Irrigation Engineering

BY

K. R. SHARMA

P S. E. Class I ; C. E. (Hons.) (Roorkee) ; M. I. E. (India)

P. W. D. Irrigation, Punjab.

VOLUME II

INDIA PRINTERS,

BOOKSELLERS, PUBLISHERS & PRINTERS,

Training College Road - JULLUNDUR CITY (India)

First Edition	1946
Second Edition	1948
Third Edition	1959

Printed & Published by D. V. Sharma
for M/s. Omex Trade Links, Jullundur City
at India Printers, Training College Road, Jullundur City.

Author's Other Technical Papers

1. Underpinning Satghara Rest House L. J. C. Paper No. 143 Punjab Engineering Congress, 1931.
2. Remodelling of Mithalak Distributary Paper No. 154 Punjab Engineering Congress. 1932.
3. Effect of outlets on the Regime of a channel "Indian Engineering Calcutta", October and November, 1932.
4. Silt conduction by Irrigation outlets, Paper No. 168 Punjab Engineering Congress, 1933.
5. Design of an A.P.M. Paper No. 176, Punjab Engineering Congress, 1934.
6. Design of irrigation channels Paper read in "Institution of Engineers, India" in 1935 and copies placed in the Punjab Irrigation Libraries received vide Superintending Engineer, U.J.C. No. 962 dated 28th July, 1937.
7. Silt selective Distributary Head regulators. Paper No. 189 Punjab Engineering Congress, 1936.
8. Standing wave or Hydraulic Jump. 'Indian Engineering Calcutta' October, 1936.
9. Design of R.C. Extensible bridges for Drains. August, 1936, printed copies placed in the Punjab Irrigation Libraries vide Chief Engineer's No. 386 dated 29th January, 1937.
10. Determination of water profiles over hydraulic works. Indian Engineering, Calcutta, 1936.
11. Experiments in seepage Losses from canals. Paper No. 209 Punjab Engineering Congress, 1938.
12. Automatic silt extractors without loss of water. Indian Engineering, Calcutta. February, 1938.
13. Cross Drainage Work (Auto-suction Weirs) Indian Engineering, Calcutta, February, 1938.
14. Improved adjustable proportional modules and open flume outlets. Punjab Engineering Congress, 1940.
15. Model Experiments to determine suitable length of skimming plateforms on models of the Head regulator at Rasul, Punjab Engineer's Journal, May, 1939, Lahore,
16. Theory and Physics of seepage flow from canals. Paper No. 231 Punjab Engineering Congress, 1940.
17. Use of Portable Flumes for watercourse discharge Measurements. February 1940 Punjab Engineer's Journal, Lahore.
18. Minimum Modular Head (M.M.H.) for an adjustable Proportional Module, January, 1940. Punjab Engineer's Journal, Lahore.
19. Experiments to determine True Basic sub-soil Pressure at R.D. 299,000 U.C.C. (W.I.R. Bib. 16).
20. Meter Experiments to determine losses in Mangtawala Feeder (W.I.R. Bib. 5).
21. Tank experiments to determine laws of seepage losses at R.D. 348,000 U.C.C. (W.I.R. Bib. 7).
22. Determination of seepage Losses in U.J.C. from Rashidpur to Shadiwala from meter discharge (W.I.R. Bib. 14).
23. Through experiments to determine Seepage Losses by Point Method in U.J.C. (W.I.R. Bib. 44).
24. Sub-soil conditions in adjoining fields of *Thur* and cultivation.

