Design and operation of irrigation systems for smallholder agriculture in South Asia
Design and operation of irrigation systems for smallholder agriculture in South Asia

by

D.E. Campbell
The FAO Investment Centre's principal function is to assist member countries in the preparation of agricultural and rural development projects for international, and sometimes local, financing. The Investment Centre is also acknowledged as a source of assistance in the development of national capacities for project preparation and execution.

Over the last 15 years, irrigation has occupied a very important position in international lending for development and has made a considerable contribution to increased agricultural production. There are, nevertheless, a number of problems in smallholder irrigation which continue to limit the rate of effective implementation of such projects and occupy much of the attention of development agencies in this field. Some of the problems are technical, but more often they relate to the small cultivator and his role in the operation of the project and the development of the irrigated area.

This FAO Investment Centre Technical Paper comprises 5 papers selected from more than 30 prepared between 1974 and 1985 for the assistance of Irrigation Department staff, mainly engineers, preparing and executing projects in India assisted by the World Bank. Although these papers were written for projects in India, the principles involved apply in neighbouring countries and further afield. However, when transposing costs to another region, account should be taken of the comparatively low cost of construction in India.

Since I believe that the papers could be of considerable use to people outside the Investment Centre who share our interest in the development of irrigation projects, I have decided to give them wider circulation by publishing them as part of our series of Technical Papers. The opinions expressed, however, remain those of the author and are not necessarily endorsed by the Organization or the Bank.

Any comments on the material or suggestions which could contribute to the greater usefulness of the paper would be most welcome and should be addressed to the Investment Centre.

Cedric Fernando
Director
FAO Investment Centre
# Table of Contents

**Volume I**

A. **Issues and Options in Design of Reservoir and Canal Systems**  
   Page 1 - 30

B. **Water Distribution from Minor Canal to the Field, and Land Shaping in Irrigation of Smallholdings**  
   Page 31 - 71

C. **Irrigation from Small Tanks**  
   Page 73 - 120
   Plates 1 to 9  
   Page 121 - 137

**Volume II**

D. **Irrigation from Supply Canals via Buried Pipe Distribution Systems**  
   Page 1 - 105
   Plates 01 to 08  
   Page 37 - 51

E. **Tubewell and River-Lift Irrigation**  
   Page 107 - 131
   Plates E1 to E6  
   Page 133 - 143
D. IRRIGATION FROM SUPPLY CANALS VIA BURIED PIPE DISTRIBUTION SYSTEMS

TABLE OF CONTENTS

<table>
<thead>
<tr>
<th>Section</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>D.1 THE APPLICATION OF BURIED PIPE TO GRAVITY SUPPLY SYSTEM</td>
<td></td>
</tr>
<tr>
<td>- BACKGROUND</td>
<td>3</td>
</tr>
<tr>
<td>Introduction</td>
<td>3</td>
</tr>
<tr>
<td>The Problems of Distribution by Open Channel</td>
<td>3</td>
</tr>
<tr>
<td>Where Holdings are Small</td>
<td>3</td>
</tr>
<tr>
<td>Types of Pipe Distribution System</td>
<td>4</td>
</tr>
<tr>
<td>Delivery at Individual Outlets Within a Distribution System</td>
<td>4</td>
</tr>
<tr>
<td>Cases to Be Considered</td>
<td>5</td>
</tr>
<tr>
<td>D.2 THE KALLADA PROJECT PIPE SYSTEM - DESIGN CONCEPTS AND OPTIONS</td>
<td>6</td>
</tr>
<tr>
<td>The Project Area</td>
<td>6</td>
</tr>
<tr>
<td>The Special Requirements of Farm Water Distribution</td>
<td>6</td>
</tr>
<tr>
<td>in the Upstream Portion of the Kallada Area</td>
<td>6</td>
</tr>
<tr>
<td>Choice Between Open Channel and Pipe Distribution for Kallada</td>
<td>7</td>
</tr>
<tr>
<td>Regulation of Flow at Delivery Valves</td>
<td>8</td>
</tr>
<tr>
<td>Selection of Pipe System</td>
<td>10</td>
</tr>
<tr>
<td>Operation of the Kallada System - The Role of the Cultivator</td>
<td>12</td>
</tr>
<tr>
<td>D.3 KALLADA PROJECT - GUIDELINES FOR LAYOUT AND DESIGN UPSTREAM (VALLEY-SLOPE) PORTION OF SERVICE AREA</td>
<td>13</td>
</tr>
<tr>
<td>Introduction</td>
<td>13</td>
</tr>
<tr>
<td>Rate of Supply per Hectare of Service Area (Irrigation Duty) at the Outlet from the Canal</td>
<td>13</td>
</tr>
<tr>
<td>Size and Location of Service Areas (Outlet Commands) and Location of Outlets</td>
<td>14</td>
</tr>
<tr>
<td>Required Capacity of Pipe System, Upper Portion of the Kallada Command</td>
<td>15</td>
</tr>
<tr>
<td>Design of Outlet Structure from Distributaries and Main Canal</td>
<td>16</td>
</tr>
<tr>
<td>Location of Delivery Points (Hydrants) and Line Layout</td>
<td>18</td>
</tr>
<tr>
<td>Determination of Pipe Size - Hydraulics of System</td>
<td>22</td>
</tr>
<tr>
<td>Control of Water-Hammer - Location of Surge Risers - Rate of Valve Closure</td>
<td>23</td>
</tr>
<tr>
<td>Operation of System</td>
<td>26</td>
</tr>
<tr>
<td>D.4</td>
<td>KALLADA PROJECT - DOWNSTREAM (NEAR-FLAT) PORTION OF SERVICE AREA</td>
</tr>
<tr>
<td>-----</td>
<td>---------------------------------------------------------------</td>
</tr>
<tr>
<td></td>
<td>Introduction</td>
</tr>
<tr>
<td></td>
<td>Irrigation Requirements - Separation of Supply</td>
</tr>
<tr>
<td></td>
<td>to Garden Lands and to Paddy Areas</td>
</tr>
<tr>
<td></td>
<td>Rate of Delivery to the Field and Size of Outlet Command</td>
</tr>
<tr>
<td></td>
<td>Alternative Distribution Systems - Open Channel</td>
</tr>
<tr>
<td></td>
<td>and Buried Pipe</td>
</tr>
<tr>
<td></td>
<td>Pipe Materials</td>
</tr>
<tr>
<td></td>
<td>Pipe System Layout</td>
</tr>
<tr>
<td></td>
<td>Points of Delivery from Pipe System -</td>
</tr>
<tr>
<td></td>
<td>Location and Area Served</td>
</tr>
<tr>
<td></td>
<td>Regulation of Flow to Pipe System</td>
</tr>
<tr>
<td></td>
<td>Structures - Canal Outlet and Delivery Points</td>
</tr>
<tr>
<td></td>
<td>Hydraulic Design</td>
</tr>
<tr>
<td></td>
<td>Preliminary Cost Estimates</td>
</tr>
<tr>
<td></td>
<td>Comparison of Buried Pipe and Lined Open Channel Systems for</td>
</tr>
<tr>
<td></td>
<td>Lower Kallada</td>
</tr>
<tr>
<td></td>
<td>Conclusions</td>
</tr>
</tbody>
</table>

**PLATES** D1 to D8 37 - 51

**ANNEXES**

<table>
<thead>
<tr>
<th>D1</th>
<th>Kallada Project - Soils, Cropping Patterns, and Water Requirements</th>
<th>53</th>
</tr>
</thead>
<tbody>
<tr>
<td>D2</td>
<td>Kallada Project - Supply of Water to Valley Bottom Lands in Upper Portion of Command Area</td>
<td>61</td>
</tr>
<tr>
<td>D3</td>
<td>Kallada Project - Head Losses in Pipe Distribution Systems</td>
<td>65</td>
</tr>
<tr>
<td>D4</td>
<td>Kallada Project - Water-Hammer in Buried Pipe</td>
<td>73</td>
</tr>
</tbody>
</table>
## Table of Contents

<table>
<thead>
<tr>
<th>Section</th>
<th>Title</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>E.1</td>
<td>BACKGROUND AND SCOPE</td>
<td>109</td>
</tr>
<tr>
<td>E.2</td>
<td>THE RANGE OF WELL CAPACITY AND TECHNOLOGY AVAILABLE IN GROUNDWATER DEVELOPMENT</td>
<td>109</td>
</tr>
<tr>
<td></td>
<td>Open Wells</td>
<td>109</td>
</tr>
<tr>
<td></td>
<td>Hand-Operated Tubewells</td>
<td>110</td>
</tr>
<tr>
<td></td>
<td>Small Tubewells with Mechanically-Driven Pump at Well-Head</td>
<td>110</td>
</tr>
<tr>
<td></td>
<td>Small Tubewells and Submerged Pumps</td>
<td>111</td>
</tr>
<tr>
<td></td>
<td>Deep Tubewells</td>
<td>111</td>
</tr>
<tr>
<td>E.3</td>
<td>RELATIVE COSTS OF TUBEWELL SYSTEMS</td>
<td>112</td>
</tr>
<tr>
<td>E.4</td>
<td>TECHNICAL AND SOCIAL FACTORS IN PLANNING OF GROUNDWATER DEVELOPMENT</td>
<td>112</td>
</tr>
<tr>
<td></td>
<td>Utilization of Aquifer Potential</td>
<td>112</td>
</tr>
<tr>
<td></td>
<td>Physical Characteristics of the Aquifer</td>
<td>113</td>
</tr>
<tr>
<td></td>
<td>Groundwater Development in an Area of Canal Irrigation</td>
<td>114</td>
</tr>
<tr>
<td></td>
<td>Power Supply for Tubewells</td>
<td>115</td>
</tr>
<tr>
<td></td>
<td>Size of Holding</td>
<td>116</td>
</tr>
<tr>
<td></td>
<td>Equity in Distribution of Groundwater</td>
<td>116</td>
</tr>
<tr>
<td></td>
<td>Existing Wells and Possible Conflict in Further Development of an Aquifer</td>
<td>117</td>
</tr>
<tr>
<td>E.5</td>
<td>PRIVATE VERSUS PUBLIC DEVELOPMENT, A SUMMARY</td>
<td>117</td>
</tr>
<tr>
<td>E.6</td>
<td>ECONOMIC AND FINANCIAL FACTORS</td>
<td>118</td>
</tr>
<tr>
<td>E.7</td>
<td>DESIGN OF TUBEWELL DISTRIBUTION SYSTEMS</td>
<td>120</td>
</tr>
<tr>
<td></td>
<td>Introduction</td>
<td>120</td>
</tr>
<tr>
<td></td>
<td>Desirable Size of Delivery Stream (to the Individual Cultivator)</td>
<td>121</td>
</tr>
<tr>
<td></td>
<td>Choice of Construction Material for Distribution System</td>
<td>121</td>
</tr>
<tr>
<td></td>
<td>Size of Service Area of Tubewell</td>
<td>122</td>
</tr>
<tr>
<td></td>
<td>Area Supplied by an Individual Outlet</td>
<td>123</td>
</tr>
<tr>
<td></td>
<td>Division of Tubewell Supply between Outlets</td>
<td>123</td>
</tr>
<tr>
<td></td>
<td>Layout of Pipe System</td>
<td>124</td>
</tr>
<tr>
<td></td>
<td>Regulation of Tubewell Output</td>
<td>124</td>
</tr>
<tr>
<td>E.8</td>
<td>OPERATION OF THE TUBEWELL SYSTEM AND THE ROLE OF CULTIVATOR GROUPS</td>
<td>125</td>
</tr>
<tr>
<td>E.9</td>
<td>AGRICULTURAL SUPPORTING SERVICES IN TUBEWELL AREAS</td>
<td>126</td>
</tr>
<tr>
<td>Topic</td>
<td>Page</td>
<td></td>
</tr>
<tr>
<td>------------------------------------------------------------</td>
<td>------</td>
<td></td>
</tr>
<tr>
<td>E.10 RIVER-LIFT SYSTEMS</td>
<td>127</td>
<td></td>
</tr>
<tr>
<td>Design and Operation</td>
<td>127</td>
<td></td>
</tr>
<tr>
<td>Private versus Public Development of River-Lift System</td>
<td>128</td>
<td></td>
</tr>
<tr>
<td><strong>TABLE E1</strong></td>
<td>131</td>
<td></td>
</tr>
<tr>
<td><strong>PLATES E1 to E6</strong></td>
<td>133 - 143</td>
<td></td>
</tr>
</tbody>
</table>
IRRIGATION FROM SUPPLY CANALS VIA BURIED PIPE DISTRIBUTION SYSTEMS
D. IRRIGATION FROM SUPPLY CANALS VIA BURIED PIPE DISTRIBUTION SYSTEMS

D.1 THE APPLICATION OF BURIED PIPE TO GRAVITY SUPPLY SYSTEM - BACKGROUND

Introduction

D.1.1 Distribution of water from the supply canal to the individual farm remains the most troublesome feature of irrigation development in the Asian area of smallholder cultivation. This is particularly the case where crops other than paddy are being grown in at least one season of the year, when the simple field-to-field approach which is appropriate only to wet-land paddy is not applicable. The latter is the situation considered in the following notes.

D.1.2 The use of buried pipes as a means of overcoming much of the difficulty was introduced in Asia for distribution from tubewells and river-lift installations. Among the advantages of pipes compared with open channels is freedom from right-of-way problems, particularly in areas of smallholdings, and the assurance of delivery at the design rate of flow to the farthest cultivator in the service area, undiminished by losses or unauthorized diversions en route.

D.1.3 With a pumped source of water the use of pipe is facilitated, as the pump can be designed to provide the additional head necessary to accommodate the hydraulic friction in the pipe system. In the case of supply from canals, however, pumped head is not available and a pipe system must be designed to operate within the limits of the gravitational head available between the water surface in the supply canal and the elevation of the point of delivery. The advantages of pipe distribution are nevertheless as real for canal supplied systems as for pumped systems, and there are many situations in which there is more than adequate gravity head to make the use of pipe possible.

D.1.4 An obvious example is a supply canal running along the side of a valley and serving an irrigation area on the slopes below it. In other special circumstances, although the topography may be near-flat, the advantages of buried-pipe distribution may be sufficiently compelling to warrant special measures to permit it, such as keeping the level in the supply canal higher than usual above ground level (up to 1 m) and designing the pipe system to operate on that very low head.

The Problems of Distribution by Open Channel Where Holdings are Small

D.1.5 The advantages of pipe distribution over open channels have been referred to briefly above. Some are especially relevant to a typical Asian situation in which there are smallholdings, often in several small parcels. Property boundaries follow no regular geometric pattern, and consolidation of holdings which would facilitate rational layout of surface channels is resisted by cultivators. Cultivators are also unwilling to give up land for channel construction and land values may be high (Rs200,000/ha is not uncommon reaching as high as Rs500,000/ha in coastal Kerala). Furthermore, poor maintenance aggravated by intentional breaching of channels by upstream cultivators makes delivery to downstream cultivators precarious. To this list of conventional problems with open channel distribution can be added special situations including distribution in semi-urban areas where
interference to property access is a major disincentive to open channels, and the similar situation in the Kallada project to be described later where the presence of mature orchard trees is a further complication.

Types of Pipe Distribution System

D.1.6 For the purpose of these notes, consideration is confined to relatively small pipe systems of capacity not more than 2 cfs (56 litres/sec), generally less. These are basically distribution systems, with the associated main canal network providing conveyance down to the point where distribution to farms begins. Pipe, in considerably larger size, may also be used for conveyance in some situations in place of an open canal network, but such systems involve substantially different technology and are not considered herein.

D.1.7 The simplest form of pipe distribution system is sometimes described as an "underground canal", and it functions in much the same manner as a canal. Runs of pipe connect a series of open delivery boxes spaced along the length of the line, from which water flows through gated turnouts to farm channels, usually from one box at a time. The delivery boxes function as drop structures when the line runs significantly down-slope, and the pipe between successive boxes is under virtually no pressure. As with the corresponding open channel system, consisting of runs of open channel between turnouts (and drops if required), this is an "upstream control" system. If all turnouts are closed, the outlet gate to the line from the supply canal must also be closed or water will spill at one of the delivery boxes. This operational feature is inherent in all open-channel distribution systems except the most sophisticated (level-top channels with level-control) and is not a major disability in most situations. It is not inherent in all pipe distribution systems, where there is also the option of downstream control, i.e. design such that when water ceases to be drawn off at downstream points, it simply backs up in the line and stops supply from the canal (as in a domestic supply system). Such downstream control systems either require the pipe to withstand pressure at the lower end, or involve the use of level-control or pressure-control valves at stations down the length of the line. These options will be discussed further in Chapter D.3.

Delivery at Individual Outlets Within a Distribution System

D.1.8 A central feature in the design of open-channel distribution systems is the means adopted to ensure an appropriate rate of flow at each turnout for efficient water management on the field. In most circumstances (Kallada is an exception), this implies a flow of from 15 to 30 litres/sec at the farm boundary, or higher in some circumstances. This is referred to as the farm stream or delivery stream. The quantity of water supplied during a particular irrigation is controlled by the duration of the period of irrigation, not by varying the rate of delivery. To achieve this, the practice in India is increasingly to design the distribution channel (a watercourse or field channel) with capacity equal to the desired farm stream and to deliver the whole of it to each turnout in rotation, each receiving the water for a period determined by the size of the area which it serves, or other factors. This avoids the problem of regulating the flow at

\[1/\] This subject is discussed in more detail in Volume I, Paper B.
each turnout, which would be encountered if more than one turnout at a time were to take water from the watercourse. Regulation is done at one point only, i.e. at the outlet from the canal. Within the area served by that outlet (the "outlet command", commonly 15 to 40 ha) the only control required is organization of rotational supply to outlets.

D.1.9 With pipe distribution systems the same question arises, i.e. how to ensure an appropriate rate of delivery at each of the turnouts (or valved outlets). One means of doing so, the simplest, is in the same manner as for an open-channel watercourse, i.e. the distribution pipeline is designed to convey the desired farm stream only, and the whole of the flow is taken at each turnout or valved outlet in turn. Where a pipe system is simply an alternative to a conventional open-channel watercourse served by an outlet from a canal, this would normally be the type of design selected. However, where the capacity of the pipe delivery system needs to be greater than the single farm stream, this solution is not available. Examples from tubewell and river-lift projects are rates of delivery (determined by economics of tubewell capacity and other factors) of 40 litres/sec (UP tubewells), 60 litres/sec (West Bengal tubewells) and 110 litres/sec (West Bengal river-lift). Equipment manufacturers have not yet developed a low-cost low-head flow controller with capacity in the range of 15 to 30 litres/sec, suitable for installation at each valved outlet on a pipeline system. Consequently, the expedient adopted in the first two cases quoted is to subdivide the flow at the tubewell between sub-systems, two of 20 litres/sec in UP and three of 20 litres/sec in West Bengal, each delivering the desired stream-flow at one delivery valve at a time on the sub-system. The sub-division of flow at the tubewell is accomplished by a weir arrangement in an elevated open distribution chamber (see Volume II, Paper E). In the case of West Bengal river-lift where a 110 litres/sec installation is divided between six sub-systems, the approach is similar, but primary and secondary distribution chambers are used. An adaptation of the same arrangement could be employed for supply from a canal in certain circumstances. For pipe system of greater capacity a different approach would be used, outside the scope of this paper.

D.1.10 In the majority of cases of pipe distribution from open supply canals, the simple arrangement of making pipe system capacity equal to farm stream is likely to be favoured. The special circumstances which make part of the Kallada project an exception are discussed below.

Cases to Be Considered

D.1.11 In the current issue of these notes, the Kallada project is considered in detail, as the project is under construction and the notes are required for operational purposes. The cases considered for Kallada are:

- Distribution in steeply sloping garden lands, with associated near-flat valley-bottom lands.
- Distribution to near-flat garden lands in an area of very high land values.
D.2 THE KALLADA PROJECT RIPE SYSTEM - DESIGN CONCEPTS AND OPTIONS

The Project Area

D.2.1 In the Kallada area, in southern Kerala, the upstream half of the command area consists of steeply-graded valley slopes largely under "garden" crops, with intervening valley-bottom lands mainly under paddy. Rainfall, averaging 2,500 mm per year, occurs principally in the monsoon season from late May to mid-November, the remaining six months of the year being mainly dry. Irrigation is required for crop production throughout the dry season, and also to bridge gaps in the monsoon, notably for paddy.

D.2.2 The garden crops include coconuts (which are particularly responsive to dry-season irrigation), arecanut, pepper, plantains, some coffee and cocoa, and inter-crops of vegetables and cassava. Dry-season irrigation of rubber is also proposed on a limited scale. The valley slopes are generally in sandy loams of lateritic origin.

D.2.3 In the valley-bottom areas, paddy has been the traditional wet-season crop, often followed immediately by a second crop early in the dry season. However, these areas are increasingly being converted to non-paddy crops (coconuts, plantains), particularly around their margins. Vegetables are also important. The valley-bottom soils are relatively free-draining loams or sandy loams originating from erosion of the valley slopes. The grade is typically about 1% down the length of the valley, which ensures good surface drainage.

The Special Requirements of Farm Water Distribution in the Upstream Portion of the Kallada Area

D.2.4 The garden lands have been under rainfed crops, principally coconuts, for many years prior to the advent of the project. The palms, now mature, were generally planted in separate small bunded basins, each without particular regard to the elevation of the neighbouring basins, in a more or less random fashion. Short of re-shaping and re-planting the whole area there is no possibility of introducing a conventional arrangement of contour channels serving rows of palms, and in much of the area irrigation has to be conveyed to each individual palm, or to its small bunded basin, separately. There are some larger basins, for instance, 15 to 20 m in length and 5 m in width with plantains planted on ridges, in which conventional furrow-in-a-basin could be practised, but these are the exception.

D.2.5 Technically, one solution could have been a sprinkler system, and there is sufficient gravity head for such operation in parts of the area, but not in most of it. Elsewhere sprinklers would require pumping, which is undesirable at this stage of development of the area. In the upstream half of the command where slopes are commonly in the range of 3% to 5% and the topography is highly irregular, there is no alternative to delivery of water to each individual palm and this implies the use of flexible hose in one form or another. It is essentially a horticultural operation.

D.2.6 A key question is the desirable rate of delivery in these circumstances. Farm streams in the range of 15 to 30 litres/sec were referred to earlier for efficient farm water management of field crops. However, a stream of that amount delivered from a hose could cause excessive erosion around the base of a palm. Furthermore, a water-filled hose of that size would be difficult to drag from one palm to another and would damage inter-crops en route. On the basis of trials in the area, a delivery stream
of nominally 2.5 litres/sec has been adopted. However, each delivery point on the line has two outlets of that capacity, in effect making available to any cultivator on the valley slopes a combined rate of delivery (two hoses) of 5 litres/sec. This is sufficient to supply the water requirements of a typical 0.5 ha holding in garden lands in about 12 hours of irrigation once per week.

D.2.7 Irrigation on the valley-bottom lands is of a very different nature. Requirements of paddy and also of perennials and vegetables must be met. Where possible, direct supply from the canal outlet down to the valley bottom, by open channel or pipe, is provided in order to permit short-term supply at a relatively high rate (about 30 litres/sec). Where this is not possible for topographic reasons, and the valley-bottom area has to be supplied from points on the pipeline extending along the valley slope, outlets of 10 litres/sec are generally provided (in very small areas down to 5 litres/sec) for this purpose. The particular problems and provisions for supply to the bottom lands are discussed in more detail in Annex 02.

D.2.8 The above discussion refers mainly to the upstream portion of the project area. Downstream, towards the coast, the topography becomes progressively flatter. It is still divided between wet-land areas devoted mainly to paddy and extensive near-level areas largely under coconuts. In this area, the problem of differences in elevation between adjacent basins serving individual palms is much reduced and supply by farm channels is possible. There is insufficient head for application by hose. However, land values have become extremely high, largely influenced by potential for residential use. In such circumstances, open-channel distribution from canal outlet to the individual holding would have to face the problems of high cost of right-of-way and cultivator resistance to the loss of access and consequent reduction in property values which could result from construction of open channels in this area. Buried-pipe distribution is likely to be the preferred arrangement, but in view of the small amount of head available a special design approach is necessary (see Chapter 0.4).

D.2.9 To summarize, irrigation of the mature garden lands on the valley slopes in the upstream portion of the project is necessarily horticultural, rather than conventional. Re-shaping of the area to simplify water distribution is not practical at this stage of development of the area. The unit of delivery, the "farm-stream" adopted, is 2.5 litres/sec (approximately 0.10 cfs), which will in most cases be applied by 50 mm flexible hose from valved outlets. These are in pairs, two per hydrant, giving a nominal rate of delivery of 5.0 litres/sec at the hydrant.

Choice Between Open Channel and Pipe Distribution for Kallada

D.2.10 This question has largely been answered, at least for the upstream portion of the project, in the above description of the area. Cost comparison of a unit length of buried pipe and a unit length of open channel (which would have to be lined, and with drop structures) would in any case favour the pipe on these slopes. However, the choice of pipe was not on economic grounds. It was imposed by two factors:

- Layout of open watercourses in much of that area would be impractical due to the presence of existing palms, dwellings, irregular stone-walled terracing, and highly irregular topography.

- A proportion of the palms on the valley slopes cannot be served by conventional open channel at ground level. The possibility of using
elevated open flume was considered, but obstruction to right-of-way in this semi built-up area would rule out such a solution.

D.2.11 Buried pipe distribution is clearly indicated in this upstream proportion of the Kallada command.

Regulation of Flow at Delivery Valves

D.2.12 With a "farm-stream" as small as 2.5 litres/sec or even considering the combined capacity of the two delivery valves (5.0 litres/sec) at a hydrant as the farm stream, it would be impractical to design the pipe distribution system for a capacity equal to a single farm stream, with rotational delivery at one point at a time only, as discussed in para D.1.9. With the design duty of the Kallada canal system of 1.0 litres/sec/ha (see Annex D1), the areas which could be served by 2.5 or 5.0 litres/sec would be simply 2.5 and 5 ha respectively. Such small "outlet commands" would require an extensive system of sub-minor canals and an undesirable number of canal outlets.

D.2.13 The nominal area of outlet command which has been adopted for the project, with the above factors in view, is 10 ha. For topographic reasons the area varies, from 5 to 20 ha, and in some cases larger. Referring to a 10 ha system, of capacity nominally 10 litres/sec, four 2.5 litres/sec delivery valves will normally be in operation at one time. This necessary departure from the simplicity of one supply point being in service at a time (para D.1.9) raises the key question of how to regulate the flow from four separate valves at possibly four points on the pipe system (at least two) with some approximation to equality, bearing in mind that points may be at different elevations and under different hydraulic pressures. This question is discussed further hereunder. The other central questions are the distance between delivery points (hydrants), the means of conveyance from a delivery valve to the individual palm or plot, and the layout and cost of the associated buried pipe system.

D.2.14 Regulation of flow between a number of delivery valves operating simultaneously is a trade-off between simplicity of design and accuracy of the result aimed at. The simplest solution, conceptually, would be a flow-control device at each of the delivery valves, limiting the discharge to 2.5 litres/sec regardless of line pressures (above a certain minimum). This may well be the eventual solution for Kallada or other similar projects. It is technically feasible and could very simply be added, but at present the industry does not have available a valve suitable to Kallada flows and pressure.

D.2.15 Alternatives, within current technology, include the following:

a) Installation of in-line pressure regulators at appropriate locations on the buried distribution line.

b) Provision of float-controlled pressure-regulating pipes at each hydrant. These could be added at any time. The same installation could also be arranged to function as a flow controller.

\[\text{For discussion of design duty of individual systems, as a function of percentage of paddy, see also Annex D1.}\]
c) Provision of the same type of float-controlled pressure-regulating stand-pipes at intervals on the main line itself.

d) For approximate regulation, the use of orifice-plates with various appropriate sizes of orifice, installed immediately upstream of the delivery valves, or varying sizes of valve.

D.2.16 As background to evaluation of these alternatives, the following factors are noted:

- The maximum rate of diversion to a Kallada pipe system is regulated at the design value by the setting of the gate or valve opening at each outlet from the supply canal. It is not influenced by factors within the pipe system itself such as location or number of valves in operation at any particular time. The latter factors may control the actual rate of diversion at any time, but not the maximum (refer to discussion of design of outlet structure, paras D.3.16-D.3.25).

