runoff events. A number of on-farm practices are capable of reducing runoff from small storms, but major off-farm structures are required for mitigating major floods.

5. Cowcowing Creek Study

5.1 Introduction

The Cowcowing Creek Catchment north of Wyalkatchem has an area of 187.5 km^2 to Butt's Crossing (Figure 5.1). Flooding of the road crossing was reported by the Wyalkatchem Soil Conservation District Advisory Committee to cause problems approximately one year in three. In order to assess the potential for soil conservation structures to mitigate this flooding, the catchment was modelled to determine tributary hydrograph timing and to assess the effect of storage structures (dams and absorption banks) on peak flow in floods with various average recurrence intervals. In the absence of any flow or flood level data on the catchment/ regional relationships were used for the RORB runoff-routing model parameters, and for rainfall loss rates. The catchment map (Figure 5.1) was schematised into sub-areas for modelling purposes.

5.2 Rainfall and loss estimation

Rainfall data were abstracted from Australian Rainfall and Runoff (ARR, 1977) and are presented in Tables 5.1 and 5.2. The time of concentration of the catchment is about six hours (ARR 1977) which may be low due to the very low grades in the channels.

Table 5.1 Rainfall probability for Cowcowing Creek (ARR, 1977)

Average recurrence Interval (yrs)	Rainfall depths (mm)	
	Duration = 0.5 hrs	Duration = 12 hrs
1	11	35
5	15	50
10	18	59
20	21	68
50	25	82
100	29	95

Table 5.2 Rainfall temporal patterns (ARR, 1977)

Rainfall duration (hrs)	Time increment (hrs)	Percentage of rainfall occurring in successive time increments (%)
0.5	0.1	13, 26, 35, 20, 6
12	2	25, 50, 13, 6, 4, 2

Rainfall losses were assumed to be 12 mm initial and 4 mm/hr continuing, the latter value being recommended for the south west of W.A. in an early draft of the revised edition of ARR. The latest edition of ARR (1987) recommends the equation:

$$IL_5 = 700\ P^{-0.47}\ L^{-0.08}$$

where P = average annual rainfall (mm) and L = mainstream length (km)

$$IL_{10} = 1.09\ IL_5$$

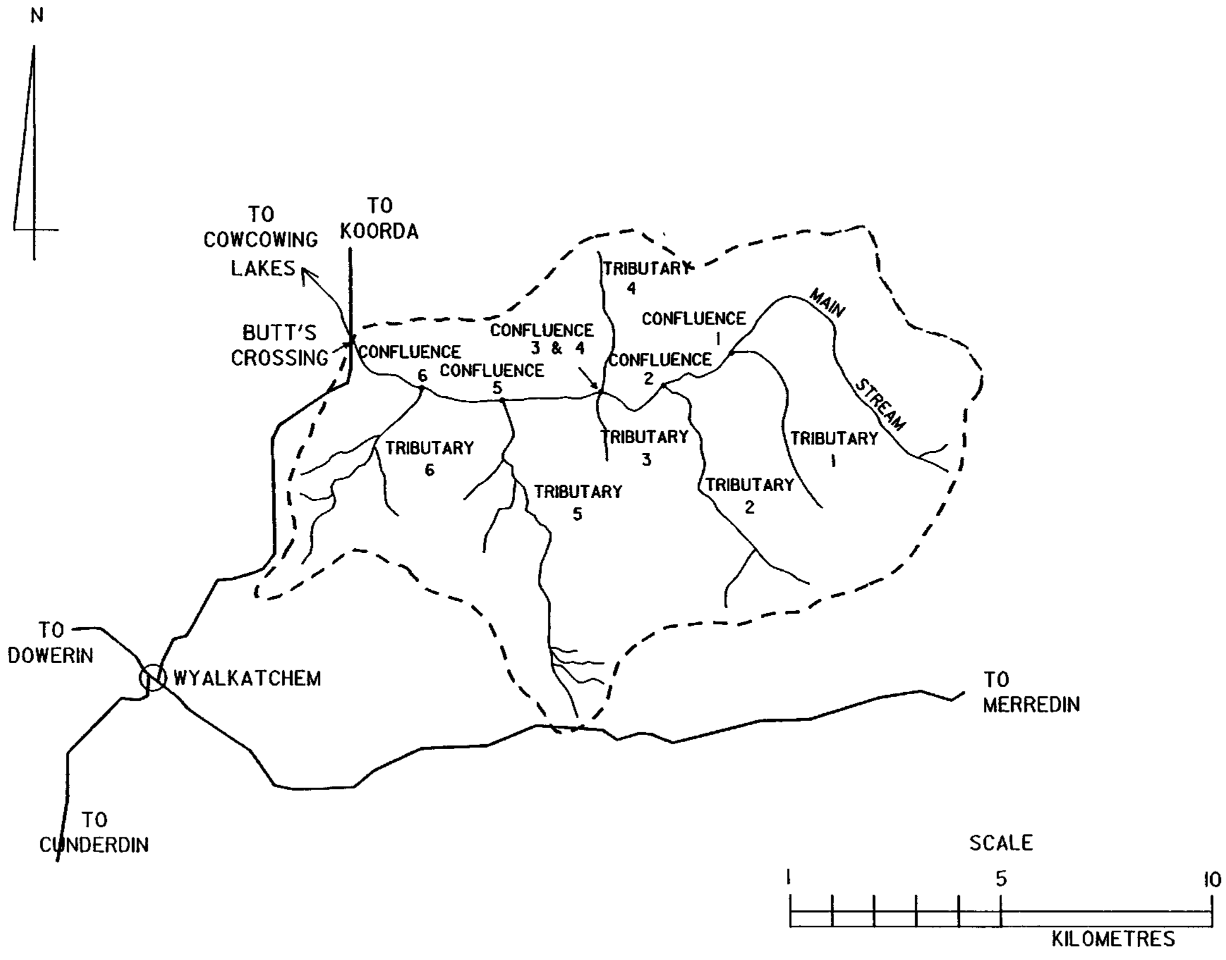


Figure 5.1 Cowcowing Creek Tributaries

For Cowcowing Creek, P = 340 mm and L = 23 km. Therefore IL10 = 35 mm. With an initial loss of 35 mm and a continuing loss of 3 mm/h (ARR, 1987), little runoff would be expected in the 12 hour, five year return period storm (Table 5.1) whereas flooding at Butt's Crossing is reported to occur about once every three years. For the following simulations, the initial loss was assumed to be 12 mm with a 4 mm/h continuing loss. The effect of using different initial and continuing losses is likely to be considerable and therefore the results obtained from the simulations are considered to be indicative rather than absolute. This problem in estimating initial and continuing losses highlights a need for information of flood levels in historic rainfall events to allow calibration of the model. A 12 hour storm duration was selected in this example as the objective was to assess the effect of structures on a runoff hydrograph rather than to accurately estimate a flood of a particular average recurrence interval.

The excess rainfall patterns for 12 hr duration storms in 2 hr time increments with average recurrence intervals of 5 to 100 years are presented in Table 5.3.

Table 5.3 Excess rainfall pattern - 12 hr storm

Average recurrence interval (years)	**Rainfall (mm)**	**Time (hrs)**						**Percentage runoff**
		0-2	**2-4**	**4-6**	**6 -8**	**8-10**	**10-12**	
5	Total	12.5	25	6.5	3	2	1	-
-	Excess	0	17	0	-	0	0	34
10	Total	14.7	29.5	7.7	3.5	2.4	1.2	-
-	Excess	0	21.5	0	0	0	0	36
20	Total	17	34	8.8	4 .1	2.7	1.4	-
-	Excess	0	26	0.8	0	0	0	39
50	Total	20.5	41	10.7	4 .9	3.3	1.6	-
-	Excess	0.5	33	2.7	0	0	0	44
100	Total	23.7	47.5	12.4	5.7	3.8	1.9	-
-	Excess	3.7	39.5	4.4	0	0	0	50

5.3 Runoff-routing parameters

As explained in Appendix A, the general runoff (Q) versus storage (S) equation of the RORB model is:

$S = Kr.\ Kc\ Q^m$

where Kr, Kc and m are model parameters for each sub-area (Laurenson and Mein, 1985).

With no stream flow data on the catchment for parameter estimation, a value of m = 0.8 was chosen as recommended by Flavell (1983). Relative delay times were based on L.S~0-5 for each sub-area where L is channel length and S is channel slope (Laurenson and Mein, 1985).

Parameter Kc was calculated from the regional equation for the Western Australian wheatbelt (Flavell, 1983):

$Kc = 3.26 \ A^{0.43} \ S^{-0.72}$

where A is total catchment area (187.5 km^) and S is mainstream slope (0.2 m/km), giving a Kc value of 96.8 for Cowcowing Creek.

5.4 Natural catchment model

The natural catchment was modelled with the ten-year return period storm and data as described above. Hydrographs of the tributaries and main stream are shown in Figures 5.2 and 5.3.

Several features are noted from the hydrographs:

1. The peak flow at Butt's Crossing is 13 m^3/s after 44 hours (Figure 5.3).
2. Tributary 6 peaks at 14 hours and has receded to 0.9 m^3/s (15 per cent of its peak value) at the peak of the mainstream hydrograph (at 32 hours). It therefore adds only 8 per cent to the peak flow at Confluence 6 (Figure 5.3) and little to the Butt's Crossing hydrograph peak.
3. Tributary 5 peaks at 16 hours, almost simultaneously with the mainstream hydrograph at Confluence 5 (Figure 5.3). Hence Tributary 5 contributes significantly to the later mainstream peak at Butt's Crossing.
4. Tributary 2 peaks at 14 hours and has reduced to 50 per cent of the peak by the time the mainstream hydrograph peaks at Confluence 2 (Figure 5.2).
5. Tributary 1 peaks at 6 hours and is coincident with the peak of the mainstream hydrograph (Figure 5.2).

Summary results in Table 5.5 on peak flows and volumes illustrate some of these points.

Table 5.5 Natural catchment results

Location on catchment (Figure 5.1)	Catchment area (km^2)	Time to peak (hrs)	Peak flow (m^3/s)	72 hr runoff volume (Mm^3)	Total runoff volume (Mm^3)
u/s* of tributary 1	30.5	6	6.08	0.63	0.66
Tributary 1	14.3	6	5.56	0.30	0.31
u/s of tributary 2	58.0	30	5.68	1.00	1.26
Tributary 2	22.8	14	4.74	0.48	0.49
u/s of tributary 3	89.0	34	7.94	1.44	1.94
Tributary 3	2.8	6	3.01	0.06	0.06
Tributary 4	12.0	8	3 83	0.26	0.26
u/s of tributary 5	111.3	40	8.32	1.73	2.42
Tributary 5	41.3	16	9.27	0.87	0.89
u/s of tributary 6	161.3	32	13.21	2.38	3.50
Tributary 6	18.5	14	6.20	0.40	0.40
at Butt's Crossing	187.5	44	13.0	2.36	4.08

* u/s = upstream.

5.5 The effect of large storages

The following results are from computer simulations of the catchment to Butt's Crossing, with portions of the catchment assumed to have zero flow. This has been modelled by placing an infinite storage at the downstream point of the area to have zero flow. The simulations show the magnitude of the changes to the peak flow and volume at Butt's Crossing which could be achieved by a large structure. They are not to be taken as a recommendation that such structures should be built, but as a useful means by which the flood response of the catchment can be better understood.

All simulations use the same input data for rainfall, losses and runoff routing parameters as used for the natural catchment described above. The Kc value was unchanged as the same total area (187.5 km2) was used.

