Soil conservation earthworks design manual

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1. INTRODUCTION

1.1 Steps in design and construction

The design and construction of soil conservation earthworks should involve each of the following steps:

(i) **Definition of the problem to be solved.** This is often not a trivial exercise. For example a water erosion problem may be the result of waterlogged soils which can best be tackled by drainage. Also the source of surface ponded water may be difficult to determine. The landholder’s observations (rather than interpretations) should be actively sought along with any observations at the site of the planned earthworks. In most cases there is no substitute for district experience as to the types of problems which occur in different landscapes and soils, and to solutions which have worked in similar situations.

(ii) **Consideration of possible solutions to the problem.** Earthworks are not always the best solution to soil conservation problems. Changes in land management (e.g. tillage, grazing, contour working, farm plans) must first be considered as they affect larger land surfaces than do individual earthworks. A wide range of possible solutions to soil conservation problems are outlined in another manual entitled 'Introductory manual to soil conservation in Western Australia'.

(iii) **Estimation of the likely costs and benefits** of the alternative solutions to the problem. This estimation can take the form of rough calculations or spreadsheet analyses which can be carried out on a microcomputer. It is advisable that personnel involved in designing soil conservation earthworks spend some time carrying out detailed cost-benefit analyses for typical situations which they face in their district, so that they can make of microcomputers and the necessary software make such analyses a simple exercise.

(iv) **Decide on the best solution** for the particular problem on the basis of (i) to (iii) above.

(v) **Design any earthworks solutions using this manual.** It may be necessary to calculate alternative designs with different probabilities of failure for presentation to the landholder.

(vi) **Survey the earthworks and supervise their construction** (or check on their construction if time constraints prevent direct supervision). An outline of the approach to surveying and constructing various soil conservation earthworks can be found in ‘Introductory manual to soil conservation in Western Australia’ and on a video entitled ‘The role, design and construction of soil conservation structures’.

1.2 Limitations of this manual

In view of the great diversity of hydrological conditions encountered in the field, and the relative paucity of information with which to design soil conservation earthworks, it is frequently not possible to quantify all aspects of hydrologic design. It remains for the practising soil conservationist to search for a reasonable solution to a problem which is most appropriate to the situation, location, information and skills. The equations in this Manual provide the best available estimates of flow rates and volumes for carrying out design but it must be recognised that the information has come from a relatively limited data set, and that a significant amount of variability in runoff estimates remains unexplained by the equations. The observations of landholders during previous runoff events and the experience of local Departmental personnel are particularly valuable and should be used in conjunction with the design equations.
It is not intended that the material in this Manual will be adequate in all situations. In complex or high risk situations, assistance should be requested from specialist personnel of the Division of Resource Management.

1.3 Responsibility for hydrological design

While an employee of the Department of Agriculture uses this Manual for the design of soil conservation earthworks, he/she is not likely to be personally liable for any legal consequences of their design. Similarly if it can be shown that a particular design would have been approved by a person’s peers who carry out similar design work, the person is not likely to be liable for any legal consequences.

This is important as innovation in the design of soil conservation earthworks is important if advances are to be made. In the case of a trial of a new soil conservation structure, the landholder should be informed (preferably in writing) of the trial nature of the proposed earthworks. A person is only likely to be personally liable for the consequences of their design if the design is clearly inferior to those carried out by his or her peers, or if it used design standards which were not current at the time that the design was carried out.

1.4 Design philosophy

It is anticipated that soil conservation structures may fail if they receive greater than the design storm. The decision of which return period to design for is determined by the consequences of failure and the additional costs of designing for successively higher return periods. When estimating the consequences of failure, consideration needs to be given to the time of the year when the design storm is expected (e.g. summer or winter) relative to the likelihood of the soil being in an erodible condition (e.g. cultivated, little ground cover).

In the design storm, level and absorption banks will fill and begin to discharge around their ends. If there are no waterways for this overflow, some erosion is to be expected. At a much higher return period, the banks may breach and release their above-ground storage which is likely to have much more serious consequences. In events exceeding the design storm, waterways will flow outside their boundaries, possibly resulting in the erosion of surrounding land. In addition, some scouring of the waterways can be expected. The present design standards for grade banks have resulted in a low percentage of failures over the past 30 years but the design return period of these structures is yet to be determined.

1.5 Some important warnings

- **Do not** divert water away from recognisable ‘natural’ flow lines if this could cause damage to a neighbour’s land, or to a public utility. If water is diverted to fill a dam or to allow gully reclamation, the overflow must be returned to its original natural watercourse before it leaves the farm.

- **Do not** allow surplus water to leave the property except in the watercourse it would have been in if no earthworks had been constructed. A contour bank system may divert water from several minor flow lines to one definite watercourse or artificial waterway. If a return to the original watercourse is not practical, the contouring should not be done without written approval from neighbouring landholders or authorities who may be affected.

- **Do not** divert water onto road verges or other public utilities unless specific written permission has been obtained by the landholder.

- **Advise** landholders to approach road-making or other public authorities reasonably, if the authorities divert water so that it could damage their land, in order to come to some arrangement with the authority. Remember that usually they have only narrow strips or limited space in which to protect the facilities which they are providing for the public.
2. SOIL CONSERVATION EARTHWORKS

2.1 Options

The decision on whether or not to construct soil conservation earthworks depends on alternative techniques which are available to achieve a similar end. Earthworks are used to reduce water erosion, waterlogging and flooding and to conduct water to farm dams. Alternative techniques (e.g. minimum tillage, stubble retention, contour working) may be used to reduce runoff which causes erosion, or to increase the soil’s resistance to erosion, or both. In order to decide on the optimum mix of methods of reducing runoff, an understanding of runoff-generating mechanisms is necessary.

2.2 Runoff mechanisms in the agricultural areas of Western Australia

The mechanisms whereby runoff is produced in Western Australian agricultural areas are rainfall-excess and saturation-excess. The two mechanisms are not mutually exclusive and the situations described below for various soils can occur simultaneously or sequentially during a storm.

2.2.1 Rainfall-excess (also called Hortonian flow)

This mechanism occurs when rainfall intensity exceeds the infiltration capacity of the soil and the soil surface storages are exceeded. This form of runoff is most common during high intensity storms and on soils with low infiltration capacities.

Rainfall-excess is an important mechanism in the following soils:

(i) **Heavy textured soils.** These soils have low infiltration capacities to begin with and these become progressively lower as the soils wet up and any shrinkage cracks close due to the swelling of clay minerals, etc. Heavy textured soils are estimated to occupy about 11 per cent of the agricultural area.

(ii) **Surface sealing soils.** Whether a soil develops a surface seal depends particularly upon the dispersive nature of the clay on the soil surface. Factors which affect soil structural properties (e.g. minimum tillage, cropping frequency, gypsum etc.) will affect the development of a surface seal. A vegetative cover on a soil will also help prevent a soil from sealing due to protection of the soil from raindrop impact. The formation of a surface seal has been reported to occur on all soils except sands with less than about 5 per cent of clay. Runoff and erosion have been reported on yellow loamy sands, presumably due to surface sealing. It has also been suggested that the seal may inhibit soil detachment and that while runoff is enhanced by the sealing, soil loss is less than expected.

(iii) **Non-wetting soils.** Soils become non-wetting if water-repellent organic matter coats the surfaces of soil particles. As sandy soils have lower surface areas, they are more predisposed to becoming non-wetting. Sandy soils occupy about 48 per cent of the agricultural area. While sands are more susceptible, the existence of large amounts of highly-repellent organic litter in mallet hill areas can result in sandy clay loams and sandy clays becoming non-wetting. Soils are more non-wetting if allowed to dry out (i.e. during summer) and also after long periods of clover ley.

(iv) **Surface compacted soils.** Surface compaction by vehicle tyres and sheep results in decreased infiltration capacities and low surface storage capacities. Runoff and erosion along combine and sheep tracks is commonly reported and can be limited by contour working (with corners not worked if running downhill) and by farm plans (e.g. sensible location of fence lines and watering points).
2.2.2 Saturation-excess

Rain falling on saturated soils results in immediate ponding and, once surface storages are exceeded, runoff commences. Saturation-excess may occur after rainfall-excess if infiltration is sufficient to saturate the soil profile. Once saturation has occurred, infiltration rates are limited by the soil’s hydraulic conductivity and hydraulic gradients. Drainage of areas susceptible to waterlogging is the best means of controlling runoff and soil loss due to saturation-excess.

Saturation-excess is an important mechanism in the following soils:

(i) **Duplex soils.** Duplex (texture contrast) soils are likely to become waterlogged due to:
- the high infiltration capacity and low soil evaporation characteristics of the sandy topsoil;
- poor internal drainage due to the clay sub-soil;
- the low water storage capacity of the shallow top-soil.

Duplex soils have been estimated to occupy about 37 per cent of the agricultural area and occur in all landscape positions. Saturation is more likely in certain landscape positions (e.g. below shedding areas, converging flow on concave hillslopes, decreased slopes adjacent to valleys) and where soils have lower transmissivities (i.e. become thinner and/or have lower hydraulic conductivities). Where the saturated zone is perched on a clay sub-soil, infiltration rates are limited to the rate of lateral drainage and drainage through the clay sub-soil.

(ii) **Soils in groundwater discharge areas.** Where there is a permanent groundwater system that discharges in an area of upward hydraulic heads, soils may remain saturated throughout the year and, due to the upward heads, infiltration rates are negligibly small. Saline seeps may occur on hillsides or in valley bottoms.

(iii) **Heavy textured soils in valley flats.** Water may pond on heavy textured soils in valley flats due to slow rates of internal and external drainage. Internal drainage is drainage within the soil profile while external drainage is overland flow or runoff. The surface water may come from in-situ rainfall, runoff and seepage waters from surrounding hillsides, or flood waters from higher up the valley. Identification of the source of the water is required when planning the best treatment (e.g. levees, interceptors, V or W drains) for these areas.

2.3 Runoff under reduced tillage

Research indicates that increased infiltration (and therefore reduced runoff) is likely under reduced tillage on all except sandy soils. Increased organic matter near the soil surface under reduced tillage results in increased aggregation of fine-soil particles, which appears to increase infiltration rates (Mechanism 2.2.1 (ii) above). Higher bulk densities at the surface of sandy soils under minimum tillage, appear to be associated with reduced infiltration rates in these normally high-infiltration-rate soils.

Where other runoff mechanisms are responsible (e.g. saturation-excess) tillage treatments may have a minimal effect on runoff. Thus runoff at the end of a wet winter may be unaffected by tillage treatment, as observed by Bligh (1984) on small plots.

Stubble mulching, in which tillage is carried out under a stubble layer which remains mainly on the soil surface, may be expected to reduce runoff caused by rainfall excess. As for reduced tillage, stubble mulching may be expected to have little effect on amounts of runoff caused by saturation excess, although stubble on the soil surface may impede overland flow, resulting in lower peak flows.
2.4 Earthworks for reducing runoff from farmland

Flood mitigation structures delay or retain runoff in order to reduce flood peaks and/or volumes.

While the same volume of runoff may occur with flood mitigation structures which only delay runoff (e.g. grade banks) it usually occurs over a longer time. On one agricultural catchment, grade banks have been observed to reduce peak flows by up to one-third (Bligh, 1982).

Level and absorption banks give a greater reduction in peak flow than grade banks, provided that they do not overtop. These banks retain initial runoff (the amount depending upon their design storage) and provide temporary storage for further runoff, while overflowing safely around their ends. Level banks store runoff only in the excavated area at overflow, whereas absorption banks flood the ground surface uphill of the excavation. Retained runoff is then allowed to evaporate or infiltrate, or it may be drained through pipes under the banks.

Seepage interceptor drains divert runoff while intercepting shallow seepage above a less permeable sub-soil. Reverse bank seepage interceptors use the spoil from a graded channel on the down-hill side, to form a bank which diverts surface runoff on the natural ground surface.

Waterspreading structures redistribute runoff across a slope. The thinner the overland flow, the more effect surface roughness has on flow rates.

2.5 Maintenance needs of banks and drains

Banks and drains are physical structures built to particular specifications and, as such, must be periodically maintained to compensate for settlement and compaction. Without maintenance, breaches may occur at low points (e.g. stock tracks) releasing diverted or stored runoff downslope. The result can be sequential failure of other structures downslope, possibly causing worse flooding and erosion than would have occurred had the structures not been built in the first place.

Because each bank is effectively only as high as its lowest point, any low points should be built up as soon as possible after they occur. When a bank has settled below its design height, it must be built up along its entire length. Excessive amounts of silt must also be removed from the channels of seepage interceptor drains. Landholders should be encouraged to set aside a time each year to spend upon the maintenance of soil conservation structures.

2.6 Surface protection of soils

A vegetative cover (pasture, crop or stubble) for soils helps prevent water erosion in two ways:

(i) it protects the soil surface from the energy of raindrop impact; and

(ii) it impedes surface runoff resulting in reduced transport capacity of the runoff that does occur, and decreases the amount of soil detached by the runoff itself. Runoff has been found to increase markedly once individual patches of bare ground become interconnected. This interconnection typically occurs once ground cover falls below about 75 per cent.

Detached soil can result from:

(i) raindrop impact: the terminal velocity of raindrops can reach nine metres per second;

(ii) tillage: it is common to observe rill erosion proceeding only to the depth of cultivation. Severe sheet erosion can occur following a summer storm on a fallowed paddock;
(iii) **animal hooves:** sheep have been found to detach between 0.5 and 1.0 tonne of soil per sheep per week.

### 2.7 Types of soil conservation earthworks

Grade banks and seepage interceptor drains divert runoff to waterways at non-erosive velocities. After runoff is diverted to a stable waterway, gullies may be filled for cropping (Carder and Spencer, 1968). Waterways must be maintained in a permanently grassed condition and, preferably, fenced in order to control grazing and access.

Where waterways are not easily definable or are unstable, or flood mitigation is desired, level or absorption banks may be constructed to retain initial runoff and discharge the remainder during extreme events. Pipes may be installed through the banks to drain stored runoff in order to minimize both recharge to saline watertables and downslope waterlogging. The maintenance requirements of absorption banks are particularly critical because they store water above the ground surface. Therefore any breach releases a large amount of stored runoff with, possibly, serious damage to any further banks or other structures downslope.

### 3. RAINFALL

#### 3.1 Rainfall intensity-frequency-duration

When designing soil conservation structures, it is necessary to know the expected frequency with which the structures will fill or fail. In design, longer return periods (e.g. up to 20 years) should be used for important structures for which failure will be serious (such as absorption banks) than for less important structures (such as grade banks). It might be helpful to present a farmer with designs for several different return periods, so that he is aware of the increased risks of installing banks further apart than may be recommended by the Department.

It is also useful to know the likely average recurrence interval (ARI) of a storm which has caused erosion or damaged a soil conservation structure when assessing what should be done to prevent a re-occurrence. If the storm has a very long return period (e.g. 1 in 100 years), the option of doing nothing should be considered.

Canterford *et al.* (1987) have developed methods of estimating the expected average rainfall intensities for storms with durations of 5 minutes to 72 hours, and with ARI’s of 1 to 100 years for all parts of Australia. To determine the intensities for a location it is necessary to refer to nine maps and use either an algebraic, graphical or computerised procedure.

Using the computerised procedure, diagrams of expected rainfall intensities have been calculated for 20 centres in South-Western Australia (Figure 3.1). For areas located between two centres it will be necessary to estimate intensities for both centres and take an average (weighted according to the closeness to each centre).

**Example 1.** Estimate the average rainfall intensity for a storm at Merredin with a duration of two hours and an ARI of five years.

Using the Merredin graph, locate the storm duration on the horizontal axis and proceed vertically until the 5-year line is encountered. The average intensity is read from the vertical axis as 13 mm/h. Therefore in a 2-hour duration storm, \(2 \times 13 \text{ mm} = 26 \text{ mm}\) will fall, with a 5-year average recurrence interval.