- The quantity of water delivered at a particular valve per irrigation depends not only upon the rate of delivery at that valve, but also upon the time for which the valve is open. The latter is a question of cultivator cooperation in sharing of supply (rotational operation) rather than of technology.

- The variation in line pressure between the valves open at any one time, typically four, and the consequent variation in their discharge unless flow regulators are used, can be minimized by appropriate selection or grouping of valves to be opened together. This again is an organizational rather than a technical matter.

- Service areas which are elongated laterally can present a problem in providing sufficient head to reach the high points on the service area farthest from the canal outlet. Supply at rather less than the design rate may be necessary in such circumstances, regardless of any provision for flow regulation, with corresponding adjustment of the duration of supply at that point to compensate for the reduced rate of delivery.

- Apart from automatic methods of flow regulation, manual regulation of delivery by adjustment of valve opening by the cultivators themselves can be assumed in circumstances in which it is in the interest of the individuals to do so. For instance, at a point of low elevation on a distribution line the locally high hydraulic head could cause a fully-open valve to deliver substantially more than 2.5 litres/sec, in the absence of a flow regulator. However, such flow, unless throttled down by partial valve closure, would be likely to blow the hose off the valve outlet, or in any case would result in too high a velocity of flow from the hose for satisfactory application to crops. It can reasonably be assumed that the cultivator would reduce the flow by partial valve closure in such circumstances.

- The individual Kallada pipe systems, due to their number, will necessarily have to function with a minimum of departmental attention. A canal supervisor with responsibility for 500 ha will have at least 50 pipe systems in his charge and will be principally occupied with the functioning of the distributary and minor canals.
and outlets, with little time for attention to operation and maintenance of the pipe systems themselves. The systems should consequently be as simple as possible technically, requiring a minimum of maintenance or adjustment following the initial commissioning period. Reliance will have to be placed upon the cultivators and their organizations for day-to-day operation.

D.2.17 To summarize the above discussion, a flow regulator suitable for each outlet valve is not yet commercially available, but probably it will be in due course and could eventually be added as an operational refinement. Other methods of automatic pressure or flow regulation in the pipeline or at hydrants could be installed at this time. However, with whichever controls were adopted, reliance would still have to be placed upon the cultivators for some aspects of operation. In the circumstances, and in the interests of simplicity and minimum maintenance, initial flow regulation at individual valves will be limited to the orifice plates referred to in para D.2.15, or use of smaller size of valve 1/ where the head is unusually high. The orifice plate is simply an annular steel disc which is inserted in the line immediately before the valve. The size of the central hole in the disc is made to suit the head reduction required at the design flow.

D.2.18 Achieving near equality in delivery between valves will be facilitated by appropriate selection of valves in operational groups (the valves which will be in operation together, at any particular time). As previously noted, the maximum rate of supply to the pipe system as a whole, and division of flow in the supply canal between pipe systems, is determined by setting of the gate or valve at the canal outlet and is independent of valve operation within the pipe system.

Selection of Pipe System

D.2.19 In the layout of a pipe system for irrigation distribution, the starting point is the desired location of the delivery points, and the means of conveyance from the delivery points to the field or plot. As discussed earlier, circumstances in the upstream portion of the Kallada command require conveyance by hose (or the equivalent portable pipe) to the individual palm. The delivery points with their supply valves, and the buried pipe serving them, are the fixed portion of the systems and the hose is the moveable portion. As all delivery points do not operate at the same time, hose (or portable pipe) may be moved from one to another. Closer spacing of delivery points and consequent shorter runs of hose make for convenience in operation but also for higher cost. The higher cost is incurred not only for the delivery points themselves (hydrants with pairs of supply valves) but in some situations for additional length of buried pipe required to accommodate the closer spacing.

1/ As discussed in Annex D4, the use of a smaller size of valve can be the preferable solution where water-hammer is an important factor.
D.2.20 An important factor is the type and cost of the moveable system to be used. Hose has been referred to. An alternative is quick-coupled pipe, in length of 2 to 4 m and with spherically-seated (flexible) joints. The pipe may itself be equipped with small gated outlets, or valved outlets suitable for hose connection. One of the arrangements considered early in the project (and assumed in the Staff Appraisal Report) was in fact based upon a down-slope buried "main stem" and portable quick-coupled laterals to be laid at right angles to and on either side of the main stem. The laterals were to be connected to a series of hydrants down the length of the main stem, being moved from one to another as irrigation progressed. The laterals were to have valved outlets at intervals along their length, from which short runs of hose would extend (down-slope) for supply to individual palms or to plots. It became evident, however, as detailed surveys proceeded that the outlet areas were very often elongated in shape along the contour and relatively short down-slope, rather than the converse. For this reason, and also the highly irregular shape of many of the areas, the systematic use of portable pipe laterals was not further pursued, although it is probable that individual cultivators will, in some circumstances, use quick-coupled pipe for distribution within their own holdings.

D.2.21 In the arrangement actually adopted, delivery is by 50 mm hose directly from the hydrant to the plot. Location and spacing of hydrants is an important consideration and is discussed further in Chapter D.3. Spacing in the horizontal direction is kept to not more than 90 m where possible, implying a horizontal distance of hose run of not more than 45 m. Down-slope runs may be of greater length, for hydraulic reasons.

D.2.22 With regard to layout of the buried-pipe system, in most cases this will be largely pre-determined by shape of service area and desired location of hydrants. In some cases, however, particularly where the service area is near-square in shape, there are options available in layout. These include multiple (two or three) down-slope runs supplied by a single upper horizontal "main stem" or, alternatively, multiple horizontal runs supplied by a single down-slope main stem. A further variant, where multiple runs are involved, is to connect the outer ends of the runs to form closed loops. In some cases, this can have considerable advantage in pipe size, also in reduced variation in head between valves. The loop system has important application in tubewell distribution systems, and some river-lifts, where the shape of the service area or sub-areas is to a greater extent open to choice and can be optimized in the interest of pipe economy. (For further discussion, refer to Paper E.) It would also be applicable in those pipe distribution systems served by open canals in which topography is relatively regular. However, in the very irregular conditions of much of the Kerala command the loop system is likely to be applicable rather infrequently.

D.2.23 Selection of pipe system layout is largely a matter of trial and comparison of costs for likely alternatives. Hydraulic criteria in layout design are discussed in Chapter D.3. Typical types of layout are shown on Plate D1.
Operation of the Kallada System — The Role of the Cultivator

D.2.24 A distribution system of the type being constructed for the upper portion of the Kallada project provides piped delivery to valved supply points throughout the service area. It also ensures that the total rate of diversion to the service area of a pipe system is not more than the design amount. Beyond those two technical provisions, the functioning of the system is largely in the hands of the cultivators, as it will not be possible for the Irrigation Department to supervise the day-to-day operation of every supply point in the Kallada command.

D.2.25 Systematic rotational schedules can, and should, be set up initially with Departmental assistance, but adherence to such schedules must be left to the responsibility of the cultivator organizations. 1/ With intelligent scheduling, each cultivator can be assured of a supply at approximately the design rate of flow (2.5 litres/sec or optionally two such streams) for a pre-arranged period each week, making for efficient water application and a minimum of time devoted to the task. Without such scheduling, and if cultivators claim the use of any or all supply points at will, the rate of delivery at most supply points would be unpredictable, and certainly too small for effective application. Some degree of cooperation between the 20 or 30 cultivators served by a pipe system is consequently very much in their own interest and can reasonably be anticipated.

D.2.26 A particular operational problem which can only be resolved by agreement between cultivators within a system is the scheduling of deliveries so that irrigation of the valley-slope lands is carried out largely in daylight hours (as constant attention to hoses is necessary) and of valley-bottom lands at night. As the distributary canals serving each system necessarily operate 24 hours (with the main canal), for a system not to take water at night would be to lose part of its allocation.

D.2.27 Joint action is also desirable in the matter of provision of hose. As hoses can be moved to the valves in operation at any particular time (generally four), it is unnecessary to provide a hose for every hydrant, much less for every supply valve. Each system will be provided with the minimum necessary number initially, and these will be regarded as communal property. The group of cultivators served by a system will be responsible for replacement of these hoses when necessary, but may also purchase additional hoses, communally, to facilitate operation (e.g. to reduce time lost in moving from one hydrant to another). It is also anticipated that cultivator groups may, in some cases, request and pay for installation of additional hydrants.

D.2.28 To summarize, the systems being installed are capable of providing excellent service if they are operated in the communal interest. It is expected that most cultivator groups will find it to their advantage to do so. However, Departmental support and guidance will be necessary initially until cultivator groups become familiar with their systems and capable of operating them.

---

1/ A registered Water Users Association is proposed for every four to five pipe systems (totalling 30 to 40 ha). Sub-committees for each individual system may also be set up.
Introduction

D.3.1 This chapter is intended to serve as guidelines for the project.

D.3.2 A number of items are dealt with in more detail than necessary for field use. For the sake of brevity in the main text, such detail is confined to the annexes.

Rate of Supply per Hectare of Service Area (Irrigation Duty) at the Outlet from the Canal

D.3.3 The design capacity of the Kallada distributary canals is 1 litre/sec/ha based upon continuous operation in the peak season (24 hours/day and 30 days/month). As it is intended that all outlets ("spouts") will also be operated continuously in the peak season, the capacity of the pipe distribution systems (averaged over the project) must be that same figure. This value is generally confirmed by the calculations of peak water demand at the outlet for various proportions of valley-slope and valley-bottom lands in Annex 01. These are summarized as follows:

<table>
<thead>
<tr>
<th>Description</th>
<th>Litres/sec/ha</th>
</tr>
</thead>
<tbody>
<tr>
<td>Project average case, with 33% of valley bottom and 67% valley slopes</td>
<td>1.00</td>
</tr>
<tr>
<td>Case of 100% valley slopes</td>
<td>0.80</td>
</tr>
<tr>
<td>Case of 60% valley-bottom lands (in this situation direct supply from the outlet to the valley-bottom portion of the area would generally be available)</td>
<td>1.17</td>
</tr>
<tr>
<td>Case of 100% valley-bottom lands (in this situation the supply to the area would probably be simply by direct diversion from the outlet, without pipe system):</td>
<td></td>
</tr>
<tr>
<td>- Estimated figure</td>
<td>1.43</td>
</tr>
<tr>
<td>- Suggested design figure with margin for higher infiltration</td>
<td>1.70</td>
</tr>
</tbody>
</table>
D.3.4 With the above figures as background, the capacity of the headreach of the main pipeline should, in all outlet commands, be taken as the higher of the following two values:

a) 1 litre/sec/ha calculated on the whole of the area served by the outlet.

b) 1.7 litres/sec/ha calculated on the portion of the area which is in valley-bottom lands.

(Criteria a) will give the higher value unless the proportion of valley lands is greater than 1 or 60% of the service area.)

D.3.5 It is emphasized that the above criteria refer to the capacity of the pipeline, not the capacity of the outlet culvert from the supply canal (the spout), which is usually greater than line capacity and which is normally reduced to desired discharge by partial gate or valve closure. Where direct diversion to the valley-bottom portion of an outlet command from the outlet box via a nullah is possible (without going through the pine-line), the full capacity of the outlet can be made available for short periods. This may be of value during land preparation. In such circumstances, the capacity of the outlet culvert (with gate or valve fully open) should be made at least 25 litres/sec. The supply of water to valley-bottom lands is discussed further in the following section, also in Annex D2, and in connection with the design of the outlet box.

Size and Location of Service Areas (Outlet Commands) and Location of Outlets

D.3.6 Size of service area in the upstream portion of the Kallada command is nominally 10 ha, a balance between cost of outlet structure per hectare served, which decreases with size of area, and cost of pipe per hectare which beyond a certain point increases with size of area, as larger pipe diameters become required. It is, however, purely a nominal figure. The actual area of each particular outlet command, or block, and the location of its boundaries must be determined with regard to local site conditions, particularly topography, the hydraulic profile (full supply level - FSL) of the supply canal, and in some cases the existence of outlets which have already been constructed. The latter factor is of particular importance in direct outlets from the main canal. Detailed topographic surveying, determination of boundaries of outlet commands, and location of outlets with regard to the latter, should be carried out in advance of outlet construction, rather than outlets being located arbitrarily and constructed at the time of construction of the distributary. Where an existing outlet, constructed earlier, is not in a suitable location for supply to the block, or cannot be adapted to it in pipe layout, a new outlet may have to be added. This is not a major construction item in a small distributary.

D.3.7 In deciding upon block boundaries in an area where there will be several neighbouring blocks, it is very desirable for the detailed topographic survey of that area as a whole to be completed before individual block boundaries are fixed, as there may be several possible alternative arrangements. Thus, a particular part of the area could be included in one or in another of the adjacent blocks, or there may be advantage in combining two small nearby areas under a single outlet.
0.3.8 In the highly irregular topography of the Kallada project, it will not be possible to provide service to the whole of the nominal gross command of every distributary, nor in fact is there sufficient water to do so. Portions of the gross command are not economically irrigable, particularly where very narrow and on excessively steep slopes. Judgement will have to be exercised as to which areas can be served and which cannot.

0.3.9 The areas served by individual outlets are likely to be in the range of 4 ha to exceptionally 20 ha. The larger areas will most often occur under direct outlets from the main canal, where the number of outlets must be limited. It is noted that this discussion applies to the upstream portion of the Kallada area, where delivery is via 2.5 litres/sec outlet valves and hose. In the downstream near-flat areas discussed later (Chapter 0.4), size of outlet service area is determined by a different set of factors.

Required Capacity of Pipe System, Upper Portion of the Kallada Command

0.3.10 Nominal design capacity, per hectare, has been defined in para 0.3.6 as the greater of either 1 litre/sec/ha of the area commanded, or 1.7 litres/sec/ha of that portion of the area commanded which is in valley-bottom lands. The criterion applies not only at the upper end of the pipe system, but also at any point within the system.

0.3.11 The capacity so derived is in effect the minimum design figure. In obtaining the actual required capacity, the following additional criteria apply, and may result in the capacity required being greater than the nominal:

a) As the nominal rate of delivery at each valve is 2.5 litres/sec, the capacity of the line at any point should be a multiple of 2.5 litres/sec. Generally, however, valves should be capable of being operated in pairs on each hydrant, and the minimum capacity of a line at any point should not be less than 5 litres/sec.

b) Where valley-bottom lands are to be supplied by an outlet from the pipe system (refer to Annex 02), it is desirable that pipe capacity at that outlet should be not less than 10 litres/sec. (This situation generally occurs only in the larger outlet commands.)

0.3.12 In stating the capacity of a pipeline at a particular point, it is necessary to define the operating conditions under which that capacity is obtainable. Capacity is determined by several factors including the elevation of the area to which water is being delivered (in relation to FSL of the supply canal), and which particular valves are in operation on the system at the time. This is particularly the case with long pipe runs under low head. The subject is discussed further in para 0.3.39.

0.3.13 Before leaving the question of line capacity, it is re-emphasized that the design maximum rate of flow at an outlet supplying a pipe system is controlled by the setting of the gate or valve on the outlet culvert or "spout", and not by the capacity of the pipe system. The latter is a variable, dependent upon the number and location of delivery valves in operation. With all valves open (not a normal method of operation), pipe system capacity would exceed the design rate of flow to the outlet command, but the actual rate of flow would be restricted by the gate or valve setting.
Design of Outlet Structure from Distributaries and Main Canal

D.3.14 Outlets may be on distributary canals, or in some cases directly on the main canal. A schematic design for an outlet on a distributary is shown on Plate D2. It has the following functions:

- Exclusion of leaves or other floating debris from the pipe system.
- Entrapment of sand which would otherwise enter the pipe system.
- Measurement and regulation of flow.

D.3.15 As the distributary canals are generally unlined and pass through erodible granular lateritic soils, a considerable amount of coarse sand will pass down the distributaries as bed-load, and must either be excluded from entry to the outlet or entrapped before it reaches the PVC pipe system. Further, as the canals traverse areas under coconuts and plantains, there will be much floating debris. With the design shown of the type on Plate D2, with centre-line of the outlet pipe about 15 cm below normal full supply level, the bottom of the intake will generally be well above the bottom of the canal. Direct entry of bottom-travelling (bed-load) sand will consequently be minimized. Such sand as does enter the outlet pipe will be trapped in the upstream compartment of the outlet chamber, from which it should periodically be removed.

D.3.16 Entry of floating or waterlogged leaves, palm fronds, or other material will be prevented by the screen. To minimize clogging of the screen, and the frequency with which it must be cleared, the velocity through the net area of openings should not be more than 0.3 m. This is easily achieved, with such small intakes, as the flow diverted is generally less than 30 litres/sec. The screen may be formed of parallel steel bars, approximately 12 x 3 mm, arranged on edge with not more than 6 mm width of opening between adjacent bars. The bars are formed into a composite structural unit which is bolted onto the intake. Although normal practice is to arrange screens so that the openings (slots) are vertical to facilitate raking of debris from the screen from the canal bank, it is suggested that the screens on distributary intakes be arranged with slots in the horizontal position. This encourages floating or waterlogged debris to slide along the screen, carried by the flow in the channel, and to continue downstream rather than to become lodged in the screen. Raking of the screen, normally simpler with vertical slots, is unlikely to be a problem in this case as the distributaries are seldom more than 0.6 m deep and clearing of screens can be performed from within the channel itself.

D.3.17 The outlet chamber serves as a means of controlling and measuring the flow to the pipe system. Measurement is effected at the 90° Vee-notch weir (a steel plate cut to the required shape and bolted to the concrete partition in the chamber). For a discharge of 10 litres/sec, the depth over the apex of the Vee will be approximately 15 cm. The flow to the chamber is regulated by the slide-gate shown or the equivalent valve. The degree of opening of the gate to give the desired flow will depend upon the actual height of canal FSL above the level of the apex of the Vee-notch. With the dimension shown on Plate D2 for a flow of 10 litres/sec, the slide-gate will be about one-third open. The gate should have a padlock arrangement (through holes in the gate frame) which normally prevents the gate from being opened beyond the pre-set amount, unless at times of direct supply to valley-bottom paddy lands when the gate may need to be fully open.
The gate can be closed by cultivators at any time. The desirable gate setting for normal operation will be determined after the distributary goes into service and final FSLs have been established. If at any time the canal runs above or below nominal FSL, the flow to the pipe system will change due to changed head on the gate. However, a temporary change of 5 cm above or below the nominal 15 cm head on the gate would vary the flow to the pipe system by not more than 15% to 20%. If the FSL of the canal changes permanently (due to siltation or weed growth), the gate opening should be readjusted accordingly.

D.3.18 Where two pipe systems are served by a single outlet, the outlet chamber should be made wider than shown to accommodate two Vee-notches and the downstream compartment of the chamber should be divided into two by a central dividing panel, each half serving one pipe system. Where optional direct diversion to valley-bottom lands via a nullah is required, a separate rectangular low-level outlet from the downstream compartment of the outlet chamber is provided (shown dotted on Plate D2). This is normally closed by a second steel-plate slide gate. When opened, it takes the whole flow of the outlet. For direct diversion of 25 litres/sec, the depth over the Vee-notch will be approximately 20 cm, and the slide gate on the outlet pipe will be almost fully open.

D.3.19 The galvanized iron wire mesh screen shown on the intake to the PVC line is particularly noted. This is a backup screen to prevent leaves from entering the line. It is also intended to prevent children from dropping stones, etc., into the pipe system.

D.3.20 The top of the outlet chamber must be carried up to above maximum water level in the canal, by a margin of at least 30 cm. With this provision, if irrigation from hydrants on a pipe system ceases, water merely backs up in the pipeline and chamber until the level in the chamber is the same as the level in the canal and flow to the line ceases. There is no spill.

D.3.21 It is emphasized that there is no gate or valve at the entry to the pipeline. This is intentional, as closure of a valve at that location could cause sub-atmospheric pressure in the line and pipe collapse. If shut-off of supply to a line is required (for instance for maintenance), the gate on the outlet is closed.

D.3.22 In normal operation, the line will run in the part-full condition for a short distance down-slope until the level is reached necessary to provide the head on the valves in operation at the time. From that point on the pipe will be full. Under critical design conditions (when all available head is required), the full/part-full inter-face moves up the line into the distribution chamber. The maximum head on the line, for the condition of discharge of design flow, corresponds to the situation in which the water-level in the downstream compartment is just above the bottom of the Vee-notch. If the level in the compartment rises above that elevation, the weir becomes partially submerged and flow is reduced below the nominal design amount.

D.3.23 A number of direct outlets from the main canal were constructed at the time of construction of the main canal. Some of these take the form of a pipe 0.3 m diameter laid in the embankment fill at an elevation a short distance above canal bed level. The upstream end of the pipe is butted against the outside surface of the canal lining, presumably with surrounding masonry support. The pipe is supplied with water from the canal via a number (10 or 12) of 25 mm diameter holes drilled through the canal lining. The arrangement needs modification in several respects. Firstly, the intake
at present formed by the group of 25 mm holes would be very susceptible to blockage from waterlogged debris, and very difficult to clear due to the head on it when blocked. It should be opened up to at least a 200 mm diameter hole, with chamfered entry (the rate of delivery to most direct outlet commands will be 20 to 25 litres/sec), provided with a fixed trash screen of the type described for the distributary outlet (Plate 02). However, in this case the screen bars should be aligned vertically (up the slope of the canal lining) rather than horizontally, to facilitate raking the screen from the surface. Secondly, a direct outlet on a major canal, particularly one equipped with downstream valve and hence under pressure, can present a hazard to the security at the canal embankment. Rupture of the embedded pipe, or its connection with the lining, could cause heavy discharge into the embankment. Some precautions should be taken. One is to provide a simple slide gate (behind the trash screen) operable from canal-side, to provide upstream emergency closure of the outlet. Without this provision discharge into the embankment could not be stopped short of drawing down the main canal. The other is a review of the design of the connection between the pipe and the back of the lining to determine its structural adequacy. If this is in any doubt, it would be preferable to take out a portion of the existing lining (possibly a 2 ft diameter opening) and reconstruct the intake. Any new direct outlets should have the pipe carried through the canal lining and sealed on the upstream side. Thirdly, at its downstream end the line (with valve) should be made to discharge into an outlet chamber similar in function to that shown on Plate 02. However, the chamber may have to be enlarged on the upstream side of the weir, to provide energy dissipation from the outlet valve, which will be under head corresponding to main canal FSL. The chamber should be carried up to canal FSL plus 30 cm, if spill from the chamber on shut-down of irrigation from the pipeline is to be avoided. In view of the head on the chamber in the latter condition, a reinforced circular section rather than the rectangular form shown on Plate 02 may be preferable. It is emphasized that the direct outlets currently provided were installed provisionally, prior to determination of the capacity required. They are of relatively low capacity and designed for manual operation (i.e. closure of the outlet valve whenever irrigation from the pipe system is not required). In new direct outlets, extension of the outlet chamber up to canal FSL, while requiring some structural work, makes such manual operation unnecessary and also provides means of regulating and measuring the rate of diversion.

Location of Delivery Points (Hydrants) and Line Layout

D.3.24 Location of delivery points is briefly discussed in relation to selection of pipe system in para D.2.21. From the viewpoint of cultivator convenience, the greater the number of hydrants (or the closer the spacing), the better. One hydrant per holding would be ideal in this respect. However, cost is a very pressing constraint. At a minimum of Rs1,200 per hydrant, a density of one hydrant per 0.5 ha presents a cost of Rs2,400/ha for hydrants alone. This is probably the maximum density (exceptionally one per 0.4 ha) sustainable on cost grounds, averaged over an outlet command as a whole. On the other hand, a particular individual cultivator may well be prepared to meet the cost of an additional hydrant for his own use or shared with a neighbour (provided that it is used only within the operational schedules established for the outlet command). He should be encouraged to make such request, with pre-payment, at the time of installation of the system. Later addition of a hydrant is also possible, at some further cost.
0.3.25 Setting aside individual requests, and returning to hydrants installed as part of the communal system, the following factors should be taken into account in deciding upon their location:

a) The maximum horizontal distance between hydrants should not exceed 90 m, and if possible consistent with other criteria (density in hectares per hydrant) it should be kept less. This spacing implies a maximum horizontal run of 45 m of hose from any hydrant. This arbitrary limit is imposed by two considerations. One is the length of hose which can easily be manipulated when full. The other is the head loss in the hose itself. With 50 mm hose delivering 2.5 litres/sec, the friction loss in 45 m of hose is approximately 1.5 m. At some valve locations such a friction loss would not be a problem. However, with hydrants in the upper portion of the service area where available head is small, it could be a limiting factor in rate of delivery and use of larger hose (60 or 70 mm) may be necessary.

b) The distance of up-slope run from the hydrant to the highest plot served should be considerably less than 50 m, particularly for those hydrants in the upper portion of the command, as in this direction land slope and friction loss in the hose are cumulative in reducing available head on the system. The up-slope distance should preferably be not more than 20 m.

c) In the down-slope direction, the opposite situation is encountered. The ground-slope assists in overcoming friction in the hose. For example, with a slope of 3% which is very common the slope almost offsets the friction gradient in the hose, and the run could be extended down-slope indefinitely from the hydraulic viewpoint. The remaining consideration in this situation is the inconvenience of a long run of hose. However, cultivators have a number of options for down-slope distribution including use of a length of fixed or moveable 90 mm PVC pipe, with hose from the hydrant discharging into the upper end of the pipe and with hose connection at the lower end for local distribution from that point. Down-slope length of run should not therefore be subject to the same limitations as horizontal or up-slope runs. It is preferable to locate a hydrant near the upper boundary of a holding, thereby requiring a relatively long down-slope run, than to locate the hydrant near the centre of the holding in order to limit the length down-slope. As an instance, a hydrant supplying an area of 100 m length down-slope might with advantage have the hydrant within 20 m of the upper boundary and 80 m of the lower.

d) Where possible, a hydrant should be located on the lateral boundary between two holdings providing direct access to both. It was decided for Kallada that every holding of 0.5 ha or greater should have direct access to one 2.5 litres/sec outlet valve. However, in some circumstances, particularly with very small holdings, one cultivator may have to cross the holding of another to gain access to a hydrant. It must be made clear to cultivators on whose holdings hydrants are installed that the hydrant is communal property, and access by others must be allowed. However, in final location of hydrants unnecessary intrusion on privacy should be avoided. Thus, a hydrant should not be located immediately adjacent to a dwelling, or within a walled homestead garden.
e) A delivery hose should not have to cross a road. It is preferable to take a branch line across (under) the road and to install an additional hydrant than to require a hose to lie across a road. Just what constitutes a road, in this connection, rather than simply an access path, is a matter of judgement.

D.3.26 A hydrant serves two 50 mm valve-controlled supply points. In view of the large number of hydrants required and their influence on project costs, it has been necessary to adopt a very simple arrangement with minimum of associated masonry and concrete work. As spill and minor leakage are to be expected, the backfill around the hydrant and extending down to the underlying PVC delivery line should be in well-compacted, free-draining laterite or murrum.

D.3.27 Selection of type of pipe system and line layout were discussed in paras D.2.19-D.2.23. It was noted that there are often alternative possible layouts and comparison of costs may be necessary. The following notes refer to technical factors of general application in line layout.

D.3.28 Three particular requirements are noted:

- Removal of air at high points in the line.

- Prevention of excessive pressure surges (water-hammer), particularly the development of negative or sub-atmospheric pressure.

- In systems such as Kallada, which are supplied from irrigation canals, prevention of accumulation of silt at low points in the line.

D.3.29 In the Kallada system the hydrants provide means of removal of air, and special provision of air bleeder valves will not generally be necessary. However, care should be taken that any high points on the line be made to coincide with location of a hydrant, or surge riser, to facilitate air venting. Valves at these particular hydrants may need to be fractionally opened during line filling for slow removal of air. Rapid evacuation of air can cause water-hammer when the water/air interface reaches the valve and there is an abrupt change in line velocity. Between hydrants the line may be level, or sloping, or it may dip downwards subject to the reservations discussed later regarding silt accumulation.

D.3.30 Pressure surges are discussed in a later section. The most important factors in the layout relating to minimizing of pressure surges are the following:

- Routing the line so as to permit installation of one or more surge risers in the downstream portion of the line. (Either directly on the line or via a short up-slope branch on the line.)
Keeping the elevation of the main line low enough, particularly on long near-level runs on the upper portion of the service area, to avoid negative pressures during passage of surges. While it was suggested earlier that hydrants should be located near the upper boundary of the area served (within 20 m distance), this is not necessarily a favourable location for the main line, from the viewpoint of negative surges. It may be desirable to route the main-line further down-slope to ensure static head of preferably not less than 2 m (less for shorter runs) and to run up-slope branches to hydrants.