The results are shown in Table 5.6. For example Tributary 1 is 8 per cent of the total catchment area and with no outflow from this tributary, the peak flow at Butt's Crossing is reduced by only 2 per cent compared with the natural catchment result (Table 5.5). The 72 hr runoff volume is reduced by only 4 per cent.

The prevention of runoff from Tributary 5 results in an appreciable reduction in peak flow of 40 per cent, nearly twice the 22 per cent reduction in the catchment area.

Table 5.6 The effect on peak flow and 72 hour volume of infinite storages on different tributaries and on the mainstream

col 1 Area assumed to have infinite storage	col 2 % of this area of total catchment	col 3 % reduction in peak flow at Butt's Crossing	col 4 % reduction in 72 hr volume at Butt's Crossing
trib. 1	8	2	4
trib. 2	12	9	11
tribs 3 & 4	8	16	12
trib. 5	22	40	32
tribs 6	10	10	17
main/s* u/s of trib. 1	16	3	6
main/s u/s of trib. 2	31	8	13
main/s u/s of trib. 3 & 4	47	21	27
main/s u/s of trib. 5	59	43	43
main/s u/s of trib. 6	86	73	78

Notes:

col 2 = 100 x col 1/187.5.

col 3 = 100 x (13 - peak flow)/13 (see Table 5.5).

col 4 = 100 x (4. 08 - 72 hr volume)/4.08 (see Table 5.5).

* main/s = mainstream.

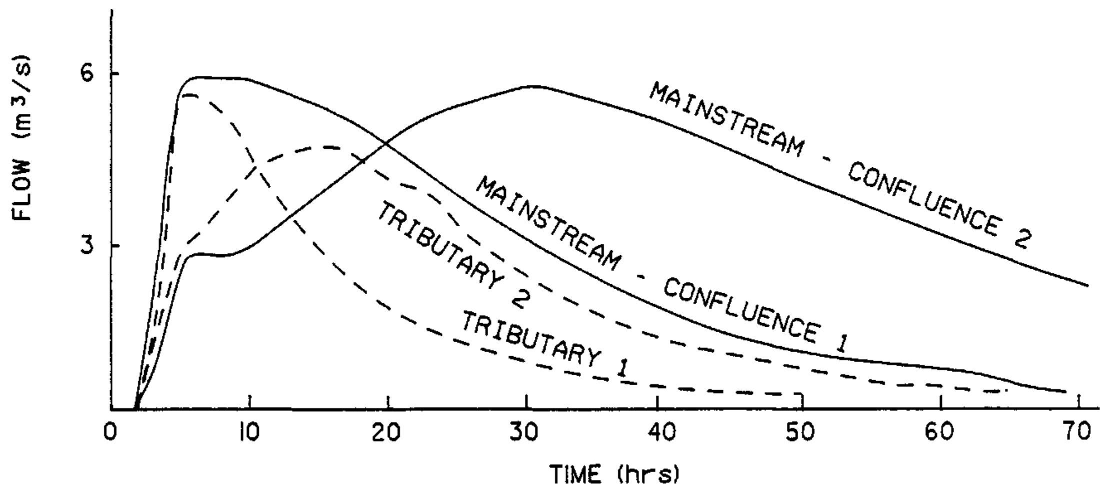


Figure 5.2 Hydrographs At Confluences 1 And 2

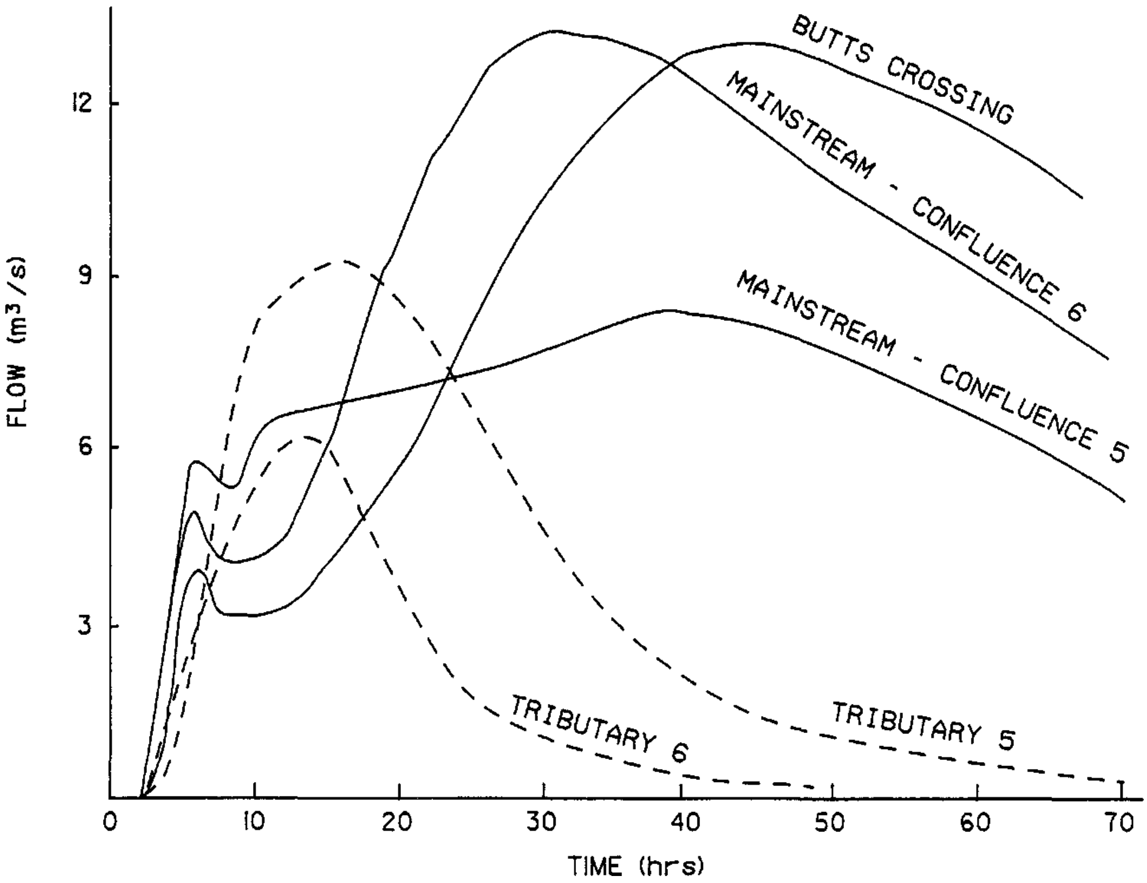


Figure 5.3 Hydrographs at Confluences 5 and 6 and At Butts Crossing

The prevention of runoff from the mainstream catchment upstream of Tributary 5 reduces peak flows comparable (43 per cent) to Tributary 5, but requires that 59 per cent of the total catchment be reduced to zero flow.

Therefore if any storage structures were to be designed, focus should be centred on Tributary 5. This is probably due to the positioning of the catchment isochrones (lines joining points with the same travel time for overland flow and runoff) and suggests that other sub-areas with similar isochrone values to Tributary 5 be considered.

Further simulations were done to test the effect of delaying the Tributary 5 hydrograph in an attempt to avoid the coincidence of peaks from this tributary and the mainstream as noted above for the natural catchment. An imposed 6 hr delay resulted in negligible change to the Butt's Crossing peak flow due to the prolonged plateau on the mainstream (Figure 5.4).

A retarding basin with spillway and low level pipe outlet was modelled. The 20 m long spillway was set at 1.5 m elevation above datum (original ground level). A 25 m long pipe with invert level of 0.5 m above datum was used to empty the structure after a flood occurred.

A flooded area of 2.0 to 2.5 km^2 was estimated from 1:50,000 topographic maps, giving a storage to overflow of 6.0 to 7.5 Mm^3, as compared with the ten-year storm runoff volume in 48 hours of 3.3 Mm^3 at this location. At Butt's Crossing the peak flow and 72 hr runoff volume were reduced by the amounts shown in Table 5.7. Assuming a 3.0 m high bank, top width 2.0 m, with 1:3 side slopes of required length 1.5 km, the cost of the retarding basin is \$250,000 at \$5/m^3 compacted fill, exclusive of spillway and pipework.

Table 5.7 Results with retarding basin just downstream of Confluence 5

Average recurrence interval (yrs)	% reduction in peak flow at Butt's Crossing	% reduction in 72 hr runoff vol. at Butt's Crossing	Max. height of water surface above spillway (m)
10	63	70	Not filled
20	63	71	0.13
50	63	70	0.34
100	60	63	0.55

Note: 12 hr duration storm used.

5.6 The effect of dams and banks

To simulate the effects of building farm dams, 36 dams each of 4000 m^3 capacity were modelled fairly evenly on Tributary 5. The total storage of these dams is 0.15 Mm^3 compared with the ten-year return period storm runoff volume of 0.89 Mm^3 from the tributary (Table 5.5). This low storage fraction means that only the rising limb of the hydrograph (Figure 5.6) is affected. The effect is that water enters the dams until they are full, after which they have negligible flood attenuation. The peak flow at Butt's Crossing in the ten-year event is reduced by only 5 per cent by these dams. At a cost of \$0.70/m^3 for excavation, the total estimated cost of 0.15 Mm^3 storage is \$105,000.

Absorption banks (storage of 5 m^3/m) on a given sub-area of the catchment were modelled by increasing the initial loss in that sub-area by an additional 20 mm. Since prior results indicated that Tributary 5 is where any works should be located, this additional loss was applied to all sub-areas in Tributary 5. Furthermore, in a second test, the loss was also applied to areas in Tributary 3 and Tributary 4. This was done because the isochrones in these locations were similar to those on Tributary 5 (Figure 5.1).

Assuming 1 km of bank for each 0.2 km^2 (20 ha) the treated catchment of 57.2 km^2 requires 286 kms of banks. Assuming cost of construction of \$500/km of bank, approximate costs are:

1. Treatment of Tributary 5 - \$103,000

2. Treatment of Tributaries 3, 4 and 5 - \$140,000

The results are summarized in Table 5.8.

Table 5.8 The effect of absorption banks on flood runoff

Treatment		% reduction Q peak at Butt's Crossing	% reduction volume of runoff in 72 hrs	% reduction total volume of runoff
1	Banks on Tributary 5	34	27	18
2	Banks on Tributaries 3, 4 and 5	42	36	24

5.7 Conclusions

The options are summarized in Table 5.9.

Table 5.9 The effect of dams and banks on flood runoff

Option no.	Option	% reduction in peak flow at Butt's Crossing	Cost ($100,000)
1	No works	0	0
2	36 x 4000 m3 dams on Tributary 5	7	1.0
3	Retarding basin d/s of Confluence 4	63	2.5 +
4	Absorption banks on Tributary 5	34	1.0
5	Absorption banks on Tributaries 3, 4 and 5	42	1.4

Considering the options in terms of the percentage reduction per unit cost, the most effective option is 4 (absorption banks on Tributary 5) followed, in order of decreasing effectiveness, by options 5, 3 and 2. There are additional benefits of water storage in option 2 which need to be considered. However, the dams had little effect on flood peaks. This result is important as farm dams are often advanced by farmers as a means of reducing flood peaks.

The modelling has highlighted the advantages of locating storage structures in that part of the catchment which contributes most water at a critical time. It may be possible to further improve the location of absorption banks by examining the isochrones for the catchment. Thus if without banks the catchment peaks after 44 hours, absorption banks should be located in each sub-catchment where the 44 hour isochrone passes. Given the likely errors in estimating travel times, it would be more realistic to locate banks between the 40 and 50-hour isochrones.