**Example 2.** Thirty millimetres of rain fell in 3 hours at Ravensthorpe. What is the ARI for the storm? The average rainfall intensity for the 3-hour storm is 10 mm/h. From the Ravensthorpe graph locate the 3-hour duration on the horizontal axis and proceed vertically until 10 mm/h intensity is reached on the vertical axis. The ARI for the storm is between 5 and 10 years (approximately 7 years ARI).
Problems with using Figure 3.1 include:

1. The ARI for a storm is often not related to the ARI for the resulting runoff. Thus a 10-year ARI storm falling on a wet and/or bare catchment may produce a 100-year ARI runoff event. Alternatively a 100-year ARI storm falling on a dry and/or heavily vegetated catchment may produce a 10-year ARI runoff event.

2. When assessing a failed structure or severe erosion, it is often not possible to know the intensity of the storm that caused the problem, as there are few pluviometers in the agricultural areas. Farmer records of rainfall intensity should be used with caution as the records have sometimes been found to be in excess of nearby pluviometers.
Figure 3.1 Rainfall Intensity curves (mm per hour) for various durations and average recurrence intervals. (Source: Bureau of Meteorology).
Figure 3.1 Continued ...
Figure 3.1 Continued ...
Figure 3.1 Continued …
Figure 3.1 Continued ...
Figure 3.1 Continued …
Figure 3.1  Continued …
Figure 3.1 Continued …
Figure 3.1  Continued …
Figure 3.1  Continued …
3.2 Probability of waterlogging

There are a number of factors which determine whether a site becomes waterlogged. These include the amount and distribution of rainfall, evaporation rates, soil type, topography and management. It is therefore difficult to produce a universal method of predicting the occurrence of waterlogging in different parts of the agricultural area.

The method that is described below assumes that the probability of waterlogging is directly related to the probability of rainfall. The method is site-specific and therefore avoids soil and topography problems. However the effect of management is ignored. Thus if the sandy topsoil of a duplex soil is non-wetting, more infiltration and waterlogging would be expected if the soil has been roughened by cultivation on the contour than if it were uncultivated.

The Bureau of Meteorology have information on the probability of receiving different amounts of rainfall in any one month or group of months for each of their rainfall stations. The information is in the form of deciles. Deciles are estimated by ranking all the rainfall amounts in ascending order and dividing the list into ten equal amounts. Thus the ninth decile is the rainfall amount which will only be equalled or exceeded one year in ten while the fifth decile is the rainfall which will be equalled or exceeded one year in two (the median).

To make use of the records it is necessary to know the rainfall that occurred during a year (or years) of waterlogging and also for a year (or years) when waterlogging did not occur at the site. From examination of the years when waterlogging occurred it can be deduced whether waterlogging results from one or more months of high rainfall and in which months the rainfall fell.

Example: The examination of rainfall records for a waterlogging-prone duplex soil at Narrogin shows that the site becomes waterlogged whenever 220 mm is received in June and July (the wettest two months of the year) but not when only 180 mm is received. It could then be assumed that 200 mm is a critical amount. Table 3.2 shows that 201 mm of rainfall is decile 7 for Narrogin for June and July. Therefore there is a 30 per cent (or 0.30) probability of receiving more than this amount during these months in any one year. If the area is cropped every second year the probability of waterlogging during cropping is 0.30 x 0.50 = 0.15 or 1 year in 7 probability. If cropped every third year the probability of waterlogging during cropping is 0.30 x 0.33 = 0.10 or 1 year in 10 probability.

<table>
<thead>
<tr>
<th>Decile</th>
<th>Rainfall (mm)</th>
<th>Probability of receiving at least this amount of rainfall</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>116</td>
<td>0.9</td>
</tr>
<tr>
<td>2</td>
<td>135</td>
<td>0.8</td>
</tr>
<tr>
<td>3</td>
<td>147</td>
<td>0.7</td>
</tr>
<tr>
<td>4</td>
<td>160</td>
<td>0.6</td>
</tr>
<tr>
<td>5</td>
<td>172</td>
<td>0.5</td>
</tr>
<tr>
<td>6</td>
<td>183</td>
<td>0.4</td>
</tr>
<tr>
<td>7</td>
<td>201</td>
<td>0.3</td>
</tr>
<tr>
<td>8</td>
<td>221</td>
<td>0.2</td>
</tr>
<tr>
<td>9</td>
<td>264</td>
<td>0.1</td>
</tr>
</tbody>
</table>

The rainfall deciles for the wettest two months of the year for twelve other agricultural centres are shown on Table 3.3. June and July are commonly the wettest months and are most likely to be correlated with waterlogging occurrence.
Table 3.2. Rainfall deciles for the wettest two-monthly period of the year for twelve agricultural centres
(Source: Bureau of Meteorology)

<table>
<thead>
<tr>
<th>Decile</th>
<th>Albany</th>
<th>Donnybrook</th>
<th>Esperance</th>
<th>Geraldton</th>
<th>Jerramungup</th>
<th>Katanning</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>148</td>
<td>277</td>
<td>90</td>
<td>100</td>
<td>52</td>
<td>104</td>
</tr>
<tr>
<td>2</td>
<td>198</td>
<td>307</td>
<td>135</td>
<td>138</td>
<td>68</td>
<td>119</td>
</tr>
<tr>
<td>3</td>
<td>210</td>
<td>341</td>
<td>146</td>
<td>161</td>
<td>82</td>
<td>127</td>
</tr>
<tr>
<td>4</td>
<td>214</td>
<td>363</td>
<td>179</td>
<td>190</td>
<td>97</td>
<td>141</td>
</tr>
<tr>
<td>5</td>
<td>217</td>
<td>386</td>
<td>186</td>
<td>208</td>
<td>116</td>
<td>152</td>
</tr>
<tr>
<td>6</td>
<td>230</td>
<td>401</td>
<td>209</td>
<td>232</td>
<td>121</td>
<td>159</td>
</tr>
<tr>
<td>7</td>
<td>237</td>
<td>425</td>
<td>218</td>
<td>256</td>
<td>127</td>
<td>167</td>
</tr>
<tr>
<td>8</td>
<td>294</td>
<td>466</td>
<td>227</td>
<td>273</td>
<td>135</td>
<td>191</td>
</tr>
<tr>
<td>9</td>
<td>331</td>
<td>546</td>
<td>271</td>
<td>343</td>
<td>159</td>
<td>227</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Months</th>
<th>June</th>
<th>June</th>
<th>June</th>
<th>June</th>
<th>May</th>
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<tbody>
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<td></td>
<td></td>
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<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Decile</th>
<th>Lake Grace</th>
<th>Manjimup</th>
<th>Merredin</th>
<th>Moora</th>
<th>Northam</th>
<th>Three Springs</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>54</td>
<td>262</td>
<td>65</td>
<td>121</td>
<td>109</td>
<td>85</td>
</tr>
<tr>
<td>2</td>
<td>67</td>
<td>289</td>
<td>71</td>
<td>139</td>
<td>123</td>
<td>98</td>
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<td>74</td>
<td>311</td>
<td>89</td>
<td>156</td>
<td>138</td>
<td>112</td>
</tr>
<tr>
<td>4</td>
<td>89</td>
<td>329</td>
<td>94</td>
<td>161</td>
<td>150</td>
<td>130</td>
</tr>
<tr>
<td>5</td>
<td>96</td>
<td>357</td>
<td>98</td>
<td>175</td>
<td>162</td>
<td>137</td>
</tr>
<tr>
<td>6</td>
<td>101</td>
<td>385</td>
<td>110</td>
<td>187</td>
<td>173</td>
<td>154</td>
</tr>
<tr>
<td>7</td>
<td>121</td>
<td>415</td>
<td>117</td>
<td>200</td>
<td>194</td>
<td>168</td>
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<tr>
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<td>434</td>
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<td>152</td>
<td>468</td>
<td>163</td>
<td>287</td>
<td>253</td>
<td>253</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Months</th>
<th>May</th>
<th>June</th>
<th>June</th>
<th>June</th>
<th>May</th>
<th>June</th>
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<tbody>
<tr>
<td></td>
<td></td>
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<td></td>
<td></td>
</tr>
</tbody>
</table>

3.3 Rainfall erosivity

The rainfall variable most correlated with soil loss is storm energy (Wischmeler and Smith, 1958). The correlation is improved if storm energy (E) is multiplied by the maximum rainfall intensity over 30 minutes EI\(_{30}\) during the storm, producing the EI\(_{30}\) index of rainfall erosivity.

The distribution of rainfall erosivity (EI\(_{30}\) in metric units) has been determined for Western Australia by McFarlane et al. 1986. As extreme storms are very important in water erosion, both the two year annual exceedence probability (AEP) erosivity and the ten year AEP erosivity were determined. The two year AEP erosive storm would be expected to be exceeded once every two years on average. This storm is also the mean annual rainfall erosivity.

Figure 3.2 shows the two-year AEP rainfall erosivity (or mean annual erosivity) for the south west. Areas to the east of a straight line drawn through Three Springs and Cranbrook, and the extension of a straight line connecting Cranbrook and Jerramungup, have a very low rainfall erosivity in comparison to the Great Southern and the coastal areas. A doubling in EI\(_{30}\) will result in a doubling in soil loss, if all the other factors affecting erosion (slope length, soil erodibility, etc.) remain constant.

Figure 3.3 shows that in inland areas the ten-year AEP erosivity is about 50 per cent higher than the two-year AEP erosivity (75 versus 50 metric units) while areas in the north east (e.g.
Three Springs) have a ten-year AEP more than double their two-year AEP (110 versus 50 metric units). Areas in the south west corner have very uniform erosivities such that their ten-year AEP erosivities are little higher than their two-year AEP erosivities.

Figure 3.2 Two-year AEP annual erosivity (metric units).
When designing earthworks, it is useful to know whether erosive storms are more likely in winter or in summer. If erosive storms are common in winter and the area is cropped, soil losses are likely to be high due to the coincidence of bare cultivated soil and erosive rain. Soil losses are also likely to be high if erosive storms are common in summer and the area is likely to be heavily grazed resulting in little vegetative cover and soils loosened by the hooves of sheep.

Figures 3.4 to 3.7 show the two and ten-year AEP erosivities over winter and summer for the south west. Summer is considered to extend from November to April (inclusive). Some discrepancies occur in the maps due to different predictive equations being used for each map. Therefore in some centres the winter erosivities slightly exceed the annual erosivities. However the overall trends shown by the maps are accurate.

There is a decrease in rainfall erosivity from the west coast to the south coast. Figure 3.8 shows the annual distribution of rainfall erosivity for three coastal stations in the west, south west and south. The locations of the stations are shown in Figure 3.3.9. The south west station has about 35 per cent of the rainfall amount of the west station but only 75 per cent of its erosivity. The south station has about 70 to 80 per cent of the rainfall amount of the other two stations respectively, but only about 30 and 40 per cent of their erosivities. Low intensity rainfall is a feature of the south coast.
Erosive rainfall near the coast is winter-dominant, but becomes progressively more summer-dominant inland and to the north. Figure 3.9 shows the annual distribution of rainfall erosivity for stations in the north, west, south and east of the agricultural areas. All except the west station show a low erosivity between May and November. Erosive rainfall is particularly common in January, February and April in the wheatbelt.

Figure 3.4 Two-year AEP winter erosivity (metric units).
Figure 3.5  Two-year AEP summer erosivity (metric units).
Figure 3.6  Two-year AEP winter erosivity (metric units).
Figure 3.7  Ten-year AEP summer erosivity (metric units).
Figure 3.8  Mean monthly EI\textsubscript{30} for three coastal areas in the south-west of Western Australia.

Figure 3.9  Mean monthly EI\textsubscript{30} for four wheatbelt centres in Western Australia.
Figure 3.10 shows the percentage of rainfall erosivity that occurred in summer over a ten year period. In the northern wheatbelt, winter-dominance only occurs within 50 kms of the coast. In the southern parts of the state, winter-dominance occurs within 150 kms of the west coast.

It can be concluded that in south-west coastal areas, erosive rainfall is likely to occur during the winter and there will be little variability in the erosivity of the rainfall from year to year compared to other agricultural areas. In inland areas, and particularly in the north-eastern areas, erosive rainfall is likely to occur during summer and be very variable from year to year.

Soil losses due to high intensity (erosive) storms are most likely on soils with low infiltration capacities (e.g. clays, loams and sandy soils with a surface seal). Erosive rain is not necessary to cause large soil losses from saturated soils (e.g. waterlogged duplex soils) as the soils have no ability to accept rainfall and have low soil strengths. Therefore the rainfall erosivity maps should be used with this limitation in mind.

Figure 3.10  Percentage of El30 that occurred during summer (November-April) during the period of pluviometer recording (average of ten years).
4. ESTIMATION OF PEAK RUNOFF RATES

4.1 Method of estimating peak runoff rates

Rainfall and runoff rates over periods longer than seven years are available from 35 catchments in the wheatbelt and 65 catchments in the south west. Relationships between peak runoff rates and catchment characteristics were developed by Flavell et al. (1987) (Australian Rainfall and Runoff, 1987). Separate formulae have been derived for different regions and soil types.

Peak flow rates for use in waterway design are calculated according to the regional flood frequency (or Index Flood) method. The following abbreviations are used in the formulae:

- **ARI** = Average recurrence interval.
- **Q_y** = Peak flow rate (m$^3$s$^{-1}$) for an average recurrence interval of y years.
- **Q_y/Q_2** = Ratio of y-year ARI peak flow to the 2-year ARI peak flow.
- **Q_y/Q_5** = Ratio of y-year ARI peak flow to the 5-year ARI peak flow.
- **A** = Catchment area (km$^2$).
- **L** = Mainstream length (km) measured from the catchment outlet to the most remote point on the catchment boundary.
- **S_e** = Equal area stream slope in metres per kilometre (m/km), obtained by equalising the areas above and below a straight line as shown in Figure 4.1.
- **P** = Average annual rainfall (mm) over the catchment area (from Figure 4.2).
- **C_L** = Percentage of the catchment area cleared of native vegetation.
Figure 4.1 Derivation of the equal-area slope of main stream.
Figure 4.2  Average annual rainfall (mm) (Bureau of Meteorology) based on complete years of record to 1979.
4.1.1 Wheatbelt Region (275-600 mm average annual rainfall) and the Esperance coast

(i) Loamy-soil catchments, 75-100 per cent cleared.
The peak flow equation is:
\[ Q_5 = 2.77 \times 10^{-6} A^{0.52} P^{2.12} \]

Frequency factors \((Q_y/Q_5)\) are:

<table>
<thead>
<tr>
<th>ARI (years)</th>
<th>2</th>
<th>5</th>
<th>10</th>
<th>20</th>
<th>50</th>
</tr>
</thead>
<tbody>
<tr>
<td>Q_y/Q_5</td>
<td>0.48</td>
<td>1.00</td>
<td>1.84</td>
<td>3.23</td>
<td>6.10</td>
</tr>
</tbody>
</table>

(ii) Loamy and lateritic, and sandy soil catchments.
The peak flow equation is:
\[ Q_5 = 3.04 \times 10^{-1} A^{0.60} 10^{0.0052 C_L} \]

Frequency factors \((Q_y/Q_5)\) are:

<table>
<thead>
<tr>
<th>ARI (years)</th>
<th>2</th>
<th>5</th>
<th>10</th>
<th>20</th>
<th>50</th>
</tr>
</thead>
<tbody>
<tr>
<td>Q_y/Q_5</td>
<td>0.50</td>
<td>1.00</td>
<td>1.76</td>
<td>3.05</td>
<td>5.65</td>
</tr>
</tbody>
</table>

4.1.2 South West Region (greater than 600 mm average annual rainfall, excluding the Esperance coast)

(i) Jarrah forest with lateritic and sandy soils.
The peak flow equation is:
\[ Q_2 = 8.22 \times 10^{-9} A^{0.73} P^{2.22} (LS_e)^{0.28} 10^{0.0064 C_L} \]