D.3.31 With regard to silt accumulation appropriate design of intake (para D.3.16) to exclude coarser material, and the entrapment of sand on the upstream compartment of the outlet chamber, are the first lines of defence. However, the fine silt encountered in supply from unlined channels in lateritic areas is likely to remain in suspension and to pass through the outlet chamber into the pipe system. Line velocities in normal operation (0.6 to 0.9 m/sec) will ensure that this fine silt is swept through the pipe system and discharged at operating valves. On shut-down of irrigation, the silt will settle but will again be picked up when operation resumes. However, precautions should be taken to avoid accumulation, particularly of the coarse silt fraction, at conspicuously low points in the line. Routing of the line to maintain near-level or uniformly downward gradient, rather than direct crossing of lower areas, is an advantage in this respect. Where the latter cannot be avoided, a blow-off valve should be provided at the low-point for de-silting, or alternatively a Tee with screwed-on end-cap may be used. This can be removed (with the line empty) and used as a blow-off for silt on temporary re-filling of the line. The end-cap is then replaced and normal operation resumed. This is not required for short dips in the line or where the up-slope gradient in the line from the low point is less than about 5%. Experience with operation of the system will indicate the need for periodic de-silting procedures such as operating with all valves open on particular branch of the system for a short period to maximize line velocity and scouring effect. Equipment is available for more positive de-silting should this ever be found necessary. This is a high pressure hose with rotary jets, which is passed into the line through a hydrant point or specially provided Tee. (Commonly used for de-silting of buried PVC corrugated drainage pipe.)

D.3.32 Within the above requirements considerable flexibility in line layout is possible, and advantage will need to be taken of it in view of the many physical obstacles to line construction in the Kallada area. An important consideration in detailed layout will be to minimize interference to property and damage to mature trees or palms. Cultivators should be consulted before alignments are finalized, and with regard to access during construction.

D.3.33 Minimum cover over the pipeline of 80 cm should be maintained, except at crossing of roads or ditches, where reduced cover may be desirable. In such circumstances the line should be protected by another concrete pipe.
D.3.34 The PVC line proposed will be solvent-welded rather than connected by rubber ring joints. The solvent-welded joint provides structural continuity, rendering unnecessary the provisions of thrust-blocks at changes in direction of the line, which are required in the case of rubber-ring connectors.

D.3.35 The lateritic soil available throughout the area will provide excellent bedding material for the pipe. It is emphasized, however, that the pipe to be used (2.5 kg/cm²) is of relatively thin wall and particular care must be exercised in bedding. The fill in contact with the pipe, and for a distance of some 5 cm around it, should be free from lumps or pebbles (if necessary being sieved). It should be carefully hand-compacted, so as to provide uniform support to the pipe without distorting it out-of-round. Careful packing around the pipe assists greatly in the ability of the line to withstand negative pressure without buckling. Above the compacted layer, the remainder of the fill may be random.

Determination of Pipe Size - Hydraulics of System

D.3.36 Determination of capacity required at any point on a pipeline (with reference to the upstream portion at the Kallada command) is discussed in paras D.3.12-D.3.15. Capacity, together with available head, establishes line size required.

D.3.37 The first step in line-sizing is to establish how hydrants will be grouped for operational purposes, each hydrant being taken as delivering a combined flow of 5 litres/sec (two 2.5 litres/sec valves). To illustrate consider the three schematic systems shown in Plate D1, each with 10 ha service area and nominal rate of supply at the outlet of 10 litres/sec. In each case there are 10 hydrants, only two of which should be in operation at any one time. The operational grouping of hydrants assumed for purposes of line-sizing is indicated by the flows shown. In Plate D1, Figure 1, for instance, it has been assumed that one hydrant will be in operation in each of the two down-slope branches. If the system were to be designed on the assumption that both the hydrants in operation at one time could be on the same down-slope branch, the required capacity and size of the branches and of the upper horizontal line would be increased. The same comment applies to Plate D1, Figure 2, where the upper and lower horizontal branches are sized on the basis of one hydrant being operated at a time on each of them (the hydrant at the farthest end of the line is likely to give the controlling condition for determination of line size). However, in this case to provide a 10 litres/sec outlet to the valley-bottom (paddy) lands, the capacity of down-slope run has been kept at 10 litres/sec rather than the 5 litres/sec which would otherwise have been required. (When 10 litres/sec is being delivered to the valley bottom, all other hydrants are closed.) Plate D1, Figure 3, is a situation frequently encountered with a long horizontal run supplied from one end rather than centrally (usually because of use of a common outlet for two such systems). In sizing the line it has been assumed that one hydrant is in operation in the upstream half of the system and one in the downstream half (remote from the outlet). If both operating hydrants were to be assumed to be in the downstream half, particularly near the end of it, line size would have to be increased considerably.
0.3.38 Having established how the hydrants are assumed to be grouped, and the critical location of specific pairs of hydrants for the purpose of the line-sizing, line size may be simply determined. Static head on the line down to the elevation of the highest plot being watered is obtained from the contour plan of the area and the watersurface profile in the supply canal. Head loss through the outlet chamber must also be allowed for. With head known, and the flow in each of the reaches of the line leading to the farthest valve, friction loss is calculated in each reach for one or two line sizes until a total head loss (including the delivery hose) is arrived at, which matches head available. The process is one of trial and error, and in many cases several different combinations of line size in the reaches of the line between outlet and the critical hydrant may need to be considered. Further, a particular reach, usually the upstream portion of the line, may contribute to friction head for more than one grouping of hydrants, and its capacity and size may have to be checked in each case.

0.3.39 Friction factors and calculation of friction loss for various flows and line sizes are discussed in Annex D3, and presented in graphical form on Plate 1 of the annex.

0.3.40 In the cases illustrated in Plate 01, the outlets are on one side of the command. More frequently an outlet is located nearer to the centre. Two alternatives are available in this case. One is to divide the outlet command into two halves, a right and a left portion each with separate pipe system but served by a common outlet chamber (as discussed in para 0.3.20). The other is to use a common system with the main line bifurcating into left and right branches a short distance from the outlet. Either can be applicable and the choice depends to some extent on the size of the command, division at the outlet chamber being more appropriate to larger commands. However, if the single main stream with bifurcation is adopted, the bifurcation should be sufficiently below the elevation of the outlet chamber that the line is full at the bifurcation under most circumstances.

Control of Water-Hammer - Location of Surge Risers - Rate of Valve Closure

0.3.41 It might appear that PVC pipe, being relatively elastic, would effectively cushion the pressure waves which result from valve closure on a long pipeline. This is the case in one respect, but the advantage is to some extent offset by the slow velocity of pressure waves in PVC pipe. Pressure at a valve which is being progressively closed continues to build up until negative pressure waves due to reflection at the upstream free surface (usually the outlet from the supply canal) return to the valve. The slow wave velocity in PVC pipe has to be countered by relatively slow rate of valve closure, to avoid excessive pressure build-up. Furthermore, in water-hammer a positive surge is immediately followed by an equivalent negative surge, which may produce (temporarily) sub-atmosphere pressure. Thin-walled PVC pipe is particularly susceptible to collapse under such circumstances. In short, PVC pipe is not immune from the effects of water-hammer, and the latter must be taken into account in system design. However, as detailed analysis of pressure transients in branching pipe networks is a highly technical subject, it is not practicable to resort to complete analysis for each of the small Kallada pipe systems. It is sufficient for the designer to have a general understanding of the cause of water-hammer and the measures which can be taken to minimize it.
D.3.42 In accompanying Annex D4 the subject is discussed using the coil-spring analogy. Simple cases of complete valve closure, progressive closure, and reflection at a close-ended branch line are then considered. Wave velocity is derived and also the relationship between change in line velocity, length of line and pressure rise.

D.3.43 Summarizing:

a) For PVC pipe in the pressure class 2.5 kg/cm² (of the type to be largely used in the Kallada project), the velocity of the pressure wave resulting from a small change in line flow is approximately \( v = 700 \text{ ft/sec} \).

b) The "round-trip time" is simply the length of time for a wave to travel at 700 ft/sec from the valve to the nearest free surface (the outlet from the canal, or in some cases a surge riser) and back to the valve. This is usually written as \( T \) (seconds).

c) The pressure rise at the valve due to a change of velocity in the pipeline of \( V \) ft/sec occurring within the round-trip time \( T \) (or any lesser time, including instantaneously) is given by:

\[
\text{Head rise } H (\text{ft}) = \frac{V \times v}{g} = V \times 22 \text{ for } 2.5 \text{ kg/cm}^2 \text{ PVC pipe}
\]

Thus a change in line velocity of 1.0 ft/sec occurring with the time \( T \) will cause a pressure rise at the value of 22 ft of head and subsequently (cyclically) a pressure fall of 22 ft.

D.3.44 The problem is to determine the amount of the velocity change \( V \) occurring within the time \( T \). As discussed in Annex D4, the rate of change of velocity in the pipeline is not uniform over the period of valve closure. It is much greater towards the end of the valve travel than initially. In the annex it is suggested that the rate of velocity change during the round-trip time \( T \) be taken for purposes of estimating pressure rise, as three times the average rate over the whole of the valve travel. The example given is a reduction in line velocity from 3 ft/sec to 1.5 ft/sec during a period of valve travel of 15 seconds. The average rate of change of velocity is 0.1 ft/sec per second. The round-trip time \( T \) in the case taken was 3 seconds. The rate of change of line velocity during time \( T \), at the moment of maximum pressure rise, is taken as \( 3 \times 0.1 \text{ ft/sec, or } 0.3 \text{ ft/sec.} \) The change in velocity during time \( T \) of 3 sec is then \( 3 \times 0.3 \) or 0.9 ft/sec. The corresponding pressure rise is \( 0.9 \times 22 \) or 20 ft of head.

D.3.45 The factor of three suggested above is a considerable approximation. Computer analysis of Kallada systems indicates the possibility of higher values. In the circumstances it is desirable to have substantial margin in valve closure time, thus reducing the velocity change during the round-trip time \( T \). The time \( T \) itself can be reduced in some situations by installation of a surge riser nearer to the valve. However, topographic factors may limit this possibility and valve closure time remains the variable which can be controlled to any degree desirable. Control must be exercised by positive mechanical means, such as geared reduction or a simple arrangement of the type shown on Plate D3.
D.3.46 It would be physically possible to close both valves on one hydrant together, but the likelihood of this being done with sufficient precision in timing to cause their peak pressures to occur simultaneously is small, and it is suggested that closure of a single valve be taken as the normal design criterion. In checking water-hammer, it must be recognized, however, that the discharge from a 50 mm gate-valve particularly on the portions of the pipe system which are under high head could be substantially more than 2.5 litres/sec. Alternatively, the valve may be only partly open in order to regulate flow to 2.5 litres/sec, and in this case the time to complete closure from the part-open condition could be considerably less than the time required for closure from the fully-open condition. In this connection, it is preferable to use smaller sized valves in the high head locations rather than limiting discharge by orifice plate. The smaller valve will have further to travel in effecting closure than a larger part-open valve, and effective closure time will be greater.

D.3.47 As discussed in Annex D4, the features which can be incorporated in a pipeline design for the control of water-hammer include the following:

a) Positive control of rate of closure of valves where required.

b) Installation of surge risers.

c) Keeping the line sufficiently below the steady-state hydraulic gradient (for long runs under low head) so that negative pressure waves do not reduce the net pressure in the line below atmospheric pressure.

d) Installation of mechanical devices such as pressure relief valves and vacuum relief valves.

D.3.48 Surge risers are a very desirable feature on a pipeline where topography permits their installation. For risers to be effective, however, they must be connected either directly to the line or via a short up-slope branch preferably not more than 20 or 30 m in length. The top of the riser must be at, or above, canal level (FSL) at the outlet, or spill will occur when irrigation from the pipe system ceases. Further, the riser should not extend more than about 5 m above ground level (for structural reasons, rather than hydraulic). The diameter of the riser should not be less than that of the pipe to which it is connected, if it is to be effective in water-hammer reflection (the riser is not simply a vent). It should preferably be not less than 200 mm in diameter if minimizing of negative pressures (down-surge) is a factor. Risers have their principal application on long near-horizontal runs as described in (c) above. A surge riser is obviously not a solution for a long down-slope run, where the line may be 10 to 20 m below canal FSL, as the height of riser would be prohibitive.

D.3.49 The relative merits of slow-closing valves and of pressure and vacuum relief valves in control of water-hammer are much debated. The latter have particular application in pump-supplied systems where power failure and sudden stopping of pumps is a common cause of water-hammer. In canal-supplied systems such as Kallada, this problem does not arise. Sudden closure of the gate on the canal outlet to a Kallada pipe system would result only in draw-down of elevation of the free surface in the pipeline (paras D.3.23 and D.3.24) and a slow oscillation of pressure in the line. The potential cause of water-hammer in the Kallada system, in normal operation, is closure or opening of delivery valves. Positive control of
rate of valve closure is a very effective counter to water-hammer, preventing its occurrence at source.

D.3.50 Negative pressures, a potential cause of line collapse, again can most effectively be controlled by slow-closing valves, as the negative pressure is usually caused by reflection of an initial positive pressure wave due to valve closure. On long low-head pipe runs, where the problem of negative pressure is most frequently encountered, the topography usually permits installation of surge risers which shorten the wave round-trip time and reduce the magnitude of pressure variations. Vacuum valves may be considered in this situation, but a vacuum valve has the disadvantage of admitting air to the line, which itself can cause water-hammer on subsequent evacuation of the admitted air. Slow valve closure, and slow opening, assured by mechanical means, is a simpler solution.

Operation of System

D.3.51 The Kallada pipe distribution systems are intended to run in concert with their parent canals. Whenever the distributary or minor is flowing it will flow at full design capacity, together with all its outlets. Rotation of supply in periods of less than full demand will be achieved by on/off rotation of the distributary (for instance operation every second week, or for a limited number of days per week). While the system is designed for continuous supply from outlets to the pipe distribution systems as long as the distributary is operating, outlets always remaining open, it is recognized that cultivators are free to stop irrigating any time, simply by closing the delivery valves. This will cause the level in the outlet chamber to back up until flow to the outlet ceases. If this occurs at a sufficient number of outlets, spill will occur from the distributary, and provision must be made to accommodate such spill safely. The most frequent cause of spill will probably be cultivators declining to irrigate at night. However, supply to valley-bottom areas at night and valley-slope areas in daylight hours is expected to become the practice, as irrigation of the bottom lands does not require constant attention.

D.4 KALLADA PROJECT - DOWNSTREAM (NEAR-FLAT) PORTION OF SERVICE AREA

Introduction

D.4.1 Towards the lower end of the Kallada project, the topography changes from valley slopes, with intervening bottom-lands, to the near-flat coastal plain. The latter area is partly in garden lands (principally coconuts and plantains) on either lateritic or sandy soils, and partly in extensive areas of paddy, also on sandy soils. The garden lands, generally not more than 1 or 2 m higher in elevation than the paddy areas, have minor topographic relief but little if any systematic slope.

D.4.2 This lower area poses two problems from the viewpoint of irrigation distribution:

- The head available between FSL in the distributary canal and the higher areas of a typical outlet command is commonly no more than 20 cm. This very limited amount of head must cover head losses at distributary outlet, regulating structures, farm turnout, friction head in the conveyance conduit, and still leave at least 5 cm of head between farm channel and field. Conservation of head is of paramount importance in this situation.
Land values in this area are exceptionally high, commonly in the range of Rs300,000/ha to Rs500,000/ha. The high value is due to potential use of land for urban development. Cost of right-of-way for construction of open watercourses or field channels and resistance to open channels because of their interference with access and adverse affect on property values are important factors.

D.4.3 From the right-of-way viewpoint, buried-pipe distribution has considerable merit and is probably the only practical solution in this area. On the other hand, the very small amount of head available requires use of considerably larger pipe than employed in the upstream portion of the Kallada command, and pipe cost becomes a key factor in design of the system.

D.4.4 Delivery to the individual palm by hose, a necessary feature of Upper Kallada, is neither necessary nor possible in the lower area. The topography is sufficiently flat to permit use of farm channels for field distribution, and the small amount of head available rules out the use of hose unless assisted by portable diesel pump set.

D.4.5 As a corollary to the use of farm channels, the rate of delivery (farm stream) must be great enough for efficiency in conveyance and field application. In these sandy soils, this implies a minimum of 10 litres/sec, preferably more, rather than the 2.5 litres/sec delivered by hose in the Upper Kallada area.

D.4.6 The following discussion and design proposals are based upon topographic surveys of a very limited sample of the Lower Kallada command. The areas surveyed are believed to be reasonably representative of the area as a whole, but situations requiring some departure from the proposals may be encountered as surveys proceed further.

D.4.7 The Lower Kallada area is very similar in most respects to the proposed Kallada Stage II command, which is adjacent, and to the west of Stage I. Design proposals and cost estimates developed in these notes for Lower Kallada (Stage I) may also be utilized in preparation of Stage II.

Irrigation Requirements - Separation of Supply to Garden Lands and to Paddy Areas

D.4.8 In the upstream portion of the Kallada project, most distributary outlets serve commands which are partly in garden lands, and partly in valley-bottom lands. Water-table is generally at shallow depth in the latter areas due to restricted sub-surface drainage, and water requirements for paddy are consequently modest. In the lower portion of the Kallada command, however, the position with regard to irrigation requirements of the paddy areas differs considerably from the upper area. Due to their low elevation and proximity to the coast, the water-table is at or above ground surface in the monsoon season. However, it falls rapidly to 2 or 3 m below ground level in the dry season and, due to the high permeability of these very sandy soils, irrigation of paddy in the dry season would require heavy water use. The question of what to grow on these soils in that season, the means of water distribution and the possible role of shallow wells in irrigation of the paddy areas are yet to be decided upon. It is consequently suggested that supply to paddy areas in Lower Kallada be separated from supply to garden lands, paddy lands being supplied directly from the distributary or minor.
0.4.9 Peak water requirements for 100% garden lands derived in Annex D1 are 0.8 litres/sec/ha. The figure is at the outlet from the distributary, assuming delivery from outlet to plant by buried pipe and hose, and 70% plant use efficiency. It will be recalled that the capacity of the distributary system is 1.0 litre/sec/ha, based upon a crop mix of 67% valley slopes and 33% valley-bottom lands. As the lower portion of the Kallada area will probably employ unlined farm channels rather than hose, overall irrigation efficiency can be expected to be fractionally lower than in the upper area. It is therefore suggested that the figure of 1.0 litre/sec/ha at the distributary outlet be retained for 100% garden lands in the lower area, providing for losses in farm channels. The figure is based on 24 hourly continuous (30 days/month) supply in the peak period.

Rate of Delivery to the Field and Size of Outlet Command

D.4.10 Delivery from pipe supply point to the field by unlined field or farm channel in sandy soils, over a distance of up to 100 m, calls for a delivery stream of at least 10 litres/sec if seepage losses in transit are to be kept within acceptable proportions. The upper limit is determined by the erodibility of these light soils and efficiency of water management on the field. A maximum of around 20 litres/sec is suggested.

D.4.11 The arrangement now widely adopted in India for distribution from canal outlet to farm by open watercourse or field channel (refer para D.1.8) involves delivery to one farm only at a time, within each outlet command. Each farm within the command area takes the whole flow in the watercourse in turn for a period of time determined by farm size and other factors. The rate of supply is consequently the same for all farms. The capacity of the watercourse, with this rotational system, is made equal to the desired delivery stream. The system has the advantage of avoiding the problem of division of the flow in the watercourse between two or more farms taking water at the same time. As discussed in para D.1.9, the same approach can be taken in pipe distribution systems, except where the delivery stream for particular reasons has to be unusually small (as in the case of delivery by hose in Upper Kallada).

D.4.12 It is recommended that the principle of rotational supply to one farm at a time, with the capacity of the watercourse system (whether open channel or buried pipe) made equal to the delivery stream, be adopted for Lower Kallada.

D.4.13 The upper and lower limits in size of delivery stream, taken together with the design water duty (1.0 litre/sec/ha), determine the range of size of outlet command. Thus, a delivery stream of 10 litres/sec continuously flowing could supply 10 ha. A stream of 20 litres/sec could supply 20 ha. Between these limits the area of outlet command may be selected to fit topographic and other constraints, and the size of delivery stream is matched to that area. Thus, for a 12 ha command, the delivery stream selected would be 12 litres/sec.

D.4.14 If all outlet commands are kept within the 10-20 ha size range, operation of the distributary and outlets is greatly simplified in that all outlets operate together, with the distributary, and opening and closing of individual outlets is not required. However, constraints of topography and property boundaries are likely to present situations in which an area of less than 10 ha cannot be combined with another adjacent area without an excessively long run of connecting channel or pipeline. As it is undesirable to reduce the delivery stream to less than 10 litres/sec, it is suggested in
these circumstances that the 10 litres/sec capacity be retained, with the intention of either eventually rotating supply to the outlet in question (involving weekly operation of the outlet gate, an undesirable solution) or simply running continuously with other outlets. In the latter case, the volume of water delivered seasonally, per hectare, would be greater for the small outlet command than for the remainder of the project, a situation which could be accepted in the interests of operational simplicity provided that such cases were exceptional. A lower limit to size of outlet command should, however, be imposed. A minimum of 5 ha is suggested.

Alternative Distribution Systems - Open Channel and Buried Pipe

0.4.15 The alternative types of construction which could be considered for distribution within the outlet command include the following:

- Lined trapezoidal channels (unlined channels for the watercourse system are not considered, due to the high seepage losses which would occur in these light soils. Unlined construction is confined to the short farm channels).

- Rectangular flumes of concrete, brick or masonry, the base of the flume being at ground level. (Flumes supported at intervals on piers, to minimize right-of-way requirements, are not applicable in this area due to the low elevation of water-level in the distributary.)

- Buried low-pressure pipe, either of PVC or of concrete.

The buried-pipe alternative is considered first and is subsequently compared with open-channels.

Pipe Materials

0.4.16 The alternative materials are PVC, or lightly reinforced (NP2 classification) spun concrete pipe. High strength unreinforced concrete pipe made by the "packer-head" process would also be suitable technically but is not available. As will be established later in these notes, PVC pipe of the pressure classification which would be used in this application (2.5 kg/cm²) has a cost advantage over concrete pipe, except possibly for the largest sizes required, if the pipe is manufactured from duty-free resin. Exemption from duty or excise on resin is applicable in the case of PVC pipe procured for World Bank financed projects in India, provided that International Competitive Bidding (ICB) procedures are followed. Pipe so procured to date has been of Indian manufacture, utilizing imported resin. As the cost of pipe procured through ICB procedure is about 40% less than through regular purchase at government-approved rates, such procurement is of particular importance. In the remainder of this discussion, ICB pipe costs are assumed. Layouts are based upon the use of PVC pipe, but cost comparisons with concrete pipe in the larger sizes are also made.
Pipe System Layout

D.4.17 As noted earlier, the head available from distributary to the field in the Lower Kallada area may be as little as 20 cm. For present purposes, it is assumed that friction loss in the pipe system when supplying the farthest field from the distributary outlet should not be more than 10 cm, the remaining 10 cm being made up of 5 cm in losses at structures and in the farm channel, and 5 cm head on the field itself. As the length of a 20 ha outlet command may be from 500 m up to 1,000 m, this requires limiting the friction gradient to as low as 0.1 per 1,000. This is a very low gradient for a pipe system.

D.4.18 One means of keeping pipe size and cost within acceptable limits is to employ, where applicable, the closed-loop type of layout. This is particularly advantageous in rotational supply systems where water is delivered at one point only at a time. In such a situation, flow to the point in question is around both arms of the loop, thereby considerably reducing the line flow in comparison with the alternative branching system in which the full flow must be taken by one line only. 1/ The loop arrangement makes particularly economical use of pipe both in respect to conveyance and in serving the desired delivery points with minimum line length. Whether or not the loop type of layout is applicable to a particular outlet command will be determined by the configuration of the command. In most cases in the Lower Kallada area, particularly for the larger outlet commands, it is anticipated that the loop type of layout will be adopted. Three such layouts are illustrated on Plate 04, also a case in which a branching system is more appropriate. Actual outlet commands are likely to be more irregular in shape than those illustrated in Plate 04 and consequently pipe length and cost per hectare will be rather greater.

Points of Delivery from Pipe System - Location and Area Served

D.4.19 In the upstream portion of the Kallada project, the spacing of delivery points (hydrants in that case) is influenced by limitations in the practical length of portable hose and the relatively small (2.5 litres/sec) delivery stream. The area served per hydrant is around 0.5 ha. In the downstream portion of the project command, such close spacing is not proposed. As the latter topography permits use of field channels for distribution from delivery points, and as the size of the delivery stream is considerably greater, an area of supply from a delivery point of around 1 to 2 ha is suggested. Density of delivery points will vary with topographic conditions, size of holdings, location of boundaries, and other factors. Length of field channel from supply point to farm is an important criterion and as far as possible should be kept at not more than 100 m. Delivery points should, where possible, be located at the intersection of property boundaries to provide direct access by irrigation. Where there is appreciable topographic slope, the delivery point must be at the high point of the area served to permit gravity distribution to all points within that area. From the viewpoint of efficiency of water delivery and cultivator convenience, the greater the number of delivery points the better. However, closer spacing of delivery points implies increased length of pipe, except where points are located along a straight run of pipe and do not require the

1/ Although the flow in the two arms will generally be unequal, and dependent upon the location at the point being supplied, the critical condition for determination of pipe size is with supply to a point at the farthest position on the loop. In this situation, the flow in each arm is half the total delivery stream.
addition of branch lines. The delivery points themselves are also a significant cost factor.

0.4.20 The question of density of delivery points will be resolved only after survey and design of a considerable number of outlet commands. As discussed for Upper Kallada there may be a case for additional outlets, and associated branch pipeline, if any, being constructed at the request and cost of individual cultivators or neighbours.

Regulation of Flow to Pipe System

0.4.21 The small amount of head available between distributary and field in the Lower Kallada area has been referred to earlier. This is a particular constraint in the regulation of flow to the outlet command. In conventional regulation where more head is available the type of control structure used in the Upper Kallada area is employed. The control in that case is a gate or valve at the downstream end of the diversion culvert functioning as an orifice operating under critical flow (non-submerged conditions). The head loss through the gate (about 15 cm) is such that any likely variation in level in the distributary will not cause a major change in flow through the outlet. Measurement of flow in the same structure is by Vee notch, also operating under non-submerged conditions implying a further head loss of some 5 cm. Such a head loss at the canal outlet is obviously not acceptable in the Lower Kallada system. In fact, any type of weir involves some head loss, and the lower the head on the weir (or orifice) the more sensitive it is to level in the supply distributary, which in this case may vary by at least 5 cm from nominal design FSL.

0.4.22 In the circumstances, the system suggested for Lower Kallada does not involve regulation at the outlet, but at the delivery point. This is practicable as each point in turn takes the whole flow. The control at the delivery point is an alfalfa-type valve, or a slide gate, the maximum degree of opening of the valve being capable of being pre-set and locked. This adjustment is made, for each delivery point in turn, at the time of commissioning of the pipe system. The use of a portable low-head "duck-billed" weir of the type shown on Volume I, Plate D6, is suggested for this purpose. The actual setting of the maximum degree of opening of each delivery valve will depend upon its elevation, and the distance from the outlet (i.e. the pipe friction loss to that point). As only one valve will be in operation at a time, correct setting of maximum opening of each valve provides control of diversion to the pipe system as a whole. With regard to the effect of variations in level in the distribution canal from nominal, it is noted that the pipe system and delivery valve function as a composite hydraulic unit, with flow proportional to square root of head. A variation of 5 cm in 20 cm of operating head would consequently result in a variation in rate of delivery of some 10% only, which is acceptable.