25. Determination of losses in Lower Chenab Canal by taking Seepage Discharge observations in a closure (W.I.R. Bib. 21).
26. Determination of losses in Upper Chenab Canal by taking Seepage Discharge observations in closure (W.I.R. Bib. 21).
27. B.S.P. Experiments at Khanki to determine B.S.P. cantours in a complicated case of sources and sinks and to determine True-Soil Pressures in River bed. (W.I.R. Bib. 17).
28. Report on Seepage losses of U.C.C. & L.C.C. (years calculated and actual compared by months). (W.I R. Bib. 3).
29. "Hump" Investigations under canal.
30. Point Method Apparatus and its use on L.C.C. Distys : to determine their losses.
31. Plotting and analysis of 39 Daily B.S.P. Stations. (W.I.R. Bib. 43).
32. Plotting and analysis of B.S.P. Pipes along canals calculations of losses by using observed S.I C. (W.I.R. Bib. 18).
33. Capillary fringe and soil evaporation studies with Hydrodynamic soil Pressure ovservations (W.I.R. Bib. 42).
    - (*a*) Containing foreward by Mr. Blench.
    - (*b*) Definitions.
    - (*c*) Note dated 2-6-39, 15-12-39 on Hydrodynamic soil pressure observations at Lahore.
    - (*d*) Note dated 8.6.39. on Experiments at R.D 180,000 L.C.C. to measure evaporation from the bed of a pit.
    - (*e*) Note dated 15 9.39. on Capillary fringe and Rise of water level in Bore Holes at R.D. 180,000 L.C.C.
    - (*f*) Note dated 20.9.39. on soil Pressure Observations to determine evaporation gradient by progressing lowering of bed of pit at R.D. 180,000 L.C.C.
    - (*g*) Note dated 16.11.39. on Hydrodynamic soil Pressure observations and Capillary Fringe studies at R.D. 150,000 L.C.C.
    - (*h*) Note dated 25.1.40 on Hydrodynamic soil pressure observations and capillary fringe studies opposite R.D. 46,000 Chichokimallian Distys.
    - (*i*) Note dated 1.2.40 on Technique of Hydrodynamic soil pressure observations below the effective saturation line.
    - (*j*) Note dated 25.3.40. on Capillary Fringe studies at Hudiara Nala.
    - (*k*) Note dated 26.3.40. on 'Hydrodynamic soil pressure observations at Seikhupura site'.
34. Division of the Punjab by an Engineer, June, 1947. C.M.G. Press, Lahore.