Frequency factors \((Q_y/Q_2)\) are:

<table>
<thead>
<tr>
<th>ARI (years)</th>
<th>2</th>
<th>5</th>
<th>10</th>
<th>20</th>
<th>50</th>
</tr>
</thead>
<tbody>
<tr>
<td>Q_y/Q_2 (0% cleared)</td>
<td>1.00</td>
<td>1.60</td>
<td>2.20</td>
<td>3.00</td>
<td>4.25</td>
</tr>
<tr>
<td>Q_y/Q_2 (50% cleared)</td>
<td>1.00</td>
<td>1.45</td>
<td>1.85</td>
<td>2.30</td>
<td>3.00</td>
</tr>
<tr>
<td>Q_y/Q_2 (100% cleared)</td>
<td>1.00</td>
<td>1.28</td>
<td>1.50</td>
<td>1.75</td>
<td>2.05</td>
</tr>
</tbody>
</table>

(ii) Jarrah forest with loamy soils.
The peak flow equation is:
\[ Q_2 = 3.68 \times 10^{-8} A^{0.68} P^{2.29} 10^{0.0081 C_L} \]

Frequency factors \((Q_y/Q_2)\) are:

<table>
<thead>
<tr>
<th>ARI (years)</th>
<th>2</th>
<th>5</th>
<th>10</th>
<th>20</th>
<th>50</th>
</tr>
</thead>
<tbody>
<tr>
<td>Q_y/Q_2</td>
<td>1.00</td>
<td>1.75</td>
<td>2.55</td>
<td>3.50</td>
<td>5.10</td>
</tr>
</tbody>
</table>
(iii) Karri forest with loamy soils (1,070-1,410 mm average annual rainfall).
The peak flow equation is:
\[ Q_2 = 6.01 \times 10^{-9} A^{0.87} p^{2.41} \]
Frequency factors (Q_y/Q_2) are:

<table>
<thead>
<tr>
<th>ARI (years)</th>
<th>2</th>
<th>5</th>
<th>10</th>
<th>20</th>
<th>50</th>
</tr>
</thead>
<tbody>
<tr>
<td>Q_y/Q_2</td>
<td>1.00</td>
<td>1.51</td>
<td>1.94</td>
<td>2.40</td>
<td>3.05</td>
</tr>
</tbody>
</table>

4.2 Worked examples - Calculation of design peak flow rates

4.2.1 Wheatbelt Region (275-600 mm average annual rainfall) and the Esperance coast

Calculate the peak flow rate for a 20 year average recurrence interval (ARI) from the Cuballing experimental catchment, (approximately 20 km north of Narrogin). This 170 ha catchment is approximately 90 per cent cleared, and soils are predominantly sandy loams.

The average annual rainfall at the Cuballing catchment, is read from Figure 4.2, as approximately 500 mm.

The peak flow equation for loamy soil catchments, 75-100 per cent cleared, is read from Section 4.1.1 as:

\[ Q_5 = 2.77 \times 10^{-6} A^{0.52} p^{2.12} \]

Substituting the catchment area (A), converted to square kilometres as 1.7 km^2, and average annual rainfall (P) of 500 mm:

\[ Q_5 = 2.77 \times 10^{-6} (1.7)^{0.52} (500)^{2.12} \]
\[ = 1.92 \text{ m}^3 \text{s}^{-1} \]

The 20-year ARI peak runoff rate is obtained by multiplying its 5-year ARI peak runoff rate (Q_5) by the 20-year frequency factor Q_{20}/Q_5 of 3.23.

Therefore \[ Q_{20} = Q_5 \times Q_{20}/Q_5 \]
\[ = 1.92 \times 3.23 \text{ m}^3 \text{s}^{-1} \]
\[ = 6.21 \text{ m}^3 \text{s}^{-1} \]

4.2.2 South West Region (greater than 600 mm average annual rainfall, excluding the Esperance coast)

Calculate the peak flow rate for a 50-year average recurrence interval, from a 1200 ha sandy-soil catchment which is 50 per cent cleared, near Balingup (approximately 30 km north-west of Bridgetown). The mainstream length from the catchment outlet to the most remote point on the catchment boundary is 8 km. The topographic profile of the main stream from the catchment boundary is as shown on Figure 4.1.

The average annual rainfall at Balingup is read from Figure 4.2 as approximately 1000 mm. The catchment area (A), expressed in square kilometres, is 12 km^2. The percentage cleared (C_L) is 50. The equal-area slope (S_e), calculated graphically as shown in Figure 4.1, is approximately 21 m in 8 km, or 2.63 m/km.
The peak flow equation for lateritic and sandy-soil catchments from Section 4.1.2 is:

\[
Q_2 = 8.22 \times 10^{-9} A^{0.73} P^{2.22} (LS)_{c}^{0.28} 10^{0.0064} C_L
\]

\[
= 8.22 \times 10^{-9} (12)^{0.73} (1000)^{2.22} (8 \times 2.63)^{0.28} 10^{0.0064}(50)
\]

\[
= 1.13 \text{ m}^3 \text{ s}^{-1}
\]

The 50-year ARI peak flow rate is obtained by multiplying the 2-year ARI peak flow \(Q_2\) by the 50-year frequency factor \(Q_{50}/Q_2\). For a catchment which is 50 per cent cleared, \(Q_{50}/Q_2\) is 3.00.

Therefore, \(Q_{50} = Q_{50}/Q_2 \times Q_2\)

\[
= 1.13 \text{ m}^3 \text{ s}^{-1} \times 3.00
\]

\[
= 3.39 \text{ m}^3 \text{ s}^{-1}
\]

5. **ESTIMATION OF RUNOFF VOLUMES**

For the design of level and absorption banks it is necessary to be able to estimate the runoff volumes which would be expected with a certain probability (or return period) and over a certain duration (e.g. one, two or five days). A storm with a return period of ten years has a 10 per cent probability of occurring in any one year while a storm with a return period of twenty years has only a 5 per cent probability of occurring in any one year. The probability of a given return period flood occurring at least once in a particular interval of time is shown on Table 5.1.

**Table 5.1. Probability of filling during a particular interval of time**

<table>
<thead>
<tr>
<th>Design return period (yrs)</th>
<th>Time interval (years)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>5</td>
</tr>
<tr>
<td>5</td>
<td>0.67</td>
</tr>
<tr>
<td>10</td>
<td>0.41</td>
</tr>
<tr>
<td>20</td>
<td>0.23</td>
</tr>
</tbody>
</table>

For example, over a 20 year interval there is an 88 per cent probability of a structure filling at least once during its lifetime if it was designed for a 10 year return period storm, and a 64 per cent probability if it were designed for a 20 year return period storm.

To estimate the runoff volumes from ungauged agricultural catchments, a regional flood frequency analysis was made of 14 catchments located throughout the agricultural areas (Davies and McFarlane 1986). The analysis has made it possible to estimate the runoff volume \(Q\) for catchment runoff events of different durations (i.e. 1, 2 or 5 days) and with different return periods (i.e. 2, 5, 10 or 20 years) - see Table 5.2.
Table 5.2. Runoff volumes (mm’s) for events of different durations and return periods

<table>
<thead>
<tr>
<th>Design return period (yrs)</th>
<th>Runoff duration</th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1 day</td>
<td>2 days</td>
<td>5 days</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>3.7</td>
<td>4.3</td>
<td>5.0</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>7.6</td>
<td>9.1</td>
<td>10.3</td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>12.0</td>
<td>14.0</td>
<td>15.8</td>
<td></td>
</tr>
<tr>
<td>20</td>
<td>19.6</td>
<td>22.5</td>
<td>26.6</td>
<td></td>
</tr>
</tbody>
</table>

The estimates from Table 5.2 were derived from catchments of up to 1.2 km² (120 ha) in area, and they should not be used for catchments greater than 2.5 km² (250 ha) when designing level and absorption banks for flood mitigation. There is no upper limit in area when designing banks for water erosion control, or drains to discharge runoff.

Table 5.2 shows that runoff volumes are only related to runoff duration and return period. Other variables (e.g. catchment area, rainfall or soil type) are not significant at present although they may become significant with more runoff and catchment information. In particular, the runoff volumes of catchments in lower rainfall areas are likely to be over a greater range for the different return periods than for catchments in higher rainfall areas. However it is not possible to quantify these differences at present.

6. WATERWAYS

6.1 Introduction

Sub. clovers and grasses increase a soil’s structural resistance to erosion. They also provide a protective blanket of leaves and stems over the soil. Heavy grazing destroys this protective cover, leaving the soil vulnerable to erosion during runoff events. Grassed waterways should be fenced in order to allow grazing to be controlled, so that at all times there is a good pasture cover to provide protection against erosion. Side levees may be constructed, in order to confine flow within the grassed waterway.

Native vegetation and trees are not usually as effective as a good grass cover in protecting waterways in areas where large summer flows are frequent. Stubble and other debris washed off paddocks during floods, catches around stems and trunks, increasing their resistance to flow and thereby causing a greater depth of flow. Flow may then overtop side levees and spill outside the waterway. Therefore in areas where summer floods are frequent, it is generally preferable to clear waterways along their entire length and establish pasture in order to provide complete ground cover before constructing banks.

Grassed waterways should never be cultivated, because the cultivated layer is vulnerable to erosion by flowing water. Top-dressings of fertiliser should be applied as required, in order to maintain a complete ground cover.

Waterways are always necessary to take the overflow from farm dams, because a dam which is designed to fill in most seasons, must also overflow in many. Therefore protection of the waterway below every dam is essential, unless the overflow is diverted by a grade bank to a nearby stable waterway.
6.2 Design of grassed waterways

6.2.1 Manning’s formula

Velocities of flow in waterways must be low enough to ensure that soil movement is negligible.

Manning’s formula is commonly used for calculating average velocities of flow in channels. Average velocity is calculated as a function of the roughness of the channel bed and sides, its hydraulic radius (the cross-sectional area of flow divided by its wetted perimeter) and the slope of the energy gradient. The equation has been developed from experimental data.

Manning’s formula is expressed as:

\[ V = \frac{1}{n} R^{2/3} s^{1/2} \]

where \( V \) = average velocity of flow \((\text{ms}^{-1})\)

\( R \) = hydraulic radius \((\text{m})\)

\[ R = \frac{\text{Cross – sectional area of flow (m}^2\text{)}}{\text{wetted perimeter (m)}} \]

where the wetted perimeter is the length of the line of contact between the water and the channel, at a right angle to the direction of flow.

\( s \) = Slope of the energy gradient (approximately the slope of the channel bed) in metres per metre.

\( n \) = Manning’s roughness coefficient.

6.2.2 Manning’s roughness coefficient

The calculated average velocity in a channel will only be as accurate as the estimated value of Manning’s roughness coefficient ‘n’, which was used in making the calculation. Rough channels have higher values of ‘n’ than smooth channels. Small changes in ‘n’ greatly affect estimated velocity. For example a change in ‘n’ of 0.01 from 0.025 to 0.035 will affect velocity by about 40 per cent.

For lined channels, specific information on the type and condition of the material forming the wetted perimeter is known. These data permit estimation of ‘n’ within reasonably well-defined limits.

For earthen channels, on the other hand, various factors affect their retardance to flow and make the estimation of ‘n’ more difficult, and therefore less accurate. These include:

- **Physical roughness**: The type of materials forming the channel bottom and sides affects ‘n’. Fine particles in smooth, uniform surfaces result in relatively low ‘n’ values. Coarse materials and pronounced surface irregularity result in higher values of ‘n’.

- **Vegetation**: The density, physical characteristics and height of vegetation in a channel will greatly affect its retardance to flow. The degree to which the vegetation blocks the cross-sectional area, particularly at shallow depths of flow, should be considered together with its ability to bend flat and effectively line the channel during deeper flows.

- **Cross section and alignment**: Abrupt changes in cross-section or relatively sharp curves result in higher ‘n’ values than more gradual changes.
• **Erosion and deposition**: Erosion and deposition of silt in the channel will affect the value of ‘n’, depending on the changed channel geometry.

• **Obstructions**: Any obstructions to flow will increase the value of ‘n’. The degree of increase will depend on the amount, size and type of the particular debris forming the obstructions.

In addition, the value of ‘n’ in an earthen channel varies with surface conditions during the growing season. It is not a fixed value.

All of these factors should be studied and evaluated, including the likely standard of maintenance and the season of the year when the design storm usually occurs, before selecting the appropriate value of ‘n’. Typical values of Manning’s roughness coefficient are shown on Table 6.1.

It has been common practice in New South Wales and the United States to use \( n = 0.04 \) for waterways which are densely grassed with perennial grasses, and for which more appropriate data are not available. It is suggested that \( n = 0.035 \) be used for waterways of relatively light though well-grassed stands of annual species in Western Australia, as shown on Table 6.1, because their hydraulic roughness will be reduced during summer since dead grass bends over easily in the flow. Careful consideration should be given in each case to the conditions likely to prevail during those seasons of the year when peak flows are most likely, and the most appropriate ‘n’ value selected.

### 6.2.3 Maximum permissible average velocities of flow

The width of grassed waterway required to carry a particular design flow without eroding depends on the average velocity of flow. Maximum permissible average velocities for different soil textures and annual grass cover conditions, are shown on Table 6.2. If the annual-grass cover is not uniform over the whole waterway, the maximum permissible velocity read from

<table>
<thead>
<tr>
<th>Soil texture</th>
<th>Vegetation</th>
<th>% Ground cover</th>
<th>( n^* )</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Sand</td>
<td>Bare</td>
<td>&lt; 40%</td>
<td>0.020</td>
</tr>
<tr>
<td>2. Sand</td>
<td>Sparse grass cover</td>
<td>&lt; 60%</td>
<td>0.020</td>
</tr>
<tr>
<td>3. Sand</td>
<td>Well-grassed annual pasture with sub. clovers</td>
<td>&gt; 60%</td>
<td>0.030</td>
</tr>
<tr>
<td>4. Loamy sand</td>
<td>Vigorous sub. clover and grasses</td>
<td>&lt; 80%</td>
<td>0.035</td>
</tr>
<tr>
<td>5. Sandy loam</td>
<td>Bush water</td>
<td>&lt; 40%</td>
<td>0.025</td>
</tr>
<tr>
<td>6. Sandy loam</td>
<td>Annual pasture</td>
<td>&lt; 60%</td>
<td>0.025</td>
</tr>
<tr>
<td>7. Sandy loam</td>
<td>Annual sub. clover and grasses</td>
<td>&gt; 60%</td>
<td>0.03</td>
</tr>
<tr>
<td>8. Sandy loam</td>
<td>Annual sub. clover and grasses</td>
<td>&gt; 80%</td>
<td>0.035</td>
</tr>
<tr>
<td>9. Sandy loam</td>
<td>Parkland cleared with sub. clover and grasses</td>
<td>&gt; 60%</td>
<td>0.035</td>
</tr>
<tr>
<td>10. Sandy clay loam</td>
<td>Well-grassed with sub. clovers or medics</td>
<td>&gt; 80%</td>
<td>0.035</td>
</tr>
<tr>
<td>11. Clay-loam</td>
<td>Sparse grass cover</td>
<td>&lt; 60%</td>
<td>0.03</td>
</tr>
<tr>
<td>12. Clay-loam</td>
<td>Well-grassed with sub. clovers or medics</td>
<td>&gt; 60%</td>
<td>0.035</td>
</tr>
<tr>
<td>13. Clayey soils</td>
<td>Well grassed</td>
<td>&gt; 60%</td>
<td>0.03</td>
</tr>
<tr>
<td>14. All soils</td>
<td>Densely-grassed with a component at perennial pasture</td>
<td>&gt; 95%</td>
<td>0.04</td>
</tr>
</tbody>
</table>

* Based on values obtained from ‘Soil and Water Conservation Engineering’ (Schwab et al. 1981), and ‘Soil Conservation’ (Hudson, 1981), with interpolations based on local experience provided by D.J. Stanton (WA Department of Agriculture).
Table 6.2 should be that for the section of the waterway with the poorest grass cover.