0.4.23 The alfalfa-type valve required should be designed for very low head loss when fully open. If a suitable valve is not available commercially, a valve along the lines indicated on Plate D7 could be fabricated by local supplier.
Structures - Canal Outlet and Delivery Points

D.4.24 As regulation and measurement of flow are not functions of the outlet structure with the arrangement proposed for Lower Kallada, provision need be only for the following:

- Trash screens, as described for Upper Kallada (para D.3.18).
- Outlet chamber cum sand-trap, also as for Upper Kallada Plate D2. (As line velocities will be much lower than in Upper Kallada, there will be a greater tendency for sand or silt to accumulate in the line. Size of sand-trap will consequently be of particular importance in Lower Kallada.)
- A simple slide gate operating in the open or closed position, and capable of being locked closed.
- A secondary fine screen covering the entry to the pipeline.
- The outlet chamber should be carried up to above distributary FSL as water-level in the chamber will stand at that level when all delivery valves are closed.

D.4.25 The delivery point could take the form of a Tee in the main line, and, a PVC riser connecting to the delivery valve, the latter being located in a concrete delivery chamber. However, in view of the cost of Tees for the sizes of pipe required in Lower Kallada, it may be found preferable simply to carry the concrete chamber down to pipe level and to provide direct connection of pipe into the chamber as indicated on Plate D6. One or more outlets from the chamber supply the field channels which it serves.

D.4.26 Changes in direction of the pipeline will generally be located at, and incorporated in, the delivery chamber.

D.4.27 The only other structures required are one or more surge risers, carried up to distributary FSL. In view of the relatively flat topography, location of surge risers should present no difficulty. Line velocities are in any case low in the Lower Kallada systems (around 0.25 m/sec) and water-hammer should not be a problem.

Hydraulic Design

D.4.28 A typical longitudinal profile of part of a loop system, with delivery points, is shown on Plate D5. Friction gradients for PVC pipe in the size range of interest for Lower Kallada are set out in Annex D3. The figures assume a value of 150 for the Hazen-Williams roughness coefficient C.
Preliminary Cost Estimates

0.4.29 Unit prices for PVC pipe vary with changing international prices for resin, and with rate of exchange. The most recent available prices based on ICB tenders were for delivery in Northern India under a contract awarded in September 1984. Prices for 2.5 kg/cm² pipe in sizes 160 and 200 mm were approximately Rs25/m and Rs39/m delivered, equivalent to a cost of Rs13.2/kg. 1/ ICB prices for larger size of pipe are not directly available at the time of writing, nor for 160 mm and 200 mm pipe for delivery in Kerala. Pending more specific information, a cost of Rs16/kg for delivery at works site in Kerala, under ICB procurement, has been assumed. Cost of trenching, installation of PVC pipe, and backfill has been estimated at Rs12/m. The resulting unit costs are as follows:

<table>
<thead>
<tr>
<th>Pipe size outside diameter (mm)</th>
<th>Weight per metre of 2.5 kg/cm² pipe (kg)</th>
<th>Cost per metre Delivered to site (Rs)</th>
<th>Cost per metre Installed (Rs)</th>
</tr>
</thead>
<tbody>
<tr>
<td>160</td>
<td>1.88</td>
<td>30</td>
<td>42</td>
</tr>
<tr>
<td>200</td>
<td>2.95</td>
<td>47</td>
<td>59</td>
</tr>
<tr>
<td>225</td>
<td>3.81</td>
<td>61</td>
<td>73</td>
</tr>
<tr>
<td>250</td>
<td>4.60</td>
<td>74</td>
<td>86</td>
</tr>
<tr>
<td>280</td>
<td>5.89</td>
<td>94</td>
<td>106</td>
</tr>
<tr>
<td>315</td>
<td>7.42</td>
<td>119</td>
<td>131</td>
</tr>
</tbody>
</table>

0.4.30 The following cost figures for concrete pipe (NP2 classification) are indicative only, but may be used for comparative purposes:

<table>
<thead>
<tr>
<th>Pipe size inside diameter (inches)</th>
<th>Cost per metre At project site (Rs)</th>
<th>Cost per metre Installed (Rs)</th>
</tr>
</thead>
<tbody>
<tr>
<td>12 (305 mm)</td>
<td>105</td>
<td>120</td>
</tr>
<tr>
<td>15 (381 mm)</td>
<td>140</td>
<td>155</td>
</tr>
<tr>
<td>18 (457 mm)</td>
<td>170</td>
<td>190</td>
</tr>
</tbody>
</table>

For larger sizes concrete pipe may prove more favourable. For present purposes, however, PVC pipe is assumed throughout.

0.4.31 Estimated direct costs of the four cases shown on Plate D5 are set out in Annex D3. Direct costs per hectare are in the range of Rs6,000 to Rs7,500 at levels.

1/ Subsequent prices (July 1985) were about 12% lower.
Comparison of Buried Pipe and Lined Open Channel Systems for Lower Kallada

D.4.32 The main factors entering into evaluation of the two alternative systems are:

- technical effectiveness;
- construction cost and maintenance;
- right-of-way cost and resistance of cultivators to provision of right-of-way for open channels.

D.4.33 For the moment it is assumed that both pipe and lined open channel or flume systems can be equally effective as far as delivery of water is concerned, provided that they deliver to the same size of sub-area, which in Lower Kallada is some 1 to 2 ha. Construction costs for pipe systems, for outlet commands of 20 ha and less, have been estimated above. The direct cost per metre of pipeline, installed, using 250 mm pipe, is estimated at Rs86/m. Distributing the cost of the valved delivery points (which are an essential feature of the pipe system) on a per metre basis, this figure increases to approximately Rs100/m.

D.4.34 The layout of an open channel system would not necessarily correspond to that of a buried pipe system. It is assumed that delivery would be made to the same points in both cases, but the open channel would probably have to follow property boundaries to a greater extent than buried-pipe, for right-of-way reasons, and might consequently be of greater length. However, for present purposes it is assumed that the length is the same in both cases (equivalent to about 55 m/ha) and comparison can be made on the basis of cost per metre.

D.4.35 Current costs for construction of lined open channels and flumes of the capacity in question are available from a number of projects elsewhere in India. They vary regionally, but are broadly in the range of Rs40/m (lined trapezoidal section) to Rs90 (half-round concrete or concrete flume). Brick flume construction is around Rs60/m. These costs are without right-of-way or earthwork.

D.4.36 The amount of right-of-way required varies with type of channel. A trapezoidal channel with minimum embankment on either side would require a minimum of 2 m width of right-of-way. A flume section could possibly be constructed on 1 m width of acquired right-of-way, the area physically occupied being possibly marginally less. Where either type of channel is in fill (crossing low areas), the width of right-of-way required is considerably greater. Right-of-way can be valued at the market rate per square metre, which in Lower Kallada is between Rs30 and Rs50. However, this does not reflect the fact that an open channel crossing or traversing the boundary of a property is a considerable impediment to access or to future subdivision for building purposes. It is not possible to evaluate this factor in financial terms but it can weigh heavily in terms of cultivator resistance to development of a project.

D.4.37 Considering market value only, and assuming this to be Rs30/m2, the cost of open channel ranges from (40 + 2 x 30) or Rs100/m for lined trapezoidal section to (90 + 30) or Rs120/m for concrete flume section. Brick flume costs approximately (60 + 30) or Rs90/m. Cost of gated turnouts from the surface channel (the equivalent of valved delivery points on the pipeline) adds approximately Rs5/m to these figures.
D.4.38 It is evident that the initial cost per metre of lined channel in the Lower Kallada situation is not less than the cost of buried pipe. Subsequent maintenance costs, and the adverse effect of open channels on property values favour pipe.

Conclusions

D.4.39 It is suggested that buried-pipe be employed in Lower Kallada for outlet commands of 20 ha or less. Lined open channels may be required in some cases (as minor canals) where required to reduce outlet commands to this size.

D.4.40 The above examination of the use of buried pipe for distribution in near-flat areas is of significance in the design of irrigation systems generally. If PVC pipe is available at a cost corresponding to internationally traded prices for resin, 1/ the use of buried-pipe for distribution within areas of up to 20 ha becomes attractive even in virtually flat areas where the only head available is the 20 or 30 cm provided by the distributary. Crops referred to are other than paddy, i.e. the irrigation duty considered is around 1 litre/sec/ha. The higher duty required for paddy in most cases would weigh against use of pipe distribution in flat areas where supply is by gravity. Use of pipe for paddy and mixed cropping is appropriate, however, for tubewell river-lift systems, or where there is significant natural gradient.

D.4.41 The use of buried-pipe for quaternary distribution should be investigated routinely as an option in the design of new irrigation systems, particularly in areas of smallholdings or high land values.

---

1/ Without major excise on imported resin, which in India is some 140% (unless exempted).
KALLADA PROJECT - Pipe Systems

FIG. 1

FIG. 2

FIG. 3
KALLADA PROJECT
OUTLET FROM DISTRIBUTARY

15 x 25cm OUTLET FOR DIRECT SUPPLY TO PADDY (OPTIONAL. SEE TEXT)
KALLADA PROJECT

Attachment to 50 mm gate valve (non-rising stem) to ensure slow closure

ELEVATION B-B

PART PLAN A-A

Key: 10 cm length

Slot engages on top valve stem

Valve closure key

Pin engages on grooves in valve wheel (See Plate 3)

Stops limit travel of key, per stroke

(Not to scale)
DESIGN AND OPERATION OF IRRIGATION SYSTEMS FOR SMALLHOLDER AGRICULTURE
IN SOUTH ASIA. VOL. II
LOWER KALLADA
SCHEMATIC LAYOUTS

CASE B
(20 Ha)

CASE A
(20 Ha)

CASE C
(10 Ha)

CASE D
(10 Ha)

BOUNDARY
PIPELINE AND DELIVERY POINT
UPPER KALLADA

LONGITUDINAL PROFILE OF PART OF LOOP

ALFALFA VALVE

SCHEMATIC OF DELIVERY POINT
DESIGN AND OPERATION OF IRRIGATION SYSTEMS FOR SMALLHOLDER AGRICULTURE IN SOUTH ASIA. VOL. II

LOWER KALLADA
DELIVERY CHAMBER

(See Plates D7 and D8)

Additional unit if required
LOWER KALLADA DELIVERY CHAMBER
(See Plates D6 and D8)

PLAN VIEW

ALFALFA VALVE ASSEMBLY
REQUIRED ONLY WHERE GROUND ELEVATION IS MORE THAN 1.5 m BELOW DISTRIBUTARY FSL
(INSTAL BETWEEN 65 cm BOTTOM BLOCK AND THE 50 cm OUTLET BLOCK)

DETAIL OF GATE GROOVE

SECTION AT BASE
(BEVEL-CUT PIPE AS REQUIRED)
DESIGN AND OPERATION OF IRRIGATION SYSTEMS FOR SMALLHOLDER AGRICULTURE IN SOUTH ASIA. VOL. II

LOWER KALLADA

DELIVERY CHAMBER
Detail of Alfalfa Valve

(See Plate D7)
SOILS, CROPPING PATTERNS, AND WATER REQUIREMENTS

Soils and Cropping Patterns in Valley Floor and Valley Slopes

1. The term "valley floor" is used here, rather than "wetland", as even under pre-project conditions 20% of the valley floor is occupied throughout the year by annuals and perennials, and a further 18% by "irrigated dry" seasonal crops in the summer. Under "with project" conditions the latter figure is expected to rise to some 68%.

2. In the upper half of the project area, most of the valley soils are colluvial formed by erosion from the adjacent slopes. The valley lands generally have a slope of at least 1% downstream. There is consequently good surface drainage, supplemented in most cases by a substantial dug channel down the middle of the valley. This is used both for drainage and for irrigation (by return flow). Infiltration rates in both upper and lower portions of the project area vary seasonally with elevation of water-table. The latter is near surface level in the wet season in valley floor lands throughout the project command. In upper areas an underlying layer of compact clay contributes to keeping the water-table under paddy relatively high even if in the dry season, restricting the rate of the infiltration. The long narrow configuration of the valleys and the low permeability of the underlying strata ensure that such infiltration as does occur in head-reaches effectively raises water-table in lower reaches. Some 7% of the valley floor is at present under paddy throughout the summer season, supplied only by drainage from higher areas.

3. With the exception of such local lower-lying areas, the valley soils of the project are sufficiently free-draining to permit either paddy or non-paddy crops to be grown, or for non-paddy crops to follow immediately after paddy (as is practised in some 20% of the valley lands). The trend to replacement of paddy in the valley lands by perennials (particularly coconuts) and annuals (plaintains) is evident throughout. Cultivation practices in such areas include use of deep drainage furrows with low intervening beds (30 to 40 cm in height). It is notable that such simple practices are effective in controlling topsoil drainage conditions even under the very heavy rainfall of the project area (2,500 mm in the six-month wet season), which in heavier soils would prohibit the introduction of these crops in such a topographic situation. The financial incentive in such conversion to plantation crops is very considerable (net returns are increased several-fold), particularly in view of the relatively high labour costs in Kerala.

4. The subject of cropping patterns in the valley floor lands of the project raises a number of design and operational questions including the following:

a) The range in crop mix to be considered including the proportion of annual and perennial crops versus kharif and rabi paddy, and the proportion of non-paddy seasonals versus summer paddy.

b) Water requirements of rabi and summer paddy, with particular reference to the influence of water-table elevation on effective infiltration rate.
c) The proportion of valley floor to valley slope lands in a "10 ha" outlet command, the range of values likely to be encountered in project design, and the influence of this proportion on water requirements.

d) The month-by-month variation in irrigation requirements of crops on valley floor lands and on valley slopes, and the extent to which supply can be tailored to this varying demand. Operational means of supplying water at a rate less than full canal capacity.

e) The basis of determining the entitlement of individual cultivators, bearing in mind the different needs of cultivators on valley floor and valley slope lands, and the associated questions of control and measurement of water use by the individual or group, and the method of charging for water.

5. Table 1 summarizes (Column a) the present and projected "with project" (average) (Column b) cropping patterns, the crop areas being expressed as a percentage of the cultivable area of valley floor and of valley slopes respectively.

6. The cropping pattern on valley slopes does not call for comment. Annual crops (tree crops and others) already occupy nearly 80% of the slopes, and this proportion will be extended to 99% under the project; the anticipated returns will derive from irrigation of currently rainfed crops. In the valley floor the project will provide water for:

- Supplemental irrigation of kharif paddy.
- Irrigation of rabi paddy, particularly in late November and December after withdrawal of the monsoon.
- An anticipated major increase in summer pulses, sesameum, groundnuts and vegetables (from 18% of the valley floor to 68%).
- A possible increase of summer paddy from some 7% to 11% of the valley floor area.

7. As discussed above, the current trend is towards an increasing proportion of perennials and annuals in the valley floor lands with corresponding reduction in kharif and rabi paddy, and possibly of summer paddy. However, summer paddy is likely to continue to occupy particularly low-lying areas of heavier soils or locally restricted drainage although at present price levels and costs of production it is much less attractive than other alternative crops. While the area under summer paddy may well remain unchanged or be reduced under project conditions, for the purpose of examining irrigation requirements in a variety of possible situations the case of summer paddy on 22% of valley floor lands has also been considered (Table 1, column d and para 10). This may be compared with the present 7% and the projected 11%.

8. The remaining variants examined are variations in the ratio of valley floor lands to valley slopes in the command of an outlet.
9. The anticipated project-wide proportions of valley floor and valley slope lands to total project command are 33% and 67% respectively. However, the command of individual outlets may range from 100% valley floor to 100% valley slope.

10. The cases considered here are the following:

Column c - Valley floor lands 100%. Cropping pattern for valley floor as in column b.

Column d - Valley floor lands 100%. Assumed percentage of summer paddy increased from 11% to 22%.

Column e - Valley slope lands 100%. Cropping pattern for valley slopes as in column b.

Column f - Valley floor 33%, valley slopes 67%. Cropping patterns as in column b. The nominal case.

Column g - Valley floor lands 60%, valley slopes 40%. Cropping patterns as in column b.

Irrigation Requirements with Varying Proportions of Valley Slopes

11. For present purposes, water requirements for all non-paddy crops have been taken, with some approximation, as numerically equal to the calculated consumptive use (ETo) of the Penman standard reference crop. The principal uncertainty in the case of non-paddy crops is with regard to coconuts, for which little information is yet available on crop/water response. However, the assumption made is in line with known consumptive use of other orchard crops.

12. Of more significance in project design is the estimation of irrigation requirements for paddy in the climatic and soils conditions of the Kallada project. Both kharif and rabi paddy are at present entirely "rainfed", although the term is not strictly appropriate as precipitation on the valley floor is substantially augmented by runoff from the adjacent valley slopes.

13. Values of mean monthly precipitation averaged between Trivandrum and Cochin, which lie on either side of the project area, are shown in Table 2. It is apparent that land preparation for kharif paddy in May and June requires little assistance from irrigation, as total precipitation in the two months is usually in excess of 600 mm. Minimal irrigation releases in these months, and through October are proposed. Harvesting of the kharif paddy crop in August and September is necessarily carried out in wet conditions, as rainfall in each of these months is likely to average some 8 mm per day. Re-puddling for the follow-on rabi paddy crop, and transplanting, occur in September and October. This is carried out under rainfed conditions (some 500 mm precipitation in the two months), assisted by diversion of runoff from the valley slopes. As the soil is likely to be at or near field moisture capacity and water-table at shallow depth

1/ 20,400 ha of valley floor and 41,200 ha of valley slopes.
following the kharif crop, water requirements to bring fields to the saturated condition for re-puddling are likely to be relatively small. Projected releases of water to supplement rainfall in September and October are consequently small and much less than system capacity.

14. Peak irrigation requirements for rabi paddy occur in December, by which time the monsoon has fully withdrawn. This is the critical month for water supply in most of the cases considered (i.e. other than 100% valley slopes). Estimation of irrigation requirements for paddy in December is consequently of particular importance. With an ETo value of 4.2 mm/day, 1/ evapo-transpiration from paddy is likely to be no more than 5 mm/day. The more difficult item to estimate, or to measure, is infiltration from the paddy field. As discussed earlier, this is influenced very much by height of water-table, and in much of the valley floor lands the latter is determined in part by a clayey horizon at shallow depth. Although the top-soils of the valley floor are relatively free-draining per se, the influence of the clay subsoil causes them to be described as being imperfectly drained and having low to medium infiltration rates. Determination of infiltration rates by double-ring infiltrometer, particularly if carried out towards the end of the dry season when water-table is at its lowest, usually gives values more than ten times as great as those determined by field ponding under large-scale irrigation. The measured values from ring infiltrometer tests carried out on sandy soils in the lower portion of the project area in April illustrate this problem. They are as high as 50 cms/day, in a situation in which actual infiltration under paddy in the wet season, constrained by high water-table, is a few millimetres only. Infiltration rates to be assumed under extensive irrigation for project design purposes remain very much a matter of judgement and evaluation of local conditions. Thus in the long narrow valleys of the Kallada command and with low permeability strata at shallow depth, down-valley sub-surface flow must be relatively small, the valley acting as a semi-closed basin. In such circumstances a high rate of infiltration would rapidly bring the water-table to the surface, at which time infiltration would virtually cease, regardless of the inherent permeability of the soil. In estimating water requirements for paddy percolation rates used were 6.5 mm/day for kharif, 7 mm/day for rabi, and 10 mm/day for summer paddy, in loam soils. These values are believed to be very conservatively high in the project conditions described. Actual rates are expected to be not more than half these figures, with the possible exception of summer paddy. However, as summer paddy is likely to be grown principally in low-lying poorly drained areas, even with this crop infiltration is likely to be low.

15. In estimating crop water requirements for the purpose of these notes, the infiltration rate has arbitrarily been taken as 6 mm/day for rabi and summer paddy (the question does not arise in kharif when the water-table is at the surface over most of the area). However, this figure is regarded as an upper limit as far as the project area as a whole is concerned.

16. Irrigation requirements for a nominal 10 ha area with the five situations listed earlier are listed hereunder. As the purpose of the computation was to check peak required rates of delivery, only the non-monsoon months have been examined. The following conclusions may be drawn:

1/ Temperature and ETo values in the project area do not vary widely throughout the year in Kerala and even in summer months ETo remains less than 5 mm/day.
Case 1  (Column b) This is the "project average" case, with regard to proportion of valley floor and valley slopes. It has the project design cropping pattern. Calculated peak water requirement at the outlet from the distributary is at the rate of 0.96 litres/sec/ha, in December, followed by 0.86 litres/sec/ha in March. This broadly confirms the project design figure of 1 litre/sec/ha at the outlet.

Case 2  (Column e) This assumes that the whole 10-ha area is in valley slopes. The peak month is March, with water requirement at the outlet from the distributary of 0.81 litres/sec/ha, almost all of which is devoted to irrigation of perennials.

Case 3  (Column c) This is the opposite case from 4, i.e. the whole 10 ha area is in valley floor lands. The peak requirement in December is 1.06 litre/sec/ha at the plant, equivalent to 1.43 litres/sec/ha at the outlet from the distributary (or minor). The figure is very much influenced by the value assumed for infiltration. As an area entirely in valley floor lands would generally be served by open channel watercourse rather than pipe, the design outlet capacity could be provided with some additional margin at little cost, possibly up to 1.7 litres/sec/ha. The rate of flow actually required, and delivered, would be determined after going into service.

Case 4  (Column d) This also has the whole 10-ha area in valley floor lands, but the area under summer paddy is doubled from 11% (Table 2) to 22%. December remains the peak month, and the required capacity is the same as in Table 2 (i.e. it is determined by the 76% of rabi paddy rather than the 22% of summer paddy).

Case 5  (Column g) This has 60% of the 10 ha in valley floor lands (compared with the project average of 33%). In this case, the peak requirement is 1.17 litres/sec/ha at the outlet from the distributary. This illustrates the fact that, for areas with more than some 50% of valley floor lands, the "project average" capacity of 1 litre/sec/ha at the outlet from the distributary may have to be increased. However, the pipe system would be limited to 50% or less of the area in valley slopes.
IRRIGATION FROM SUPPLY CANALS VIA BURIED PIPE DISTRIBUTION SYSTEMS

KALLADA PROJECT

Present and Projected Cropping Patterns in Valley Floor and Valley Slopes

<table>
<thead>
<tr>
<th>Present and Projected Cropping Patterns in Valley Floor and Valley Slopes</th>
</tr>
</thead>
<tbody>
<tr>
<td>% of area of total area (%)</td>
</tr>
<tr>
<td>Present</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>VALLEY FLOOR</th>
<th>Present</th>
<th>Projected</th>
</tr>
</thead>
<tbody>
<tr>
<td>Kharif - Paddy</td>
<td>72 74 74 74 0 24 44</td>
<td>74 76 76 76 0 25 46</td>
</tr>
<tr>
<td>Paddy</td>
<td>72 74 74 74 0 24 44</td>
<td>74 76 76 76 0 25 46</td>
</tr>
<tr>
<td>Vegetables</td>
<td>3 1 1 1 0 0 1</td>
<td>2 2 2 2 0 1 1</td>
</tr>
<tr>
<td>Rabi - Paddy</td>
<td>74 76 76 76 0 25 46</td>
<td>74 76 76 76 0 25 46</td>
</tr>
<tr>
<td>Paddy</td>
<td>74 76 76 76 0 25 46</td>
<td>74 76 76 76 0 25 46</td>
</tr>
<tr>
<td>Vegetables</td>
<td>2 2 2 2 0 1 1</td>
<td>2 2 2 2 0 1 1</td>
</tr>
<tr>
<td>Summer - Paddy</td>
<td>7 11 11 22 0 4 7</td>
<td>7 11 11 22 0 4 7</td>
</tr>
<tr>
<td>Paddy</td>
<td>7 11 11 22 0 4 7</td>
<td>7 11 11 22 0 4 7</td>
</tr>
<tr>
<td>Vegetables, sesamum, groundnuts,</td>
<td>18 68 68 68 0 22 41</td>
<td>18 68 68 68 0 22 41</td>
</tr>
<tr>
<td>vegetables</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Two seasons (Kharif &amp; Rabi)</td>
<td>0 1 1 1 0 0 1</td>
<td>0 1 1 1 0 0 1</td>
</tr>
<tr>
<td>Ginger</td>
<td>21 20 20 20 0 7 12</td>
<td>21 20 20 20 0 7 12</td>
</tr>
<tr>
<td>Perennial and annuals</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>VALLEY SLOPES</th>
<th>Present</th>
<th>Projected</th>
</tr>
</thead>
<tbody>
<tr>
<td>Annuals - Tree crops</td>
<td>63 80 0 0 80 54 32</td>
<td>63 80 0 0 80 54 32</td>
</tr>
<tr>
<td>Other</td>
<td>16 19 0 0 19 13 8</td>
<td>16 19 0 0 19 13 8</td>
</tr>
<tr>
<td>Summer season - Pulses and Vegetable</td>
<td>2 1 0 0 1 0 0</td>
<td>2 1 0 0 1 0 0</td>
</tr>
</tbody>
</table>

Crop calendar for paddy (Current practice)

- Kharif paddy: Transplant May - June
  Harvest Aug. - Sept.
- Summer paddy: Transplant Jan.
  Harvest March - April
IRRIGATION FROM SUPPLY CANALS VIA BURIED PIPE DISTRIBUTION SYSTEMS

KALLADA PROJECT

Mean Monthly Precipitation for 30-Year Period (mm)
as at Trivandrum and at Cochin

<table>
<thead>
<tr>
<th></th>
<th>J</th>
<th>F</th>
<th>M</th>
<th>A</th>
<th>M</th>
<th>J</th>
<th>J</th>
<th>A</th>
<th>S</th>
<th>O</th>
<th>N</th>
<th>D</th>
<th>Year</th>
</tr>
</thead>
<tbody>
<tr>
<td>Trivandrum</td>
<td>20.1</td>
<td>20.3</td>
<td>43.5</td>
<td>122.1</td>
<td>248.6</td>
<td>331.2</td>
<td>215.4</td>
<td>164</td>
<td>122.9</td>
<td>271.2</td>
<td>206.9</td>
<td>73.1</td>
<td>1839</td>
</tr>
<tr>
<td>Cochin</td>
<td>9.6</td>
<td>34.2</td>
<td>50.0</td>
<td>139.5</td>
<td>364.3</td>
<td>755.9</td>
<td>571.9</td>
<td>385.7</td>
<td>234.8</td>
<td>332.7</td>
<td>183.7</td>
<td>36.8</td>
<td>3099</td>
</tr>
<tr>
<td>Mean of Trivandrum and Cochin (rounded)</td>
<td>15</td>
<td>27</td>
<td>47</td>
<td>131</td>
<td>306</td>
<td>543</td>
<td>394</td>
<td>275</td>
<td>175</td>
<td>302</td>
<td>195</td>
<td>55</td>
<td>2469</td>
</tr>
<tr>
<td>Mean mm/day (rounded)</td>
<td>0.1</td>
<td>1</td>
<td>1.5</td>
<td>4.4</td>
<td>10</td>
<td>18</td>
<td>13</td>
<td>9</td>
<td>7</td>
<td>10</td>
<td>6.5</td>
<td>1.5</td>
<td></td>
</tr>
<tr>
<td>ETo mm/day (Trivandrum)</td>
<td>4</td>
<td>4.4</td>
<td>4.9</td>
<td>4.7</td>
<td>4.3</td>
<td>3.5</td>
<td>3.7</td>
<td>4.2</td>
<td>4.3</td>
<td>3.8</td>
<td>3.6</td>
<td>4.2</td>
<td></td>
</tr>
</tbody>
</table>

1/ These two stations lie immediately to the south and to the north of the project area. Rainfall records which have been maintained at four stations within the command area are not to hand at the time of writing.
SUPPLY OF WATER TO VALLEY-BOTTOM LANDS IN UPPER PORTION OF COMMAND AREA

1. The area supplied from a canal outlet can be of the following types:

   a) Purely valley slopes (garden lands), supplied entirely by pipe system.

   b) Purely valley-bottom lands supplied entirely by simple direct diversion from the outlet, without pipe distribution system.

   c) Mixed valley-slope and valley-bottom lands, both being supplied via a buried-pipe system (i.e. there is no opportunity for direct diversion to the valley-bottom land without use of the pipe system).

   d) Mixed valley-slope and valley-bottom lands, with opportunity for supply of the valley-bottom lands by direct diversion from the outlet, and alternatively if desired from hydrants on the buried pipe system.