The relative effectiveness of the different options outlined in Table 5.9 are unlikely to be changed by using different initial and continuing losses for the storms, but the percentage reduction and cost-effectiveness will be affected. Had the higher initial losses now recommended for wheatbelt catchments been used, in addition to the lower continuing losses, the absorption banks on Tributary 5 may be cost effective in terms of reducing flooding at Butt's Crossing.

6. Beacon Catchment Study

6.1 Introduction

The Beacon Catchment (1375 km2) drains into a salt lake south west of Beacon township (Figure 6.1). In the eastern and southern parts of the catchment, drainages are indistinct due to the flatness of the valley floor.

A reach of the natural drainage channel west of Beacon has been confined by the construction of levee banks (Figure 6.1). The altered reach is approximately 1.5 km long and discharges into a reach approximately 10.0 km long which drains to the salt lake. The channel gradient is slight (about 0.001) and probably varies locally along the total channel length of 11.5 km, leading to large areas of surface detention in wet years. The simulation was performed to determine the effect of the levee banks on the flood wave downstream.

This problem is not an easy one to provide answers to without considerable field work to determine channel cross sections and bed levels. Also the lack of any stream flow data makes the estimation of likely flood magnitudes difficult. As a first step, an idealized channel was analysed hydraulically as set out below. In the remainder of this section, comments are made on the general effects of catchment and channel changes.

Catchment alterations involving clearing, grazing, cropping, urbanization and any conservation practices may cause changes in the delivery of water and sediment to the drainage channel. Adjustments may then occur to the channel itself in the form of changes in width, depth, meander wavelength and slope. Channel alterations may also be applied directly through levee building and channel straightening for flood protection and through weir or dam construction for water supply off take.

Warner (1985) noted that the consequences of these direct channel alterations have not been predicted with any certainty. In the case of channel straightening or levee construction the desired effect (generally lower flood level) may be achieved in the treated reach, but upstream and downstream effects may be present also. In particular, velocities of flow are likely to be higher in the treated reach, enabling higher local sediment transport and resulting in increased sedimentation downstream where velocities are lower. The imposed reduction in overbank, floodplain storage may also increase flood peaks downstream. Clearly where land values are high (e.g. urban areas) these consequences may be tolerated if high value areas are protected. However, in rural areas, with more uniform land values, the downstream consequences of levee building may be viewed differently.

The lack of data in most catchments, before and after channel improvements, makes these effects difficult to quantify except in general terms. However by identifying the main physical factors, general statements can be made about the consequences of a particular channel treatment and this has been the procedure adopted here. The hydraulics of open channel flow have been used to calculate the effect of channel alterations on flood waves for idealized situations. In the absence of any stream flow data for the catchment this is considered to be a realistic approach. Considerable

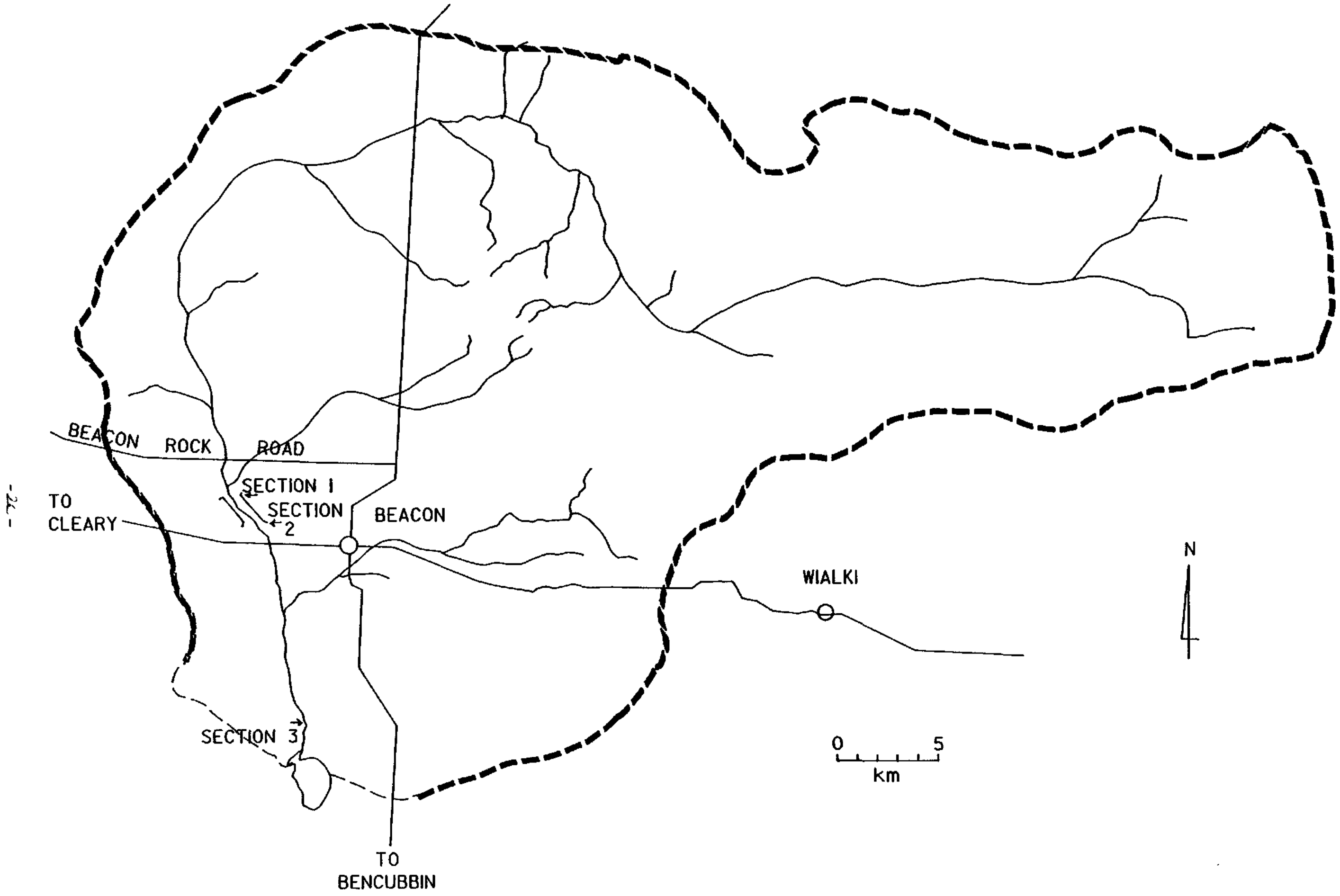


Figure 6.1 Beacon Catchment Showing Sections I, 2 & 3

field work would be needed to simulate the hydraulic conditions for the actual drainage channel geometry and it is believed that the results presented show the typical magnitude of the alterations to a flood wave. No analyses of sediment loads have been made.

When a flood wave passes through a channel reach, the inflow and outflow hydrographs at the upstream and downstream ends of the reach, respectively, are as shown in Figure 6.2. Assuming a negligible loss or gain of water in the reach, the total areas under the hydrographs are equal, as the volume of flood water is unchanged. As shown in Figure 6.2, the flood peak is reduced and delayed. The difference between the ordinates of the inflow and outflow hydrographs, is equal to the rate of change of storage of water in the reach:

i.e: $\Delta S/\Delta t = I-0$

Where $\Delta s/\Delta t$ is the change in storage during a period of time At, I is the average inflow during Δt and 0 is the average outflow during Δt. The value of $\Delta s/\Delta t$ is positive when storage is increasing and negative when storage is decreasing. This equation forms the basis for a hydrologic procedure of flood routing.

6.2 Rainfall and loss estimation

A ten-year average recurrence interval rainfall of 12 hours duration was modelled on a 800 km2 catchment above the levee banks. The time of concentration of the catchment is about 10 hours (ARR 1977) which may be low due to the low gradients. The total rainfall was 42.2 mm and an initial loss of 24 mm together with a continuing loss of 2.5 mm/hr, resulted in 6.2 mm excess rainfall or 15% runoff, which is considered realistic.

6.3 Runoff-routing parameters

The RORB model parameters for the catchment were calculated from a mainstream length of 80 km, a slope of 1 m/km. Using equations given by Flavell et al. (1983), m = 0.8 and Kc = 48.0.

Downstream of this catchment a simplified channel was modelled. The channel has a 1.5 km reach followed downstream by a 10.0 km reach. Each of the two reaches has a constant width and rectangular cross section (Figure 6.3).

The effect of levee bank construction was simulated by varying the width of channel in each reach. A constant river bed slope and bed roughness were assumed for both channels. The values of K and m were calculated for uniform flow in a wide rectangular channel using Manning's formula (Mitchell and Laurenson, 1983). The expressions are

$$K = Ln^{0.6}B^{0.4}/S^{0.3}$$

where L = reach length (m)
n = Manning's n value
B = width of channel (m)
S = bed slope
m = 0.6

Manning's n value has been taken as 0.035 throughout and the bed slope S was taken as 0.001. Values of K corresponding to the reach length and channel widths studied are presented in Table 6.1. It is assumed that the present natural channel has an equivalent width of 600 m and that the present levee banks have an equivalent channel width of 100 m. Other widths have been included for comparison.

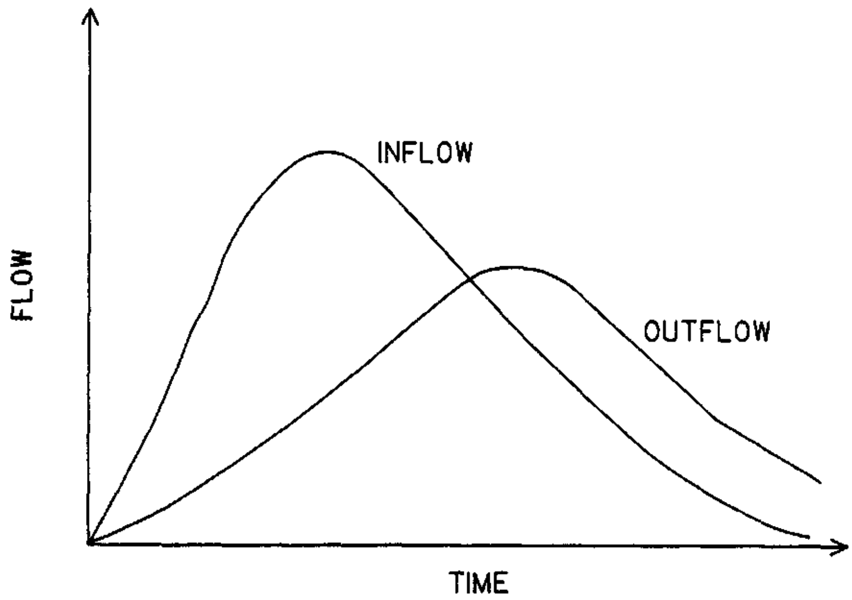


Figure 6.2 - Relationship Between Inflow And Outflow In A Channel During A Flood

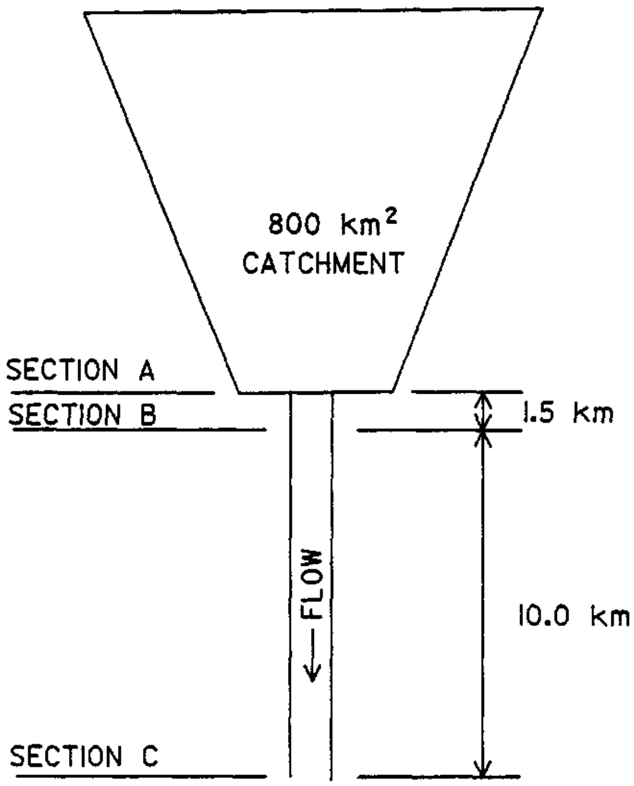


Figure 6.3 Layout Of The Catchment And Channels Studied

Table 6.1 Values of parameter K

Channel width (m)	Reach length (m)	
	1500	**10000**
50	2.1	14.0
100	2.8	18.5
300	4.3	28.7
600	5.7	37.9

6.4 The effect of levees

The peak flow at the upstream end of the 1.5 km reach (Section A, Figure 6.3) is 62.0 m3/s and the runoff volume in the first three days (known as the three-day runoff volume) is 4.63 Mm^3, (1 Mm^3 = 1 million cubic metres), compared with a total runoff volume of 5.04 Mm^3.