The maximum average velocities shown on Table 6.2 represent those considered permissible on average earth surfaces. These average velocities may be exceeded without significant erosion on some soils, where sediment deposited by streamflow has produced a well-graded channel bed which is resistant to erosion.

In some instances, average velocities shown on Table 6.2 may be found to be safely exceeded in natural waterways, particularly those draining large catchments. In such cases, it is necessary to be perceptive, enquiring about historical flow depths from local landholders and calculating their average velocities of flow using Manning’s formula, and relating these calculated average velocities to reported erosion. Under these circumstances, it is permissible to design for an average velocity appropriately in excess of the relevant value shown on Table 6.2.

6.2.4 Procedure for determining waterway dimensions

For wide, shallow channels where the width exceeds the average depth of flow by a factor of 5 or more, the hydraulic radius may be substituted by the depth of flow in Manning’s formula, without much loss of accuracy (Skurlow, 1975). Average velocities are overestimated by approximately 12 per cent when the factor is 5, and by approximately 6 per cent when the factor is 10. Most grassed waterways and shallow drains may therefore be safely designed substituting depth for hydraulic radius, though the exact hydraulic radius should be used for relatively deep and narrow drains.

Substituting average depth \( (d_{av.}) \) for hydraulic radius \( (R) \), Manning’s formula is approximated as:

\[
v \approx \frac{1}{n} \left( \frac{d_{av.}}{s^{1/2}} \right)^{2/3}
\]
Table 6.2. Maximum permissible average velocities of flow for waterways with uniform vegetative cover

<table>
<thead>
<tr>
<th>Soil texture</th>
<th>Vegetation</th>
<th>Ground cover</th>
<th>Permissible velocity *(ms⁻¹)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Sand</td>
<td>Bare</td>
<td>&lt; 40%</td>
<td>0.5</td>
</tr>
<tr>
<td>2. Sand</td>
<td>Sparse grass cover</td>
<td>&lt; 60%</td>
<td>0.6</td>
</tr>
<tr>
<td>3. Sand</td>
<td>Well grassed annual pasture with sub. clover</td>
<td>&gt; 60%</td>
<td>0.75</td>
</tr>
<tr>
<td>4. Loamy sand</td>
<td>Vigorous sub. clover and grasses</td>
<td>&gt; 80%</td>
<td>1.2</td>
</tr>
<tr>
<td>5. Sandy loam</td>
<td>Bush waterway</td>
<td>&lt; 40%</td>
<td>0.75</td>
</tr>
<tr>
<td>6. Sandy loam</td>
<td>Annual pasture</td>
<td>&lt; 60%</td>
<td>1.0</td>
</tr>
<tr>
<td>7. Sandy loam</td>
<td>Annual sub. clover and grasses</td>
<td>&gt; 60%</td>
<td>1.0</td>
</tr>
<tr>
<td>8. Sandy loam</td>
<td>Annual sub. clover and grasses</td>
<td>&gt; 80%</td>
<td>1.0</td>
</tr>
<tr>
<td>9. Sandy loam</td>
<td>Parkland cleared with sub. clover and grasses</td>
<td>&gt; 60%</td>
<td>0.9</td>
</tr>
<tr>
<td>10. Sandy clay loam</td>
<td>Well-grassed with sub. clovers or medics</td>
<td>&gt; 80%</td>
<td>1.2</td>
</tr>
<tr>
<td>11. Clay-loam</td>
<td>Sparse grass cover</td>
<td>&lt; 60%</td>
<td>1.0</td>
</tr>
<tr>
<td>12. Clay-loam</td>
<td>Well-grassed with sub. clovers or medics</td>
<td>&gt; 60%</td>
<td>1.2</td>
</tr>
<tr>
<td>13. Clayey soils</td>
<td>Well-grassed</td>
<td>&gt; 60%</td>
<td>1.2</td>
</tr>
<tr>
<td>14. All soils</td>
<td>Densely-grassed with a component of perennial pasture</td>
<td>&gt; 95%</td>
<td>1.5</td>
</tr>
</tbody>
</table>

* Based on values obtained from ‘Soil and Water Conservation Engineering’; (Schwab et al. 1981), and ‘Soil Conservation’ (Hudson, 1981), and interpolations based on local experience provided by D.J. Stanton (WA Department of Agriculture).

Re-arranging terms, the average depth may be calculated as:

\[
\frac{d^{2/3}}{\text{av.}} \approx \frac{v.n}{s^{1/2}} = v.n.s^{-1/2}
\]

and

\[
d_{\text{av.}} \approx (v.n.s^{1/2})^{3/2}
\]

\[
\approx v^{1.5} n^{1.5} s^{-0.75}
\]

Therefore, the average flow depth corresponding to a particular average velocity, may be calculated for a waterway of known slope and Manning’s Roughness Coefficient.

The peak flow (Q) in a waterway may be calculated as:

\[
Q = A.v
\]

where \(A\) is the cross-sectional area of flow, which may be calculated as:

\[
A = w.d_{\text{av.}}
\]

where \(w\) is the maximum width of flow.
Therefore the peak flow may be calculated as:
\[ Q = A \cdot v = w \cdot d_{av} \cdot v \]

Re-arranging terms,
\[ w = \frac{Q}{d_{av} \cdot v} \]

allowing the flow width to be calculated when the peak flow, average depth and average velocity are known.

Field survey information is required in order to determine average depths of flow, if the waterway cross-section does not have a level bottom. As an approximation, a parabolic shape may be assumed for un-eroded natural waterways without side-levees. The average depth of flow may then be calculated as two-thirds of the maximum depth of flow, which occurs in the middle of the parabolic cross-sectional area of flow.

Side-levees built from the outside of the waterway, may be required in order to confine the flow. A freeboard allowance of 0.2 m must be included in their design height, in order to allow for surcharge above the design depth because of variation from peak flow estimates and other factors.

6.2.5 Worked example: Design width of a grassed waterway

Calculate the design width of a uniformly-grassed waterway of annual species, on sandy loam soil in a natural depression of 3 per cent slope, to carry a design peak flow of 1.5 m$^3$s$^{-1}$.

Follow the initial procedure for the design of the waterway outlined in flow-chart form on Figure 6.1.

**Step 1**: If the waterway is not fenced out, it will probably be relatively bare and unprotected during autumn and early winter, when peak flows are likely. Therefore calculate the design width for the worst possible condition, namely, not fenced out and a negligible grass cover.

**Step 2**: The value of Manning’s roughness coefficient is read from Table 6.1 for a bare, sandy loam as $n = 0.025$.

**Step 3**: The maximum permissible average velocity for this sandy loam soil is read from Table 6.2 as 0.75 ms$^{-1}$.

**Step 4**: The waterway width will probably be very much greater than five times its average depth. Therefore calculate the average flow depth which will result in the maximum permissible velocity of flow. (The slope is 3 per cent, which must be expressed in units of metre per metre, as $s = 0.03$).

\[ d_{av} \approx v^{1.5} \cdot n^{1.5} \cdot s^{-0.75} \]
\[ = (0.75)^{1.5} \cdot (0.25)^{1.5} \cdot (0.03)^{-0.75} \]
\[ = 0.036 \text{ m} \]

The design peak flow ($Q$) was given as 1.5 m$^3$s$^{-1}$.
Figure 6.1. Initial procedure for design of waterways.
Step 5: The width of flow is calculated as:

\[ w = \frac{Q}{V_{dav}} \]

\[ = \frac{1.5}{(0.036)(0.75)} \]

\[ = 55.6 \text{ m} \]

Step 6: Survey the cross section of the waterway in the field. Calculate its average depth across the design width of 55.6 m. (In order to build up experience with shapes of un-eroded natural waterways, it is a good idea to compare the measured average depth with that calculated from the parabolic approximation of two thirds of the maximum depth).

If the available width at the design average depth is about 55.6 m, accept the design.

Step 7: Suppose that the available width at the design average depth were greater than 55.6 m.

Step 8A: Transfer to the procedure outlined on Figure 6.2.

Step 9A: Are side-levees required in order to limit the area lost to cultivation? If not, accept the design.

Step 10A: If side-levees are required, obtain the average depth when the waterway is just flowing at width \( w \) by field survey. Suppose that this average depth is 0.020 m.

Figure 6.2. Design of waterways if the available width is greater than the initial design width.
**Step 11A:** The flow depth at the side levees will therefore be:

\[
\frac{3}{2} (d_{av.} - 0.020) = \frac{3}{2} (0.036 - 0.020)
\]

\[
= \frac{3}{2} (0.016) = 0.024 \text{ m}
\]

**Step 12A:** Adding 0.2 m freeboard, the minimum height of the side levees required will be

0.024 + 0.200 = 0.224 m

Suppose, on the other hand, that the available width at the design average depth were less than 55.6 m. The waterway must be re-designed following the procedure outlined on Figure 6.3.

**Step 8B:** Suppose that field survey information shows that flow will occur over only a 10 m width at an average depth of 0.036 m. The cross-sectional area of flow is therefore:

\[
A = w \cdot d_{av.}
\]

\[
= (10) (0.036)
\]

\[
= 0.36 \text{ m}^2
\]

**Step 9B:** The peak flow for a 10 m wide waterway is therefore:

\[
Q = A \cdot v
\]

\[
= (0.36) (0.75)
\]

\[
= 0.27 \text{ m}^3 \text{s}^{-1}
\]

which is less than the 1.5 m$^3$s$^{-1}$ peak flow.

**Step 10B:** Select an average depth, which is more likely to result in the 1.5 m$^3$s$^{-1}$ peak flow, and obtain its relevant flow width from the field survey. Try an average depth of flow of 0.1 m. Suppose that survey information on the shape of the natural waterway in the field shows that this flow will occur over a 30 m width.
Figure 6.3  Design of waterways if the available width is less than the initial design width.
**Step 11B:** The cross-sectional area of flow is therefore:

\[ A = (30) (0.1) \]

\[ = 3 \text{ m}^2 \]

The average velocity of flow at 0.1 m average depth is:

\[ V \approx \frac{1}{n} \frac{d}{\text{av.}} \frac{2/3}{s^{1/2}} \]

\[ \approx \frac{1}{0.025} (0.1)^{2/3} (0.3)^{1/2} \]

\[ \approx 1.49 \text{ ms}^{-1} \]

**Step 12B:** The peak flow (Q) in this 30 m wide waterway flowing at an average depth of 0.1 m is therefore

\[ Q = A \cdot v \]

\[ = (3) (1.49) \]

\[ = 4.47 \text{ m}^3 \text{s}^{-1} \]

**Step 13B:** This is more than the 1.5 m$^3$s$^{-1}$ design peak flow. Therefore revert to Step 10B of Figure 6.3.

**Step 10B:** Select likely average depth between the 0.036 m (for 0.28 m$^3$s$^{-1}$ peak flow) and the 0.10 m (for 4.47 m$^3$s$^{-1}$ peak flow)

Try an average depth of 0.07 m. Suppose that field survey information shows that the width of flow at an average depth of 0.07 m is 20 m.

**Step 11B:** The cross-sectional area of flow is therefore

\[ A = (20) (0.07) \]

\[ = 1.4 \text{ m}^2 \]

The average velocity of flow is:

\[ V \approx \frac{1}{n} \frac{d}{\text{av.}} \frac{2/3}{s^{1/3}} \]

\[ \approx \frac{1}{0.025} (0.07)^{2/3} (0.03)^{1/2} \]

\[ \approx 1.18 \text{ ms}^{-1} \]

**Step 12B:** The peak flow is:

\[ Q = A \cdot v \]

\[ = (1.4) (1.18) \]

\[ = 1.65 \text{ m}^3 \text{s}^{-1} \]

**Step 13B:** This is slightly more than the 1.5 m$^3$s$^{-1}$ design peak flow.

Therefore a 20 m wide waterway flowing at an average depth of 0.07 m would discharge the design peak flow in this bare sandy-loam drainage line, with a small margin for safety.
**Step 14B:** However its average velocity of 1.18 ms\(^{-1}\) is considerably higher than the 0.75 ms\(^{-1}\) maximum which is permissible for a sandy-loam soil with negligible grass cover, shown on Table 6.2.

**Step 15B:** Therefore design for improved waterway conditions. If the waterway were to be fenced and maintained in a well-grassed condition, the maximum permissible average velocity would be increased to 1.0 ms\(^{-1}\). Manning’s Roughness Coefficient for a well-grassed waterway is \(n = 0.035\) (from Table 6.1).

**Step 16B:** Its average velocity would be:

\[
V \approx \frac{1}{n} \frac{d}{d_{av}} \frac{2}{3} s^{1/2} \\
\approx \frac{1}{0.035} (0.07)^{2/3} (0.03)^{1/2}
\]

\[
\approx 0.84 \text{ ms}^{-1}
\]

Its peak flow would then be

\[
Q = A.v = (1.4)(0.84) = 1.18 \text{ m}^3 \text{s}^{-1}
\]

**Step 17B:** This is less than the 1.5 m\(^3\)s\(^{-1}\) design peak flow.

**Step 18B:** Therefore try a deeper average flow of 0.08 m in the well-grassed waterway. Suppose that field survey information shows that the width of flow at an average depth of 0.08 m is 23 m.

**Step 16B:** Its average velocity would be:

\[
V \approx \frac{1}{n} \frac{d}{d_{av}} \frac{2}{3} s^{1/2} \\
\approx \frac{1}{0.035} (0.08)^{2/3} (0.03)^{1/2}
\]

\[
\approx 0.92 \text{ ms}^{-1}
\]

Its peak flow would then be:

\[
Q = A.v = w d_{av} \cdot v = (23)(0.08)(0.92) = 1.69 \text{ m}^3 \text{s}^{-1}
\]

**Step 17B:** This is only 0.19 m\(^3\)s\(^{-1}\) (or 13 per cent) more than the design peak flow of 1.5 m\(^3\)s\(^{-1}\).

Therefore a 23 m wide well-grassed waterway flowing at an average depth of 0.08 m would discharge the design peak flow, with a small margin for safety.

**Step 14B:** Its average velocity of 0.92 ms\(^{-1}\) would be less than the 1.0 ms\(^{-1}\) permissible for a well grassed sandy loam (from Table 6.3), and is therefore acceptable.
Therefore a permanently well-grassed, fenced waterway 20 m wide would safely discharge the design peak flow. Note that its average depth of 0.08 m compared with the 0.07 available m for flow 20 m wide in this waterway without side levees, means that flow at its edges would be 0.01 m deep. A side levee maintained at a height of 0.21 m (to allow the required 0.20 m freeboard allowance for surcharge) would be required in order to keep flow within the 20 m wide waterway. Alternatively, the design could be re-calculated for a wider waterway without side levees, using further field survey data.

Unless higher historic flow peaks with little erosion indicate that higher velocities are acceptable, the design must therefore be for a fenced waterway which is fertilised and grazed in a controlled manner to maintain a permanently, well-grassed condition. From the above calculations a 20 m wide waterway, with side levees maintained at least 0.21 m high, would be adequate.

6.3 The need to maintain grassed waterways

The maximum protection against erosion must be maintained on grassed waterways at all times, in order to minimise the risk of rilling and gullying. A grassed waterway may be difficult and expensive to stabilise once eroded, particularly if flow cannot be diverted. Therefore, maintaining a healthy grassed waterway is a wise investment for the future.

6.3.1 Fencing for controlled access

Fencing along both sides and ends of a waterway allows controlled access, and may best be incorporated at a paddock boundary in a conservation farm plan. Fenced waterways can be grazed in a controlled manner in order to maximise the vigour of the sward. ‘Crash’ grazing by a relatively large number of stock for a day or two will minimise stock tracks which can cause channelling of flow and eventual gullying. Alternatively, grassed waterways may be slashed or mown for hay.

Fenced waterways should not be used for farm access, because flow may occur down vehicle tracks, which can then initiate gullying.