2. The purely valley-slope case calls for no comment. In the case of mixed valley-slope and valley-bottom lands supplied by pipe (case (c)), the capacity of a line should be the greater of 1.0 litre/sec/ha over the whole area served by it, or 1.7 litres/sec/ha over the area of valley-bottom land only served by it. Further, line capacity should not be less than 5 litres/sec at any point. These criteria are in conformity with overall project design cropping pattern and projected water utilization. However, additional criteria are suggested. They arise from some unpredictability as to likely cropping patterns in the valley-bottom lands under irrigation. The bottom lands to be supplied by an outlet are very frequently in the form of a long narrow strip (e.g. 600 mm length x 50 mm in width across the valley). In the monsoon season and also in rabi when water-table in the valley bottom is likely to be near the surface, particularly under irrigation, water distribution in the bottom lands whether under paddy or non-paddy crops is unlikely to be a problem. Direct diversion from the canal outlet into the upstream end of the valley bottom, and distribution downstream by field-to-field flow or by groundwater flow is likely to be adequate, where there is possibility of such direct diversion. However, in the summer season when water-table in the valley bottom is likely to be low and when non-paddy crops will probably be of particular importance, diversion into the upstream end of the valley-bottom and distribution downstream by unlined field channel would be less effective, due to high seepage loss. In such circumstances, supply to the valley bottom from hydrants located at intervals along the length of the pipe system (extending down-valley) will probably be more efficient.

3. In considering the desirable rate of such supply, a distinction is noted between irrigation of garden crops on the valley slopes and irrigation of the same crops on valley-bottom lands converted from paddy. Thus, on the slopes, the use of hose and consequently delivery at a small rate of flow, but at a number of points simultaneously, is necessary. On the other hand, on bottom lands generally with ridge-and-furrow cultivation delivery at a higher rate and at one point at a time is desirable. Where paddy and non-
paddy crops both occur together in the valley bottom, as is commonly the case, a high rate of application is particularly required during land preparation for the paddy.

4. With the above discussions as background, the following additional criteria are suggested:

a) In the case of purely valley-bottom lands supplied by direct diversion from a canal outlet, without pipe system, substantial capacity should be provided. This can be done at nominal cost. Capacity should be at least 25 litres/sec (for instance a 250 mm diameter pipe outlet) for a 10-ha command in purely bottom land. With the necessary provision of a slide gate or valve (capable of being locked partly open), the actual rate of diversion at any time can be controlled and also the seasonal total. However, the capability of supplying at a relatively high rate during particular periods (for instance in land preparation for paddy, or in the early years of project development when a relatively high proportion of summer paddy may be accommodated) is thereby built into the system.

b) When there are both valley-slope and valley-bottom lands in the outlet command, but where it is physically possible to provide direct diversion to the land as well as the alternative of supply via hydrants on the buried pipe system, both should be provided. The gate-controlled or valve-controlled "spout" will discharge into an outlet box from which both a pipeline and a gated outlet to the direct diversion can be supplied (generally one or the other at any time, not both). The outlet valves which are intended to supply the bottom lands from points along the length of the pipe system should be of full line capacity at each point of supply. This may necessitate providing a special hydrant or outlet box with valve sized for 10 litres/sec at such points (unless line capacity at the point in question is in any case limited to 5 litres/sec which is the capacity of a regular two-hose hydrant). It is proposed to provide additional line capacity, above that called for in the criteria referred to earlier, but rather to make the outlet capable of delivering full line capacity at points of supply to bottom land.

c) When there are both valley-slope and valley-bottom lands in the outlet command, but it is not possible to provide direct diversion to the bottom land, outlets to bottom land from the pipeline should be of full line capacity. Under these circumstances, and where it can be done at little additional cost, it may be warranted to provide additional capacity in the portion of the line from its upstream end to the first outlet to the bottom land (e.g. to provide 15 litres/sec in a system otherwise designed for 10 litres/sec, down to the first outlet).
5. The importance of providing a flow of not less than 10 litres/sec where valley-bottom lands are supplied from a pipe system is emphasized. Water distribution to non-paddy crops will be by unlined open channel in the bottom lands, not by hose, and a flow of not less than 10 litres/sec is desirable for efficient distribution and application in such circumstances. In the case of paddy, distribution may be by semi-continuous field-to-field flow, but even with paddy a flow of 10 litres/sec is desirable during land preparation. The duration of the supply at that rate of the flow will of course depend upon the number of hectares of valley-bottom land and the water entitlement. Finally, as night irrigation is likely to be confined to valley-bottom areas, the pipe system should be capable of delivering the whole flow from the spout (i.e. the design capacity of the pipe system) to the valley floor during the night hours.

6. The above discussion of irrigation of valley-bottom lands refers to the middle and upper portions of the Kallada command where such lands are in alluvial soils. Irrigation of near-flat alluvial sandy soils in the lower portion of the area is discussed in Chapter 0.4.
A. Friction Gradient in PVC Pipe and PE (Polyethylene) Hose

The following figures for friction gradient have been computed by the Hazen-Williams formula, using a friction factor \( C = 150 \). The gradients arrived at are similar to, although not identical with, gradients calculated by the Manning formula using a friction factor \( n = 0.011 \).

PVC nominal pipe sizes are Outside Diameter. Equivalent Inside Diameters have been calculated from wall thicknesses derived from manufacturer's figures for pipe weight per metre, using a Specific Gravity of 1.4. These thicknesses correspond to the upper end of manufacturing tolerance, and friction gradients so calculated are fractionally higher than if average values of wall thickness are used. The wall thicknesses refer to a working pressure rating 2.5 kg/cm\(^2\) (25 m of water) for diameters of 90 mm and above, and 4.0 kg/cm\(^2\) (40 m of water) for 75 mm diameter, for which the lower rating is not available.

The Hazen-Williams formula is as follows:

\[
0.54 \quad s = \frac{V}{1.318 \times C \times r}^{0.63}
\]

\( s \) = friction gradient (in feet per foot); \( C \) = friction factor (150)

\( r \) = hydraulic mean radius; \( D \) where \( D \) is internal diameter in feet

\( V \) = Velocity in ft/sec

Or

\[
0.54 \quad s = \frac{Q^{1.85}}{d^{4.87} \times 2.266}
\]

When \( Q \) is flow in cusecs, \( d \) is diameter in feet and \( C = 150 \).
This can also be expressed as:
\[
    s = Q^{1.85} \times 15.56 \times d^{-4.87}
\]

When \(Q\) is flow in litres/sec and \(d\) is actual internal diameter in centimetres.

The following useful ratios are derived directly from the above formula:

For a given pipe size the friction gradient is proportional to \((Q)^{1.85}\), where \(Q\) is flow.

For a given flow the friction gradient is proportional to \(D^{4.87}\) where \(D\) is the Inside Diameter of the pipe.

Thus, if the gradient is \(S_1\) with diameter \(D_1\), then the gradient \(S_2\) with diameter \(D_2\) will be

\[
    S_2 = S_1 \left( \frac{D_1}{D_2} \right)^{4.87}.
\]

(For comparison, with the Manning formula friction gradient is proportional to \((Q)^{2}\), and for given flow but varying diameter it is proportional to \(D^{5.33}\).

Dimensions of PVC pipe and polyethylene hose, for purposes of hydraulic calculations, and calculated hydraulic gradients for a range of flows are set out in the attached Tables 1 and 2.

B. Head Losses at Structures and Fittings in Pipe Distribution Systems

In addition to friction losses in the pipe and delivery hose, head losses occur at all structures (intake, valves, tees, bends, etc.). Of these, the most significant are the losses in the main line at the tee serving each hydrant, and in the hydrant riser (assuming it is flowing) both at the junction with the main line and at the top of the riser where a second tee serves the two outlet valves. Energy losses at a tee, in line and riser very with the proportion of the main line flow being diverted to the riser, and can be estimated approximately only. The following factors are suggested for design purposes. The value of the factor is multiplied into the velocity head at each structure \((v^2/2g)\) to give the head loss at that point. Typical values for head loss at that point. Typical values for head loss are given in each case, for illustration. Velocity is taken as 0.9 m/sec except where noted.
- **Intake at Canal**  
  Assumed bell-mouthed  
  \[ \frac{2}{0.5} \frac{v}{2g} \]  
  typical value 0.02 m

- **Gate Valve on Main Line**  
  Fully open  
  \[ \frac{2}{0.2} \frac{v}{2g} \]  
  typical value 0.01 m

- **Standard Long-Radius Bend**  
  90°, Radius more than 4 x pipe diameter  
  \[ \frac{2}{0.15} \frac{v}{2g} \]  
  typical value 0.01 m

- **Tee in Main Line at Hydrant**  
  90 mm to 75 mm  
  If no diversion to hydrant  
  Loss in main line  \[ 0.25 \frac{v}{2g} \]  
  typical value 0.01 m

  If half flow diverted to hydrant  
  Loss in main line  \[ 0.5 \frac{v}{2g} \]  
  typical value 0.02 m

  Loss in lower end of riser  \[ 1.0 \frac{v}{2g} \]  
  typical value 0.04 m

- **Tee at Upper End of Hydrant**  
  75 mm to 50 mm each side  
  \[ \frac{2}{1.0} \frac{v}{2g} \]  
  typical value 0.04 m

- **Gate Valve on Outlet**  
  (Hydrant to Hose)  
  Fully open  
  \[ \frac{2}{0.2} \frac{v}{2g} \]  
  Velocity assumed 4/ft/sec  
  typical value 0.02 m
- Head Loss at Downstream End of Hose
  Velocity assumed 4 ft/sec

  \[ \text{Head loss} = 1.0 \frac{k}{v^2} \frac{v^2}{2g} \]
  typical value 0.08 m

- Total Losses at Structures
  (Compared with Friction Loss in Pipe)

- For purpose of illustration, the following case is taken:

<table>
<thead>
<tr>
<th>Structures</th>
<th>Metres</th>
</tr>
</thead>
<tbody>
<tr>
<td>Intake</td>
<td>0.02</td>
</tr>
<tr>
<td>Gate valves on main line</td>
<td>0.02</td>
</tr>
<tr>
<td>Bends</td>
<td>0.02</td>
</tr>
<tr>
<td>Tees in main line</td>
<td>0.04</td>
</tr>
<tr>
<td>Hydrant closed</td>
<td>0.04</td>
</tr>
<tr>
<td>Hydrant open</td>
<td>0.02</td>
</tr>
<tr>
<td>Hydrant (two in operation, with four hoses)</td>
<td></td>
</tr>
<tr>
<td>Lower end</td>
<td>0.04</td>
</tr>
<tr>
<td>Upper end</td>
<td>0.04</td>
</tr>
<tr>
<td>Outlet valve</td>
<td>0.02</td>
</tr>
<tr>
<td>Vel. head lower end of each hose</td>
<td>0.08</td>
</tr>
<tr>
<td></td>
<td>0.30</td>
</tr>
</tbody>
</table>

Pipe and Flexible Hose

See layout shown on Plate D1, Fig. 1. The common 10 litres/sec line and one 5 litres/sec line is taken for purpose of illustration.

<table>
<thead>
<tr>
<th>Pipe and Flexible Hose</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>110 mm pipe</td>
<td>1.1 m</td>
</tr>
<tr>
<td>90 mm pipe</td>
<td>2.43 m</td>
</tr>
<tr>
<td>50 mm pipe</td>
<td>1.65 m</td>
</tr>
</tbody>
</table>

Total Structures, Pipe, and Hose

(Head available at 2.5% slope 6.25 m)

5.48 m

5.18 m

It is apparent from the above figures that head losses at structures will generally be a relatively small proportion of total losses (in this case 5%). In most cases a blanket allowance of say 7% may be added to pipe and hose friction losses, to cover structure energy losses. Where head is critical, substitution of larger diameter hose may be indicated, or delivery at slightly reduced rate (e.g. 2 litres/sec) for a correspondingly longer period.
## IRRIGATION FROM SUPPLY CANALS VIA BURIED PIPE DISTRIBUTION SYSTEMS

### Dimensions of PVC Pipe and PE Hose

<table>
<thead>
<tr>
<th>Polyethylene Hose</th>
<th>PVC Pipe</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Pipe outside diameter (mm)</td>
<td>75 90 110 140 160 200</td>
</tr>
<tr>
<td>2. Pressure rating (kg/cm²)</td>
<td>4 2.5 2.5 2.5 2.5</td>
</tr>
<tr>
<td>3. Weight per m (kg)</td>
<td>0.663 0.606 0.894 1.413 1.884 2.945</td>
</tr>
<tr>
<td>4. Approximate mean perimeter (mm)</td>
<td>229 278 339 433 494 618</td>
</tr>
<tr>
<td>5. Thickness (mm) derived from (2) and (3) using 4. gravity of 1.4</td>
<td>2.1 1.6 1.9 2.3 2.7 3.4</td>
</tr>
<tr>
<td>6. Inside diameter (mm)</td>
<td>50.0 55.0 70.8 86.8 106.2 135.4 154.6 193.2</td>
</tr>
<tr>
<td>7. Inside diameter (cm)</td>
<td>5.0 5.5 7.08 8.68 10.62 13.54 15.46 19.32</td>
</tr>
<tr>
<td>8. d (10 x)</td>
<td>2.54 4.03 13.8 37.2 99.4 324 619 1.832</td>
</tr>
</tbody>
</table>
IRRIGATION FROM SUPPLY CANALS VIA BURIED PIPE DISTRIBUTION SYSTEMS

Hydraulic Gradients - PVC Pipes and PE Hose

<table>
<thead>
<tr>
<th>PE Hose</th>
<th>Rigid PVC Pipe</th>
</tr>
</thead>
<tbody>
<tr>
<td>Nominal dia. (mm)</td>
<td>50</td>
</tr>
<tr>
<td>Internal dia. (cm)</td>
<td>5.00</td>
</tr>
<tr>
<td>4.87 d</td>
<td>2.540</td>
</tr>
</tbody>
</table>

Hydraulic Gradient (%)

| Q (l/s) | Q | (
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>3.61</td>
<td>2.21</td>
</tr>
<tr>
<td>2.25</td>
<td>4.49</td>
<td>2.75</td>
</tr>
<tr>
<td>2.50</td>
<td>5.46</td>
<td>3.34</td>
</tr>
<tr>
<td>2.75</td>
<td>6.51</td>
<td>3.99</td>
</tr>
<tr>
<td>3</td>
<td>7.85</td>
<td>4.87</td>
</tr>
<tr>
<td>4</td>
<td>13.03</td>
<td>1.47</td>
</tr>
<tr>
<td>5</td>
<td>19.70</td>
<td>2.22</td>
</tr>
<tr>
<td>6</td>
<td>27.60</td>
<td>1.15</td>
</tr>
<tr>
<td>8</td>
<td>47.00</td>
<td>1.97</td>
</tr>
<tr>
<td>9</td>
<td>58.50</td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>71.10</td>
<td>1.11</td>
</tr>
<tr>
<td>12</td>
<td>150.60</td>
<td>2.36</td>
</tr>
<tr>
<td>15</td>
<td>190.00</td>
<td>1.01</td>
</tr>
<tr>
<td>17</td>
<td>211.00</td>
<td></td>
</tr>
<tr>
<td>18</td>
<td>257.00</td>
<td>0.218</td>
</tr>
<tr>
<td>20</td>
<td>388.00</td>
<td>0.330</td>
</tr>
</tbody>
</table>
IRRIGATION FROM SUPPLY CANALS VIA BURIED PIPE DISTRIBUTION SYSTEM

FRICTION GRADIENTS FOR PVC PIPE 2.5 Kg/cm² rating

(Hazen-Williams formula. Value of Coefficient C=150)

FLOW - Litres/second

0 0.2 0.4 0.6 0.8 1.0 1.2 1.4 1.6 1.8 2.0 2.2 2.4 2.6

FRICTION GRADIENT - Percent

Nominal Diameter 160 mm

FLOW - Litres/second

0 10 20 30 40 50 60 70 80 90 100 110 120

FRICTION GRADIENT - Percent
WATER-HAMMER IN BURIED PIPE IRRIGATION DISTRIBUTION SYSTEMS

TABLE OF CONTENTS

A. Introduction 75
B. The Rigid Column Approach to Analysis of Pressure Rise in a Pipe on Valve Closure 75
C. The Non-Rigid (Elastic) Approach to Analysis of Pressure Rise 77
D. Evaluation of Pressure Rise and of Velocity of Pressure Wave on Instantaneous Valve Closure 80
E. Pressure Rise on Slower Rates of Reduction in Flow 85
F. Rate of Change of Flow during Valve Closure 86
G. Water-Hammer in Branch Lines, Particularly Closed-Ended Branches 88
H. Control of Water-Hammer in Pipe System Design 90

PLATES 1-4 93-99

APPENDIX 1 - Derivation of Equations for Head Rise and Velocity of Pressure Wave 101
WATER-HAMMER IN BURIED PIPE IRRIGATION DISTRIBUTION SYSTEMS

A. Introduction

1. The mathematical analysis of water-hammer in other than a simple system is complex and is usually approached with the assistance of substantial computer programmes, supplanting the pre-computer era mathematical approximation or graphical methods.

2. However, the physical factors entering into the phenomenon of water-hammer are relatively easily understood, and with such an understanding and with the assistance of a few easily-derived basic equations the engineer can design relatively simple systems with reasonable assurance of avoiding the classical water-hammer problems. For complex or critical systems recourse should be had to the computer services of specialized agencies.

3. The following notes are aimed at providing such an understanding. Approximations in the treatment are acknowledged, but it is believed that they do not detract from the usefulness of the discussion. The notes are directed at the Kallada project (gravity buried-pipe distribution from open-channel supply canal), but they are relevant to gravity and pumped-supply irrigation distribution systems in general.

4. There are two traditional approaches to water-hammer:

- The simple rigid-column approach. This is a gross approximation, and is discussed herein principally to point out its limitations.

- The elastic approach, which takes into account elastic deformation of both the fluid being conveyed (water in this case) and the pipe wall. While both water and pipe wall are normally conceived as relatively rigid materials, in fact their elastic deformation is the determining factor in most water-hammer events.

B. The Rigid Column Approach to Analysis of Pressure Rise in a Pipe on Valve Closure

5. The situation is illustrated in Plate 1, Fig. 1. A pipe supplied from a reservoir is discharging through a valve at its lower end. In the initial steady state, the discharge is a function of the static head at the valve, friction loss down the length of the pipe (determining net head on the valve), and area of valve opening and its discharge coefficient. For a given size and length of pipe (and
pipe roughness) and a given area of valve opening, the flow may be calculated (by trial and error) dividing the available static head between friction loss in the pipe and velocity head through the valve. On valve closure the force required to cause the resulting deceleration of the flow in the pipe must be provided by a rise in net head at the lower end of the pipe. Two other factors to be considered are the increased velocity through the valve due to this pressure rise (partially offsetting the reduction of area of valve opening on partial closure) and the reduction in friction loss in the pipe due to the reduction in rate of flow. However, by trial and error the net head of each small interval of time during the period of valve closure may be calculated. A principal item in this calculation is the force required to cause the rate of deceleration of the water column. This is arrived at from the classic equation:

\[ \text{Force} = \text{Mass} \times \text{Acceleration} \]

For the pipe in question this relationship can be written:

\[ \text{Decelerating Head} = \frac{\text{Length of Pipe} \times \text{Rate of Deceleration}}{g} \]

6. The calculation is simple enough. However, it ignores two physical factors which largely invalidate the answers obtained. These are the following:

(a) As the pressure rises in the pipe (initially at the valve), part of the flow arriving at that point is temporarily stored by elastic expansion of the pipe, and part by elastic compression of the water itself.

(b) A pressure wave (discussed subsequently) originating at the valve travels rapidly up to the reservoir and back to the valve, effectively offsetting the pressure rise which caused it a few moments earlier.

To illustrate the difference between the results obtained with the rigid column approach and the elastic approach which reflects the actual situation:

- With the rigid column approach pressure rise due to deceleration is proportional to length of pipe, whereas with the elastic approach, beyond a certain length (discussed later), it is not.

- With the rigid column approach pressure rise is independent of pipe material or wall thickness in relation to diameter. With the elastic approach these items have an important bearing on actual pressure rise.

7. In view of the gross approximation involved with the rigid column approach, in most situations it is more satisfactory to adopt the elastic approach even if simplifying assumptions have to be made in the latter case.
C. The Non-Rigid (Elastic) Approach to Analysis of Pressure Rise

8. In discussing elastic effects in water-hammer, it is convenient to consider first the case of instantaneous total valve closure. Subsequently, slower closure is treated as a series of small instantaneous valve movements at regular intervals of time. As the main purpose of these notes is to generate an appreciation of the physics rather than of the mathematics of water-hammer, an analogy is first presented.

Instantaneous Total Valve Closure - The Coil Spring Analogy

9. The effect of sudden total closure of a valve at the end of a pipeline may be likened to the behaviour of a coil spring thrown end-wise against a wall, and the end held there (Plate 1, Fig. 2a). On meeting the wall (analogous to the suddenly closed valve) the spring compresses at that point, and progressively outwards (Fig. 2b), until finally the whole spring is momentarily stationary in the compressed state (Fig. 2c). Rebound then begins, the coils expanding to their original uncompressed state progressively from the outer end (Fig. 2d) until momentarily the whole spring is in the original uncompressed condition but is travelling outwards from the wall (except for the end which is held there) with its original velocity reversed in direction (Fig. 2c). The outward-moving spring will immediately come into tension at the wall and progressively along its length (Fig. 2f) until it is again stationary but in tension throughout its length (Fig. 2g). It will then progressively relax its tension from the outer end back towards the wall (Fig. 2h) until the whole spring is momentarily in the original untensioned (relaxed) condition and moving towards the wall with its original velocity (as in Fig. 2a). The cycle then repeats itself. The following points are noted:

- The energy of the coil spring cycles between entirely kinetic (due to velocity and mass only) as in Fig. 2a and 2e, and entirely strain energy (due to tension or compression) with zero velocity as in Fig. 2c and 2g.

- The inter-face between the portion of the spring in tension (or compression) and the portion in the relaxed condition moves back and forth twice in each cycle. It may be likened to a wave of compression or tension.

- The outward rebound velocity (Fig. 2e) is equal and opposite to the initial inward velocity (Fig. 2a).

Each of the above points has a parallel in the elastic behaviour of a flowing pipe on sudden valve closure.

Instantaneous Total Valve Closure - Elastic Behaviour of Pipe and Fluid

10. The sequence of events on instantaneous complete valve closure in a pipeline, taking into account elastic effects, is shown diagrammatically in Plate 2, Fig. 2a through 2h, each of which may be compared with the corresponding Fig. 2a through 2h of Plate 1. (In fact, if the concept of a flexible rubber rod is substituted for the coil spring, the analogy with the actual events in the pipeline is a very direct one).
11. In the situation shown in Plate 2, Fig. 2a, the friction gradient in the pipe is ignored, or taken as small in comparison with the velocity head through the initially open valve. This is for the sake of simplicity in presentation.

12. On instantaneous closure of the valve there is an instantaneous pressure rise immediately adjacent to the valve. This causes the pipe in that immediate area to enlarge (fractionally), and the water within it and entering it in that small instant of time to compress (also fractionally), as indicated in Fig. 2b. The area of increased pressure and zero velocity moves progressively upstream (a "positive pressure wave") until the velocity is arrested throughout the length of the pipe, and the pipe is momentarily under increased pressure throughout (Fig. 2c). Immediately thereafter the increased pressure is relieved at the upstream end of the line (where it is controlled by the reservoir surface elevation), and the pipe at that point returns to its original unstressed diameter causing water to flow back towards the reservoir from that portion of the line. This sequence progresses all the way back to the valve (Fig. 2e) (a "zero pressure wave") at which time, momentarily, the whole length of the line has returned to its original diameter, the pressure throughout is back to the original pre-closure level, and the flow in the pipe is at its original velocity but in the reverse direction. In the immediate vicinity of the valve this causes an instantaneous drop in pressure as the flow in the negative direction is arrested at that point. This causes an elastic reduction in pipe diameter at that point, and an elastic expansion of the water within it. The area of reduced pressure, and zero velocity, moves upstream (Fig. 2f) (a "negative pressure wave") until it reaches the reservoir (Fig. 2g), at which point the pressure returns to normal and the pipe locally expands to its normal unstressed diameter, requiring water to flow from the reservoir into that portion of the line. This sequence progresses back to the valve (Fig. 2h) and upon arriving, there the situation of Fig. 2b is replicated and the cycle begins again.

13. The following points are noted:

- As in the case of the coil spring, the energy within a particular element of the line cycles between wholly kinetic (due to the mass and velocity of water within the line) and wholly strain energy (due to elastic deformation of the pipe wall and water, velocity then being zero).

- The interface between the condition of velocity $V_0$ and original pressure on the one hand, and zero velocity and increased or decreased pressure on the other, moves up and down the length of the pipe twice in each cycle.

- The velocity of the pipe changes from the original velocity $V_0$ to the same velocity in the reversed direction during the cycle.

The variation in pressure during the cycle, at three particular points, is shown below. (The time for the pressure wave to make one round trip is taken as $T$ seconds.)
(a) At a point midway between the valve and the reservoir

<table>
<thead>
<tr>
<th>Time</th>
<th>Pressure</th>
</tr>
</thead>
<tbody>
<tr>
<td>At time 0</td>
<td>Po (the original pressure)</td>
</tr>
<tr>
<td>At time ( \frac{T}{4} )</td>
<td>Suddenly rises to ( Po + \frac{P}{2} ) and remains there for ( \frac{T}{2} ) seconds</td>
</tr>
<tr>
<td>At time ( \frac{3T}{4} )</td>
<td>Suddenly reduces to original ( Po ) and remains there for ( \frac{T}{2} ) seconds</td>
</tr>
<tr>
<td>At time ( \frac{5T}{4} )</td>
<td>Suddenly reduces to ( Po - \frac{P}{2} ) and remains there for ( \frac{T}{2} ) seconds</td>
</tr>
<tr>
<td>At time ( \frac{7T}{4} )</td>
<td>Suddenly returns to ( Po ) and remains there for ( \frac{T}{2} ) seconds</td>
</tr>
</tbody>
</table>

(b) At a point immediately adjacent to valve

<table>
<thead>
<tr>
<th>Time</th>
<th>Pressure</th>
</tr>
</thead>
<tbody>
<tr>
<td>At time 0</td>
<td>Pressure suddenly rises to ( Po + \frac{P}{2} ) and remains there for ( T ) seconds</td>
</tr>
<tr>
<td>At time ( T )</td>
<td>Pressure suddenly reduces to ( Po ) and immediately thereafter reduces to ( Po - \frac{P}{2} ), where it remains for ( \frac{T}{2} ) seconds. This is in effect an instantaneous reduction from ( Po + \frac{P}{2} ) to ( Po - \frac{P}{2} )</td>
</tr>
<tr>
<td>At time ( 2T )</td>
<td>Pressure instantaneously rises from ( Po - \frac{P}{2} ) to ( Po + \frac{P}{2} )</td>
</tr>
</tbody>
</table>

The rapidity of change from positive to negative waterhammer at or near to the valve is particularly noted.

(c) At a point immediately downstream from the reservoir

<table>
<thead>
<tr>
<th>Time</th>
<th>Pressure</th>
</tr>
</thead>
<tbody>
<tr>
<td>At time 0</td>
<td>Po</td>
</tr>
<tr>
<td>At time ( \frac{T}{2} )</td>
<td>Pressure suddenly rises to ( Po + \frac{P}{2} ) but almost immediately returns to ( Po ) where it remains for ( \frac{T}{2} ) seconds</td>
</tr>
<tr>
<td>At time ( \frac{3T}{2} )</td>
<td>Pressure suddenly falls to ( Po - \frac{P}{2} ), but almost immediately afterwards returns to ( Po ) where it remains for ( \frac{T}{2} ) seconds</td>
</tr>
</tbody>
</table>
14. Two reservations are noted in regard to the above discussion. Firstly, as previously pointed out, friction is neglected. Secondly it is assumed that the reduced pressure $P_0 - P$ is above the vapour pressure of water. If $P_0 - P$ falls below that level at any point separation of the water column occurs, a highly undesirable situation due to the pressure wave generated by subsequent collapse of the vapour bubble when the pressure again rises. This situation is likely to occur at the upstream end of a sloping line where $P_0$ may be relatively small.

D. Evaluation of Pressure Rise and of Velocity of Pressure Wave on Instantaneous Valve Closure

15. The analysis in the previous section is descriptive. It is now necessary to proceed to actual evaluation of the above items. For ease in reading, however, the physical factors determining pressure rise and wave velocity are discussed qualitatively in the first part of the section, derivation of actual equations being relegated to the annex.