There are two effects which will be considered, namely, the effect of channel width on peak flow at a downstream point and the effect of channel width on three-day runoff volume at a downstream point. The total runoff volume at this point over a longer period would equal that at the upstream point, since it has been assumed that there is no loss or gain of water in the course of flow through a reach.

Effect of channel width (in the 1.5 km reach) on flooding at the downstream end of the 1.5 km reach

Figure 6.4 shows that the peak flow at the downstream end of the 1.5 km reach (Section B, Figure 6.3) increases as the channel width in this reach is reduced. The peak flows at Section B are 53.3, 55.6, 58.5 and 60.0 m3/s for widths of 600, 300, 100 and 50 m respectively, representing peak flow increases of 0, 4, 10 and 13 per cent relative to the 600 m wide channel value.

The three-day volume at Section B also increases as the channel width in this reach is reduced (Figure 6.5). The three-day runoff volumes at Section B are 4.56, 4.58, 4.60 and 4.61 Mm3 for widths of 600, 300, 100 and 50 m respectively, representing volume increases of 0, 0, 1, and 1 per cent relative to the 600 m wide channel value. These increases in three-day runoff volume are obviously less marked than the increases in peak flow.

Effect of channel width (in the 1.5 km reach) on flooding at the downstream end of the 10.0 km reach

The conditions at the downstream end of the 10.0 km reach (Section C, Figure 6.3) are affected by the channel widths in both the 1.5 km reach and the 10.0 km reach as discussed above.

Assuming a certain channel width in the 10.0 km reach, the effect of varying the width of the 1.5 km reach in isolation was determined. With a 600 m wide channel in the 10.0 km reach, the peak flow at Section C increases only marginally as the channel width in the 1.5 km reach is reduced (Figure 6.4). Peak flows at Section C are 5.2, 5.3, 5.3 and 5.3 m3/s for channel widths of 600, 300, 100 and 50 m respectively in the 1.5 km reach, representing peak flow increases of 0, 1, 1, and 1 per cent relative to the 600 m wide channel value.

The three-day runoff volume at Section C also increases slightly as the channel width in this reach is reduced (Figure 6.5). The three-day runoff volumes at Section C are 0.90, 0.91, 0.93 and 0.94 Mm^3 respectively, representing increases of 0, 1, 3 and 4 per cent relative to the 600 m wide channel value. These increases (0 to 4 per cent) are slightly greater than those above at the downstream end of the 1.5 km channel (0 to 1 per cent), because after three days, the peak flow has only just reached the lower section. After a longer period of time (e.g. six days) the increases would be much less, and as stated earlier, over the whole hydrograph there is no increase.

Effect of channel width (in the 10.0 km reach) on flooding at the downstream end of the 10.0 km reach

The peak flow at the upstream end of this reach is affected by the channel width in the 1.5 km reach as discussed above.

Assuming a 100 m wide channel in the 1.5 km reach, the peak flow at the downstream end of the 10.0 km reach (Section C, Figure 6.3) increases as the channel width in this reach is reduced. Peak flows at Section C are 5.3, 7.2, 11.4 and 14.6 m^3/s for channels widths of 600, 300, 100 and 50 m in the 10.0 km reach respectively, representing peak flow increases of 0, 36, 115 and 175 per cent relative to the 600 m wide channel value.

The three-day runoff volume at Section C also increases as the channel width in this reach is reduced (Figure 6.7). The three-day runoff volumes at Section C are 0.93, 1.33, 2.13 and 2.67 Mm3 for widths of 600, 300, 100 and 50 m respectively representing volume increases of 0, 43, 129 and 187 per cent relative to the 600 m wide channel value.

The general result emerges that a reduction in channel width produces both an increased peak flow and an increased three-day runoff volume at a downstream section. In terms of Figure 6.2, the effect of a reduced channel width is to reduce the attenuation of the inflow hydrograph, so that the outflow hydrograph resembles the inflow hydrograph more closely.

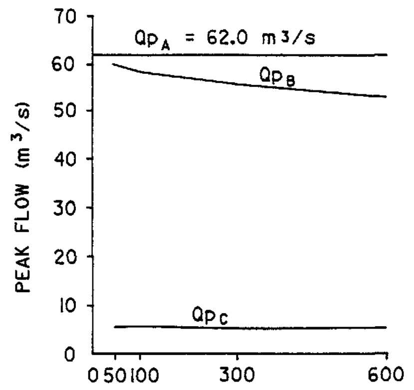
The magnitude of the calculated increases in peak flow appears to be dependent on (a) the length of reach in which the channel width has been reduced and (b) the distance downstream of that reach to the point of interest. As an example of (a), the results presented above show that a reduction in channel width of 600 to 50 m in the 10.0 km section has a larger effect at Section C (175 per cent increase) than a similar width reduction in the 1.5 km reach has at Section B (13 per cent increase).

As an example of (b). Figure 6.4 shows that reducing the channel width in the 1.5 km reach has an effect at Section B but a lesser effect at Section C. Whereas the effects on peak flow at Section B are between 0 to 13 per cent, the effects at Section C are between only 0 to 1 per cent, within the range of channel widths studied.

All the results presented assume no loss or gain of water to the reaches studied. In practice there would commonly be lateral inflow to the reaches so that the peak flow downstream may well be greater than that upstream. Such inflow is likely to occur (Figure 6.1). However even in these circumstances the attenuating effects of the channels would depend on the channel geometry. The analysis has only considered the effects of channel width on a flood hydrograph. There may be further effects including increased sediment transport due to the increased velocity between levee banks, as discussed by Warner (1985).

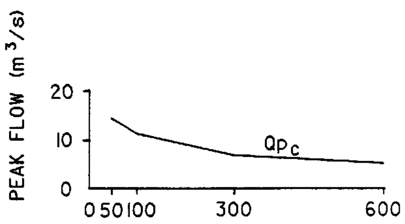
NOTE Qp_A, Qp_B, Qp_C ARE PEAK FLOWS AT SECTIONS A, B AND C RESPECTIVELY.

V_A, V_B, V_C ARE 3 DAY RUNOFF VOLUMES AT SECTIONS A, B AND C RESPECTIVELY



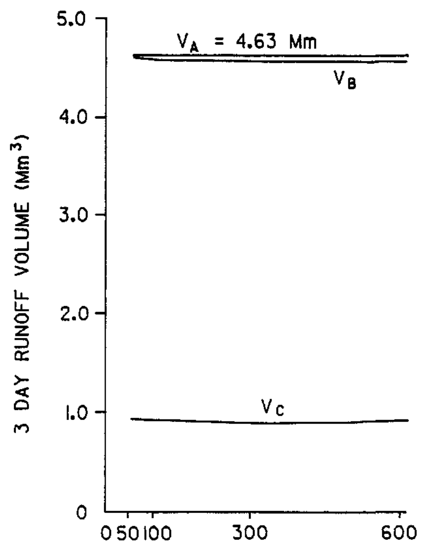
CHANNEL WIDTH, SECTION 1-2 (m)
Qp_C ASSUMES 600m WIDE CHANNEL IN 10km REACH

FIGURE 6.4
PEAK FLOWS DUE TO WIDTH CHANGES IN 1.5km REACH



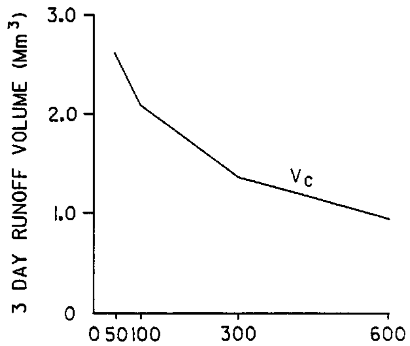
CHANNEL WIDTH, SECTION 2-3 (m)
Qp_C ASSUMES 100m WIDE CHANNEL IN 1.5km REACH

FIGURE 6.6
PEAK FLOWS DUE TO WIDTH CHANGES IN 10.0km REACH



V_C ASSUMES 600m WIDE CHANNEL IN 10km REACH

FIGURE 6.5
3 DAY RUNOFF VOLUMES DUE TO WIDTH CHANGES IN SECTION B (1.5 km REACH)



V_C ASSUMES 100m WIDE CHANNEL IN 1.5km REACH

FIGURE 6.7
3 DAY RUNOFF VOLUMES DUE TO WIDTH CHANGES IN SECTION C (10.0 km REACH)

The levee banks as constructed will lead to a reduction in flood plain storage and therefore increase flood peaks discharge downstream. Assuming that equivalent channel widths of 600 and 100 m are applicable to the natural reach and the levee bank reach respectively, the following conclusions are made.

At the downstream end of the present levees there will be a 10 per cent increase in the flood peak in the event studied (approximately a ten-year average recurrence interval event). However 10.0 km downstream of the present levees the increase will be only 1 per cent. The volume of runoff in the first three days of this flood will be increased by about 1 per cent at the downstream end of the levees, but it will be about 3 per cent greater 10.0 km downstream. In a longer period of time (e.g. six days) these increases would be less and over the total hydrograph there would be no increase. If similar levee banks were constructed in the future in the 10.0 km reach then the flood peak would be increased by about 115 per cent and the three-day runoff volume by about 129 per cent relative to their estimated values at present.

6.5 The effect of a road crossing

The analysis in Section 6.4 started 2 km downstream of the existing road crossing at Beacon Rock Road (Figure 6.1). The analysis therefore ignored the effect of the crossing on flood hydrographs. This section assesses the effect of the crossing on a ten-year average recurrence interval flood hydrograph by extending the study upstream to include both the crossing and the 2 km of natural channel downstream (Figure 6.8).

The same flood hydrograph as in Section 6.4 was used at the upstream end of the modelled creek but as the upstream end has changed between the two sections the results are not comparable. The comparison which can be made is between the computer results presented below which show the effect of the present road crossing (Cases 1 and 2) and the effect of removing the crossing (Case 3). The reason for there being two cases for the present crossing is that no data are available for the size of the pond caused by the crossing nor for the overflow characteristics of the road. The two cases therefore relate to different assumptions about these conditions.