6.4 Gully treatments

Where a waterway has degenerated into a gully, the best treatment may be to help nature reclaim it by vegetative methods following gully filling, or by using gully head structures incorporating level sill outlets. Gully head treatments require immediate maintenance in times of flood, in case weaknesses develop which may allow the gully head to again advance. Otherwise gully head structures may be undercut in a matter of hours during a flood.

6.4.1 Gully filling

Where a waterway has degenerated into a gully, stock-piling topsoil for replacement after reshaping, followed by re-seeding, can be effective in again stabilising the waterway.

Temporary diversion of flow may be required, particularly where catchment areas are greater than 100 ha. Design dimensions of reshaped waterways should be re-calculated as shown in Section 6.2.

6.4.2 Gully head sills

Gully head sills spread flow in a shallow sheet so that it enters the gully not only at its head, but along its sides as well. This reduces the power of the falling water per unit width of overfall into the gully, which can allow grasses to survive and protect the soil.

Shallow flow into the gully is achieved by building a check bank at the gully head, in order to divert the overflow to level sill outlets on either side. The check bank must extend beyond the maximum width of flood flows. Most failures of gully head sills can be attributed to insufficient width of check bank.
The check bank should be maintained at a height of at least 0.7 m above ground level. If pushed from uphill, the design must be such that the excavation must be submerged when the sills overflow, so that there is no free overfall to initiate gullying above the check bank.

The total length of level sill required may be obtained using the design flow for the waterway, calculated for the design average return period (as shown in Section 6.3.1). The total sill length of 10 m is required for each cubic metre per second of flow.

The sill channels on either side of the bank must have sufficient cross-sectional area, to conduct the design flow and allow for deposition of any silt from erosion upstream. Silt deposits should be removed immediately after major runoff events in order to safeguard channel capacity. Deposition in the sill channels may diminish in later years, as the waterway and channel of the check bank become grassed, causing any silt to be deposited upstream in the waterway before it reaches the channel.

7. GRADE BANKS

7.1 Types of grade banks
Grade banks are used in soil conservation to intercept and divert runoff into waterways at non-erosive velocities.

7.1.1 Narrow-based grade banks
Narrow-based grade banks have a ridged peak, and are constructed from earth removed from a flat-bottomed channel on the up-hill side. The areas occupied by the bank and channel are not cropped.

7.1.2 Broad-based grade banks
Broad-based grade banks have gently sloping batters and channel bottoms, which allow cropping of the bank and channel. Because depth of seed placement affects crop yields, batter widths of broad-based banks should be wide enough to allow sowing with existing and anticipated future machinery.

Broad-based banks should not be constructed on land slopes of greater than 4 per cent, because of reduced channel capacity on steeper slopes. Construction costs of broad-based banks are significantly higher than those of narrow-based banks, because of the larger amounts of soil moved and the stockpiling and replacing of topsoil which may be required. On soils with a shallow, less permeable subsoil layer, broad-based banks may also function as seepage interceptors.

Broad-based banks should be maintained at a height of at least 0.5 m above the channel bottom. Maintenance may be conveniently carried out using a disc plough, moving soil upslope on both sides of the bank.

7.2 Functions of grade banks
Grade banks are used to:

- **Divert surface runoff.** Grade banks divert runoff across a slope from an eroded or potentially erodible area to a stable waterway or farm dam, or conduct the overflow from a farm dam to a stable waterway.

- **Reduce effective slope length.** Grade banks effectively break up a long-slope into a series of shorter-slopes. The maximum depth of runoff is therefore reduced, together with its velocity and erosivity.
• **Increase runoff duration and decrease peak flow rates.** The longer route taken by the diverted runoff, and its generally lower flow velocities, usually increase the time taken for runoff to reach the catchment outlet. Consequently the peak flow rate at the catchment outlet or critical design point is often reduced.

• **Increase infiltration and reduce runoff.** Because farming operations are carried out parallel to banks, each furrow and cultivation ridge stores a small additional amount of runoff, which can infiltrate into the soil during and after rainfall.

### 7.3 Channel outlets

Channel grades increase abruptly where grade banks discharge into waterways. Therefore, flow velocities also increase, with the potential to cause erosion. Velocities should be reduced by spreading the flow in order to make it as shallow as possible, by means of a level sill at the outlet of each grade bank.

To form a level sill, the channel is constructed beyond the end of the bank, with the downhill edge of the channel on a true contour. The flow is therefore discharged along the length of the sill. At least a 10 m length of level sill is required at each grade bank outlet.

Spoil from the extended channel is used to form a bank uphill of the channel, which diverts flow from upslope. Level sills at each successive bank downslope should terminate just short of the waterway, so that flow in the waterway from upslope does not enter the sill channel.

It is important that sills should not develop low points, such as those caused by stock tracks or trickle flow. A short length of fence along the sill from the bank to the waterway can help reduce such damage.

### 7.4 Bank spacing

Ideally, banks are spaced on a slope, such that runoff is diverted before it develops enough kinetic energy to erode soil. Banks must at least be spaced at such intervals that rilling of the inter-bank area is greatly reduced. The main factors determining bank spacing are:

• **Soil type:** soil texture and structure at the surface and in the subsoil affect infiltration rates and, therefore, runoff. The chemical and physical nature of soil materials also affects its ability to resist erosion.

• **Slope** affects runoff velocity and therefore its ability to erode soil.

• **Land use and treatment:** the frequency and intensity of tillage affects the infiltration of rainfall into the soil and, therefore, runoff. Tillage also loosens the soil, making it more vulnerable to erosion.

• **Land management:** contour working and stubble retention reduce the velocity and erosivity of runoff flows.

• **Outlet stability:** if safe outlets to a waterway are limited, critical disposal points may dictate the starting points for surveying grade banks.

• **Rock outcrops:** large rock outcrops require a bank immediately down-slope, in order to divert surface runoff.

• **Farm dams:** grade banks increase the effective catchment areas of farm dams. Existing or proposed farm dams therefore dictate the starting points for surveying grade banks.
Where a system of banks is necessary to reduce erosion, a sound knowledge and experience of the local district is invaluable in deciding on bank spacings between those necessarily located at fixed landscape positions, such as waterway entry points and paddock features.

Maximum bank spacings to reduce erosion have been determined through experience in the Wheatbelt and Great Southern regions. Experienced soil conservation personnel in these regions and at Head Office, have contributed to determining maximum bank spacings.

On other than sandplain soils, bank spacings (averaged along the length of each bank) should not exceed those shown on Figure 7.1. These have been described as linear functions of average land slope for the three regions shown on Figure 7.2.

(i) The South-Eastern Wheatbelt receiving less than 450 mm of average annual rainfall has less intense rainfall than the other regions, largely because of a lesser incidence of extreme summer storms. It therefore has the widest maximum spacings.

(ii) The Northern and Eastern Wheatbelt receiving less than 450 mm average annual rainfall is subject to higher intensity summer rainfall than the South Eastern Wheatbelt, and hence requires lower maximum bank spacings.

(ii) Areas receiving more than 450 mm average annual rainfall are more likely to experience saturation-excess runoff than the other areas, necessitating the closest maximum bank spacings in order to reduce erosion.
Figure 7.1. Maximum bank spacings for all except sandy soils, varying with land slope for the regions shown on Figure 7.2.
Figure 7.2  Maximum bank spacings for all except sandy soils for different regions, bounded by 450 mm average annual rainfall isohyet, and the extension of a line between Brookton and Gorrigin.
Large peak flows may occur from sandy soils during high-intensity summer storms. Erosion can be satisfactorily reduced with bank spacings of up to 400 m on sandy soils in areas receiving less than 450 mm average annual rainfall, south of the extension of a line through Brookton and Corrigin (shown on Figure 7.2) and up to 300 m north of this line, provided that there is reasonable vegetative cover to reduce overland flow velocities.

In designing a bank system, the top-most bank must be placed at least as close to the watershed divide as the average bank spacing downslope. Where waterways are unstable or unavailable, level or absorption banks of sufficiently large capacity to store expected, long-return-period runoff should be considered (see Section 8).

### 7.5 Design procedure for grade banks

Grade banks should be designed to safely discharge peak flows from their effective catchment areas. Ideally, average recurrence intervals of high flows would be calculated, in order to design the channel capacity required for a flow of known recurrence-interval. However, such calculations are not yet considered to be sufficiently accurate for reliable use in the design of grade banks.

An alternative design method uses bank heights and grades which have previously been found to be adequate over several decades.

#### 7.5.1 Bank grades

Grade banks are surveyed on the ground surface for subsequent construction of a uniform-cross-section channel on the uphill side, and a uniform-height bank immediately downslope.

The channel grade at the outlet end must be low enough to carry peak flows without erosion of the channel surface. Maximum channel grades of up to 0.5 per cent have been found satisfactory for narrow-based banks on some erosion-resistant soils, while maximum grades of broad-based banks should not exceed 0.3 per cent. Experience on similar soils in the locality is a necessary guide in selecting maximum bank grades which do not result in unacceptable channel erosion.

##### 7.5.1.1 Uniform-grade banks

Banks which are surveyed with a uniform grade along their entire length will have shallower flow depths upstream, because of progressively reduced catchment. Overtopping of the banks during extreme runoff events is therefore less likely upstream. Uniform-grade banks are required where concentrated flow is diverted, such as from a road culvert or farm dam overflow to a waterway.

##### 7.5.1.2 Variable-grade banks

If a uniform depth of flow is required in order to make use of the full channel capacity along its entire length, grades must be progressively increased towards the outlet end (from Manning’s formula). The maximum grade at the outlet end should be multiplied by factors shown on Table 7.1, calculated as the square of the ratio of the distance of each quarter point from the closed end, to the total bank length.

<table>
<thead>
<tr>
<th>Ratio of</th>
<th>Distance from closed end (m)</th>
<th>0</th>
<th>1/4</th>
<th>1/2</th>
<th>3/4</th>
<th>1</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Total bank length (m)</td>
<td>0</td>
<td>1</td>
<td>2</td>
<td>3</td>
<td>4</td>
</tr>
<tr>
<td>Multiplier of maximum grade (square of the ratio)</td>
<td>0</td>
<td>0.06</td>
<td>0.25</td>
<td>0.56</td>
<td>1</td>
<td></td>
</tr>
</tbody>
</table>
If a variable-grade bank is to be surveyed so that the depth of flow will be approximately uniform along its entire length of 800 m, with a maximum grade of 0.4 per cent, the design grades must be calculated as the grades upstream of at least each quarter-length of bank. From Table 7.1, the design grades of each quarter-length of bank will be:

<table>
<thead>
<tr>
<th>Quarter</th>
<th>Grade Calculation</th>
<th>Design Grade</th>
</tr>
</thead>
<tbody>
<tr>
<td>First Quarter</td>
<td>0.06 x 0.4%</td>
<td>0.02%</td>
</tr>
<tr>
<td>Second Quarter</td>
<td>0.25 x 0.4%</td>
<td>0.1%</td>
</tr>
<tr>
<td>Third Quarter</td>
<td>0.56 x 0.4%</td>
<td>0.22%</td>
</tr>
<tr>
<td>Fourth Quarter</td>
<td>1 x 0.4%</td>
<td>0.4%</td>
</tr>
</tbody>
</table>

Continuously-varying grades may be similarly calculated for surveying, using automatic surveying equipment.

7.5.2 Height of grade banks

Narrow-based grade banks should be constructed and maintained so that, after settling, their minimum height is at least 0.5 m above the natural ground surface. Broad-based grade banks should be maintained at a minimum settled height of 0.5 m above their channel bottom.

Banks which divert runoff from steep, non-arable areas should be maintained at a height greater than 0.5 m above the natural surface, since any failure of such a diversion bank may cause serious erosion on arable land downslope. A bulldozer-built bank constructed more than 1.2 m high is recommended in such situations, in order to provide long-term protection against serious erosion downslope.

7.5.3 Length of grade banks

The length of a grade bank affects the peak flow rate which it discharges, since its catchment area increases with length. Grade banks should not exceed 800 m in length, unless their height is increased above 0.5 m, in order to allow them to discharge higher peak flows.

In situations where lengths in excess of 800 m are unavoidable, the possibility of designing one bank shorter than 800 m, with another bank discharging in the opposite direction to a second waterway, should be investigated. Such a lay-out may require re-alignment or replacement of fencing in accordance with a conservation farm plan. Such paddock re-arranging may allow the area between banks which can be worked as one piece, to be doubled.

If such overlapping, opposite-flowing banks are not feasible in a particular situation, the settled height of grade banks more than 800 m long must be increased to greater than 0.5 m above the ground surface. If grade banks up to 1000 m long are necessary, they should be maintained at a minimum settled height of at least 0.6 m above the ground surface.

7.6 Maintenance requirements of grade banks

Grade banks should be maintained at their design height along their entire length, because overtopping at any low point may breach the bank and cause serious erosion downslope. Therefore it is particularly important that any low points, which may develop at stock or vehicle crossings, should be checked at least once a year, and built up again as required.
8. LEVEL AND ABSORPTION BANKS

8.1 Introduction

Level and absorption banks are soil conservation and flood mitigation structures used to control runoff waters when a waterway cannot be safely maintained. The banks are commonly used in low rainfall areas of the wheatbelt (less than 350 mm average annual rainfall) and on steep ground below areas that produce a lot of runoff.

Both level and absorption banks are surveyed on the contour. Level banks are open at one or both ends whereas absorption banks are turned up-hill at both ends. Water storage in level banks is almost all in the bank channel with little water being stored above ground level. Absorption banks however can have two-thirds or more of their water stored above ground level, making them more efficient (but more risky) structures than level banks. Level banks are only recommended where widespread leakage or piping through the bank spoil is expected.

Where water is stored above ground, as behind absorption banks, safeguards must be included in the design. Such safeguards may include:

(i) Designing for a long return period (low probability) runoff volume.
(ii) Providing adequate freeboard (at least 0.4 m after settling) when the banks are full to overflowing around their ends.
(iii) Ensuring bank lengths are not excessive (e.g. not more than two kilometres) so that water does not have to flow long distances to reach an end overflow.
(iv) Constructing channel stops in the bank channel to isolate sections of banks thought to be prone to breaching or losing freeboard (e.g. sandy soils).

Flow around the ends of level and absorption banks can result in rills or gullies where no waterways are provided. Such damage is likely to be less than that which will occur if the bank breaches and releases all its above-ground water. Level and absorption banks should be constructed on a hillside so that the overflow from the ends of one bank does not become inflow to any banks further down-hill. Thus a strip of land is left on the hillside which can carry the over flow from all the bank ends. If feasible, this land should not be cultivated or heavily grazed, in order to prevent erosion during the rare storm events when the bank capacities are exceeded. Where overflow around the ends is expected fairly frequently, level sills may be constructed between adjoining bank ends to spread the overflow more evenly.

8.2 Estimating the storage capacity of level and absorption banks

Bank capacities are affected by:

(i) The amount of end turn-up.
(ii) The slope of the land.
(iii) Soil type.
(iv) The machinery used for construction.
(v) The number of pushes.
(vi) The soil moisture content when the banks are built.

The last four factors determine the dimensions of the bank channel and the bank height. These factors are most important for level banks where most storage is in the bank channel. The amount of end turn-up and the slope of the land are the most important factors controlling the capacity of absorption banks.
McFarlane and Ryder (1985) have outlined a method of surveying bank capacities. Figure 8.1 shows the cross-section of a typical absorption bank.