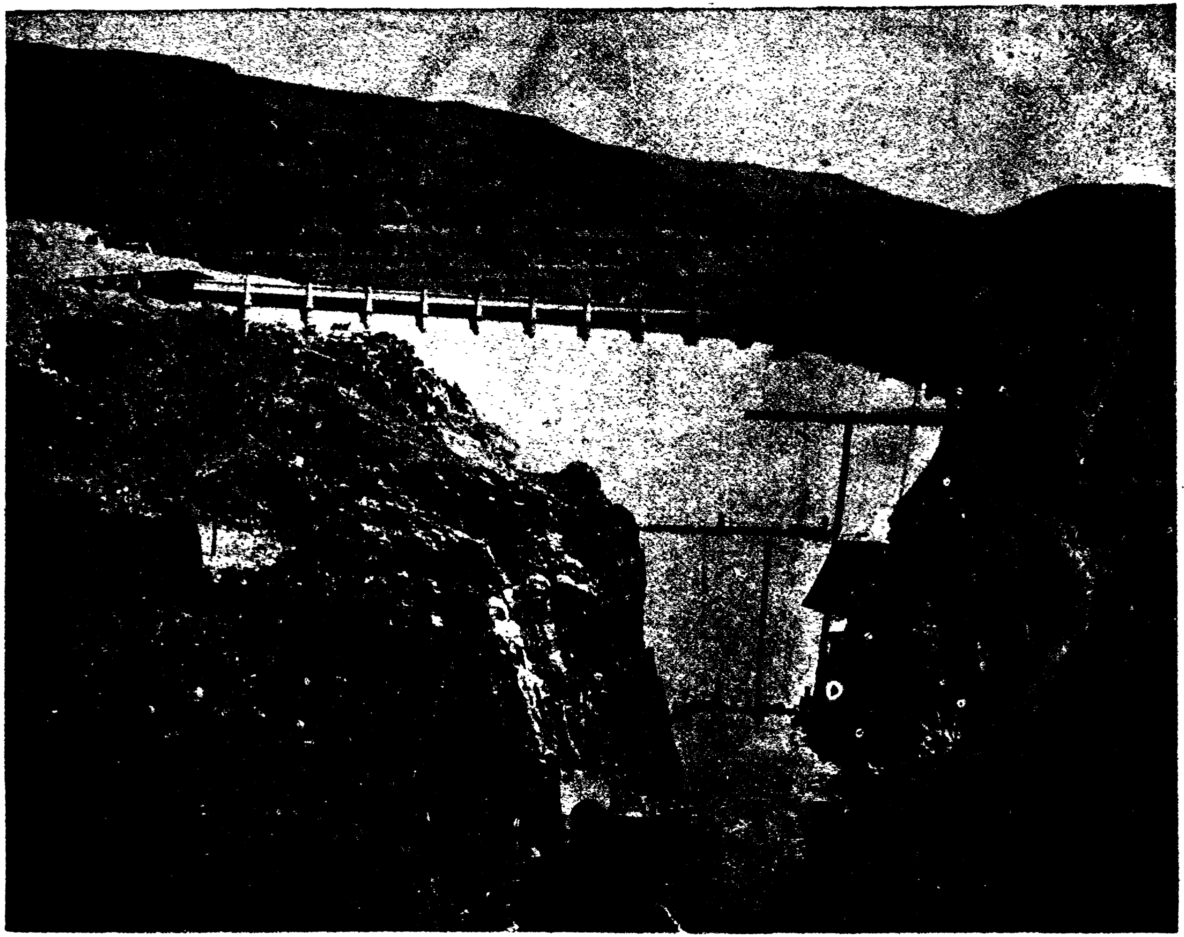
IRRIGATION ENGINEERING

Volume 1.