Both Cases 1 and 2 have the same assumed width of overflow along the road (30 m), but Case 2 has a much larger assumed storage upstream of the road, due to the greater width of the flooded area (30 m and 600 m in Cases 1 and 2 respectively) (Table 6.2). In both cases the culverts are assumed to be blocked and a 1:1000 bed slope assumed to give the elevation/storage curves (Figure 6.9). In Case 3 the crossing is assumed to be removed and a floodway constructed along the line of the creek bed so that the flood hydrograph is unaffected.

Table 6.2 Assumed values of parameters for the existing road crossing

Parameter	Case 1	Case 2
Overflow width (m)	30 m	30 m
Width of flooded area	30 m	600 m
Initial water level (see Figure 6.9)	1m	1m

The peak flow and three-day runoff volume at cross-sections on Figure 6.8, are presented in Table 6.3 (a and b) for all three cases. The differences between Cases 1 and 3 are small (less than 1 per cent), due to the assumption that the flooded area upstream of the existing crossing is only 30 m wide (Table 6.3(a)). The result is that removal of the road, as in Case 3, means that the small ponded area assumed in Case 1 is no longer available and the flood hydrograph is hardly affected. Similar comments apply to the runoff volume results presented in Table 6.3(b).

The differences between Cases 2 and 3 are, however, much greater. The large ponded area assumed in Case 2 has a substantial attenuation effect on the flood hydrograph reducing the peak to 42.5 m^3/s at Section E and 39.6 m^3/s at Section A, compared with 62.0 m^3/s and 47.2 m^3/s in Case 3 with the crossing removed.

The natural channel between Sections B and C substantially reduces the peak flow in both cases to 5.0 and 5.1 m^3/s so that there is little difference at Section C. The three-day runoff volume is however, still approximately 17 per cent greater in Case 3 than Case 2 at this section (0.83 Mm^3 compared to 0.71 Mm^3).

Field data are necessary to determine the actual volume of water which would be ponded upslope of the road crossing in a flood. This depends on the road crest level relative to contours upstream of the road. A survey to allow 0.25 m interval contours to be plotted in this area up to 0.5 m above the road crest would be necessary.

Table 6.3(a) Peak flows (m3/s) at each section

Location	Case 1 (little ponding)	Case 2 (substantial ponding)	Case 3 (no ponding)
Section D	62.0	62.0	62.0
Section E	61.3	42.5	62.0
Section A	47.0	39.6	47.2
Section B	46.8	39.5	46.9
Section C	5.1	5.0	5.1

Table 6.3(b) Three-day runoff volumes (Mm3) at each section

Location	Case 1 (little ponding)	Case 2 (substantial ponding)	Case 3 (no ponding)
Section D	4.63	4.63	4.63
Section E	4.63	4.47	4.63
Section A	4.50	4.32	4.50
Section B	4.46	4.28	4.47
Section C	0.83	0.71	0.83

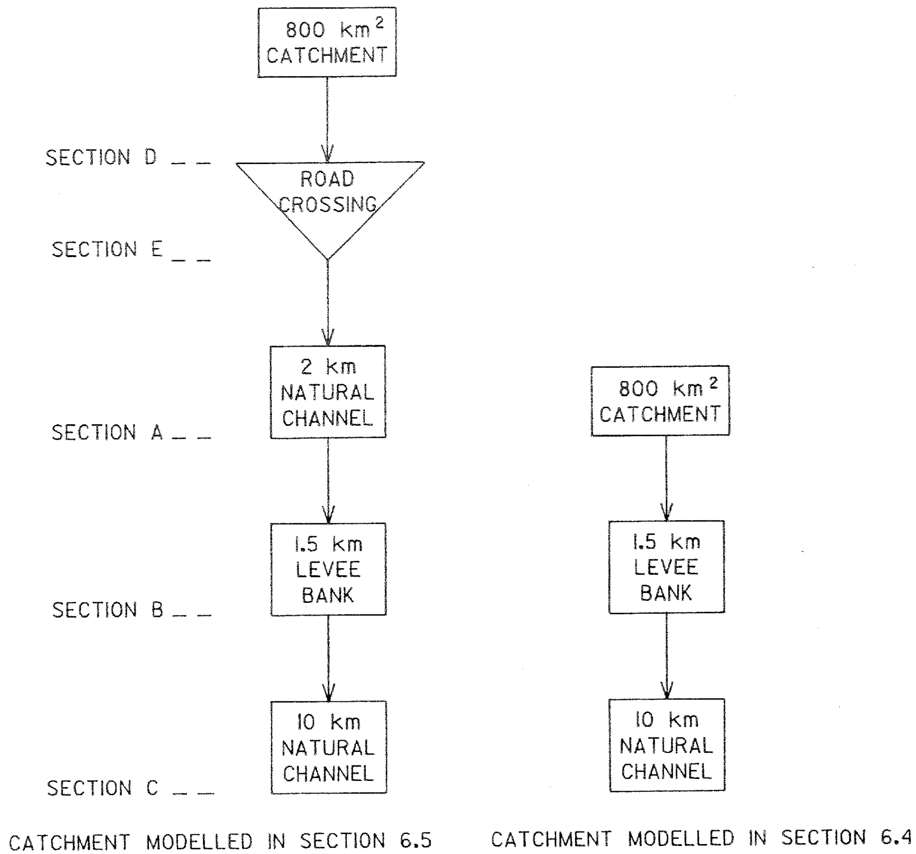


FIGURE 6.8
CATCHMENTS MODELLED

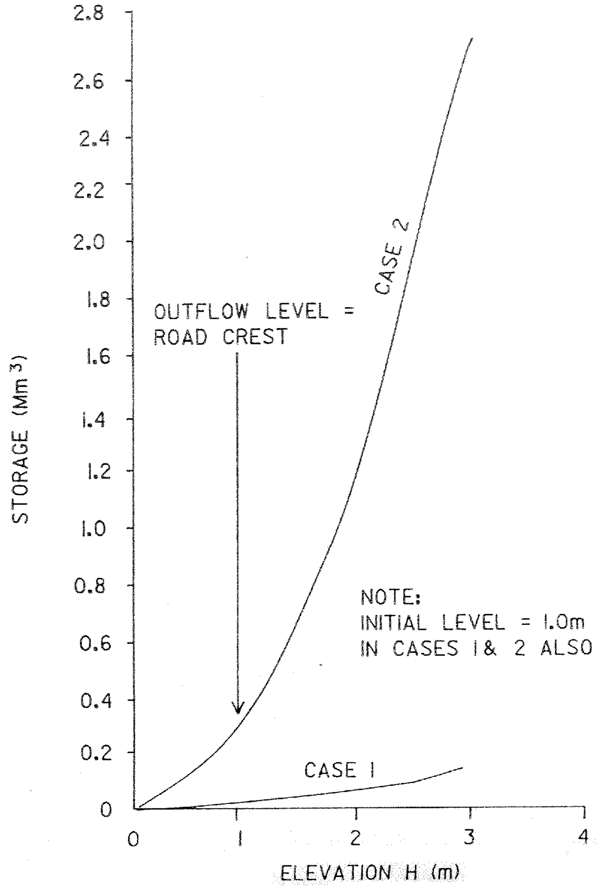


FIGURE 6.9
ELEVATION / STORAGE CURVES

6.6 Conclusions

Levee banks can greatly increase flood peaks when the banks are close together resulting in a large decrease in floodplain storage. However the effect of the levees on peak flows is mainly at their discharge end with their effect 10 kilometres downstream being greatly attenuated for the situation simulated at Beacon. Over a long period (e.g. six days) the levee have little affect on flood volumes. Road crossings can cause a substantial attenuation in flood peaks and therefore can greatly affect the results predicted for the levee bank system. This effect of road crossings has been referred to in several flood reports reviewed in Section 3. The effect of the road crossings on flood volumes (Table 6.3(b)) is substantially less than for flood peaks for the assumptions used in this simulation.

7. West Nugadong Catchment Study

7.1 Introduction

The West Nugadong Soil Conservation District is a natural catchment of about 380 km2 in the northern wheatbelt of Western Australia (Figure 7.1). More than 95 per cent of the area has been cleared of native vegetation for agriculture.

During recent years up to 10 per cent of the area has become salt affected and is no longer suitable for cereal production. Since 1983 experimental drains have been constructed in the area. Pumps have been installed to drain an experimental area, although low conductivity subsoils make this difficult. The pressure of the deep and shallow watertables have been monitored (Ross George, pers. comm.).

Improving the surface drainage has been one recommendation for alleviating waterlogging in the area. However the drains may worsen the flooding of the arable land downstream. This section looks at the effect of surface drainage on the flooding above a road crossing, at the downstream end of the Soil Conservation District (Figure 7.1).

The effect of channel improvements and surface drainage on catchment flood hydrology has been the source of much discussion over many years, but of relatively little scientific research. Most related work, undertaken in North America and the USSR, has tended to concentrate on the hydrological consequences of agricultural operations, such as major changes in land use or cultural practices, rather than drainage alone.

Intensive studies in 12 catchments in Northern Ireland showed that post-drainage flood peaks can increase by between 3 and 100 per cent relative to pre-drainage flood peaks (Bailey and Bree, 1980). Flood peaks increased with increasing mean annual rainfall and decreased with the proportion of the catchment draining through lakes (Figure 7.2). Although the data are for Northern Ireland and may not be representative of Western Australian conditions, they are the only data known to the authors. Considering the above mentioned relationship, an increase of between 0 and 10 per cent of the flood peak could be expected for the study area (Figure 7.2).

7.2 Rainfall and loss estimation

The method for estimating storm rainfall and loss rates in the West Nugadong area was as follows:

1. The duration of the design storm (in hours) was taken as the time of concentration of the catchment (tc), which is the time taken for runoff to travel from the most remote point in the catchment to the catchment outlet. tc was estimated from ARR (1977):

tc = 0.76 A0-38

where A is catchment area in km^2

For A.= 322 km^2, tc = 6.8 hours

2. The ten-year average recurrence interval event was used for the design storm.

3. The ten-year average recurrence interval, 6-hour rainfall intensity for the area was estimated to be 8 mm/h (McFarlane, 1986).

4. For the temporal pattern of rainfall bursts, the graphs provided in ARR (1977) were used.

5. The Initial Loss (IL) was calculated, using the equation (Flavell and Belstead, 1986):

$$IL = 460 P^{-0.41} L^{-0.08}$$

where L = mainstream length (km) and P = mean annual rainfall (mm)

For P = 320 mm (mean annual rainfall for Wubin) and L = 28.0 km, IL = 33.12 mm

6. 3 mm/h was assumed as the Continuing Loss (CL) (Flavell and Belstead, 1986).

7.3 Rainfall-runoff parameters

For calculating the peak flood values from rainfall data, the runoff routing model RORB (Laurenson and Mein, 1985) was used.

The parameters adopted for the RORB model were as follows:

Time increment: 1.00 hour
Rainfall pattern: 3.8, 12.0, 20.0, 6.7, 3.8, 1.7 (mm), total = 48.0 mm
Routing parameters: M = 0.80, $Kc = 1.06 L^{0.87} s^{-0.46}$
where L = mainstream length (km), S = equivalent uniform slope (m/km)
For L = 28.0 km and S = 1.3 m/km, Kc = 17.06 (Flavell and Belstead, 1986)
Duration of calculation = 36 hours.

The pre-drainage peak flow, as calculated by RORB is 40.6 m3/s and occurs 5 hours after the start of the design storm. The post-drainage peak flow is therefore estimated as 10 per cent higher, namely 44.7 m^3/s.

7.4 7.4 Backwater calculation data

For a given flow, the elevation of the water surface along the creek is known as the "backwater" and is determined by consideration of the hydraulic controls. The backwater curves have been determined by the direct step method (Chow, 1959).