Above ground capacity (AGC) in cubic metres per metre of bank can be estimated by:

\[ \text{AGC} = \frac{50T^2}{i} \]

where \( T \) = turn-up (metres)
and \( i \) = land slope (per cent)

Thus if a bank has a 0.5 m turn up on a 2 per cent slope,

\[ \text{AGC} = \frac{50(0.5)^2}{2} = 6.25 \text{ m}^3/\text{metre length of bank.} \]

The width \( W \) of water in metres which will back-up behind an absorption bank when it is full can be calculated by:

\[ W = \frac{100T}{i} \]

In the above example, \( W = \frac{100(0.5)}{2} = 25 \text{ metres} \)

Below ground capacity (BGC) of a bank can be estimated from channel dimensions if it is assumed the channel is parabolic in shape.

\[ \text{BGC} = \frac{2}{3} \text{ C.D.} \]

Where

\[ C = \text{channel width (metres)} \]
\[ D = \text{maximum channel depth (metres) below ground level} \]
Thus for a bank with $C = 6$ m and $D = 0.6$ m,

$$BGC = \frac{2}{3} (6) (0.6) = 2.4 \text{ m}^3/\text{m}$$

### 8.3 Design of level and absorption banks

#### 8.3.1 Systems of level and absorption banks on catchments

The procedure to follow when designing level and absorption banks is shown as a flow diagram in Figure 8.2 and explained in point form below. In the design event the banks will be just filled (i.e. flow will commence around the ends of the banks).

Firstly it must be determined whether the main purpose of the banks is erosion control or flood mitigation (or both). For erosion control, section A is used; for flood mitigation, Section B is used while if both are relevant, the design should be done using both A and B with the final design being the one which meets both criteria.

Section A: Level-and absorption-banks for erosion control

A1 Decide on the bank spacing which is required to prevent unacceptable inter-bank rilling. This decision is based on site conditions (soil type, management) and previous experience in the district. Maximum bank spacings are shown on Figures 7.1 and 7.2.

A2 Decide on the return period flood(s) being designed for. This decision is based on the consequences of flow around the bank ends and can be made jointly with the farmer. When flow around the ends is likely to be onto cultivated soil, a return period of at least 10 years should be used.
Figure 8.2. Procedure for designing level and absorption banks.
A3 Decide whether the design flood will be of 1, 2 or 5 days duration. This decision is based upon the likelihood of water being retained in the bank channels for long periods. In clayey soils, the 5 day flood is more appropriate to use. In sandy soils, the one day flood may be appropriate.

A4 Determine the runoff depth (mm) during the design flood from Table 5.2.

A5 Calculate the required bank cross-sectional area from knowing bank spacing (A1) and design runoff (A5) using:

\[
\text{Cross-sectional area (m}^2\text{)} = \text{Bank spacing (m) x Design runoff (m)}.
\]

A6 From a knowledge of bank cross-sectional areas in your district, decide whether the area calculated in A5 is feasible. If it is feasible, A5 is the bank size which should be built on the site to contain the design flood.

A7 If the calculated area is greater than a feasible area, calculate the inter-bank spacing for a feasible cross-section using:

\[
\text{Bank spacing (m) = \frac{\text{feasible cross-sectional area (m}^2\text{)}}{\text{design runoff (m)}}.}
\]

Alternatively, improve or provide a waterway (if feasible) and install grade banks.

Section B: Level and absorption banks for flood mitigation

B1 Decide on the return period flood being designed for. This decision is based on the consequences of flow around the ends and can be made jointly with the farmer and any other people likely to be affected. When flow around the ends is likely to be onto cultivated soil, a return period of at least 10 years should be used.

B2 Decide whether the design will be of 1, 2 or 5 days duration. This decision is based upon the likelihood of water being retained in the bank channel for long periods. In clayey soils, the five day flood is more appropriate to use. In sandy soils, the one day flood may be appropriate.

B3 Determine the runoff depth (mm) during the design flood from Table 5.2.

B4 Estimate the bank cross-sectional area that is possible for the site from a knowledge of similar banks in similar areas.

B5 Calculate the bank spacing (m) from:

\[
\text{Bank spacing (m) = \frac{\text{Bank cross-sectional area (m}^2\text{)}}{\text{Design runoff depth (m)}}.}
\]

Example 1: Erosion control

Calculate the bank cross-sections required to hold the 10 and 20 year return period storms on a catchment with sandy loam soils. District experience has shown that unacceptable inter-bank rilling occurs when banks are more than 250 m apart on sandy loam soils.
Steps
A1 Bank spacing = 250 m
A2 Return periods = 10 and 20 years
A3 2 day duration flood appropriate due to sandy loam soils
A4 From Table 5.2.
    10 year return period runoff volume = 14.0 mm
    20 year return period runoff volume = 22.5 mm
A5 Cross sectional area (m$^2$) = bank spacing (m) x design runoff (m).
    10 year return period $A_{10} = 250 \times 0.0140 = 3.5 \text{ m}^2$
    20 year return period $A_{20} = 250 \times 0.0225 = 5.6 \text{ m}^2$

The average of 8 cross-sections across large absorption banks in the Merredin catchment is about 8 m$^2$, which shows banks of this size would be more than adequate for both the 10 and 20 year return period flood. If there is concern that the bank channels may retain water, the 5 day duration flood should be calculated. The required cross-sectional areas for the 10 and 20 year floods in this case are 4.0 and 6.7 m$^2$ respectively.

Example 2: Flood mitigation

Calculate the bank spacing required for banks with a 6 m$^2$ cross-section to contain the 20 year flood runoff from a sandy catchment.

Steps
B1 Return period = 20 years.
B2 One day duration flood as it is considered likely that the sandy channels will be empty during the 20 year storm.
B3 From Table 5.2
    20 year return period runoff = 19.6 mm.
B4 Cross-sectional area = 6.0 m$^2$.
B5 Bank spacing (m) = \[
\frac{\text{Cross sectional area (m}^2\text{)}}{\text{Design runoff depth (m)}} = \frac{6.0}{0.0196} = 306 \text{ m}
\]

Bank spacing for the 2 and 5 day duration floods are 267 and 225 m respectively.

The calculation of bank spacings can be represented in graphical form as shown in Figure 8.3 (from D. Bennett, Merredin Regional Office).
Figure 8.3. Graphical estimation of bank spacings from runoff depth in the design storm and bank cross-sectional areas.
8.3.2 Single level and absorption banks below water shedding areas

In areas where waterways are available in the lower sections of catchments, it is often only necessary to build a single level or absorption bank below water shedding areas at the top of a catchment (e.g. below lateritic breakaways or outcropping rock). Grade banks and/or seepage interceptors are then built where the waterway is available. The design of level and absorption banks in the previous section is based upon gauged runoff from whole catchments and Table 5.2 is inappropriate for designing level and absorption banks around areas known to have enhanced runoff properties.

The procedure to follow when designing single level or absorption banks around shedding areas is:

(i) Estimate the maximum distance from the proposed bank to the top of the shedding area (L) in metres.

(ii) Examine the material into which the bank will be constructed. If the bank is to be in permeable gravels, the channel is likely to empty quickly and a shorter duration design storm (e.g. 24 hours) is more appropriate than if the channel is to be in heavy clay, when a longer duration storm (e.g. 72 hours) is required.

(iii) Calculate the amount of rain likely during the design storm using Figure 3.1 and Table 3.1.

(iv) From an examination of the shedding area, estimate the proportion of runoff that is likely to occur during the design storm. If the shedding area is predominantly rock outcrop, a high percentage of runoff can be expected (e.g. 70 per cent). On other areas, it is much harder to estimate percentages. Table 8.1 shows the percentage of storm rainfall that is predicted to become runoff on whole catchments in areas where the 12 hour duration, 10 years return period storm intensity is 5.05 mm/hr (e.g. Narrogin, Merredin). Table 8.1 has been calculated for different design storms (Figure 3.1 and Table 3.1) and estimated runoff volumes (Table 5.2). Using the figure and tables it is possible to calculate percentage runoff for different parts of the agricultural area. Runoff from shedding areas will be greater than those shown on Table 8.1.

Table 8.1. Percentage of storm rainfall becoming runoff from whole catchments in areas with a 12 hour duration, 10 year return period storm intensity of 5.05 mm/hr

<table>
<thead>
<tr>
<th>Return period (years)</th>
<th>Storm duration (days)</th>
<th>1</th>
<th>2</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>5</td>
<td>4</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>10</td>
<td>9</td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>16</td>
<td>14</td>
<td></td>
</tr>
<tr>
<td>20</td>
<td>26</td>
<td>23</td>
<td></td>
</tr>
</tbody>
</table>

(v) Estimate the size of the required bank by:

Cross-sectional area (m²) = L (m) x design runoff depth (m) = L (m) x design rainfall (m) x \( \frac{\text{percentage runoff}}{100} \)
Example

Determine the capacity of an absorption bank to be built in gravelly soils below a lateritic breakaway at Narrogin to handle the 10 years return period storm. The maximum distance of the bank from the top of the breakaway is 50 m.

Steps

(i) \( L = 50 \) m.

(ii) Permeable gravels, design storm duration = 24 hours.

(iii) 24 hour duration, 10 year return period storm intensity at Narrogin = 5.05 \times 0.63 = 3.18 mm/h. Storm amount = 3.18 \times 24 = 76 mm or 0.076 m.

(iv) Table 8.1 shows about 16 per cent of one day storm rainfall becomes runoff on whole catchments in 10 year return period storms. Non-wetting lateritic soils are likely to shed two or three times as much (i.e. 32 to 48 per cent runoff).

(v) Bank cross-sectional area (m\(^2\))

\[
= L \times \text{design rainfall} \times \frac{\text{percentage runoff}}{100}
\]

\[
= 50 \times 0.076 \times \frac{32}{100}
\]

\[
= 1.22 \text{ m}^2 \text{ for a 32 per cent runoff}
\]

(and 1.82 m\(^2\) for a 48 per cent runoff)

The average of 16 surveyed cross-sections across single push level banks on the Cuballing experimental catchment was 2.2 m\(^2\). Such banks would be more than adequate for the above example.

8.4 Maintenance requirements of level and absorption banks

Some landholders prefer large bulldozer-built banks to small grader-built banks as the former are considered to require no maintenance. However, large banks do require maintenance if they are to retain their freeboard. The damage resulting from a bank (or series of banks) breaching may be greater than in the case of no banks.

It is recommended that freeboard be maintained at 0.4 m or above. Thus if an absorption bank has a turn-up of 0.5 m, the total bank height (at its lowest point) should be 0.9 m after settling. It is common for a bank to settle by 25 per cent following construction. Therefore, a bank which should be at least 0.9 m after settling will need to be at least 1.2 m immediately after construction.

Where maintaining freeboard is thought to be a problem (e.g. unstable soils, likelihood of stock crossings forming, a likelihood of poor maintenance), the banks should be shorter in length and/or contain channel stops which will prevent all the water leaving through any breaches. Level banks are preferable to absorption banks in such cases as less water is stored above ground in the former.
9. SEEPAGE INTERCEPTOR DRAINS

9.1 Types and functions

Seepage interceptors are (usually) open drains which intercept shallow seepage flows in duplex (sand over clay) soils, thereby controlling waterlogging on their down-slope side. Being open drains on a grade across the slope, interceptors also collect surface waters helping prevent soil erosion and ensuring contour working. By draining the soil profile, interceptors enable more rainfall to infiltrate the soil. The drainage also allows deeper rooting by crops and pastures and decreases the time that water is ponded on the clay sub-soil, both of which may decrease the amount of recharge to saline groundwaters. In some cases seepage interceptors have been used to protect lower lying areas from discharges from saline hillside seepages.

The basic requirement for interception of shallow seepage waters is for the channel of the drain to be within the clay sub-soil. It is possible to build seepage interceptors as grade banks are built, by ensuring that the channel penetrates the clay sub-soil by at least 15 centimetres. However it is common for the up-slope batter of the drain to become severely rilled due to the following:

(i) There is a steep drop into the channel so that surface waters erode the batter face.
(ii) The batter face is frequently saturated and rainfall on this area will immediately run off.
(iii) There is an outward pressure in the seepage discharge area and it is common to find the rilling of the up-slope batter to be due to sapping and the collapse of short tunnels. This form of rilling will continue even when a vegetative cover is present on the batter face.

To overcome some of these rilling problems, reverse bank seepage interceptors have been developed (Negus 1983a, b). In these drains, surface storm waters are carried on a grassed strip up-slope of the bank while seepage flows are carried in a channel on the down-slope side of the bank. By excluding storm flows from the drain channel it is possible to increase the grade of seepage interceptors from about 0.4 per cent to 0.8 or even 1.0 per cent. The higher grade ensures the flow rate in the drainage channel is fast enough to carry any silt that may enter the drain from the channel batters and decreases the need for maintenance. The procedures for surveying and constructing reverse bank seepage interceptors are outlined in Farmnote 20/83. It has been found that the ends of the drains need not be turned downhill to discharge the water at waterways as is outlined in the Farmnote, if a small amount of ponding in the channel is acceptable. When the depth to the clay sub-soil is greater than 50 centimetres, interceptors have to be constructed with a bulldozer which greatly increases their cost. Fortunately shallow sub-soils are common in waterlogging-prone areas and construction using a road grader is often feasible.

The effect of seepage interceptors on down-slope waterlogging and crop yields has been found to be very variable (Negus, 1983c; Cox and Negus, 1985). An average drained distance in waterlogging years in the Upper Great Southern is 50 metres. It is recommended that interceptors be initially installed on a wider spacing (e.g. 120 metres) to assess their effectiveness at a particular site before deciding on a final spacing.

While it is recommended that reverse bank seepage interceptors be used for waterlogging/soil erosion control in areas with shallow duplex soils, some success has been achieved with interceptors with the spoil on the downslope side of the channel. These have been called conventional interceptors (McFarlane 1984a, b). The problems of batter rilling and channel scouring with conventional interceptors means this form is not recommended for most situations although they may be more practical to build in steeply sloping country. Buried tube drains have also been used to intercept seepage flows but are about ten times more expensive than open interceptors and do not have the advantages of surface water control and of ensuring contour working.
9.2 When to use seepage interceptor drains instead of grade banks

Seepage interceptors are used when shallow seepage in duplex soils is a problem. The most usual case is waterlogging of areas used for cereal cropping. As the interceptors only cost an additional $50 to $80 per kilometre to construct (in comparison with grade banks), if they help a waterlogging problem during a cropping year at least once every ten years, they are probably cost-effective (see Section 3.2). The main disadvantages of interceptors, apart from their slightly higher cost, are reduced vehicular access on paddocks (as they are difficult to cross) and the need for more frequent maintenance to ensure the seepage channels are free of obstructions.

Seepage interceptors should be used above saline areas to prevent waterlogging of crops, pastures and halophytes, all of which fail rapidly when waterlogging and salinity occur together (Barrett-Lennard, 1986; McFarlane, 1985). On discrete hillside seeps, interceptors should be used below the seep to pick up the saline discharge and carry it off to a safe disposal site (e.g. a well defined creek channel or an already saline waterway) if this is possible (Nulsen, 1985). This interception of saline discharge will protect lower lying areas if the interceptor is correctly sited and cuts off all seepage from the hillside seep. Seepage interceptors can also be used to divert fresh seepage waters into dams. Experimental data have shown the volume of seepage flows in duplex soils far exceeds surface flows and therefore interceptors will not need to be as long as grade banks to produce the same flow volume to dams.

9.3 Conversion of existing grade banks to seepage interceptor drains

It is not uncommon to see waterlogging occurring on a hillslope irrespective of the presence of grade banks, as grade banks only intercept seepage waters in very shallow duplex soils. If waterlogging is recognised as a problem in an area with grade banks, the following options are available:

(i) Fill in the grade banks and construct reverse bank seepage interceptors on a higher grade. This option is the most expensive, especially if the grade banks have been incorporated into a farm plan. However, if the grade banks have been allowed to deteriorate through lack of maintenance it may be the best solution.

(ii) Convert the grade banks into reverse bank seepage interceptors by cutting a channel into the clay on the downslope side of the bank. This option has been tried with some success in an area with duplex soils alternating with deeper, sandier sections. The converted banks had about half the drainage effect of specially constructed reverse interceptors, perhaps due to loss of channel water in the sandy areas. The lower grade of converted grade banks (0.4 versus about 0.8 per cent) will result in more leakage or bridging across the drain where sub-soils are permeable.