16. Subsequently, the two equations required in practice are recalled from the annex, and some typical situations are evaluated. Reference to the annex is not essential at first reading.

Pressure Rise

17. In Plate 2, Fig. 2b, the situation a very short time after sudden total valve closure is shown. The water reaching the face of the valve has obviously been brought to rest, while a short distance upstream it is still moving with its original velocity. Evaluation of pressure rise due to this change in velocity is based on conservation of energy, it being assumed that the kinetic energy of flow $(\text{Mass} \times \text{Velocity})^2$ is fully converted to energy stored in the pipe wall (and the water within it) during elastic deformation due to the pressure rise.

18. The energy stored within the water due to elastic compression is simply a function of its modulus of elasticity and the head rise (Appendix 1B). It is independent of considerations of pipe elasticity.

19. On the other hand, the amount of elastic deformation of the pipe and the head rise due to arresting the flow depend very much on the stiffness of the pipe wall. This is a function of both the modulus of elasticity of the pipe material and the ratio wall thickness to pipe diameter.

20. To illustrate, referring again to Plate 3, the strain energy stored within the pipe wall can be regarded as a function of either:

(a) The product of the amount by which the wall stretches circumferentially and the average tension within the wall during the process, or
(b) The work performed by the water, under pressure, in pushing the pipe wall outwards, i.e. the product of the mean head during the pressure rise and the amount by which the wall is deformed (radially).

21. The two approaches give the same result, as the work done by the water in deforming the pipe is equal to the energy stored within the pipe wall due to this deformation. However, the second approach (b) is the more convenient for the purpose of this discussion.

22. For a given incoming velocity, and consequently a given amount of kinetic energy per unit of length of the water column, the equivalent amount of work required to be performed by the water in expanding the pipe could be provided by the product of either a small pressure rise and a large radial deformation of the pipe wall, or a large pressure rise and a small deformation of the pipe wall. The first is the case with a flexible pipe (small pressure rise), and the second with a stiff pipe (large pressure rise).

23. To illustrate, for a given velocity of flow the head rise on sudden closure is likely to be some five times greater with a pipe of relatively stiff material (steel) compared with a relatively flexible material (PVC) \(1\). Offsetting disadvantages of flexibility are discussed later. As pressure rise is a function of both elasticity of pipe wall and elasticity of water, it is also of interest to note that with a flexible material such as PVC only some 2% of the strain energy is accounted for by elastic compression of the water, which is relatively much more rigid than the pipe. On the other hand, with a steel pipe of typical wall thickness the amounts of energy accounted for by elastic compression of the water and by elastic deformation of the pipe wall are more nearly equal. A notable exception would be the case with water conveying entrapped air bubbles, where the effective elasticity of the flow is much reduced.

24. A factor expressing the relationship of head rise on sudden closure and the parameters of elasticity of water (\(e\)), of pipe material (\(E\)), and wall thickness to diameter (\(p\)), is derived in Appendix I (A to D). The equation is:

\[
\text{Head rise } H = \frac{\text{Original velocity } V_0 \times \left( \frac{1}{g} \left( \frac{1}{w/g} \left( \frac{1}{\frac{1}{e} + \frac{1}{pE}} \right) \right) \right)}{g}^{\frac{1}{2}}
\]

or

\[
H = \frac{V_0}{g} \times C
\]

where \(C\) is the composite term in brackets conveying the above elastic factors.

\(1\) In these notes the term "PVC" is used to convey unplasticized PVC, also referred to as "UPVC". This is the material used in commercial production of PVC pipe for irrigation.
Velocity of Pressure Wave

25. This is the velocity of movement of the "inter-face" between the portion of the pipe in which water is still flowing at its original velocity, and the portion in which the flow is stopped. Hereafter, it is referred to simply as the wave velocity, although it is not a wave in the sense of progressive change from trough to peak. The change, as the wave passes a particular point on the pipeline, is virtually instantaneous (in the case of rapid closure).

26. While wave velocity of itself is not an important factor in calculating head rise for instantaneous valve closure, it becomes very important in calculation of head rise for slower valve closure, as will be evident later.

27. Referring again to Plate 2, it is evident that as the pipe wall stretches and the water within it compresses under the effect of head rise on valve closure, space is provided for a portion of the incoming flow. For simplicity, both pipe wall stretch and elastic compression of water are considered for the moment as combined into an effective increase in the cross sectional area of the pipe. If the inter-face moves upstream at $v$, the additional volume provided by elastic deformation in the portion of the pipe to the right of the inter-face per unit time is:

$$v \times \text{the effective elastic increase in sectional area}$$

Due to head rise

This volume must be equal to the incoming rate of flow at $V_0 \times \text{area of pipe}$.

Thus $v \times \text{effective elastic increase in area of pipe} = V_0 \times a$

I.e. wave velocity $v$

$$= V_0 \times a$$

Elastic increase in area

28. It is apparent from this relationship (also intuitively) that the more flexible the pipe wall and the greater the space provided for incoming flow by pipe expansion, the slower the inter-face has to move. It would also appear at first sight that the wave velocity is proportional to the rate of flow in the pipe before valve closure ($V_0$). However, as previously determined, the head rise on closure and consequently the elastic increase in area are also proportional to rate of flow $V_0$, and the two effects cancel. Thus, a higher rate of flow at valve closure causes greater pressure rise and greater elastic deformation, leaving the velocity of the inter-face unchanged. The only items influencing wave velocity are in fact the parameters of elasticity of water and of pipe material and the relative thickness of pipe wall to pipe diameter.

29. The effective elastic increase in sectional area (or increase in volume per unit length) due to a head rise $H$ is derived in Annex El and is equal to:

$$wH \frac{1 + \frac{1}{e}}{pE}$$
The term within the brackets (the elasticity factor) also occurred in the earlier calculation of head rise, where a parameter C was introduced to cover this factor. Using the value of C as defined in that calculation, the expression for wave velocity (Appendix 1) becomes \( v = V_0 C^2 \). Substituting H in terms of \( V_0 \), this expression then becomes \( v = C \). To summarize, for instantaneous valve closure against a velocity \( V_0 \),

\[
\text{Head rise } H = \frac{V_0 C}{g} \\
\text{Wave velocity } v = C \\
\text{Where } C = \left( \frac{1}{w/g \left( \frac{1}{e} + \frac{1}{pE} \right)} \right)^{\frac{1}{2}}
\]

It is the practice in the literature, although not particularly conducive to intuitive understanding of the subject, to dispense with C and to simply write wave velocity

\[
v = \left( \frac{1}{w/g \left( \frac{1}{e} + \frac{1}{pE} \right)} \right)^{\frac{1}{2}}
\]

and head rise \( H = \frac{V_0 x v}{g} \)

The wave velocity \( v \) in fact does not vary significantly over a range of sizes of pipe (all being of the same material) as the wall thickness ratio \( p \) does not vary markedly with pipe size. For design within a particular project situation the wave velocity, once evaluated, becomes virtually a constant for the project. Head rise for instant closure then is simply a function of the flow velocity \( V_0 \) closed against.

30. It should be noted that the above analysis of head rise and wave velocity (Appendix 1) is subject to the approximation that only circumferential deformation (or increase in diameter) of the pipe, due to pressure rise, is considered. There may also be longitudinal deformation, the amount depending upon the conditions of end restraint. Ignoring the latter leads to an inaccuracy of less than 10%, and is acceptable in most water-hammer calculations in view of other unavoidable approximations.

Typical Values of Pressure Rise and Wave Velocity

31. In evaluation of the above expression for wave velocity, the following values for the elastic constants are used:
Typical figures for wave velocity $v$, using the above values are as follows:

For PVC pipe, with ratio ($p$) of wall thickness to diameter taken as 1/60, or .017 (corresponding to 2.5 kg/cm² pressure rating):

$$v = \frac{1}{\sqrt{\frac{\frac{1}{w/g} + \frac{1}{e}}{\frac{1}{pE}}}}$$

$$= 700 \text{ ft/sec or } 200 \text{ m/sec}$$

Steel pipe with same ratio of wall thickness to diameter

$$v = 4,000 \text{ ft/sec (rounded) or } 1,200 \text{ m/sec}$$

Typical values of head rise for instantaneous closure against an initial velocity $V_o$ of 1 ft/sec (0.3 m/sec) and alternatively 3 ft/sec (0.9 m/sec) are as follows:

$$H = \frac{V_o \times v}{g}$$

<table>
<thead>
<tr>
<th>Head rise</th>
</tr>
</thead>
<tbody>
<tr>
<td>$V_o = 1 \text{ ft/sec}$</td>
</tr>
<tr>
<td><strong>PVC pipe with wave velocity</strong></td>
</tr>
<tr>
<td>$v = 700 \text{ ft/sec (200 m/sec)}$</td>
</tr>
<tr>
<td><strong>Steel pipe with wave velocity</strong></td>
</tr>
<tr>
<td>$v = 4,000 \text{ ft/sec (1,200 m/sec)}$</td>
</tr>
</tbody>
</table>

---

1/ Equivalent velocities for 4 kg/cm² and 6 kg/cm² pipe are approximately 870 ft/sec (265 m/sec) and 1,050 ft/sec (320 m/sec).
E. Pressure Rise on Slower Rates of Reduction in Flow

32. The figures immediately above illustrate the destructive magnitude of the head rise (and equally of fall in head) which can result from instantaneous valve closure against even modest rates of flow. Two questions immediately arise:

(a) How rapid is "instantaneous" in the sense of valve closure.
(b) How is the head rise affected by slower rates of reduction in flow.

33. The answer to (a) is apparent from Plate 2. If the valve closure is complete before the "zero pressure wave" returns to the valve, the head rise at the valve has already reached the "instantaneous" value before this relief arrives. Thus, from the point of view of pressure at the valve instantaneous closure against a particular velocity means closure in any time less than the "round trip" time of the pressure wave. This in turn depends upon the wave velocity and the length of the pipe from the valve to the free surface at the reservoir. To illustrate, and using the wave velocities calculated above, if the pipe were of length $L = 2,000$ ft (600 m) the round trip wave time for the PVC pipe would be:

$$T = \frac{2L}{\text{wave velocity}} = \frac{4,000 \text{ ft}}{700 \text{ ft/sec}} = 5.7 \text{ seconds}$$

Closure against a velocity of flow $V_0$ (or reduction in velocity by an amount $V$) in less than that time would result in full "instantaneous" pressure rise at the valve, followed immediately thereafter by the full negative pressure.

34. By comparison with a steel pipe of the same length, but with wave velocity 4,000 ft/sec (1,200 m/sec), the round trip wave time would be:

$$T = \frac{2L}{V} = \frac{4,000 \text{ ft}}{4,000 \text{ ft/sec}} = 1 \text{ second}$$

Thus, the "instantaneous" closure time for a steel pipe is much less than for a PVC pipe.

35. An answer to the second question (b) is the purpose of most water-hammer analysis. As discussed earlier and as illustrated in Plate 2, the result of valve closure is a cyclic induced pressure changing from positive through zero to negative, returning through zero to positive, and so on until dampened by friction. For convenience the effect is referred to as a "pressure wave" although it represents an abrupt change (rather than gradual) in pressure at any point on the pipeline as the wave passes. As used herein a "positive wave" increases the head at the point in question. A "zero wave" returns the head to the original pre-closure valve.

36. In analysis of head rise for valve closure rates less than "instantaneous" the closure (for purposes of calculation) may be conceived as being divided into steps, at uniform intervals of time. Each step is regarded as a small instantaneous closure, and the time sequence of pressure waves resulting from it is worked out. The net total effect at a particular point (i.e. the variations in head with time at that point) is obtained by summation of the pressure-time effects of the individual closure steps. At a particular point in
time, and referring to a particular location on the pipeline, the changes in head due to the individual closure stages may be positive or negative. Thus, they may partially offset each other, or alternatively they may be additive.

37. As a simple example the closure of a valve in four steps each reducing the flow by $V_o$, is illustrated in Plate 3. The complete closure takes place within one-half of the round-trip time $T$. The example illustrates the method generally, and also the fact that although closure against $V_o$ in any time period up to $T$ (or in stages within a total period $T$) produces full head rise of $V_o$ at the valve, the head rise at other points along the line is smaller, unless the closure against $V_o$ is in fact in one single instantaneous step.

38. The step method of analysis, which can be extended to any rate of valve closure, is very simple in concept. However, with any but the most elementary systems, keeping track of pressure waves being initiated in a series of time steps, and reflected from a number of nodal points (branches, changes in pipe diameter etc.), presents a major problem in data processing, unless recourse is made to a computer. The method is not a convenient design tool, except for analysis of simplified typical situations.

F. Rate of Change of Flow During Valve Closure

39. For simplicity it has been assumed in the above discussion that equal steps in valve closure are synonymous with equal steps in reduction of flow. In most situations this is not, in fact, the case. The final stages of valve closure may produce much larger changes in flow than the initial stages, a feature which has considerable influence on the magnitude of the resulting water-hammer. The factors which contribute to this effect are inertia of the flowing water, hydraulic friction, and changes in the energy loss coefficient as the valve closes.

40. To illustrate, consider first the limiting case of a valve directly connected to a tank, without intervening pipe. Neither inertia nor hydraulic friction are factors in this case. The valve in this case is virtually an orifice of variable size in the side of the tank. The head on the valve is constant, regardless of degree of opening of the valve. In this case flow is directly proportional to area of opening of the valve.

41. In a more typical case, however, there is a pipe hundreds of metres in length between the valve and tank (or canal outlet). The head on the valve is influenced both by the static head between tank and valve and friction loss in the pipe. As the flow is progressively reduced during valve closure the friction component is also reduced and pressure head at the valve increases, increasing the velocity through the part-open valve and partly offsetting the effect of progressive valve closure.
The inertia effect of deceleration of the flow in the line by valve closure also progressively increases head on the valve, to a degree dependent upon length of line and speed of closure (more specifically wave "round trip" time in relation to time of closure), and has the same result of partially offsetting the effect of valve closure on flow reduction in the initial stage of closure. Finally, the coefficient of energy loss (relating head loss through the partially open valve to velocity through it, or velocity in the pipeline upstream of it) also changes with degree of closure. The effective head loss for the first 25% of closure is very small, particularly if there is pipe (or hose) also on the downstream side of the valve as there may be considerable recovery of velocity head. On the other hand, during the last 25% of closure the coefficient of energy loss is high and there may be complete loss of velocity head.

42. In general, uniform rate of valve closure does not result in uniform rate of change of velocity in the pipeline. Consequently, it is inappropriate to calculate velocity change during the round-trip time by simple proportion to the velocity change over the whole period of closure. Pressure rise computed on this basis (by assuming "instantaneous" closure against the change in velocity in the round-trip time) is likely to result in considerable underestimation.

43. It is obviously impractical to carry out a full mathematical analysis of each valve installation to determine maximum change of velocity during the round-trip time, and the resulting pressure rise. Approximation must be resorted to. It is suggested that for purposes of estimating pressure rise the change of velocity during the round-trip time should be taken as three times the rate averaged over the whole period of closure. For example, the velocity in a 90 mm pipeline is reduced from 0.9 m/sec to 0.45 m/sec by closure of one 2.5 litres/sec Valve in a period of 15 seconds. The round-trip time for a distance of 2 x 300 m is approximately 3 seconds. The change in velocity in 3 seconds, if calculated on the rate averaged over the closure period, is 0.9 x 1.5 or 0.09 m/sec. Applying the above factor of three, the estimated maximum change in velocity in the line during a 3 seconds round-trip time becomes 0.27 m/sec (a head rise of 6 m). Maximum possible head rise is obtained when the whole change in velocity occurs within the round-trip time (0.45 m/sec in the above case), and this should be taken as the upper limit to the pressure calculated by the suggested procedure.

44. It is noted that a valve delivering 2.5 litres/sec may be only partially open in that initial condition, depending upon the net head at the valve. The closure time in such circumstances is that taken for closure from the initially part-open condition, rather than for closure from fully-open to closed position. Thus, if the valve requires 20 seconds for complete closure but is only 25% open when discharging 2.5 litres/sec, the effective closing time is only 5 seconds. This emphasizes the need for selection of valve size to match available head, avoiding having unnecessarily large valves operating at small part opening, with possibility of undesirably rapid closure.
45. The analysis of water-hammer in a branching system is more complex than the analysis of a simple single-line system. The following discussion is consequently limited to physical description rather than numerical calculation. In the situation considered (Plate 4) in the initial steady state there is constant flow in the main line and no flow in the branch (the valve at the end of the branch remains closed throughout). Flow in the main line is suddenly reduced due to partial closure of a valve on the main line at some point downstream from the branch. This causes a pressure wave to travel up the main line. As it passes the junction with the branch a secondary pressure wave is caused within the branch. This travels down to the closed valve at the end of the branch and is reflected back. Successive stages in this process are illustrated on Plate 4, Figs. 4a to Af. Fig. 4a shows the initial steady state. Fig. 4b shows the situation shortly after the pressure wave in the main line reaches the junction with the branch. The pressure in the branch in the immediate vicinity of the junction has risen with the rise in pressure in the main line by an amount $H_1$, causing elastic expansion in the branch and flow from the main line into the branch. Thus a pressure wave $H_1$ begins to travel down the branch, flow also following behind the pressure wave. It is emphasized that this is a different situation from that discussed earlier in connection with valve closure in the main line itself. In that case the velocity and pressure on one side of the wave interface were the pre-closure values, and on the other side zero flow with full pressure rise, i.e. kinetic energy only on one side, changing to strain energy only on the other, a typical "conservation of energy" transformation.

In the case of the branch line energy is supplied to the branch by flow from the main line, and there is a rise in both kinetic and strain energy in the portion of the branch behind the pressure wave. Figure 4c shows the situation immediately before the pressure wave $H_1$ reaches the closed end of the branch. At this moment the increased pressure reaches the closed end but also the moving column of water behind the pressure wave encounters the closed end and causes a further rise in pressure to $2H_1$ (this time by conservation of energy) as the flow is brought to rest at that point. A pressure wave $2H_1$ of double the amount of that in the main line then travels back towards the junction (Fig. 4d) bringing the flow in the branch to rest as it does so. When this $2H_1$ pressure wave reaches the junction the pressure in the branch at that point fails to equal the pressure rise in the main line $H_0 + H_1$ and flow from the branch to the main line begins (Fig. 4e). A pressure wave $H_1$ then travels down the branch line with flow towards the main line following behind the wave front. On reaching the closed valve the head at the valve reduces to $H_1$ but immediately afterwards reduces further to $H_0$ in bringing to rest the velocity.

46. Summarising, the pressure at the closed end of the branch line cycles between $(H_0 + 2H_1)$ and $H_0$ for a head rise in the main line of $+H_1$. Similarly for a head drop of $-H_1$ in the main line at the junction the pressure at the end of the branch cycles between $(H_0 - 2H_1)$ and $H_0$. 
47. There are, however, two factors which in actual practice result in considerable departure from the simple situation discussed above. Firstly the diversion of flow from the main line into the branch (through elastic deformation of the latter) itself reduces the deceleration and the magnitude of the pressure wave in the main line at the junction upstream of it. The extent of this effect depends upon the diameter of the branch line in relation to the diameter of the main line. Secondly, and probably of more significance, in the above discussion it is assumed that the pressure rise in the main line is a single step, and that no further changes occur in the pressure in the main line during the pressure cycle in the branch. This is seldom the case, as with progressive (or step-wise) closure of the valve on the main line the head rise at the branch (or any other point in the line) varies with time, as illustrated in Plate 3.

48. Thus, a pressure wave initiated at the upstream end of the branch due to an increment in head rise in the main line at that point may not have had time to complete its round trip to the closed end of the branch and return to the upstream end before a further change occurs in the pressure in the main line. The factor determining the amount of pressure rise in the closed-ended branch is in fact the increment in head rise which can occur in the main line at the junction within the round-trip wave time of the branch. This is the positive rise which is doubled on reaching the end of the branch. For a given rate of valve closure in the main line it is apparent that the increment in head rise which can occur within the round-trip time of a very short branch is much smaller than for a larger branch. Hence pressure rise in a branch (above the pressure in the main line) depends upon the length of the branch. It is nil for a very short branch (such as a valved Tee), where the pressure is virtually identical with that in the main line at all times.

49. It is re-emphasized that the pressure rise in the branch line is dependent upon the pressure rise in the main line at the junction of the branch, not at the main line valve. As illustrated in Plate 3, for progressive closure the head rise at points upstream from the main line valve is generally less than at the valve.

50. To summarize, the pressure rise in a "closed-ended" branch line depends upon:

- Rate of valve closure in the main line.
- Diameter of the branch in relation to the main line.
- Length of the branch.
- Location of the branch (the junction), with regard to distance from valve and from reservoir (or other free surface) in the main line.
ANNEX D4

H. Control of Water-Hammer in Pipe System Design

51. It is evident from the previous discussion that although the physical factors contributing to water-hammer are well defined, estimation of actual values of water-hammer pressures generally can be approximate only. In the circumstances, designs should focus on minimizing the primary causes of the problem rather than sophistication in analysis.

52. The means available for limiting pressure surges due to water-hammer are:

- Keeping the design velocity in the pipe system relatively low (as water-hammer pressure rise is proportional to initial velocity V). There is, however, an economic limit to pipe size.

- Ensuring slow rate of closure of valves. This is a very effective means of limiting water-hammer, provided that the slow rate of closing is assured by mechanical means (some simple form of geared reduction or the use of a key with limited angular travel per stroke – see Plate D4 of the main text).

- Limiting the distance between valve and free surface by installation of surge risers (stand-pipes). This is particularly applicable where the height from ground surface to static water level is relatively small (length of riser is not more than some 3 m), or where high enough ground can be reached by a short up-hill branch (20-30 m) with surge riser at its upper end. This reduces the effective round-trip time T and consequently the amount by which the velocity is reduced (for a given rate of closure) during that period, which determines the extent of pressure rise at the valve.

- The use of mechanical devices such as pressure-relief valves and vacuum valves. As such valves can induce one problem while removing another, considerable care should be exercised in their selection and use. Pressure relief valves should be capable of opening very rapidly if they are to be effective in reducing water-hammer, and should be slow-closing if further water-hammer (and possibly chatter) is to be avoided on closure. Vacuum valves control negative pressure by admitting air when the line pressure falls below atmosphere. This is effective in meeting the primary purpose of the valve, but the admitted air can become a problem, particularly if an associated valve permits rapid evacuation of air. The water following the air/water inter-face is then suddenly arrested at the air valve, with consequent further water-hammer. There are
situations in which pressure or vacuum valves have to be used for water-hammer control, but such valves should be specifically designed for this application. The means of control first discussed (rate of valve closure, stand-pipes) are preferred, where circumstances permit their use.

53. Of the above measures the one which is most readily available in all situations is the incorporation of simple mechanical means to ensure slow rate of closure.
FIG. 1 - ANALOGY OF COIL SPRING
(Refer to text for discussion)

HYDRAULIC GRAD:
AFTER CLOSURE
BEFORE CLOSURE
OF VALVE

Valve

FIG. 2

Vel. Vo → Compr. Nil

Wave

Vel. Vo → Compr. Nil

Vel. Nil

Vel. Nil Compressed

Vel. Vo → Compr. Nil

Vel. Nil

Vel. Vo → Compr. Nil

Vel. Nil Tension

Vel. Vo → Compr. Nil

Vel. Nil Tension

Vel. Vo → Compr. Nil

Vel. Nil Tension
INSTANTANEOUS CLOSURE

FIG. 2a
$t = 0$ sec

FIG. 2b
$t = 0.1 = T$

FIG. 2c
$t = 0.45T$

FIG. 2d
$t = 0.65T$

FIG. 2e
$t = 1.0T$

FIG. 2f
$t = 1.1T$

FIG. 2g
$t = 1.45T$

FIG. 2h
$t = 1.75T$
The initial velocity in the line $V_0$ is reduced to zero in four equal steps, at intervals of time $0.125T$ (T being the round-trip time from valve to reservoir).

The steps are as follows:

1. $V_0$ to $0.75V_0$ at time $t = 0$
2. $0.75V_0$ to $0.5V_0$ at time $t = T$
3. $0.5V_0$ to $0.25V_0$ at time $t = 2T$
4. $0.25V_0$ to $0$ at time $t = 3T$

The diagrams trace the pressure waves due to each of the four closure steps over the period $t = 0$ to $t = 1.625T$, at time intervals of $t = 0.125T$. They also show the total pressure at each point on the line, summing the effects due to the four closure steps.
REFLECTION AT CLOSED BRANCH

FIG. 4a

FIG. 4b

FIG. 4c

FIG. 4d

FIG. 4e

FIG. 4f
DERIVATION OF EQUATIONS FOR HEAD RISE AND VELOCITY OF PRESSURE WAVE

Calculation is based on equating the kinetic energy of the flowing water in a unit length of the pipe immediately upstream of the valve before valve closure to the strain energy in the unit length of pipe after closure. The latter includes elastic deformation of the pipe wall itself and also of the water contained within it.

Terms used are the following:

- $V_0$: the velocity before closure (m/sec)
- $a$: the cross-sectional area of the pipe (m$^2$)
- $d$: the diameter of the pipe (m)
- $w$: the unit weight of water (kg/m$^3$)
- $H$: the head rise due to valve closure
- $t$: the thickness of the pipe wall (m)
- $p$: the ratio of pipe thickness to pipe diameter (t/d)
- $g$: acceleration due to gravity (m/sec$^2$)
- $E$: modulus of elasticity of the material of the pipe wall (kg/m$^2$)
- $e$: modulus of elasticity of water (kg/m$^2$)
- $C$: a constant determined by elastic properties of pipe and water (defined in text).

(A) Kinetic Energy of the stream before valve closure

For unit length of pipe, this is simply:

$$KE = MV_0^2 = awV_0^2$$

(B) Strain energy in the elastically compressed water immediately after closure

The amount of elastic deformation in the water contained in the unit length of pipe is given by:

$$\text{Final pressure (head rise } H \times w) \times \text{Volume (a x l)}$$

$$\text{Modulus } e$$

The average pressure exerted on the water during the head rise is $wH$.

The stored elastic energy is the product

$$\left(\frac{H \times w \times a}{e}\right) \times (wH) \text{ or } (wH)^2 \frac{a}{2e}$$

(C) Strain Energy due to elastic deformation of the pipe wall, in circumferential tension (axial effects are excluded herein)

A unit length of pipe is considered. The increase in tensile force on the pipe wall (each side) due to head rise $H$ is equal to

$$wH \times \frac{\text{dia}}{2}$$
The increase in tensile stress on the pipe wall due to head rise $H$ is

$$w H \times \text{dia} \times \frac{d^2}{4} \times \frac{2 t E}{\pi}$$

The elastic deformation around the perimeter of the pipe due to the head rise is

$$IV \times \left( \frac{\pi \times \text{dia}}{2 t E} \right)$$

The mean tensile force on the pipe wall during the rise in head from zero to $H$ is

$$\frac{III}{2} \text{ or } w H \times \text{dia} \times \frac{d}{4} \times \frac{2 t E}{\pi}$$

The work done in elastic deformation of the pipe wall is the product of $V$ and $VI$

$$i.e. \frac{w H \times \pi \times d^2 \times w H \times d}{2 t E}$$

or

$$(w H)^2 \times \pi \times d^2 \times d \times 1 = \frac{1}{t \times E}$$

Writing $a$ (area of pipe) for $\frac{\pi \times d^2}{4}$ and $p$ (the wall thickness ratio) for $\frac{t}{d}$

this reduces to $\frac{(w H)^2 a}{2pE}$

(D) Energy balance, before and after valve closure

Summing the strain energy components (II and VII) and equating to the original kinetic energy (I) gives the following relationship between velocity arrested and head rise.

$$\frac{a w V_0^2}{2g} = \frac{(wH)^2 a}{2e} + \frac{(wH)^2 a}{2pe}$$

$$\frac{V_0^2}{g} = \frac{w H^2 (1 + l)}{e pE}$$

or

$$H = \frac{V_0}{g} \left( \frac{1}{w H \left( \frac{1 + l}{e pE} \right)} \right)^{\frac{1}{2}}$$
The term \( \left( \frac{1}{\left( \frac{w}{g} \left( \frac{1}{e} + \frac{1}{pE} \right) \right)} \right)^\frac{1}{2} \) is entirely dependent upon the elasticity of the water, the elasticity of the pipe wall, and the thickness ratio. It is constant for a particular pipe. Writing \( C \) for diameter this term, the above relationship becomes:

\[
H = \frac{V_0}{g} \frac{C}{\pi} \text{ VIII (E) Velocity of the pressure wave}
\]

Water continues to approach the element of pipe considered with a velocity of \( V_0 \) m/sec. Thus the amount of water crossing the inter-face between flowing and stationary water is

\[
V_0 \times a \text{ m}^3/\text{sec IX}
\]

The amount of incoming water which may be stored in unit length of pipe in which velocity has been converted to pressure is determined by two factors.