The average slope of the creek is 0.0002. The typical cross-section of the floodway was idealized as a rectangular channel with a bed width of 50 m, as estimated from 1:50,000 topographic maps and aerial photographs (Figure 7.3). The channel side slope was estimated to be 1:200.

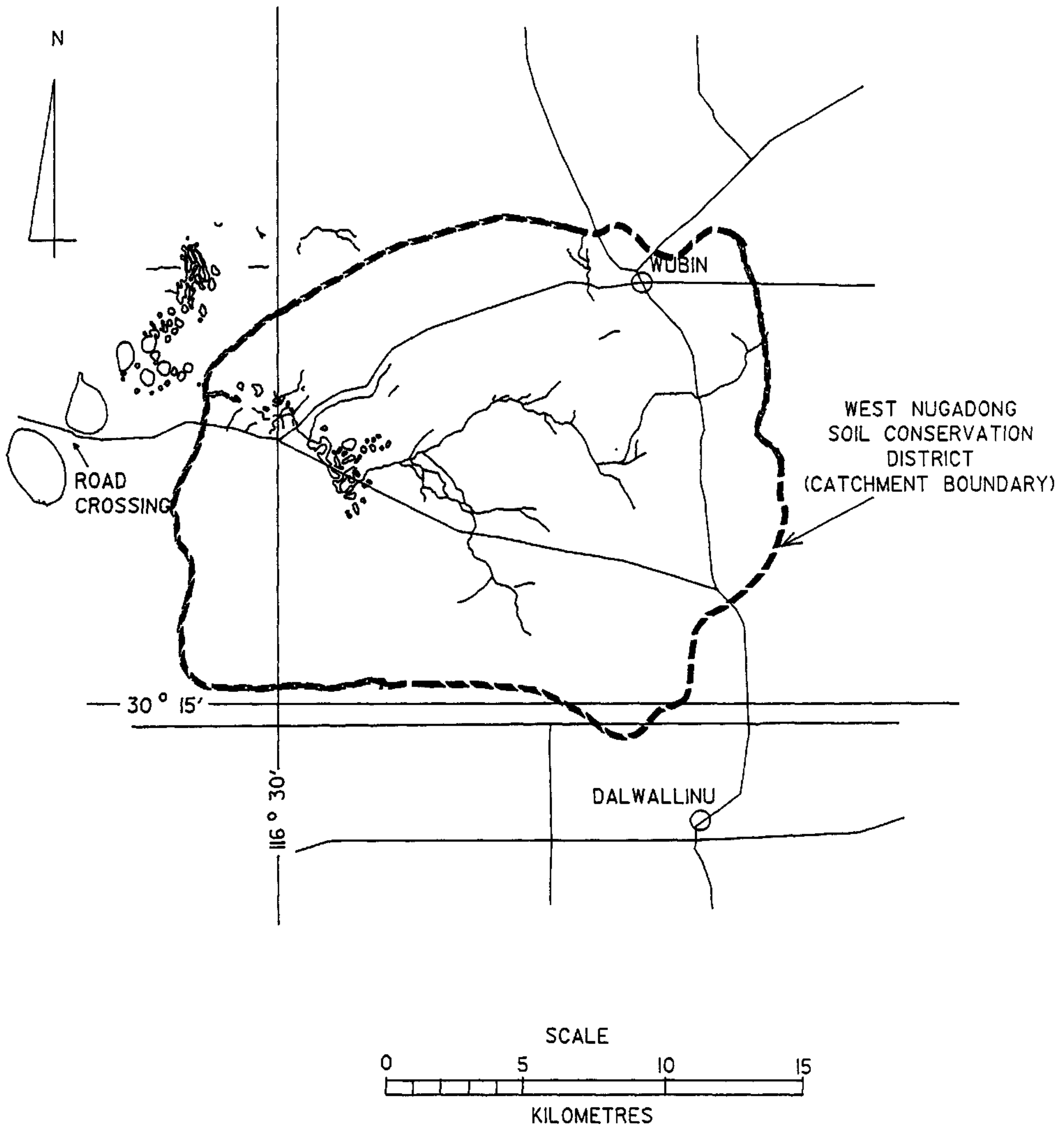


Figure 7.1 Location Map - West Nugadong

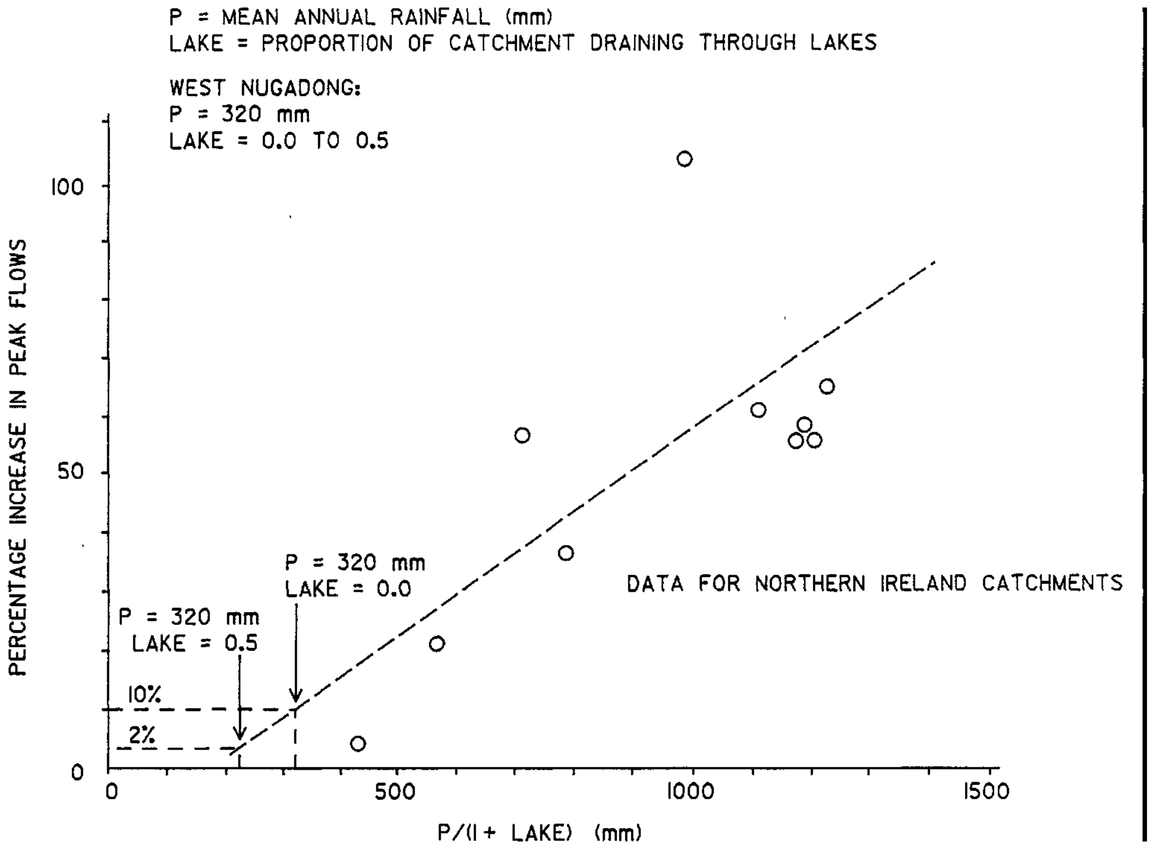


Figure 7.2 Effect Of A Lake And Rainfall On Peak Flow

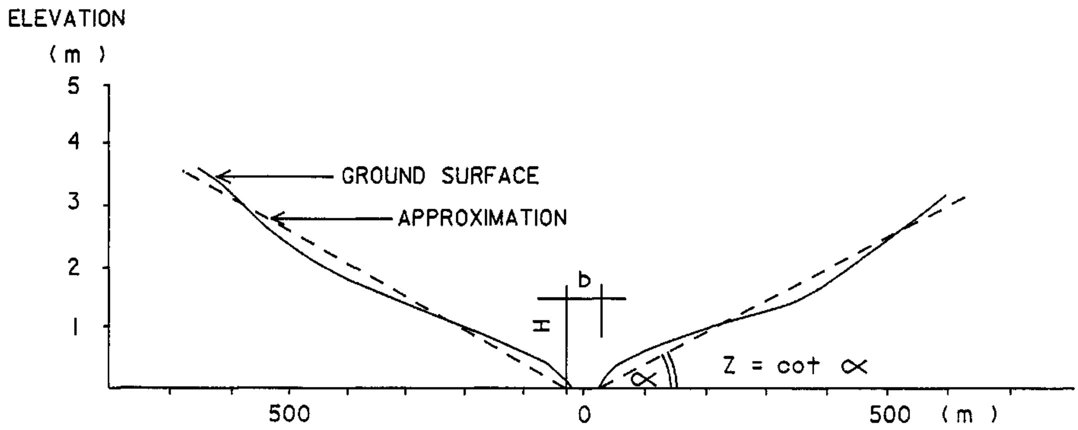


Figure 7.3 Typical Cross-section of the Main Channel

The road crossing was considered to be a broad-crested weir, with a length of 400 m. The depth of water above the crest was calculated for both pre- and post-drainage situations. The weir equation is:

$$Q = 1.71\,BH^{1.5}$$

where Q = flow (m^3/s), H = depth of water above the crest (m), and B = weir length (m).
Manning's formula for open channel flow was used,

$$Q = \frac{A\,R^{0.67}\,S^{0.5}}{n}$$

where Q = flow (m^3/s), A = cross sectional area (m^3).
R = hydraulic radius of flow cross-section (m)
S - friction slope
and n = Manning's roughness coefficient, assumed to be 0.04.

7.5 Water levels with existing drainage

The pre-drainage normal depths are shown in Figure 7.4. The normal depth of flow for the ten-year average recurrence interval flood is 0.85 m. This depth applies both downstream of the road and at a distance upstream of the road beyond the influence of the road backwater.

The road culverts are assumed to be blocked by debris so that flow occurred over the road crest only. Under these conditions the road crossing will act as a broad-crested weir and will increase the depth of water in the creek above the normal depth. At West Nugadong the road crest is approximately 1.9 m above bed level (Figure 7.5). The road crossing will cause a rise in water surface upstream over a certain distance.

The width of the flooded area depends on the bed width, depth of water and side slope of the creek. This width is at a maximum immediately upstream of the road crossing and gradually decreases to a minimum as the effect of the road crossing diminishes. At a distance upstream of the road crossing, the width of the flooded area can be calculated by the formula:

$$w = b + 2z\,(\Delta x + h)$$

where w = width of the flooded area (m)
b = bed width of the idealized channel (m)
z = side slope of the channel
h = the normal depth of water without the effect of the road (m)
Δx = depth of the backwater at the cross section (m)
No numerical values are included here.

7.6 Water levels with improved drainage

The maximum increase of the flood peak due to an improved surface drainage system, is estimated as 10 per cent (or 4.1 m^3/s) of the ten-year average recurrence interval flood peak. Such drainage improvements increase the normal depth of flow in the natural floodway by about 4 cm (Figure 7.4).