(iii) Convert the grade banks into conventional seepage interceptors by cutting the existing channel into the clay. The resulting interceptor will have the same grade as the grade bank, while batter rilling may take place, and serious scouring of the channel should not occur.

Which of the above options is exercised depends upon the site (e.g. likelihood of bridging in permeable sub-soils) and the farmers’ attitude.

9.4 Maintenance requirements of seepage interceptor drains

As any blockage in the channel of seepage interceptors is likely to result in ponding behind the blockage and bridging across the drain, regular maintenance is required. Some hard setting soils are very stable and will require little maintenance whereas areas with sandy top-soils will probably require channel maintenance every time the paddock is cropped. Generally maintenance only requires one to three cleaning runs with a grader or a blade/disc
on a three point linkage and is therefore not a major cost, considering the likely benefits of waterlogging control.

The maintenance of the height of the bank to contain storm flows on the top side is very important, as the entry of storm flows into the drainage channel will result in scouring. Surface water flow rates will be greater in seepage interceptors (and therefore shallower) than in grade banks, due to their higher grade. This rate will be decreased somewhat by the flow retardance of the grassed strip. The channel of grade banks increases their freeboard above that of seepage interceptors. Given these considerations, the bank height of seepage interceptors should be at least 0.5 m above the natural ground surface in areas with low vegetative retardance (i.e. grass pastures) and at least 0.6 m in areas with high vegetative retardance (e.g. cape weed pastures). If the catchment area above the top interceptor is very large it is best to construct it with a bulldozer, thereby ensuring the bank is at least one metre high.

10. WATERSPREADING STRUCTURES

10.1 Spreader banks

Spreader banks are used partly to check erosive run-off, and partly to redistribute surface run-off, and partly to redistribute surface run-off so that moisture available for plant growth is increased. Their use should be restricted to permanent pastures on gentle slopes (below 5%). Their use below extensive areas of untreated catchment should, be approached with caution. In such situations, a line of absorption banks may be required above the spreaders to steady the impact of large glows.

There are three types of spreader banks: gap, gap absorption and diversion spreaders. While each performs the same basic waterspreading role, they have different ancillary functions.

10.1.1 Gap spreader banks

Gap spreader banks intercept run-off and redistribute it over contour sills (Quilty, 1972a). They are level along their entire length, so have no water storage role but simply perform a steadying and spreading function. They are most commonly used where large volumes of water are to be controlled, and any attempt to store as well as spread this water would engender a high risk bank failure.

Gap spreader banks are surveyed on a contour, pegged at 20 metre intervals, but allowing for a gap in the bank very 100 to 150 metres. The gaps would be 3 to 5 metres wide, with each gap surveyed accurately at both sides.

Gap spreader banks are built from below using a road grader or bulldozer. They should have a settled height of at least 0.4 m above ground level and a base width of about 2 m.

The surveyed line represents the downhill edge or sill of the channel. Soil is pushed uphill from this line, so forming a bank with a channel on its lower side. This channel has a finished width of 2 m to 3 m, and a depth of 0.15 m to 0.2 m.

The channel edge should be sharp and the adjoining ground surface smooth and clear of surplus soil or wheel indentations. This requires back-blading of the sill as the final operation when a bulldozer is used to build the bank. In any subsequent paddock cultivation, a 1 m strip should be left undisturbed below the sill to provide a stable spilling area.

The channel also should be clean and smooth, which may require a final channel-cleaning run, especially for bulldozer-built structures.

At each location where a gap has been surveyed in the bank, the bank building operation is reversed to push soil downslope and form a check bank opposite the gap below the spreader.
channel. This check bank should extend beyond the gap to overlap the ends of the spreader bank on either side. This overlap may be achieved by pushing additional spoil from below the check bank.

At both ends of a spreader bank line, a crescent-shaped check bank is built, overlapping the spreader channel at one end and the topside of the spreader bank at the other.

If it is necessary to provide a vehicle crossing on a spreader line, this should be located in a gap, enlarging the gap check bank to a low, broad-based structure. On no account should vehicles be driven over the channel sill, as their wheel tracks are low points where water sill subsequently discharge preferentially.

Gap spreaders built in the above manner check run-off flowing down a paddock. The run-off then passes through the gap into the spreader channel from which it is discharged on a wide front. Run-off velocities are reduced and any concentrations in rills or minor gullies are dispersed.

10.1.2 Gap absorption spreader banks
Gap absorption spreader banks not only intercept runoff, like gap spreaders, but also store a certain amount before allowing any excess to overflow through gaps into spreader channels.

Surveying and construction are the same as for gap spreader banks, but each section of bank between successive gaps is turned uphill at both ends by up to 0.1 m, in order to pond water. It is recommended that freeboard should be in excess of 0.3 m. Thus if ends are turned up 0.1 m, the total bank height should be at least 0.4 m above ground level after settling.

10.1.3 Diversion spreader banks
Diversion spreader banks are used where it is desired to take water out of an erosion gully or other flow line, and spread it on adjacent slopes. They combine a grade bank with a gap spreader bank (Quilty, 1972b).

A grade bank is surveyed on a grade of up to 0.5% away from the flow source to the desired disposal area. At this point, called the changeover, the grade bank changes to a spreader bank.

From this changeover point the grade is first increased, to encourage diverted flows to move rapidly through the changeover into the spreader channel, before reducing to a low grade (0.1% or less) or a level section, depending on the spreading width desired.

For example, over successive 20 metre intervals from the changeover point the grade may commence at 1.0%, reducing to 0.7%, 0.5%, 0.3%, 0.1% and finally to 0.05%. If space for spreading is limited, on the other hand, the successive grades may be 0.5% to 0.3% to 0.1%. Whatever levels are chosen, they should not be such as to entail sharp changes in the direction of the spreader channel, as major flows will tend to break out of the channel at such points.

In both cases there is the option of continuing on a very low grade or levelling out, leaving gaps at 100 to 150 metre intervals as for gap spreader banks. A 0.1 m to 0.15 m turn-up at the end of the spreader section will prevent diverted flows from overshooting this terminal point.

In constructing the diversion section, the bank is pushed downhill to sit below the surveyed line. For the uphill push or spreader section, this surveyed line marks the spilling edge of the channel (i.e. the sill). There is thus a continuous channel from diversion to spreader sections, above the bank for the former, and below it for the latter. The change in bank construction type occurs at the changeover point, which may be marked during survey with a cross-rip.
10.2 Contour sills

Contour sills are the simplest of waterspreading structures. While they have a limited function spreading runoff in their own right, they also act as a guide for contour cultivation on sloping land, where the erosion potential is not serious enough to justify banks, but achievement of improved water retention and control of minor erosion justifies contour working.

Contour sills are generally surveyed level, but where rills and minor flow lines traverse a paddock the sill may be surveyed uphill to divert water away from such concentrations.

The survey of contour sills commences approximately 2 m vertically below either the highest point in the landscape, or the lowest bank in an upslope region of more intensive works. Further sills downslope are usually surveyed at vertical intervals of 1.5 m to 2.5 m (Quilty, 1972c).

Contour sills may be constructed using a road grader or disc plough. Using a grader, the soil should be moved uphill only to obtain a sharp spilling edge (Marsh, 1975), though using a plough soil may, for ease of construction, be moved to both sides (Quilty 1972c). Two or three runs with the grader or plough cut a channel, while successive runs spread the soil removed during the channel cutting operation.

10.3 Maintenance requirements of waterspreading structures

Maintenance may be required in the early months following construction of waterspreading structures. Low spots can occur in sills in spite of all precautions. Sills should be checked following rain storms and, if necessary, built up in low spots with a few shovel fulls of soil, or one or two sandbags.

On no account should vehicles be driven over the edge of sills, as wheel tracks create low spots.

After heavy storms, silt deposits may build up in channels, reducing their effectiveness. The deposits can be removed easily using a front-end loader, a grader or a tractor or dozer blade.

Particular attention must be given to removing sediment from channels of diversion spreader banks. Rank growth should also not be allowed behind the spreader banks, as major diverted flows will split at the changeover, with some water flowing down the spreader channel and the balance flowing along the topside of the spreader bank. If the passage of the latter is impeded by rank growth, it may overtop and break the bank.

If rilling develops around the ends of spreader banks or in their gaps, it may be controlled by depositing gravel in the trouble spots or, in areas receiving adequate summer rainfall, by sowing or sprigging grass species such as kikuyu or couch, which effectively bind the soil.

If, after excessive rains, extensive rilling develops along the length of a sill, repair may be effected by cultivating up to its edge. Cultivation in this case will level the uneven surface and restore the sill to its original condition.
11. OPEN RELIEF DRAINS

11.1 Introduction

Open drains are used in dryland farming areas of Western Australia to lower groundwater levels, and to reduce flooding by removing surface water.

In order to maximize cost-effectiveness, open drains are usually constructed in the middle of drainage depressions. Therefore they act as relief drains, since they receive water from both sides. Surface runoff is carried either in the drain channels or behind levees along both sides of the channel.

The most useful type of drain will depend on the drainage requirements of the particular site.

11.1.1 Drains to relieve groundwater levels

Drains to relieve groundwater should be constructed as deep as soil and outlet conditions permit, in order to maximize their effectiveness. Therefore, they are usually constructed using an excavator.

11.1.2 Surface drains to reduce flooding and waterlogging

Drains to remove surface floodwater in a designated time period, and reduce waterlogging, may be relatively wide and shallow. Therefore they are usually most economically constructed using a bulldozer or road grader.

11.2 Construction and Operation of Open Relief Drains

11.2.1 Excavator-built drains

In order to avoid sidewall slumping, drains constructed using an excavator must have battered sides above any indurated subsoil, (Figure 11.1). Slumping can be minimised in cohesive clayey subsoils by ensuring that side-batters are flatter than approximately 1.5 vertical to 1.0 horizontal (approximately 56°to the horizontal). Existing drains in subsoil materials similar to those at the proposed drain site should be inspected, and their side-batter gradients measured, before a design side-batter is decided on. If detailed calculation of design batter slopes is required, a consulting engineer may be employed to carry out analyses of tests on undisturbed samples of representative soil in an engineering laboratory (for further information, see USDA 1977).
Figure 11.1. Cross section and area formulae for a drain cut into an indurated subsoil layer.
Because of the danger of excessive slumping when dry, lower gradient batters are generally required in soils of coarser texture than clayey sand, than in finer-textured soils. Slumping is one of the factors which will limit the usefulness of deep open drains in coarse-textured soils.

Potential wind erosion from surrounding paddocks, can fill open drains with drift debris. Buried, perforated-pipe drains may be more effective and cheaper in the long term at sandy-soil sites, provided that iron is not present in the groundwater to form a gel which can block pipes. An oily appearance where groundwater flows out on the soil surface may indicate the likelihood of gel blocking pipes. Specialist advice on blocking of perforated pipe drains should be sought from a Salinity and Hydrology Branch Research Officer.

Drain silting caused by side-wall erosion has proven catastrophic when surface waters have been allowed to enter deep drain channels. Special desilting operations may be required after major floods at considerable cost. It is therefore desirable to keep runoff from the catchment surface outside deep drains. Continuous levees along both sides of the channel keep surface runoff outside, with the added advantage of not increasing the velocity of flow. Therefore flooding downstream is generally not increased. Road crossings, however, must have low floodways on both sides of a deep central culvert in order to discharge both surface and subsurface drain flow.

Surface ponding at low spots outside the levees may be relieved by installing pipes through the levee banks. Care should be taken to ensure that the pipes extend sufficiently far into the drain that the inflow falls directly onto the drain bottom, and not onto its erodible side-batters.

Erosion of the levee bank by raindrop impact and runoff can result in channel silting, particularly in saline areas where vegetative cover is sparse. Shaping of the levee banks as described in Section 11.3.2, to minimise the size of rill catchments flowing towards the drain, can reduce the silting.

Rain-splash erosion also occurs on the bare side-batters of deep drains. The growth of protective vegetation on the subsoil materials of the side-batters is likely to be negligible. Little can be done to reduce rainsplash erosion of side-batters, other than to minimise their exposed area by maximising their slope, consistent with avoiding sidewall slumping. Concentrated surface flow over the side-batters from rill catchments on the levees will cause rilling and should be avoided.

Vertical sides are usually stable without slumping in hard subsoils. Problems in this zone include:

1. Penetration of the hard subsoil can be slow even using the largest excavators available; and
2. It is sometimes desirable not to penetrate through to the pallid zone soils beneath. These can be extremely erodible and may contain groundwater under significant upward pressure. Specialist advice should be obtained from Salinity and Hydrology Branch personnel concerning drainage requirements of such sites.

Hard subsoils and the usual pallid zone beneath therefore pose a practical limit to the depth of open drains constructed using an excavator.

Excavator-built drains may require desilting within a few years of construction, as loose soil material from the side-batters is washed or blown into the drain. Erosion of soil in the saturated flow zone on either side of the drain bottom may also cause silting, particularly as watertables recede soon after construction.

Desilting is usually most economically carried out using an excavator which reaches over the drain levee. Special scoop buckets several metres wide may be used. The silt is then placed outside the levee.
11.2.2 Bulldozer or grader-built drains

Open relief drains to reduce flooding and waterlogging are usually most economically constructed using a bulldozer or road grader, particularly where the required drain depth is less than one metre and the ground is not too boggy. Their purpose is to improve surface drainage. A typical cross-section of a bulldozer-built drain is shown on Figure 11.2.

Cross-sectional areas may be calculated similarly for grader-built drains with adjustments of the various parameters as required, (bottom width, being zero for V-drains, for example).

Spoil is typically moved to one side using a bulldozer or grader. Either a gap is left in the bank or the shuttle direction is reversed if using a bulldozer, placing the spoil on the opposite side for the next section. Surface runoff then has access to the drain from either side of the drainage line. Smaller feeder drains may be constructed (e.g. using a grader or plough) to discharge into the main drain.

Spoil is placed on both sides of grader-built drains for reasons of economy, with gaps subsequently pushed out to allow water to enter.
Figure 11.2. Cross section and area of formulae for a bulldozer built drain.

**LEGEND**
- B  Drain bottom width
- W1  Drain top width
- D  Drain depth
- d  Flow depth
- b1  No. 1 batter slope
- b2  No. 2 batter slope
- b3  No. 3 batter slope
- H  Levee peak height

**FORMULAE**

- Cross-sectional area of Drain = DB + \( \frac{d}{2} \) \( \frac{b_1^2 + b_2^2}{b_1 + b_2} \)
- Cross-sectional area of Flow = db + \( \frac{d}{2} \) \( \frac{b_1^2 + b_2^2}{b_1 + b_2} \)
- Cross-sectional area of Levee = \( \frac{H^2}{2} \) \( \frac{1}{b_2} + \frac{1}{b_3} \)

Top width of Drain, \( W_1 = B + D \left( \frac{1}{b_1} + \frac{1}{b_2} \right) \)
11.3 Design of open relief drains

The design of a drainage system should begin at its downstream end. Drainage water should be dispersed of without detriment to downstream landholders, and with their written agreement. Such documentation is particularly important in cases of change of ownership. Drains at some sites may not be feasible, because of anticipated detrimental effects downstream.

11.3.1 Excavator-build drains

Hydrological design techniques must be applied, in order to calculate the required levee heights. Designs to ensure the effectiveness of drains in removing groundwater are not available, as the hydraulic properties of soils are very variable and difficult to estimate accurately. Each individual drain should be designed to safely discharge peak catchment flows with a particular average recurrence interval, calculated as shown in Section 4.

Levees constructed on both sides of open drains must be high enough to discharge the design flow. A freeboard allowance must be included to cater for any low points caused by excessive settlement.

Drains should be designed large enough to cope with peak storm flows at their downstream ends. Peak flows should be recalculated, and drains re-designed upstream, as the catchment area decreases. A worked example of the design of an excavator-built drain is demonstrated in Section 11.3.3.