(a) The increase in cross-sectional area of the pipe due to elastic deformation of the pipe wall when the flow is arrested.

(b) Elastic compression of the water already within the unit length of pipe, in effect providing space for more water.

These two factors are evaluated as follows:

(a) Increase in cross-sectional area:

This amounts to:

\[
\frac{\pi}{4} ((d + \text{change in } d)^2 - d^2)
\]

or expanding,

\[
\frac{\pi}{4} \times (\text{change in } d) \times 2 d \text{ X}
\]

The change in dia due to the pressure rise \( H \) can be obtained from the previously calculated change in perimeter of the pipe wall, as change in dia \( = \frac{1}{\pi} \times \text{change in perimeter} \)

Substituting from line V

\[
\text{Change dia } = \frac{1}{\pi} x \frac{wH \times \pi \times d^2}{2 \times t \times E}
\]

\[
= \frac{wHd^2}{2tE}
\]
Substituting in line X, the change in area becomes
\[ \frac{\pi}{4} \left( \frac{wHd^2 \times 2d}{2tE} \right) \]

This may be written as change in area (and also change in volume of unit length of pipe) = \( wH \frac{a}{P} \frac{m^3}{p} \) - - - XI

(b) Elastic compression of water:

This is simply \( wH \frac{a}{e} \) - - - XII

The total effective change in volume per unit length of pipe (available to store incoming water) is XI plus XII or

\[ wH \frac{a}{e} \left( \frac{1}{C} + \frac{1}{pE} \right) \] XIII

It is convenient to substitute the parameter \( C = \left( \frac{1}{w/g \left( \frac{1}{C} + \frac{1}{pE} \right)} \right)^{1/2} \), into XI, giving the effective change in volume per unit length as

\[ \frac{wHg}{C^2} \] XIV

The rate \( (v \text{ m/sec}) \) at which the inter-face must move upstream to accommodate the incoming flow is equal to:

\[ \frac{\text{Rate of inflow (m}^3/\text{sec)}}{\text{Volume of effective storage in unit length}} \]

Thus \( v = \frac{IX}{H a g} = \frac{Vo x a x C^2}{H a g} = \frac{Vo C^2}{g H} \) XV

However, \( H \) is itself a function of \( Vo \), and substituting from VIII

\[ v \text{ becomes } \frac{Vo C^2}{g x Vo x C} \]

or simply \( v = C - - - XVI \)

(The above relationship can be derived more directly from change of momentum considerations, the force \( wH \) equating to a change of momentum at the rate of \( (Vo w a x v) \).)

Thus \( \frac{wH a}{g} = Vo w a x v \)

and \( v = \frac{H g = Vo C \frac{1}{g} = C}{Vo} \)

However, the approach via change in volume (continuity) is more appropriate to the purpose of these notes).
Summary

The significance of the above results (see also in text) is as follows:

- The velocity of the "pressure-wave" $v$ is independent of the velocity $V_0$ in the pipe at the time of closure. It is determined only by elasticity of water and of pipe wall, and the ratio of wall thickness to pipe diameter.

- The pressure rise, $H$, for instantaneous valve closure against a velocity $V_0$, is $\frac{V_0 \cdot v}{g}$ which may equally be written (and usually is) $H = \frac{V_0 \cdot v}{g}$

- It is re-emphasized that for instantaneous valve closure the pressure rise $H$ is independent of the length of the pipe-line.
PAPER E

TUBEWELL AND RIVER-LIFT IRRIGATION
E. TUBEWELL AND RIVER-LIFT IRRIGATION

E.1 BACKGROUND AND SCOPE

E.1.1 Groundwater provides a large part of the supply of water for irrigation in the major alluvial basins of the Indian sub-continent, and is being turned to increasingly elsewhere in Asia. Choice of method of development of groundwater resources poses a number of sociological, administrative, and financial questions, as well as technical issues. The respective roles of the public and private sectors, financing and cost recovery, deep wells versus shallow wells, technology of water distribution from tubewells, direct irrigation versus conjunctive use of groundwater with surface supply, electrification of tubewell areas versus use of diesel sets, and the nature of the agricultural development to be anticipated or aimed at in tubewell areas, are all relevant questions in planning for groundwater utilization.

E.1.2 Most of the issues and options referred to above with regard to groundwater development apply equally to planning of river-lift irrigation systems, the river serving as the source of supply in place of the aquifer. It is consequently convenient to discuss the two types of development together.

E.1.3 It is emphasized that the paper refers to the smallholder environment common to much of Asia, in which individual farm ownership is preserved. Groundwater use in communal or estate types of development is not considered herein. Also excluded from consideration are the subjects of hydrology and the design of the well itself.

E.2 THE RANGE OF WELL CAPACITY AND TECHNOLOGY AVAILABLE IN GROUNDWATER DEVELOPMENT

E.2.1 Before discussing a number of the questions noted above, the types of wells available for consideration and their technical areas and limitations are briefly described.

Open Wells

E.2.2 Although not strictly a tubewell, the open well or dug-well occupies an important place in the lower end of the range of groundwater development. It is usually constructed in areas of hard rock, rather than into alluvium. To achieve sufficient yield for irrigation of 2 or 3 ha, a well 2 to 4 m in diameter is required. This may be compared with the very much deeper tubewell of much smaller diameters (10 to 40 cm) used in alluvial aquifers. Lifting of water from open wells may be manual (or bullock-powered), or by small diesel or electric pumps.
Hand-Operated Tubewells

E.2.3 This is the smallest tubewell proper, usually sunk by relatively primitive means by local contractors. It may be simply a perforated 50 mm diameter iron pipe jetted or driven into the ground (a "filter-point" well), or alternatively a 10 - 15 cm hole sunk by simple "churn drill", with fibre-wrapped bamboo filter installed. Water is lifted from the well by manual pumping, using either a small suction pump at the well-head or more generally a delivery pipe installed in the well with positive displacement pump at its lower end. The latter permits lifting of water from considerable depth.

E.2.4 Such wells are most commonly used for drinking water supply rather than for irrigation, in view of the small yield in relation to cost of this type of pump. However, there are possibilities for low-cost installation of small tubewells with simple means of baling at a cost which would encourage their use for limited agricultural purposes, particularly household gardens. Unless the height of the lift is small, however, the application of manual pumping to irrigation of regular field crops is also limited by the relatively small amount of energy which can be expended daily by one person (about 0.25 kWh). For a lift of 5 m, for instance, the amount of water which could be lifted daily would be about 12 m³. This is sufficient for irrigation of one-fifth hectare, if a daily requirement of 6 mm is assumed. Some 25% of the irrigated crop production, or its equivalent value, would be consumed in providing the necessary energy in this case (refer to Paper 68 of the Investment Centre Water Management Briefs). For small amounts of water for horticultural or garden application, however, the practical height of lift is considerably increased.

Small Tubewells with Mechanically-Driven Pump at Well-Head

E.2.5 This is the type of tubewell most commonly employed by the private farmer. The well itself may be simply a driven or jetted iron pipe, either perforated to provide an intake screen, or projecting through a silt-clay layer into a cavity formed in a sand horizon (a "cavity well"). Alternatively, the well may consist of a larger hole with conventional perforated screen and casing, the annular space around the casing being filled with graded gravel (a "gravel pack" well).

E.2.6 The centrifugal pump at the well-head is either connected directly to the pipe forming the well, or to a separate small suction pipe installed within the latter.

E.2.7 The advantage of this type of installation is its relatively low cost and mechanical simplicity. The pump is single-stage centrifugal driven either by small diesel or other engine, or by electric motor. Small "high speed" air-cooled diesel engines with rated output as little as 1.5 hp are now available. With coupled centrifugal pump the cost is of the order of Rs4,000 or US$350. The connection to the well is at the surface and the pump can be moved from one well to another if desired. As an individual well of this type has a capacity of about 7 litres/sec or 25 m³/hr, it is likely to serve an area of some 4 or 5 ha. With such a small area, special provisions for water distribution (e.g. lined channels or pipe) are not generally necessary.
E.2.8 There are two technical limitations in the application of the type of installation described. 
Firstly, if the aquifer is at considerable depth, with a thick overlay of clay or silt, the cost of well sinking becomes a significant item and operates against an installation of such small capacity. Secondly, a pump installed at ground level can draw water from a depth of no more than 5 or 6 m (the maximum "suction-head"). If the lift required is greater than that amount, the pump must be installed below ground level in a well or caisson, losing some of the advantages described. When the pump is diesel driven the engine must either remain on the surface with shaft or belt-drive down to the pump, or a caisson must be constructed large enough also to accommodate the engine. Where a portable diesel pump is to be used moving between several wells, a ramp may be constructed down to the necessary level at each well, provided that this is not more than some 2 or 3 m below ground surface. The alternative which may be resorted to, if the required depth of pump-setting below ground level would be impractical, is to employ the type of system referred to next, the submerged pump or turbine pump. However, the cost then becomes considerably greater than for a simple surface-mounted centrifugal.

E.2.9 Circumstances in which the height of lift is likely to become greater than can be reached by centrifugal pump and their implications in planning of groundwater development are discussed later.

Small Tubewells and Submerged Pumps

E.2.10 This system, which is free from all limitations in height of lift, has the pump installed within the well casing itself at whatever depth is necessary to avoid suction-head problems. The pump may be driven by a shaft extending down from a motor or diesel set at the surface or, in the case of an electrically driven installation, the motor also may be within the casing structurally incorporated with the pump. The latter is usually referred to as a "submersible pump" (although the pump itself is submerged in either case) and the former as a "turbine pump". At present price levels the "submersible" pump is likely to be the choice if electric power is available and the long-shaft "turbine pump" with diesel drive is necessarily the solution if it is not.

E.2.11 Submersible or turbine pumps have the merit of avoiding the "suction head" problem. However, they are in a price range beyond the reach of many small farmers, and require more expert maintenance than the well-head centrifugal. Small submersibles and turbine pumps (of capacity 25-50 m3/hr) are nevertheless being used by private well owners in some areas, and maintenance service is being made available by pump manufacturers.

Deep Tubewells

E.2.12 The wells referred to are in the range of 100 - 400 m3/hr, which puts them beyond simple private ownership. They are either "submersibles" or long-shaft turbine pumps as described in the previous section. The former are necessarily electrically driven; the latter may be either electrically or diesel powered.
E.2.13 Where a deep tubewell is used for direct irrigation (rather than for conjunctive use) the area supplied by the well may typically range from 0.7 to 3 ha/sec, depending upon crops to be grown and other factors to be discussed, but in any case the service area is sufficiently large to make a formal water distribution system an essential feature of the installation.

E.3 RELATIVE COSTS OF TUBEWELL SYSTEMS

E.3.1 As costs of materials, equipment and construction vary widely between countries, it would be inappropriate to propose firm figures for groundwater development. Furthermore, cost is not the only criterion in selection of system; physical and other factors may limit the viable alternatives. However, for the purpose of present discussion an indication of relative costs of the systems referred to earlier is desirable.

E.3.2 The figures quoted in Table E1 are based upon recent (1984) experience in India. It is emphasized that costs per hectare of area served are greatly influenced by the irrigation duty (litres/second/hectare), which should be taken into account in making any comparisons. It is also noted that Indian costs are generally lower than costs elsewhere - in some cases markedly so. However, the relative costs of the several types and sizes of wells are generally applicable. Conversion has been made at the rate of Rs10 to US$. Costs for the medium and high capacity deep tubewells include the cost of a buried pipe distribution system with valved outlets and semi-automated well controls. Two examples are given for high capacity deep tubewells, a 200 m3/hr installation serving 40 ha, and a 150 m3/hr installation serving 100 ha. These represent upper and lower limits of intensity of water application (the former case having a high proportion of dry season paddy). Costs in the latter case are based upon large scale current construction. Costs for the small diesel-operated filter point are much influenced by the type of diesel set employed and whether the set is used on more than one well. It is assumed not to be used on more than one well in the case shown. Capital costs in the case of electrically operated wells include the cost of transmission where indicated, but not the capital cost of generating capacity in the power system supplying the well. The latter is included in the determination of the cost of energy. The operating costs shown are exclusive of repayment of capital costs.

E.4 TECHNICAL AND SOCIAL FACTORS IN PLANNING OF GROUNDWATER DEVELOPMENT

Utilization of Aquifer Potential

E.4.1 Potential in this case refers to the available recharge, and also to the over-yearly storage capacity of the aquifer. To illustrate where there is little or no draw-down of the aquifer in the dry season, the water-table may rise to the surface early in the monsoon rains. In such circumstances much of the precipitation during the remainder of the season may be lost by surface runoff. If, on the other hand, the aquifer can be drawn down significantly in the dry season the storage available for recharge in the wet season is correspondingly increased. With regard to over-year storage, the capacity effectively available is also influenced by the ability of the tubewells in the area to follow the water-table down in a dry year. 1/

1/ In both cases it is assumed that the aquifer being drawn upon is largely "unconfined", i.e. it is effectively continuous up to the surface over much of its area. This is most likely the situation.
In view of the small percentage of drainable pore space in most aquifers (5% - 10%, commonly less) a draw-down of 5-10 m may be necessary to provide storage for 50 cm depth of irrigation if the whole area of the aquifer is to be irrigated by groundwater. Small tubewells with surface centrifugal pumps, limited to 5 or 6 m of "suction head", are not adaptable to this type of operation. They may, however, be suited to an interim period of partial development of a major aquifer (e.g. 10 to 15 years or more), a period which may exceed the physical life of such wells. Eventually, however, choice may have to be made between continued partial development of the resource by small tubewells using surface centrifugal pumps, and fuller development via submersible or turbine pumps. It is acknowledged that there is also an energy consideration in this choice. Limited draw-down of the water-table implies less energy in pumping than deeper draw-down. However, over the range of head in question the value of the additional water developed is likely to exceed the additional cost of energy, resulting from deeper draw-down.

E.4.2 A similar case is the situation in which there is limited artesian flow from wells, requiring no pump installation or pumping costs, but sufficient to irrigate a small proportion only of the potential irrigable area. Even low-head pumping from wells causes the artesian flow to cease. The value of the major additional supply obtained by pumping is likely to exceed considerably the additional cost incurred, although the few individuals previously enjoying artesian supply at little cost are placed at some disadvantage.

Physical Characteristics of the Aquifer

E.4.3 The characteristics referred to are the disposition of the water-bearing horizons within the formation, depth to water-table, the yield available from a well per unit of draw-down, rates of recharge of the aquifer and the quality of the groundwater.

E.4.4 With regard to the first item, a deep overlying layer of non-water-bearing silts or clays is a disincentive to construction of small wells, due to the high cost of well construction relative to yield. A depth to water-table of 4 or 5 m, with prospect of seasonal draw-down below that level, also operates against the small surface-mounted centrifugal pump, as previously discussed. Conversely an aquifer of relatively shallow depth and low yield per unit of draw-down is obviously unsuited to the installation of high capacity tubewells, but may be well adapted to development by small wells.

E.4.5 Rate of recharge of an aquifer is not generally capable of particularly close estimation. There is an element of trial in most groundwater development, data on aquifer performance being gathered progressively, as development proceeds. Caution is consequently indicated in the design of a major installation, particularly of large capacity wells. Prior introduction of a number of exploration-cum-production wells and their operation over a sufficient period to permit evaluation of the aquifer are desirable.
E.4.6 The acceptable quality of groundwater for agricultural use depends upon the type of soil to be irrigated, the crops to be grown and the alternative sources of water available. The latter point is particularly emphasized. Relatively saline waters can be utilized, if no other source of water is available, with adoption of particular techniques of cultivation and water application and appropriate choice of crops. However, the long-term reaction between the water and soil in question should be considered to avoid a situation of slow soil deterioration.

Groundwater Development in an Area of Canal Irrigation

E.4.7 Groundwater development in an area nominally served by surface canals may be either as a supplement to canal irrigation, or for water-table control with irrigation as a secondary benefit. The tubewell systems in the two cases may differ substantially.

E.4.8 In the first situation the small privately-owned tubewell may make an important contribution if aquifer conditions permit its use. Recharge from canal irrigation will generally ensure that the water-table is within the range of the small centrifugal pump. Alternatively, development may be through cooperatively or publicly-owned larger tubewells serving an area located within the gross canal command, but functioning separately from it (the original canal supply for the area being reallocated elsewhere). This is referred to as a Direct Irrigation Tubewell. Finally, the tubewell may discharge into the surface canal for "conjunctive use" in the canal system.

E.4.9 Groundwater development for water-table control may become necessary when canal irrigation is introduced into an area of relatively flat topography, or diversion to such an area is increased. In either case the water-table may rise due to increased seepage from the irrigated area, aggravated by the flat drainage gradient, until it reaches the surface, and waterlogging results. This is a major problem in some areas. Alternative solutions are deep open drains at relatively close spacing, or sub-surface tube drainage, or "vertical drainage", i.e. tubewells. In the latter case the tubewell delivery is used consumptively (for irrigation) if of suitable quality. Otherwise it may be blended with canal water for similar use, or simply discharged into drainage channels if quality is unusable.

E.4.10 Where privately-owned wells can be relied upon to keep pace with the increased need for groundwater extraction for drainage reasons, private wells may be the answer to the problem. However, the interest of farmers in private tubewells usually diminishes with increased availability of canal water, particularly if waterlogging is already an incipient problem. In such circumstances public sector tubewells may be necessary for control of the balance between canal supply and groundwater use. As previously noted, these may be operated either for direct irrigation or conjunctively. Conjunctive wells are the simplest to install, as they require no separate distribution system or operational organization and in many cases such wells are the obvious course. However, it must be pointed out that tubewell water discharged into a canal inherits whatever disabilities, limitations, or inefficiencies the canal distribution system may have downstream from that point. In contrast, the same water delivered via a direct irrigation system may have the advantages of delivery close to the field, demand operation, etc., which may have a considerable influence on the type of agriculture which develops in the area in question.
E.4.11 Comparison of the relative merits of conjunctive and direct irrigation tubewell development in a particular area requires consideration of a number of factors including:

i) The density of the surface canal network.

ii) The technical standards and delivery efficiency of the minor canal and watercourse systems.

iii) The seasonal irrigation intensity being provided through canal supply and the anticipated proportionate increase due to groundwater supply.

iv) The reliability of power supply (particularly in the case of conjunctive wells).

v) The likely level of agricultural development under the two alternative systems.

Power Supply for Tubewells

E.4.12 The alternatives are supply from a public transmission network where such is available, or use of individual diesel sets for each tubewell. Centralized diesel generating installations supplying local groups of electrically-powered tubewells, while technically feasible, are not widely used.

E.4.13 Electric power, where the supply is reasonably reliable, is certainly most convenient. The cost of transmission may, however, be a limiting factor in the use of electric power. If a transmission network of sufficiently close density already exists or is regarded as a part of the rural infrastructure, then the cost of connection to tubewells, particularly to larger wells, may be a relatively small component of cost per hectare served (refer to Table El). However, the cost of connection to individual small wells is higher per unit of area, unless the transmission infrastructure is already of high density.

E.4.14 In the case of individually owned wells the small diesel set (1.5 to 5 hp) does not present a problem of operation and maintenance. For larger wells, however, either publicly or cooperatively owned, maintenance of diesel sets requires more skilled attention and certainly more frequent attention than for electric motors of comparable size. This has a bearing on the question of small private wells versus larger publicly owned or cooperatively owned wells, and the choice between public or cooperative ownership in the latter case.

E.4.15 Where electric power supply is available, the routine attention required by even a large capacity motor is relatively minor. Where it is not available and diesel sets must be used, the logistics of fuel supply and of maintenance of such sets becomes an important factor in choice of system of well ownership and operation. One system in use has the Government agency owning and maintaining the diesel set (and well), with the group served by it procuring the fuel and providing the operator. A variant of this system is the rental of the diesel set to the user group, with maintenance provided (at low cost) by Government workshops. In either case, Government is faced with owning and maintaining a large number of diesel sets and operating the necessary workshops and stores. Efforts to transfer ownership and responsibility for maintenance of diesel sets to user groups, permitting
Government to withdraw from this activity, have not been particularly successful as users prefer the subsidized Government service.

E.4.16 To summarize, the availability of electric power for tubewell operation may have an influence on the type of well ownership and operation adopted. Small diesel sets for small individually owned wells do not present a problem. Nor do electrically operated larger wells, where the degree of support by Government agency for pump maintenance, either to cooperatively owned or to Government owned wells, is not a major factor. With larger diesel-operated wells maintenance becomes a more substantial item and there is less possibility of cooperative ownership functioning effectively, except in a relatively highly developed community.

Size of Holding

E.4.17 Size of holdings and degree of sub-division of holdings (i.e. size of "parcels") are primary factors in the design of groundwater development in a smallholder situation.

E.4.18 Where the nature of the aquifer permits the use of small wells with centrifugal pumps, either diesel or electrically operated, privately owned wells are likely to supply a proportion of the area. The farmer with a sufficiently large parcel of his own (e.g. about one-half hectare) and prospect of sale to neighbours to bring the total to 2 or 3 ha, is in a position to install a small well and probably has access to credit for that purpose. However, if the average size of holding is one-half hectare and average size of parcel as little as one-tenth hectare the proportion of the area likely to be supported by individually-owned pumps is much reduced. Cooperative ownership of a small well is a possibility in such circumstances, although difficulties are commonly encountered in organizing the 10-20 owners who would be served by such a well, either for purposes of obtaining credit or in subsequent sharing of the water. Development by private wells owned by the larger farmers may stop considerably short of meeting the whole potential need in such circumstances. The small farmers not so supplied can be expected to look to Government intervention for the solution to their problem.

E.4.19 Where the nature of the aquifer prevents development by small centrifugal units, or considerations of future full utilization of the aquifer weigh against it (as previously discussed), use of submersible or long-shaft turbine pumps becomes necessary. In this case, development by private wells is likely to be restricted to a few large farms. Government ownership or cooperative ownership of wells are the primary options for the remainder in such circumstances. However, as such wells are necessarily of larger capacity than small centrifugals, the number of cultivators served by a well can be very large, particularly if the size of parcels is small. Purely cooperative management of such a system without at least minimal Departmental supervision is likely to be difficult.

Equity in Distribution of Groundwater

E.4.20 In the most general case of groundwater development, with recharge largely from precipitation (i.e. in the absence of lateral recharge), there is insufficient groundwater available for full irrigation of the area overlying the aquifer. Formal licencing of groundwater use, in the interest of equitable distribution, is not generally practiced in Asia.
E.4.21 A particular issue is the intensity of irrigation to be provided when it is apparent that with the recharge available not all of a gross area can be covered. The options are either to size the service area of each tubewell so that 100% of it can be irrigated in the critical season, or in the interests of equity to increase the area served by a well so as to spread the benefits over more cultivators but providing a seasonal irrigation intensity of less than 100%. This is a social rather than an economic question. Even with reduced irrigation intensity a proportion of the gross area may have to be excluded from service.

Existing Wells and Possible Conflict in Further Development of an Aquifer

E.4.22 A very pressing example of the problem of equity is the case of the owner of existing small well (centrifugal type, with a maximum suction lift of 1.2 m) whose well may be put out of service by a programme of deep well construction, particularly a Government-financed programme. Where the programme is specifically aimed at draw-down of the aquifer in the interests of greater utilization of the resource, the broader public interest may be considered to prevail, although incorporation of areas currently supplied by small wells into the new supply system should be provided for as far as possible.

E.4.23 However, where the reasons for installation of the new system do not involve systematic draw-down of the aquifer (for instance, where the objective is supply to areas of very small holdings) the impact of the new wells on existing wells should be given particularly close attention. A single large capacity well will locally depress the water-table, even where a general lowering of the water-table is not aimed at. Consequently, such a well should not be located in an area where there are a considerable number of existing privately-owned small wells. Just what constitutes a "considerable number" is again a question of equity. If the existing wells supply only 15% of the area and the remaining 85% is without supply for good and sufficient reasons, then there is probably a case for a large capacity well to supply the whole area including (at owner option) the areas currently supplied by existing wells. If, however, the existing wells already supply 30% or 40% of the area such introduction may not be justified.

E.5 PRIVATE VERSUS PUBLIC DEVELOPMENT, A SUMMARY

E.5.1 Most of the factors, technical and sociological, bearing upon this question have been discussed above. There are, in fact, three degrees of well ownership to consider, private, cooperative and public. The position is summarized as follows:

a) Where small centrifugal pumps may be used, privately owned wells (diesel or electric) are likely to play a major part in groundwater development of an area. However, where holdings are very small, and particularly where they are divided into even smaller parcels, the individually-owned small well is unlikely to be the complete solution. Cooperative ownership of small wells is a possibility, but has not always been successful. Public ownership of larger wells may be desirable in those portions of the area with predominantly very small cultivators.
b) Where small centrifugal pumps cannot be used for technical reasons, privately-owned wells will play a much lesser role in development unless holdings are generally large. The alternatives are cooperative or public ownership. The smaller the holdings, the greater the difficulty with cooperative ownership.

c) Where electrical power is available, the amount of expert (Departments) maintenance required is relatively small and cooperative ownership, or cooperative operation of Departmentally owned wells is facilitated. Where electric power is not available and diesel sets must be used (other than very small units), the amount of expert maintenance required is greater and Departmental ownership is likely to be necessary, except with more advanced cultivators. However, cooperative operation of a Departmentally owned set, including provision of fuel and possibly payment of rental for the set, may be a practical solution.

d) Where wells of capacity 30 litres/sec or greater are indicated, and particularly where crops other than paddy are to be grown in at least one season of the year, a formal distribution system from well to furthest fields is desirable if agricultural production is to be maximized and operational problems minimized. Such a distribution system is unlikely to be provided with a cooperatively owned well, where initial cost is a primary consideration. Provision of such a distribution system for a well of medium or large capacity usually implies Departmental construction and probably Departmental ownership. Cooperative operation, including payment for power and for services of an operator, is nevertheless possible in such circumstances.

e) Much of the debate over private versus public ownership of tubewells in fact relates to cost recovery. With privately owned wells all costs, including capital charges, are necessarily met by the owner (except for capital subsidy which is often considerable). With wells owned and operated cooperatively the situation is similar. On the other hand, where wells are publicly owned the degree of direct cost recovery is generally less, often much less, although this need not necessarily be the case. The subject is discussed further in the next section.

E.6 ECONOMIC AND FINANCIAL FACTORS

E.6.1 As an indication of the order of economic viability of a public tubewell, and its sensitivity to costs and operating hours, values for the 42 litres/sec well listed in Table E1 may be of interest. Crops assumed are basic cereals, except for some 6% in potatoes and 12% in pulses. Assumed annual intensity of irrigated crops is 150%. The estimated Economic Rate of Return is approximately 25%, assuming operation for 3,300 hours per year (37% of time). "Switching values", i.e. variations in costs or returns which would bring the rate of return down to 12% (assumed to be the opportunity cost for capital), are an increase of 110% in construction cost, or a decrease in operating hours to 1,600 per year.
E.6.2 The following are the costs per hectare for the above case of a 42 litres/sec well serving 100 ha, and also for the case of a 56 litres/sec well with much more intensive irrigation of service area of 40 ha (also shown in Table E1):

<table>
<thead>
<tr>
<th>Service area (ha)</th>
<th>Total capital cost/ha (including trans.) (US$)</th>
<th>Cost of buried pipe distribution system and valved outlets/ha (US$)</th>
<th>Distribution system as a percentage of total (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>100</td>
<td>480</td>
<td>250</td>
<td>52</td>
</tr>
<tr>
<td>40</td>
<td>910</td>
<td>340</td>
<td>38</td>
</tr>
</tbody>
</table>

E.6.3 The above capital costs per hectare for the complete systems are less than one-third of the current capital costs for reservoir and canal projects in the same locality. Operating costs on the other hand, including energy and also maintenance and depreciation of mechanical equipment, are higher with tubewells. However, if annual operating costs of the latter are capitalized at 10% and added to capital cost, the totals (US$1,230/ha and US$1,950/ha for the 42 litres/sec well and the 56 litres/sec well respectively) remain less than the current capital cost of reservoir and canal projects. This comment is not intended as a reflection on the viability of reservoir and canal systems, which will certainly continue to be constructed where sites are available, but rather to put the cost of tubewell systems (particularly large capacity public wells) in the context of prevailing costs of providing irrigation by other means.