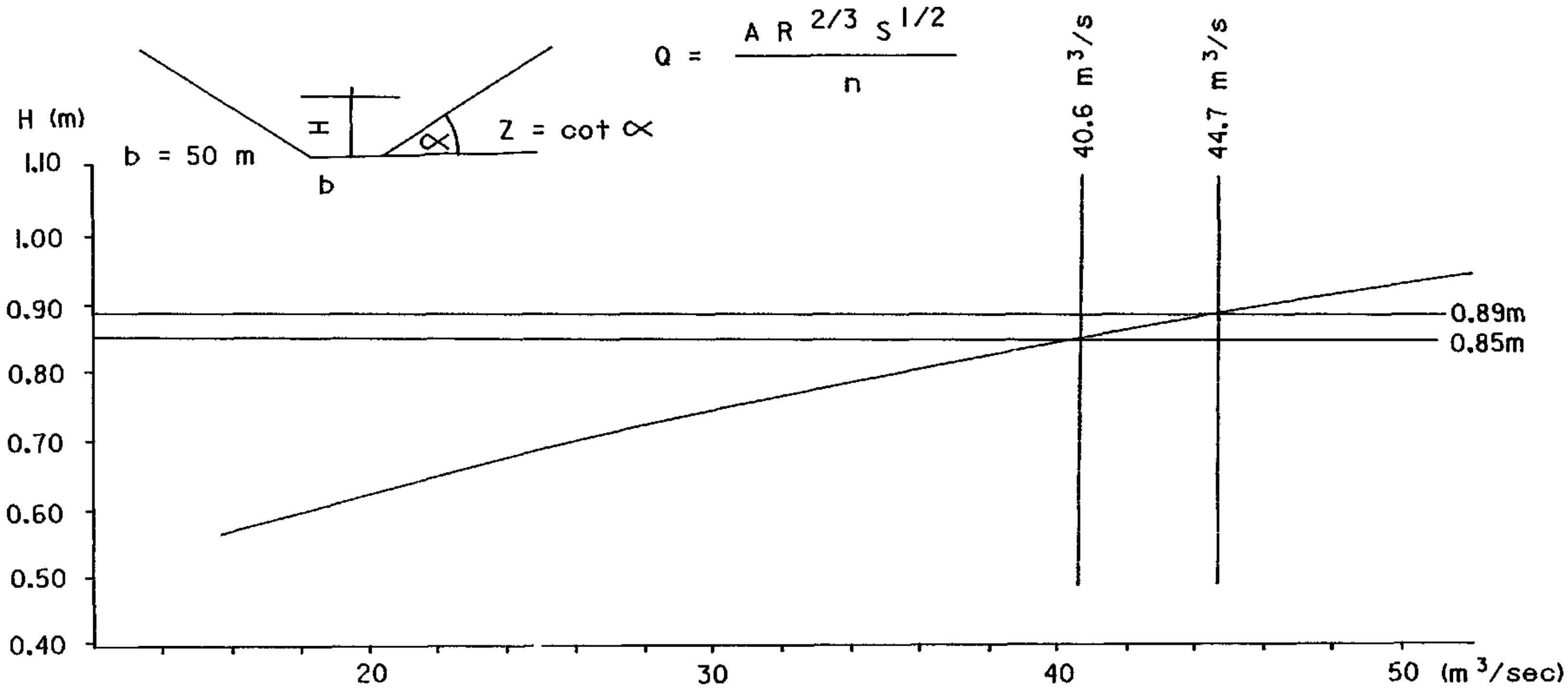


Figure 7.4 - Depth Of Flow As a Function Of Discharge for a Slope of 0.0002, b of 50m and z of 200

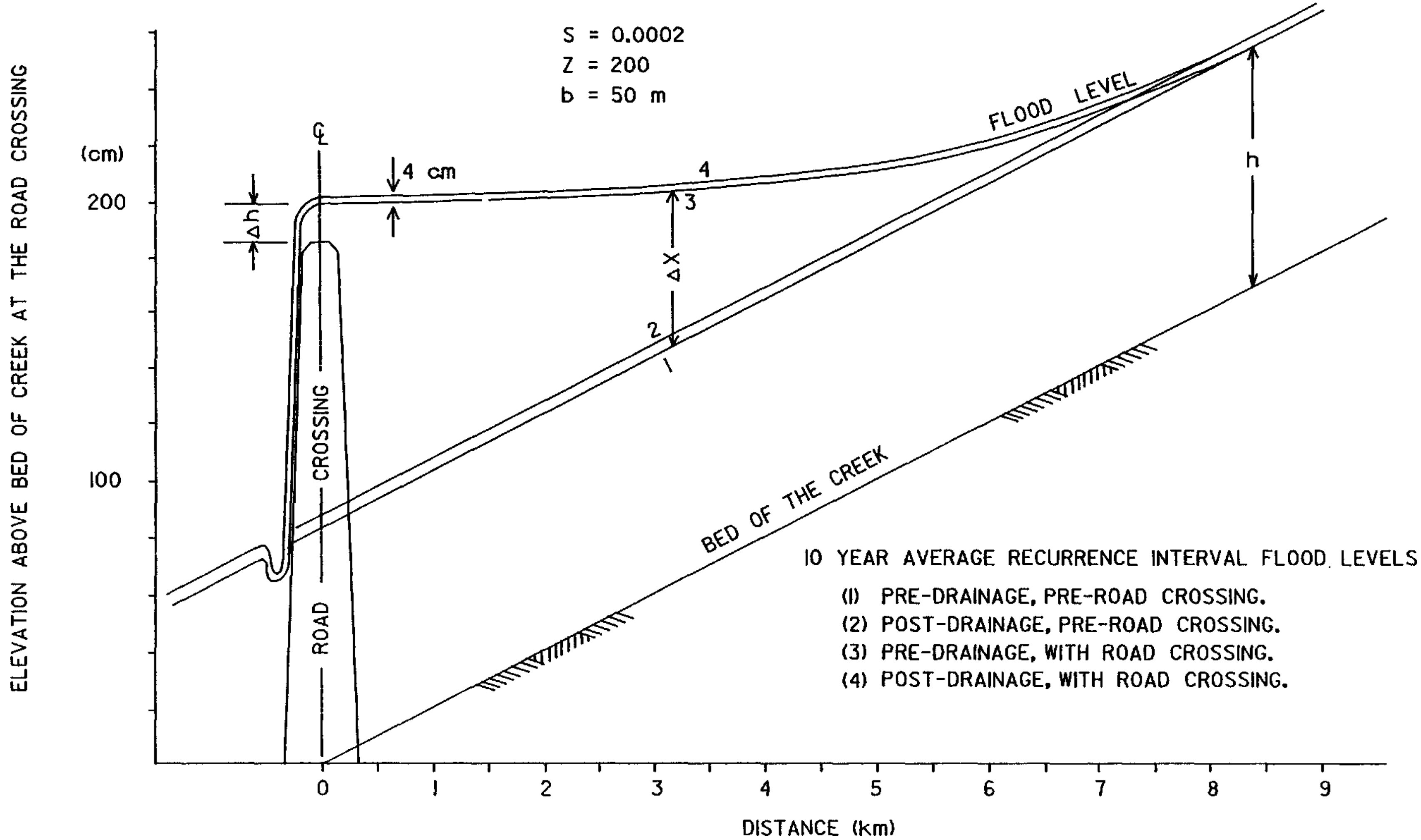


Figure 7.5 Effect Of Drainage And The Road Crossing On Flood Levels

7.7 Conclusions

Improved drainage in the West Nugadong catchment is estimated to increase a ten-year average recurrence interval flood peak by 10 per cent from 40.6 m^3/s to 44.7 m^3/s.

For the assumed drainage channel dimensions the increase in normal depth of flow at these discharges is from 0.85 m to 0.89 m. This is a relatively small increase.

In comparison, the existing road crossing at the downstream end of the catchment may cause flood levels to rise by as much as 1.0 m (if the culverts are blocked by debris) as the crest of the road is 1.9 m above the bed elevation.

It is therefore concluded that culverted road crossings can result in far larger increases in flood depth than is likely to be due to improved catchment drainage.

8. General Discussion

The conclusions drawn from the three case studies at Cowcowing, Beacon and West Nugadong are similar to those reached after the literature review in Section 4.

At Cowcowing, a large number of absorption banks appear to be required to substantially decrease flood peaks. While it is possible to locate the banks in strategic parts of the catchment, the costs are still high for significant reductions in the ten-year average recurrence interval peak flow. Large reductions can be achieved using a retarding basin in the main channel, but the costs are also high. The banks are therefore only likely to be fully effective in low average recurrence interval events (e.g. one year in five). While the banks may have limited effectiveness, the same amount of money spent on dams is likely to have even less effect on flood peaks. When considering banks and dams, other costs and benefits need to be considered. Absorption banks ensure contour working and can mitigate water erosion on the catchment. However they can also increase recharge to underlying saline groundwaters (McFarlane et al., 1986). This consideration is important in catchments with high watertables, and piped outlets to the banks (such as in the Katanning Catchment) need to be considered. Recharge can also occur through the clay soils in the valley floors in some catchments (McFarlane et al., 1987) making the channelling and draining of flood waters across the flats of considerable benefit in catchments with a salt problem.

The implications of failure also need to be considered. The downstream effect of a bank or series of banks failing is likely to be less than if a retarding basin fails. However it is more likely that a bank will fail.

At Beacon it was shown that levee banks can increase flood peaks immediately below the banks but the increase becomes greatly attenuated with increasing distance downstream. However if comprehensive levee bank systems are constructed, flood peaks can be greatly increased and the system needs to be fully engineered for safety. Previous reports on levee bank systems in wheatbelt catchments have shown the systems commonly fail if they are too small for major floods and if they are not continuous throughout the valley. The Beacon study also showed how important it is to know the amount of storage available in the natural channel in estimating the effect of road crossings on flood hydrographs.

The importance of adequate culverts at road crossings was also shown in the West Nugadong study. This conclusion can also be reached from several of the earlier flood studies (Section 3). Road and rail crossings pose problems for most wheatbelt flood mitigation projects. Improving the flow in one part of the valley may overload culverts further downstream, causing a worse problem than before any channel improvements were made. Warner (1985) cited cases from eastern Australia of this problem.

A difficulty in all wheatbelt flood studies is the need to properly define the flooding problem. In many cases the initial problem is thought to be one of high flood peaks causing damage to roads, railway lines, fences and towns. However flood volumes are also important in many farmland areas due to poor internal and external drainage. While it may not be considered important that minor rural roads are closed for several hours or days once every ten years, the continued ponding of water in paddocks for days or weeks every two years is often of great concern to landholders. Therefore the calculations of flood volumes and the conversion of flood levels to ponded areas is as important in flood studies as are peak levels.

Given the expense of structures in mitigating flood peaks, alternatives need to be considered. It is likely that conservation practices such as contour working, vegetative cover and minimum tillage will have little affect on peak flows in major floods, but they are still necessary for mitigating water erosion. Maintaining a tree cover on water shedding areas (e.g. mallet hills, rocky areas) is likely to help in certain situations. In many flood prone areas it may be necessary to accept the flood peaks and initial ponding but design drains which will speed the removal of the ponded waters once they have accumulated.

Methods of slowing flood runoff which have received little attention in Western Australia include water spreading (Quilty, 1986) and strip cropping. Water spreading had advantages in improving soil water conservation. Gap absorption spreaders should delay and attenuate flood peaks by providing storage and by converting channel flow to overland flow. Strip cropping is not applicable in Western Australia as summer crops are not grown.

9. Conclusions

Most soil conservation structures (grade-, absorption- and level-banks) and treatments (contour working, minimum tillage, vegetative cover) only have a significant mitigating effect on small to moderate floods and are less effective in controlling major flood events (e.g. ten year average recurrence interval and greater). If structures are to be used for flood mitigation, absorption banks should be strategically placed in a catchment so that they hold back waters which contribute peak runoff at the time that the main drainage line peaks. If there are salt problems in the catchment, piped outlets need to be considered to lessen recharge. Farm dams are generally ineffective in controlling major floods. However retarding basins located on main drainage lines can be effective in mitigating major floods. Unfortunately retarding basins are very expensive to construct and maintain, particularly if they have to meet strict standards necessary when they are located upstream of houses or towns.