The design storm flow and required levee height will increase with the average recurrence interval selected. Since a consequence of a levee overtopping may be the need for immediate desilting, average recurrence intervals may best be calculated on economic criteria. Estimated construction costs may be calculated for different average recurrence intervals, and balanced against estimated maintenance costs following overtopping in each case.

In general, side batter slopes flatter than approximately 1.5 vertical to 1 horizontal (i.e. less than 56 to the horizontal) have been observed to be stable in materials texturing finer than clayey sand, and which do not slake or disperse excessively.

Clayey sand and coarser-textured soils may be so unstable that drain sides collapse when dry unless relatively flat side-batters are sued (approximating the angle of repose of loose, dry soil). The stability of drains cut in similar soils should be observed before design batter slopes are decided on. If required, a measurement of the angle of repose of representative dry soil samples may be carried out in an engineering laboratory, for use in drain design. An estimate can be obtained by pouring thoroughly loosened samples of dry soil through a funnel onto a horizontal surface, and measuring the angle at which the soil comes to rest. Alternatively, buried perforated-pipe drains may be considered for such sites, thereby avoiding both side-batter and possible wind erosion problems.

Drain spoil should be placed in a continuous levee bank on both sides, with the toe a minimum 0.7 m from the top of the drain side batter. A small ledge or berm on the natural surface traps coarser debris washed from levee banks by run-off. Vegetation such as barley grass then usually helps to stabilise it, minimising the movement of spoil back into the drain.

Levee banks should be shaped so that as much run-off and erosion debris as possible flows to the outer sides of the levees. Spoil placed in a continuous levee should be shaped so that surface run-off flows towards the outer waterways, in order to minimise sedimentation of the drain by spoil from the levee. A single run-of a bulldozer along each levee bank, with the blade sloping toward the outer waterway, can conveniently achieve such a shape, taking care that the design height is maintained.
An allowance of at least 30% of bank height should be included for settlement of the uncompacted spoil. Therefore a levee with an initial height of 1 m above the natural surface should be considered to settle to 0.7 m.

Levee heights should always include an additional 0.2 m as freeboard, in order to allow for wave action and surcharge above the design depth of flow. Since any overtopping may have serious effects on siltation in the drain, low points such as at stock tracks should be built up to the design level whenever they are observed, and routinely at least once each year. When the levee has settled below its design height, it should be built up again along its entire length.

The slope of pipes installed through levee banks to discharge water from outside should preferably be 5% or higher, in order to achieve high flow rates relative to the size of pipe used. A 100 mm internal-diameter pipe may then be expected to discharge in excess of 10 litres per second when its inlet is completely under water, and a 150 mm diameter pipe, 35 litres per second. The size of pipe should be selected on the basis of discharging ponded runoff outside the drain in the required time. Beware that use of many pipes along a length of drain, or pipes of large diameter, may cause excessive peak flows in the excavated drain, and undercutting of its side-batters.

11.3.2 Worked example: design of an excavator-built drain

Example: Design an excavator-built open drain with continuous levees to exclude a peak surface flow rate with a 20-year average recurrence interval, of 69.2 m$^3$ s$^{-1}$. The surface soil at the drain site is sandy loam. The average surface slope of the drain site is 0.8 m per kilometre.

The height of the levees required on both sides of the drain to exclude surface runoff is calculated by repeatedly calculating average velocities and flow rates for different depths, until the design flow rate is achieved.

The slope (s) at the drain site is 0.8 m per kilometre. This must be expressed in units of metres per metre as $s = 0.0008$, in order to design the side waterways using Manning’s formula, as shown in Section 6.2.

The Manning’s Roughness Coefficient ‘n’ is read from Table 6.1 for the sandy loam soil at the site, as $n = 0.025$.

The average velocity of flow (v) is calculated, as in Section 6.2.4 as:

$$ v \approx \frac{1}{n} \cdot d_{av.}^{2/3} s^{1/2} $$

Where $d_{av.}$ is the average depth of flow.

Arbitrarily-select an average depth of flow ($d_{av.}$) of 0.2 m in the first instance, say, and calculate the average velocity of flow (v) and flow (Q).

Therefore

$$ v \approx \frac{1}{0.025} \cdot (0.02)^{2/3} \cdot (0.0008)^{1/2} $$

$$ \approx 0.39 \text{ m s}^{-1} $$

which will not erode the bare sandy loam soil, since its maximum permissible velocity is shown on Table 6.2 as 0.75 m s$^{-1}$.

The flow rate on either side of the drain is calculated as the average velocity of flow, multiplied by its cross-sectional area. Cross-sectional area is calculated as the average depth times the width. In order to obtain the width of flow, the maximum depth of flow is required.
Since most valley bottoms are approximately parabolic in cross-section, the maximum flow depth may be taken as the average depth multiplied by \( \frac{3}{2} \) - a characteristic relationship of parabolic cross-sections (see Section 6.2.4).

Therefore the maximum depth of flow is:
\[
0.2 \times \frac{3}{2} = 0.3 \text{ m}
\]

Field survey information is now required on widths of flow at various depths. Measure the widths of flow on either side of the proposed drain levees, as the average distances across the valley from each levee which would be inundated by flow 0.3 m deep at the levee. Suppose these distances are surveyed in the field, and measured as approximately 100 m on each side.

The cross-sectional area of flow on each side is now calculated as the maximum width of flow multiplied by the average depth of flow. For average widths of flow of 100 m, the cross-sectional area of flow is calculated as:
\[
100 \text{ m} \times 0.2 \text{ m} = 20 \text{ m}^2 \text{ on each side}
\]

The flow (Q) may now be calculated as the average velocity of flow multiplied by its cross-sectional area,
\[
Q = 0.39 \times 20 \text{ m}^3 \text{ s}^{-1} = 7.8 \text{ m}^3 \text{ s}^{-1}
\]
on each side of the drain. Therefore the total flow on both sides will be
\[
7.8 \text{ m}^3 \text{ s}^{-1} \times 2 = 15.6 \text{ m}^3 \text{ s}^{-1}
\]

However the design flow rate is 69.2 m³ s⁻¹. Therefore the design flow will be wider and deeper than the 15.6 m³ s⁻¹ flow in these calculations. In order to determine how much wider and deeper, try a deeper average flow than the 0.2 m calculated above.

Try an average depth of flow of 0.3 m.

Therefore
\[
\frac{1}{0.025} (0.03)^{2/3} (0.0008)^{1/2} \approx 0.51 \text{ m} \text{s}^{-1}
\]

which still should not erode, because the maximum permissible velocity is 0.75 m s⁻¹.

At 0.3 m average depth of flow, the maximum depth of flow is calculated as above, at:
\[
0.3 \text{ m} \times \frac{3}{2} = 0.45 \text{ m}
\]

Measure how wide a flow of 0.45 m maximum depth will be in the field. Suppose it is measured as approximately 150 m on each side of the drain levees.

The cross-sectional area of flow on each side is now:
\[
150 \text{ m} \times 0.3 \text{ m} = 45 \text{ m}^2
\]

and its peak flow rate:
\[
0.51 \text{ m} \text{s}^{-1} \times 45 \text{ m}^2 = 22.95 \text{ m}^3 \text{ s}^{-1}
\]
The total flow rate on both sides is now:
22.95 m$^3$/s x 2 = 45.9 m$^3$/s

Since the design flow is 69.2 m$^3$/s, the waterways will be deeper and wider than that calculated above.

Try a deeper average depth of flow of 0.35 m.

Its average velocity is therefore:

\[
\text{Therefore } v \approx \frac{1}{0.025} (0.35)^{2/3} (0.0008)^{1/2} \\
\approx 0.56 \text{ m s}^{-1}
\]

which should still not erode, because the maximum permissible velocity is 0.75 m s$^{-1}$.

The maximum depth of flow is now:

0.35 m x \( \frac{3}{2} \) = 0.53 m

Measure the average distances from each levee that would be inundated by flow of 0.53 m maximum depth. Suppose these distances are each measured as approximately 180 m.

The cross-sectional area of flow on each side is now

180 m x 0.35 m = 63.0 m$^2$

and its peak flow rate:

0.56 m s$^{-1}$ x 63 m$^2$ = 35.28 m$^3$/s

The total flow rate on both sides is now:

35.28 m$^3$/s x 2 = 70.56 m$^3$/s

which is approximately the design flow of 69.2 m$^3$/s.

Therefore, the levees on each side of the drain must achieve a **settled** height of the maximum depth of flow of 0.53 m plus the freeboard allowance of 0.2 m or:

0.53 m + 0.2 m = 0.73 m

Because settlement must also be allowed for, since an uncompacted levee, built 1.00 m high should be considered to settle to 0.7 m, the height of the uncompacted levee before allowing for freeboard must be:

0.73 m x \( \frac{1.0}{0.7} \) = 1.04 m

the cross-sectional area of each levee ($A_L$) may be calculated using the relevant formula shown on Figure 11.1. For a minimum one metre wide top width ($T_L$) and minimum design side-slopes ($b_L$) of 2 vertical to 3 horizontal (decided on following inspection of drains in similar soils) and settled height ($H$) of 0.73 m:

\[
A_L = H \left( T_L + \frac{H}{b_L} \right) = 0.73 \left( 1.0 + \frac{0.73}{0.67} \right) \\
= 1.53 \text{ m}^2
\]
The cross-sectional area of both levees will be:

\[ 2 \times 1.53 \, \text{m} = 3.06 \, \text{m}^2 \]

An equivalent cross-sectional area should be cut from the drain, in order to provide sufficient spoil for constructing both levees without the additional need to build them up from the outside.

From Figure 11.1, the cross-sectional area of the drain excavation \((A_D)\) for drain side-batter slopes \((b_D)\) of 1.5 and an indurated layer \((d_i)\) at 1.8 m, is:

\[
A_D = DB + \frac{d_i^2}{b_D} = DB + \frac{(1.8)^2}{1.5}
\]

\[ = DB + 2.16 \]

the value of the product of drain depth \((D)\) and bottom width \((B)\) may now be found by equating the cross-sectional areas of cut and fill (for drains as deep or deeper than the indurated layer), as:

\[ DB + 2.16 = 3.06 \, \text{m}^2 \]

Therefore: \[ DB = 3.06 - 2.16 = 0.90 \, \text{m}^2 \]

If the excavator cannot penetrate the indurated layer, the drain depth \((D)\) is then the depth of the indurated layer \((d_i)\). Substituting \(d_i = 1.8 \, \text{m}\):

\[ 1.8 \, B = 0.90 \, \text{m}^2 \]

Therefore: \[ B = \frac{0.90 \, \text{m}}{1.80} = 0.50 \, \text{m} \]

Therefore sufficient fill material would be available to construct adequate levee banks with a drain as deep as the indurated layer at 1.8 m, and with a bottom width of 0.50 m.

Other combinations of drain depth and bottom width may be considered, provided that their product is greater than 0.90 m². Alternatively, different side-batter slopes may be inserted and the product \(DB\) recalculated. The product may also be calculated for drains cut into the indurated layer, as shown on Figure 11.1. Appropriate combinations of depth and bottom width should then be selected for construction.

Estimated desilting costs may warrant a selection of shorter or longer average recurrence interval than 20 years. Re-design of the drain for a different average recurrence interval then enables a direct comparison of total costs to be made, in order to arrive at an optimum economic design.

11.3.3 Bulldozer or Grader-built Drains

Bulldozer or grader-built drains should be designed to discharge floodwaters with the design average recurrence interval in a certain time in order to reduce the time that land is flooded. Flow in the drain is calculated using cross-sectional areas of flow and average velocities. The size of drain required is calculated from the volume of floodwater to be discharged in the required time period.

The maximum volumes of runoff to be discharged in 1 or 2 day periods may be obtained from Section 5 (Estimation of Runoff Volumes). It is generally considered that surface water should not be allowed to remain on cropland for longer than 2 days during the growing season.
11.3.4 Worked example: Design of a Bulldozer or Grader-built Drain

Example: Calculate the design top and bottom widths of a bulldozer-built drain 0.5 m deep with 1:5 side-batters to remove the two-day surface runoff with a 10-year average recurrence interval from the valley floor of a 1500 ha catchment. The drain site has a clay-loam soil at 0.5 m depth, and a slope of 0.2%.

The 2-day, 10-year runoff depth from Table 5.2 is 14.0 mm. Its volume is therefore:

\[ 14.0 \text{ mm} \times 1,500 \text{ ha} \times \frac{10,000 \text{ m}^2 \text{ ha}^{-1}}{1,000 \text{ mm m}^{-1}} = 210,000 \text{ m}^3 \]

This surface runoff must be removed in 2 days, or 2 \times 24 \times 60 \times 60 = 172,800 seconds. Therefore the drain must be capable of discharging a flow of:

\[ \frac{210,000 \text{ m}^2}{172,800 \text{ s}} = 1.22 \text{ m}^3\text{s}^{-1} \]

The width of a bulldozer-built drain to discharge the design flow is then calculated as described in Section 6.2.4.

Since the site has a clay-loam soil at the drain bottom, where high erosion forces occur, its Manning’s Roughness Coefficient ‘n’ is read from Table 6.1, as:

\[ n = 0.03 \]

The design flow depth (d) equals the drain depth.

The slope of the drainage line is 0.2%, which must be expressed in units of metres per metre as \( s = 0.002 \text{ m m}^{-1} \).

Since the average width of the drain is likely to exceed the 0.5 m depth by more than a factor of 5, the average velocity of flow may be calculated using the approximation to Manning’s formula (see Section 6.2.4):

\[ v \approx \frac{1}{n} \frac{d_{av.}}{s^{1/2}}^{2/3} \]

\[ \approx \frac{1}{0.03} (0.5)^{2/3} (0.002)^{1/2} \text{ m s}^{-1} \]

\[ \approx 0.94 \text{ m s}^{-1} \]

Check whether the drain will erode and deepen during the design flow. From Table 6.2, the maximum permissible velocity for a clay-loam soil with a negligible grass cover is 1.0 m s\(^{-1}\). Therefore the design flow which has a velocity of 0.94 m s\(^{-1}\), should discharge safely without scouring.

A drain discharging the flow will therefore require the product of cross-sectional area of flow, times the average velocity of flow, to be 1.22 m\(^3\) s\(^{-1}\). The average velocity has been calculated above as: Therefore:

Cross-sectional area of flow \[ = \frac{1.22}{0.94} \text{ m}^2 = 1.29 \text{ m}^2 \]

Since the design is for the drain flowing full, the formula for the cross-sectional flow is read from Figure 11.2 as:
Cross-sectional Area = DB + \( \frac{1}{2} \) D\(^2\) \( \frac{1}{b_1} + \frac{1}{b_2} \)

The flow depth (d) is equal to the design depth of the drain (0.5 m). The side-batter slopes are both 1:5 (i.e. \( b_1 = b_2 = \frac{1}{5} \) or \( \frac{1}{6_1} = \frac{1}{6_2} = 5 \)). Therefore substituting in the above formula.

\[
1.29 = 0.5 B + \frac{1}{2} (0.5)^2 (\frac{1}{5} + \frac{1}{5})
\]

\[
= 0.5 B + 0.05
\]

Therefore \( 0.5 B = 1.29 - 0.05 = 1.24 \)

\[
B = \frac{1.24}{0.5} = 2.48 \text{ m}
\]

The top width of the drain (which will be required for construction purposes) may be calculated as shown in Figure 11.2 as:

\[
W_t = B + D \left( \frac{1}{b_1} + \frac{1}{b_2} \right)
\]

Substituting for the bottom width (B) of 2.48 m, side-batter slopes of 1:5 (i.e. \( b_1 = b_2 = \frac{1}{5} \) or \( \frac{1}{6_1} = \frac{1}{6_2} = 5 \)) and drain depth (D) of 0.5 m in the formula.

\[
W_t = 2.48 + (0.5) (5 + 5)
\]

\[
= 2.48 + 5
\]

\[
= 7.48 \text{ m}
\]

Therefore the drain top and bottom widths required are 7.48 m and 2.48 m, respectively.
12. REFERENCES