E.6.4 While capitalization of operating costs is informative from the viewpoint of the agency constructing and operating the wells, operating costs are a recurrent charge on annual Departmental budgets, while capital construction costs are not. Collection of operating costs from the irrigators is consequently of vital concern to operating Departments and to Government. At the levels of O & M costs indicated in Table E1 for the high capacity wells (42 and 56 litres/sec) it is reasonable to seek recovery of full O and M costs through water charges, certainly by the time production from a tubewell area has reached full potential. Cost recovery is very much facilitated by maximizing the use of the well, i.e. running hours, particularly by encouraging hot weather crops. Thus, in the case of the 56 litres/sec well an increase in annual running hours of 20% would increase agricultural production (and associated water charges) by at least 20%, at the cost of an increase in annual O and M costs (primarily energy) of less than 8%. Recovery of full O and M costs from tubewell cultivators is equivalent to recovery of a much higher percentage of total O and M and capital costs (about 50%) than is ever attempted for reservoir and canal systems.
E.6.5 The methods of recovery of O and M costs from public wells, in practice or as proposed, include the following:

- Levying an appropriate volumetric water rate (where circumstances permit, recording of volume of water used by the individual).

- Levying crop water charges calculated to recover the required amount.

- Requiring that cultivators served by a well pay power (or fuel) charge directly to the supplier and provide their own operator. This leaves maintenance and depreciation to be covered by an annual levy on the cultivator group.

E.7 DESIGN OF TUBEWELL DISTRIBUTION SYSTEMS

Introduction

E.7.1 Before discussing distribution in tubewell systems in detail, it is of interest to make general comparison with conveyance and distribution of water in canal irrigated areas. There are three factors which particularly distinguish tubewell (and river-lift) irrigation from canal irrigation:

a) Subject to the capacity of the well and supply of power, irrigation from a tubewell is available virtually on demand throughout the year. It is not subject to seasonal limitations in supply, as is commonly the case with canal systems.

b) A direct irrigation tubewell is generally located within its own relatively small service area. The distance from well to furthest field is seldom more than 1 km, and is generally less. Viewing the area of development as a whole, the underlying aquifer may be regarded as the primary conveyance and storage system. Supply points (wells) are distributed throughout the developed area. By comparison the distance of conveyance from reservoir to field in the case of a canal system may be hundreds of kilometres. River-lift systems located along the banks of a stream are similar to tubewells in this respect, the river providing primary conveyance and pondage. The distribution distance within each river-lift service area is relatively short.

c) Distribution from a canal system to the field is by gravity and, with some exceptions, is necessarily by open channel. In the case of tubewell or river-lift systems, however, delivery at the well or pump can be made under sufficient head to permit the use of pipe distribution systems. Pipe systems may be under-sufficient head for sprinkler operation at outlets or, alternatively, the pipe may be simply an efficient method of conveyance to low-head outlets from which field application is by gravity. This paper is directed at the latter, although not ruling out the merits of sprinkler systems in appropriate situations.

d) To summarize, a tubewell brings the point of supply relatively close to the field. In the case of a typical small "filter point" well serving 1 or 2 ha only, the well is very close indeed to any field served by it, probably no more than 50 or 60 m. On the other hand, a larger well, typical of public supply, may be 700 m from
the furthest field. While this distance is small compared with the kilometres traversed by the flow in a canal system, it is nevertheless a very important factor in the design of tubewell distribution. A tubewell provides a well-regulated, reliable, year-round supply with the possibility of very flexible scheduling of irrigation to the individual plot, in short all of the ingredients for intensive high-value agricultural development, provided that these advantages are not lost between the well and the field. The latter may very well be the case if delivery is via a precarious open channel, traversing boundaries between 40 or 50 plots all of different ownership, and subject to theft, water seepage, and interruption generally, en route.

e) In the remainder of this paper, attention is confined to "high capacity" tubewells, generally of capacity 40 litres/sec or greater.

Desirable Size of Delivery Stream (to the Individual Cultivator)

E.7.2 This depends upon factors such as size of plot, infiltration rate of soil, length of farm channel, etc., but in the situation under discussion it is likely to be in the range of 14 to 21 litres/sec. The size should be a multiple of the tubewell capacity (indicating delivery at two, three or generally not more than four outlets simultaneously).

Choice of Construction Material for Distribution System

E.7.3 The first question is open channel or buried pipe. Operationally, buried pipe has a number of advantages. It solves the right-of-way problem (a perennial issue with smallholdings); it is not subject to tampering and theft of water; maintenance is negligible; and it remains full and delivery is resumed instantly on well start-up after interruption (of considerable importance where power supply is subject to frequent outages). Finally, delivery efficiency is very high. On the negative side, pipe systems require head (commonly 2.5 - 3.5 m) which represents an energy cost, although partly offset by the high delivery efficiency.

E.7.4 In balance, for tubewells of the capacity considered and in a smallholder situation, buried pipe is operationally preferable, and with appropriate design it can be provided at a cost no greater than for a lined open channel system. It is the choice assumed herein.

E.7.5 The second question is pipe material. The options include concrete plastic (PVC), asbestos cement, steel, aluminium, and cast iron. At present price levels and assuming the use of light duty pipe (pressure rating 2.5 kg/cm²) the cost of PVC pipe installed is less than that of the equivalent alternative materials for sizes up to 200 mm diameter. Actual procurement price of PVC pipe is influenced very much by whether or not duty is levied on the resin. On the basis of international prices for resin (without duty) and current prices for "conversion" or manufacture, typical costs at rail-head are approximately as follows (September 1985 price levels):
## Size of Service Area of Tubewell

### E.7.6
Both agronomic and sociologic considerations can contribute to determination of size of service area. Estimation of nominal irrigation requirements follows the same procedure (e.g. modified Penman, or other) as for canal irrigation. As in the latter case, however, actual water use may differ considerably from the nominal "optimum". In areas of water scarcity, traditional practice in canal irrigation is usually to maximize crop yield per cost of water rather than per unit of area (by "sub-optimal" water application) and this approach may well be appropriate to tubewell irrigation in some situations. In the case of paddy, the questions most debated are water requirements in land preparation, 1/ and the possibilities for economy in water use by periodic withdrawal of standing water from paddy fields.

### E.7.7
To summarize, in determining the size of service area of a tubewell there is scope for considerable departure from conventional calculated "optimum" crop water requirements. An associated question previously referred to is the design intensity of irrigation - i.e. should the system be developed to irrigate the entire area of each holding in a particular season of the year or, alternatively, should it be designed to irrigate a proportion only of that area so that a greater number of holdings can benefit from the well.

### E.7.8
A similar question is encountered when paddy is the indicated crop in one season of the year, and non-paddy crops for the remaining seasons. If the size of the service area is limited to that which can be supplied by the tubewell during the critical period for paddy (for instance in land preparation), assuming 100% of the area is under that crop, then in non-paddy seasons there will be surplus unused capacity. It may be more appropriate, at least on social grounds, to make the service area larger, possibly sizing it on the basis of non-paddy crops, and to accept less than 100% irrigation intensity in the paddy season.

---

1/ Land preparation and cultivation procedures for paddy which are now being practised in some areas result in reductions of around 50% in water use, compared with traditional methods.
E.7.9 No particular recommendations are made on the above questions at this time, as their resolution is likely to be specific to each project situation. However, as the pipe system under discussion is capable of very efficient water distribution, economy in field application of water can reasonably be expected of the cultivators concerned and should be presumed in sizing of the well command.

E.7.10 A further factor requiring some judgement in determining size of service area is the number of hours per day for which power supply can be relied upon. This is not necessarily 24 hours. Curtailment of power supply for tubewell operation during hours of peak demand on the supply system may be in the national interest, also during outage of generating plant. Such curtailment may be primarily through systematic rostering of supply, but some unscheduled interruptions are also to be expected in most systems. An average of some 16 hours per day availability of power in the period of maximum water demand is adequate for economic operation of tubewells. The use of "dedicated feeders" (supply lines devoted to project tubewells only) can improve supply, but is not a universally available expedient.

E.7.11 Area commanded per litres/second of well capacity in recently designed systems varies from 2.3 ha (principally non-paddy crops, with seasonal irrigation intensity less than 100%), down to 0.7 ha (paddy an important item in the cropping pattern and 100% seasonal intensity).

Area Supplied by an Individual Outlet

E.7.12 An important design objective is delivery from the pipe system at points as close as possible to the individual field. Ideally, each field would have its own outlet, but some compromise is necessary in the interests of cost of pipe and of outlet structures. The area served by an outlet should be in the range 2 - 4 ha, factors influencing choice of area, including minimizing losses in travel from outlet to furthest field, and keeping the number of cultivators supplied by an individual outlet reasonably small.

Division of Tubewell Supply between Outlets

E.7.13 With a tubewell of capacity typically in the range 40 to 80 litres/sec and the desirable size of delivery stream at an outlet 15 to 20 litres/sec, it is apparent that two or more outlets (typically either two, three or four) must be in operation at one time. Division of the tubewell discharge equally between multiple outlets is a key question in tubewell distribution system design. The problem centres on the fact that individual outlets may be at different elevations and at different distances from the tubewell, i.e. the hydraulic head at the outlet varies from one outlet to another. Possible methods of regulation of flow include the following:

a) Manual adjustment of flow at each outlet.

b) Automatic flow control at each outlet.

c) Sub-division of the distribution system into sub-units, each of which has one outlet only in operation at a time, and division of flow between sub-units at the tubewell.
E.7.14 Alternative (a) is operationally less than satisfactory, and is unlikely to result in the degree of assurance of equitable delivery necessary to establish cultivator confidence. Alternative (b) awaits development by the industry of a suitable low-cost, low-head, flow controller. Alternative (c) is the one currently in use in India, this being accomplished by sub-division at the well by means of a flow-dividing weir in a control chamber. Under conditions of normal operation, i.e. one outlet open in each pipe sub-unit, the well discharge is automatically divided equally between the outlets in operation at the time, regardless of their location or elevation.

Layout of Pipe System 1/

E.7.15 The pipe sub-units, assumed to be two for the purpose of this discussion, are independent systems, each of capacity one-half (in this case) of the tubewell discharge and each having a number of outlets along its length. The number of outlets is likely to be at least six, and maybe more, of which only one at a time is normally in operation. Layout begins with division of the tubewell command into areas to be supplied by each outlet, followed by location of the outlets in the most convenient position to serve each of those areas. The pipeline is then laid out so as to supply the outlets. (One of the advantages of the buried pipe system is complete freedom of choice in alignment without the need for consideration of right-of-way or property boundaries.)

E.7.16 There are, however, two possible choices in layout. One is the open branching system, in some cases amounting to a single line. The other is the closed loop system (see Plate E1) the loop being connected to the tubewell at one point and the outlets disposed around the loop (which may be of any shape required). The loop system has considerable advantages in pipe size and cost, where applicable, as discharge at the single outlet in operation at any one time is supplied by flow around both sides of the loop. However, size and shape of the area supplied by the sub-unit may indicate use of the branching system (or straight line) in some situations. Choice is made in designs simply by trial layout and comparison of cost.

Regulation of Tubewell Output

E.7.17 Apart from the necessary protective relays at the well, the normal functions to be provided for are the following:

a) Start-up on commencement of irrigation.

b) Re-start after unintentional outage, if there is still demand for irrigation.

c) Shut-down of well where irrigation ceases.

d) Reduction of supply from the well if irrigation ceases on one of the sub-units ("loops").

E.7.18 All four of the above functions can, of course, be provided manually. However, with operations spread throughout the 24 hours this either requires constant attendance of an operator, or poses the likelihood of operation suffering from lack of his presence when needed. With the system now in use in India all four functions are automatic. It is, in principle, a system of downstream control applied to pipe distribution. Briefly, when irrigation is initiated by opening an outlet, this is signalled to the control chamber near the well (Plates E2 and E3) by a fall in level in the chamber, which in due course will cause the pump to start (by the functioning of a lower-level probe). Conversely, on shut-down of all outlets the level in the chamber will rise until an upper-level probe shuts off the pump. In the intermediate case of irrigation continuing on one loop only, requiring only half the output of the well, the upper and lower probes will cause the pump to cycle on and off, over a period of some 10 - 12 minutes. An alternative to the latter arrangement has a float-controlled sleeve-valve in the control chamber on the discharge line from the well (Plate E4). This throttles the flow from the well to half normal discharge if one loop only is in service. The latter arrangement is particularly suited to larger capacities of well, when there may be three or four loops, with possibility of only one loop being in operation at one time for short periods (this is not a normal operating condition). The sleeve valve, which operates in conjunction with upper and lower level probes, permits use of a smaller control chamber, and has other operational advantages.

E.8 OPERATION OF THE TUBEWELL SYSTEM AND THE ROLE OF CULTIVATOR GROUPS

E.8.1 While the design of an irrigation system is usually based upon certain assumptions as to how it will be operated, the actual method of operation is dependent upon agronomic and economic factors and cultivator attitudes which are not entirely predictable at the design stage. It is consequently of particular importance to build into a system sufficient flexibility to cater to a variety of possible modes of operation and to review the prospective performance of the system under those possible operational demands. This philosophy makes a clear distinction between "hardware" features of a system, i.e. physical items built into it and not capable of change, and "software". The latter involves operational programmes which may be revised or radically changed in the future.

E.8.2 The "hardware" provided by the system in question is two (or three, or four) buried pipe sub-systems, each receiving an equal share of the well discharge under normal operations. Each sub-system has a series of valve-controlled outlets serving small near-equal portions of the sub-system command. If operated in rotation, one outlet at a time, the flow at an outlet is of the same amount regardless of where the outlet is located in the system. If several, or all, outlets are opened at one time the subdivision of flow between them may not be equal, although the total flow into the sub-system remains at the design amount.
E.8.3 Whether allocation of water between cultivators is based upon equal entitlement for equal area, or upon agreed allotment for particular crops and area of crop, the delivery system functions equally well. Where there is equality in entitlement between outlet commands and each takes water once a week, operation is simplified, and in some situations each of six or seven outlets can operate on a fixed-day-of-the-week basis. Such operation is not, however, essential, nor does a fixed-day group necessarily have to be associated with a particular single outlet. It may operate for part of the day on one outlet, and for the remainder of the day (possibly in a different location for reasons of social homogeneity) from a different outlet.

E.8.4 Operationally there are three levels of water allocation or rotation; to the two or more separate sub-systems or loops, to the individual outlets on each loop; and finally to each cultivator or plot within the area served by an outlet.

E.8.5 A central issue is the level down to which rotation should be directed or supervised Departmentally, i.e. below which cultivator groups take over water management. As the system is designed to ensure equality of well discharge to the two or more loops, the question of rotation at the primary level does not normally arise. Rotation between the outlets within a loop, may, in some circumstances, be left to the cultivator organization representing the loop command (which may be from 15 to 50 ha). In other circumstances, external authority may be involved, at least in seasonal scheduling if not in day-to-day operation. However, for rotation within the outlet command a 2 to 4 ha area should be organized by the cultivators themselves, as informal exchange of weekly entitlement in accordance with individual farming operations is essential. Particularly where power supply is liable to interruption and fixed rotation schedules may consequently be disrupted, a considerable degree of autonomy in management by the cultivators is desirable. The design of the system facilitates a variety of methods of water management. However, the success of the actual operation depends upon either effective self-management by water-user groups or close supervision, preferably the former.

E.9 AGRICULTURAL SUPPORTING SERVICES IN TUBEWELL AREAS

E.9.1 A tubewell system of the type described provides facility for supply of water throughout its service area, immediately upon completion. It does not necessarily follow that cultivators will promptly take up irrigation throughout the area. Irrigation, from the cultivators' viewpoint, simply becomes an additional option now open to him. Encouraging cultivators to take up that option is a function of agricultural extension. Also essential is the provision of the necessary supporting services (supply of fertiliser, seeds, credit, marketing, etc.) necessary for conversion from limited rainfed agriculture to full irrigation.

E.9.2 The same comment could, of course, be made in the case of canal irrigation, but the gestation period is longer with a canal system, particularly the associated land development (watercourses, field channels, land shaping). With a public tubewell development the potential for very rapid conversion to irrigation warrants a pre-planned campaign of agricultural extension and services, implemented in parallel with well construction.
E.10 RIVER-LIFT SYSTEMS

Design and Operation

E.10.1 Much of the above discussion of tubewells applies equally to river-lift systems, particularly the requirements of distribution within the service area. There are significant differences, however, in the roles of the private and public sector. With regard to the first item, whereas tubewells can be distributed throughout an irrigation area, river-lift pumping stations are necessarily confined to the banks of the river in question, or its tributary channels. This often leads to strip development, with width of the area reached by irrigation being dependent upon the capacity of the individual pumping stations and the nature of their distribution systems. An example is the small privately-owned diesel or electrically operated pump set, with reach limited to 200 or 300 m from the river.

E.10.2 To generalize, the options available are the following:

- **a)** Small-scale, privately-owned pump sets located on the river bank.
- **b)** An extension of the application of small sets by the construction (at public cost) of channels branching from the river and serving small pump sets along their length. This course is feasible only when the depth of channel required is relatively small.
- **c)** Installation of publicly-owned pumping plants on the river bank, lifting water into open canals or aqueducts which serve the irrigation area by gravity.
- **d)** As in (c) but with a buried-pipe distribution system. This solution is usually limited to the intermediate range of size of irrigation area.

E.10.3 Systems of all four types are in use, and have their place in irrigation development. Discussion herein is confined, however, to the fourth type, simply referred to as river-lift irrigation.

E.10.4 The size of area which may be served by a single pumping station and pipe distribution system is influenced very much by considerations of pipe system hydraulics and pipe cost. While there are economies of scale in the cost of pumping plant, larger installations being cheaper per unit of water delivered, this is not the case beyond a certain point with pipe distribution systems. A pump station capacity of about 100 litres/sec is probably optimum, where geography permits development of the particular area with units of that size. However, much larger pump installations are called for in certain circumstances. A pump installation of 100 litres/sec capacity is assumed for the remainder of this discussion.

E.10.5 Determination of sizes of service area, size of delivery stream and area served by an individual outlet, follow the same reasoning as for a direct irrigation tubewell. In the case under discussion it is assumed that the delivery stream (the outlet discharge) is 16 litres/sec, the 100 litres/sec pump station output being divided between six outlets at any one time. As in the case of a tubewell system, there are three ways of achieving this division:
a) manually;

b) by flow controller on each outlet; and

c) by division of the distribution system into six separate units ("loops") and controlling the flow to each at a distribution chamber (in this case a primary and secondary chamber).

In an Egyptian installation with which the World Bank is involved, the second alternative, i.e. flow control at each outlet, is being adopted. In a river-lift system to be constructed in India (West Bengal) the third alternative will be used, i.e. division of the system into six sub-units, or loops, each supplying one outlet at a time.

E.10.6 The system includes a distribution structure on a high point near the pumping station at which the flow is divided between two sub-units, each taking one-sixth of the flow, and two trunk lines each of which takes one-third of the flow. Each of the latter is later divided between two further sub-units at two secondary distribution structures (Plate E6). The specific arrangement of distribution structures and of "loops" (or branching sub-systems) depends upon the shape of the well command area. In each case, however, the arrangement is down-stream control, as in the case of the tubewell system previously described. Pump control (matching output to the number of loops in operation) and automatic start and shut-down follow the same in principle as for the tubewell system. In the case of diesel operation (rather than electric) control at the pump is manual, although semi-automatic is also possible.

E.10.7 The role of cultivator groups in operation of the system, and procedures in water allocation and rotational supply, also follow the same pattern as for the direct irrigation tubewell system.

Private versus Public Development of River-Lift System

E.10.8 As noted earlier the area which can be served by small river-lift pump sets is confined to a relatively narrow strip along either bank of a river channel. Private ownership or cooperative ownership of small diesel sets is common, also Departmental rental of the small sets to individuals or groups. Maintenance of such small diesel sets is not a problem.
E.10.9 For larger installations covering a considerably greater area the position regarding ownership is similar to that discussed for large tubewells, except that the capacity of pumps, size of area served, and number of beneficiaries may be somewhat larger with river-lift system. The options are generally limited to cooperative ownership of the installation, public ownership and operation, or public ownership and maintenance of the facilities but operation by the user group. As discussed in connection with tubewells, the availability of electric power considerably simplifies the problem of maintenance, compared with large diesel sets, and minimizes the extent of Departmental technical support. Cooperative operation of Departmentally owned and maintained equipment is thereby facilitated. In more highly developed agricultural areas with a tradition of cooperative ownership of facilities (such as sugar mills) the cooperative ownership of large river-lift installations, whether diesel or electric, is entirely practical. In less developed areas, particularly where large numbers of small farmers will be served by a river-lift system, public ownership and operation (at least initially) is likely to be required. A fully effective distribution system of the type described herein, designed for equitable service to small farmers, is unlikely to be installed by a cooperative. However, it may well be operated by a cooperative once experience has been gained in its use.
## TUBEWELL AND RIVER-LIFT IRRIGATION

### Comparative Costs of Tubewells

<table>
<thead>
<tr>
<th>Filter Point</th>
<th>Low Cap. &quot;Shallow&quot; Tubewell 20 m³/hr or 0.2 ft³/s each</th>
<th>Medium Cap. &quot;Deep Well&quot; 100 m³/hr</th>
<th>High Cap. &quot;Deep Well&quot; 200 m³/hr (2 ft³/s)</th>
<th>High Cap. &quot;Deep Well&quot; 150 m³/hr (1.5 ft³/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Area Served:</td>
<td>2.4 Ha</td>
<td>4.0 ha each</td>
<td>20 Ha</td>
<td>40 Ha</td>
</tr>
<tr>
<td>Capital Cost</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Total US$</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>With transmission</td>
<td>N.A.</td>
<td>N.A.</td>
<td>21,600</td>
<td>31,800</td>
</tr>
<tr>
<td>Without transmission</td>
<td>1,300</td>
<td>2,200</td>
<td>12,500</td>
<td>22,800</td>
</tr>
<tr>
<td>Capital Cost</td>
<td>US$/ha</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>With transmission</td>
<td>N.A.</td>
<td>N.A.</td>
<td>900 3/</td>
<td>1,330 3/</td>
</tr>
<tr>
<td>Without transmission</td>
<td>540</td>
<td>550</td>
<td>520</td>
<td>950</td>
</tr>
<tr>
<td>Operating Cost</td>
<td>US$/ha/yr</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Energy</td>
<td>118</td>
<td>71</td>
<td>26 1/</td>
<td>26 1/</td>
</tr>
<tr>
<td>Maintenance and depreciation</td>
<td>45</td>
<td>44</td>
<td>40</td>
<td>83</td>
</tr>
<tr>
<td>Night guard</td>
<td>-</td>
<td>-</td>
<td>10</td>
<td>10</td>
</tr>
<tr>
<td>Operator</td>
<td>-</td>
<td>-</td>
<td>40</td>
<td>40</td>
</tr>
<tr>
<td>Total</td>
<td>163</td>
<td>115</td>
<td>116</td>
<td>159</td>
</tr>
</tbody>
</table>

1/ Dynamic head 5 m.
2/ Dynamic head 8 m.
3/ Min 600 m. of 11KV per cluster and 1,000 m. of 400 v.
4/ February 1983 appraisal report.
SCHEMATIC LAYOUT FOR A DISTRIBUTION SYSTEM
FOR TYPICAL IMPROVED STANDARD TUBEWELL COMMAND

<table>
<thead>
<tr>
<th>Area Day Commands</th>
<th>Symbol</th>
<th>Irrigated by Outlet Number</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Loop A</td>
</tr>
<tr>
<td>Monday</td>
<td>I</td>
<td>1,2,3</td>
</tr>
<tr>
<td>Tuesday</td>
<td>II</td>
<td>2,3</td>
</tr>
<tr>
<td>Wednesday</td>
<td>III</td>
<td>3,4,5</td>
</tr>
<tr>
<td>Thursday</td>
<td>IV</td>
<td>3,5,6</td>
</tr>
<tr>
<td>Friday</td>
<td>V</td>
<td>1,6,7</td>
</tr>
<tr>
<td>Saturday</td>
<td>VI</td>
<td>7,8,9</td>
</tr>
<tr>
<td>Sunday</td>
<td>VII</td>
<td>1,0,10</td>
</tr>
</tbody>
</table>

Tubewell and Distribution Chamber
Outlet Valve 6" P.V.C. Areas A and B are supplied simultaneously with 0.75 Cusec Each.
Standpipe
Feeder Pipe
Distribution System-buried P.V.C. Pipe Loop.
Boundary of 'Area Day' Commands (about 7 ha).
REGULATING DISTRIBUTION CHAMBER
(To Serve a Two Loop Distribution System)

"Exploded" View to Show Internal Channels

1/ The electrical probes are activated by upper and lower water levels and operate the pump switch relays.

2/ See Plate E1 for layout of a buried pipe distribution system.
DESIGN AND OPERATION OF IRRIGATION SYSTEMS FOR SMALLHOLDER AGRICULTURE

IN SOUTH ASIA. VOL. II

DISTRIBUTION CHAMBER

(200 cub. m./h TUBEWELL)

1-CHAMBER (REINF. BRICKWORK)
2-STAND-PIPE (300m ID REINF. CONCRETE)
3-FROM TUBEWELL
4-TO DISTRIB. LOOP
5-FLOAT OPERATED SLEEVE VALVE
6-STEEL WEIR PLATE
7-OUTLET BOX
8-NORMAL LEVEL
9-MAX. LEVEL (WELL SHUTS OFF)
10-DRAIN VALVE
11-SPILL

Section A - A

Section B - B

1.5 m
FLOAT OPERATED SLEEVE VALVE
(For Regulation of Tubewell Delivery. Capacity 2 c. f. s.)

(1) 255 mm heavy duty PVC (11 mm wall).
(2) Cast iron, outer cylindrical surface and flange machined. Radial clearance between (1) and (2) should be between 1.3 mm and 1.5 mm.
(3) 10 mm guide slot machined in (2).
(4) Guide pin. See Plate 5.
(5) 10 mm steel flange (to be supplied with valve), field welded to pump delivery line.
(6) Eight bolts, 8 mm diameter.
(7) Solvent weld to (13) and (14).
(8) 315 mm diameter, 2.5 kg PVC.
(9) 4 mm to 5 mm PVC sheet.
(10) 32 mm PVC pipe.
(11) 10 mm diameter x 150 mm steel bolt.
(12) Nut with welded 22 mm washer.
(13) End cap screwed to (16) or alternatively push-fit with retaining set screw. (This cap prevents entry of air to under-side of float during valve opening. Without the cap the valve could close too rapidly.)
(14) Four 40 mm long sections cut from 200 mm diameter flanged PVC tail-piece and solvent welded to float.
(15) Sleeve begins to lift with water at this level.

(Roller guides not shown)

SEE PLATE 5 FOR ROLLER GUIDES
NYLON ROLLER
(Brass if nylon is not available)

TOP ROLLER
(Three required at 120°)

BOTTOM ROLLER
(Three required at 120°)

ROLLER BRACKET
( Brass casting or fabricated)

SLEEVE GUIDE
(One required)

5mm ID sleeve loose fit on 5 OD pin
(End of pin peened over to retain sleeve)

Radial adjustment of roller bracket to bring roller into light contact with valve sleeve is obtained by tightening the 6mm bolt against the rubber spacer.

NOTE: Rollers optional.
TYPICAL LAYOUT OF RIVER-LIFT SYSTEM (72 ha.)

- Pump station
- Distribution chamber
- Pump delivery line
- Outlet valve
- Boundary of service area
- Boundary of sub-command
- 140 mm PVC
- 200 mm PVC
- 12 Rein. concrete
- 1 ft Contour

0 50 100 150 200 m