Levee bank systems can be effective in containing flood waters on wheatbelt valleys but they need to be comprehensive and built to an engineering standard. Piecemeal construction of levees can cause problems both upstream and downstream. A major problem with any flood training works is the safe and effective crossing of roads and railways. Flood waters pond behind roadways and railways with inadequate culvert capacity. The high velocity of flood waters inside levee banks can result in scouring of the channel and deposition of silt in drainage lines which are used as outfalls. Levee bank systems (and associated drainage works) may lessen recharge in some wheatbelt catchments by reducing the time that water is ponding.

There are no inexpensive control methods for major flooding in wheatbelt catchments. The present technique of using soil conservation structures and treatments in upland areas for mitigating erosion and minor flooding seems justified, as does the diversion of flood waters around townsites. Efforts to drain away ponded flood waters once formed after a major flood need to be improved as they are more likely to be economic than methods of controlling major flood peaks.

10. Recommendations

1. The effect of structures on flooding in wheatbelt catchments needs to be studied further to determine how valid the conclusions reached in this report are to other catchments. Analyses need to be carried out on the sensitivity of parameter changes on model predictions.

2. When soil conservation structures and treatments are evaluated for soil conservation purposes, some investigation of their effect on flooding also needs to be carried out. A more detailed analysis of the effect of contour banking on runoff in the Berkshire Valley Catchment is required. The effect of water spreading structures on flooding also needs to be assessed.

3. Methods of removing ponded flood waters from wheatbelt valleys need further evaluation as they are likely to be cost effective in some situations.

11. References

Adamson, C.M. (1974). Effects of soil conservation treatments on runoff and sediment loss from a catchment in south western New South Wales, Australia. Proc. Paris Symp. "Effects of man on the interface of the hydrologic cycle with the physical environment." IAHA-AISH Publ. No. 113.

Adamson, C.M. (1976). Some effects of soil conservation treatments on the hydrology of a small rural catchment at Wagga. J. Soil Cons. N.S.W., 32: 230-249..

Armstrong, F.W. and Maschmedt, D.J. (1984). A soil conservation study tour to the Darling Downs (Qld) and northern N.S.W., Tech. Rep. No. 79, S. Aust. Dept. Agric.

ASCE (1948). Effects of conservation practices on runoff. Agricultural Engineering, 29: 114-116.

Australian Rainfall and Runoff (ARR) (1977). Flood analysis and design. Inst. of Engin. Aust.

Australian Rainfall and Runoff (ARR) (1987). A guide to flood estimation. Inst. of Engin. Aust.

Bailey, A.D. and Bree, T. (1980). Effect of improved land drainage on river flows. Flood studies report - five years on. Inst. Civil. Engin.

Baird, R.W. (1929). The relation of runoff to vertical interval of terraces. Agricultural Engineering, 10: 188-189.

Belstead, B.S., Glenister, T.R. and Goh, J.J. (1984). Flooding in the Great Southern, January 1982. Tech. Rep. No. 38, Main Roads Dept. W. Aust.

Bettenay, E. and Mulcahy, M.J. (1972). Soil and landscape studies in Western Australia. (2) Valley form and surface features in the South West Drainage Division. J. geol. Soc. Aust., 18: 359-369.

Bird, J.F. (1980). Geomorphological implications of flood control measures, Lang Lang River, Victoria. Aust. Geog. Stud., 18: 169-183.

Bligh, K.J. (n.d.). The effects of contour banking on surface runoff at the

Berkshire Valley Experimental Catchment, 1961-82. D.R.M. Tech. Rep. No. 11, W. Aust. Dept. Agric.

Bretnall, R. (1984). Merredin flood mitigation, multiple retarding basin concept, hydraulic feasibility report. File F11408, folio 54, Water Authority of Western Australia.

Burch, G.J., Moore, I.D., Barnes, C.J., Aveyard, J.M. and Barker, P.J.

(1986). Modelling erosion hazard: a total catchment approach. Proc. Hydrology and Water Resources Symposium, Brisbane, Institution of Engineers (Aust.): 345-349.

Carder, D.J. (1971). Flooding problems on farmland in Western Australia. Proc. Aust. Soil Conserv. Conf., Melbourne.

Chow, Ven Te (1959). Open channel hydraulics. McGraw-Hill.

DRM Annual Report (1982). Annual report of the Division of Resource Management, Western Australian Department of Agriculture.

Flavell, D.J. (1983). Design flood estimation in Western Australia. NAASRA, EEC Seminar, Waterway analysis and design. Adelaide.

Flavell, D.J. and Belstead, B.S. (1986). Losses for design flood estimation in Western Australia. Proc. Hydrology and Water Resources Symposium, Brisbane, Institution of Engineers (Aust.): 203-208.

Flavell, D.J., Belstead, B.S., Chivers, B. and Walker, M.C. (1983). Runoff routing model parameters for catchments in Western Australia. Proc. Hydrology and Water Resources Symposium, Hobart, Institution of Engineers (Aust.): 22-27.

Kratchler, R.S. (1980). Eastern wheatbelt flood report - severe thunderstorms - February 1978. Tech. Rep. No. 13, Main Roads Dept. W. Aust.

Lang, R.D. (1979). The effect of ground cover on surface runoff from experimental plots. J. Soil Cons. N.S.W., 35: 108-114.

Lang, R.D. and McCaffrey, L.A.H. (1984). Ground cover - its effects on soil loss from grazed runoff plots - Gunnedah. J. Soil Cons. N.S.W., 40: 56-61.

Laurenson, E.M. and Mein, R.G. (1985). RORB - Version 3 runoff routing program users manual. Monash Uni.

Mein, R.G., Laurenson, E.M. and McMahon, T.A. (1974). Simple non-linear model for flood estimation. J. Hydraul. Div., Amer. Soc. Civil Engin., 100: 1507-1518.

McFarlane, D.J. (1986). Rainfall. In Soil Conservation Earthworks Design Manual. W. Aust. Dept. Agric.

McFarlane, D.J. and Ryder, A.T. (1987). Water erosion survey in the Northam District after storms in June 1986. DRM Tech. Rep. No. 57, W. Aust. Dept. Agric.

McFarlane, D.J., Engel, R. and Ryder, A.T. (1987). The location of recharge areas responsible for valley salinity in the Lake Toolibin catchment. Western Australia. Groundwater Recharge Conference, Mandurah, West. Aust.

McFarlane, D.J., Engel, R., Ryder, A.T. and Bales, M. (1986). The effect of level banks on groundwater recharge. Technote 27/86. W. Aust. Dept. Agric.

Mitchell, D.J. and Laurenson, E.M. (1983). Flood routing in a complex flood plain. Proc. Hydrology and Water Resources Symposium, Hobart, Institution of Engineers (Aust.): 315-318.

Mulcahy, M.J. (1967). Landscapes, laterites and soils in south western Australia. In: Landform Studies from Australia and New Guinea. Jennings, J.N. and Mabutt, J.A. (eds), Canberra A.N.U. Press.

Negus, T.R. (1968). Preliminary report on the Lake Toolibin flood mitigation survey, Wickepin Shire. W. Aust. Dept. Agric. Report (unpubl.).

Negus, T.R. (1970). Report on flood control works in the Wogalin Creek Catchment, Wickepin/Kulin Shires. W. Aust. Dept. Agric. Report (unpubl.).

Peterson, E.T. (1954). Big Dam Foolishness (Publisher unknown).

Quilty, J.A. (1986). Waterspreading and waterponding in New South Wales and their relevance to Western Australia. D.R.M. Tech. Rep. No. 51, W. Aust. Dept. Agric.

Ryan, K.T. (1981). Assessment of catchment soil conservation - a case study. Proc. Third Aust. Soil Consev. Conf., 281-283.

Ryan, K.T. (1982). Red Creek hydrologic study. I. Physical assessment, equipment and measurements. N.S.W. S.C.S. Tech. Bull. No. 23.

Ryan, K.T. (1986). Red Creek hydrological study. In: 1985/86 Annual Research Report, Soil Conservation Service of New South Wales.

Sastry, G. and Narayana, V.V.D. (1984). Watershed response to conservation measures. J. Irr. Drain. Res., 110: 14-21.

Sinclair Knight and Partners (1987). Narembeen Flood Study. Consultants report to the Water Authority of Western Australia.

Stanholtz, V.O., Ross, R.B. and Carr, J.C. (1981). Effect of spatial

variability on the simulation of overland and channel flow. Trans. ASCE, 24: 124-133.

Swanson, T.J.B. (1983). Letter to Quairading Shire Clerk. G.H.D-Dwyer (W.A.) Pty Ltd. File 84SCPF46, W. Aust. Dept. Agric.

Warner, R.F. (1985). Man's impact on Australian drainage systems. Aust. Geog., 16: 133-141.

12. Appendix A The RORB Model

The RORB runoff routing model developed by Laurenson and Mein (1983) has been used extensively in Australia for routing storm runoff through catchments. Losses can be divided into initial losses and continuing losses or as a volumetric runoff coefficient. A catchment storage model routes the excess rainfall through the catchment storages to produce surface runoff hydrographs.

The non-linear storage function which routes the excess rainfall is:

$$S = 3600\ kc\ kr\ Q^{m}$$

Where: S is the reach storage (m3)
Q is the discharge (m3/s)
kc is an empirical coefficient that is applicable to the entire catchment (dimensionless)
kr is the relative delay time, generally taken as being proportional to the length of reach between two nodes (dimensionless)
m is an empirical coefficient which is a measure of the non-linearity in the catchment (dimensionless). A value of unity implies a linear catchment.

Parameter values

Flavell et al. (1983) developed several methods of estimating the empirical coefficients in the above equation for Western Australian catchments. The case studies detailed in this report use appropriate equations. For the wheatbelt, Flavell et al. (1983) gave the following equations for estimating kc when m = 0.8.

$$kc = 3.00\ L^{0.71}\quad S^{-0.76}$$

and

$$kc = 3.26\ A^{0.43}\quad S^{0.72}$$

where: L is mainstream length (km)
S is equivalent uniform slope (m/km)
And: A is catchment area (km^2).

Flavell and Belstead (1986) subsequently developed the following regional relationship for kc for the wheatbelt, arid interior, north west and interior:

$$kc = 1.05\ L^{0.87}\ S^{-0.46}$$

Using this relationship, the following equations were developed for estimating the initial loss from wheatbelt catchments with loamy soils and 85 to 100 per cent cleared:

$$IL2 = 35\ L^{-0.123}$$
$$IL5 = 464\ P^{-0.41}\ L^{-0.077}$$
$$IL10 = 1400\ P^{-0.60}\ L^{-0.042}$$

Where: IL is the initial loss (mm) and the subscript is the storm average recurrence interval
P is the average annual precipitation (mm)
and L is mainstream length (km).

The continuing loss from the catchments was assumed to be 3 mm/h. Frequency factors for scaling IL5 were also estimated as 0.78, 1.09, 0.95 and 0.66 for 2, 10, 20 and 50-year average recurrence intervals respectively.

A later development (ARR, 1987) recommended a continuing loss of 3 mm/h with the five year return period initial loss being calculated from:

$$1L5 = 700\ P^{-0.47}\ L^{-0.08}$$

The multipliers on IL5 for 2, 10, 20 and 50 year return period storms are 0.78, 1.09, 0.95 and 1.00 respectively.