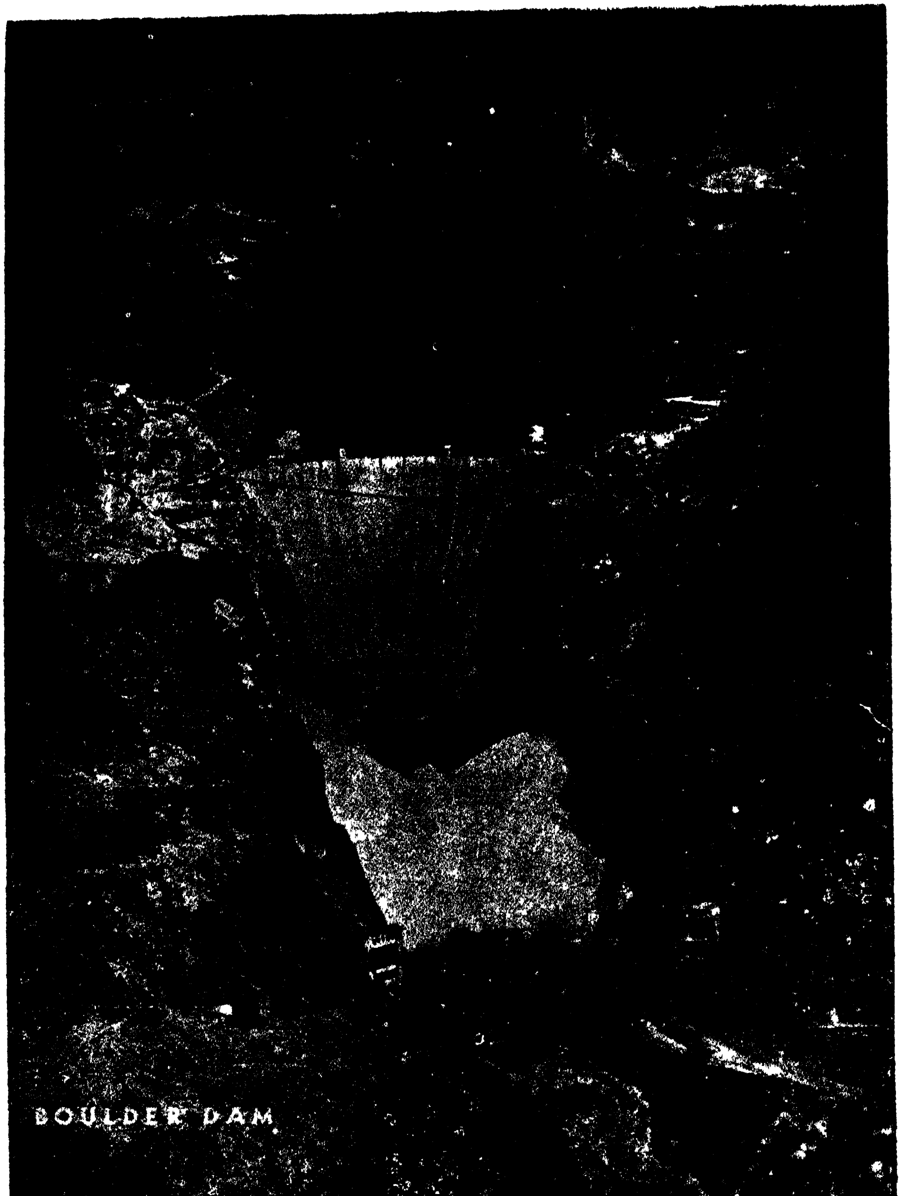
TABLE OF CONTENTS — Page

Volume II

PART III—TANK IRRIGATION (STORAGE & DAMS)



Owyhee Dam Completed in 1932. Concrete Arch-Gravity Section.



Boulder Dam 796′. Highest in the world. 1934
Concrete Arch-gravity type

PART III

TANK IRRIGATION

(Storages And Dams).

CHAPTER I

Storage Reservoirs or Tanks

1. Introduction.

The supplies of the large rivers of Northern India are very low in winter months. They have all been practically utilized in the Punjab for the purpose of irrigation by canals by constructing weirs across the rivers. An idea of the amount of supplies utilised in this way is given in chapter I of part II. All possible weirs (Diversion) have already been constructed but there are still large areas in the Bist Doab, Hissar, Gurgaon Districts and Bikaner State which need water for irrigation very badly. The supplies of rivers available in winter can be increased only by constructing storage tanks and releasing the water down when the normal low supplies in rivers fall below the requirements. The future Engineers shall have mostly to deal with new Projects of Dams and Storage reservoirs The subject matter of this part of the book is, therefore, extremely important for the coming generation of Engineers in India, especially in the Punjab.

Storage reservoirs are used to controls floods, to conserve water supplies for irrigation or domestic purposes, and to regulate stream flow. Hydroelectric power generation plants can be erected in all cases. Flood control storage serves to protect the downstream area against loss of life and property in high floods. Storage or ponding up of river supply is sometimes used to control the erosive tendency of a river in the sub-mountaneous areas where the river bed slope is very steep.

Storage may be single purpose or multipurpose. Usually the storage is a complex problem in a multipurpose scheme when it is designed for flood control, for hydro-purposes, electric power and for conserving monsoon period supplies for domestic use or for irrigation during dry periods of the year.

2. D fi iitions.

(*a*) **Conservation storage** is the term applied when the river water is impounded for subsequent release as required for some useful purposes such as municipal supply, power or Irrigati n.

(*b*) **Flood control storage** is classed as such when its object is solely to reduce downstreams flows in floods. Water stored during floods is later released as rapidly as the capacity of the downstream channel permits.

(*c*) **Valley storage** is the volume occupied in stream in floods after it had overflowed its banks. This is shown in fig 7.

(*d*) **Effective storage** is the storage available for the designed purposes. Storage below outlet levels is not effective in conservation schemes. In power reservoirs only storage above the minimum draw down level is effective. In a flood control reservoir, effective storage is the difference between actual capacity above outlet elevation and valley storage, or the storage that the flood water would have utlilised, had the reservoir not been constructed.

(*e*) **Su r-charge storage** is that which is available above an established reservoir level sometimes used to control floods.

(*f*) **Bank storage** is that which is developed in the voids in the soil cover in the reservoir area and is available as seepage water when the water level drops down in the reservoir.

(*g*) **Storage cycle** is the period at the end of which the reservoir content is the same as it was in the beginning. This peroid may be a few hours, a day, a year or a series of years. The outflow conditions from a reservoir may be vastly different from the conditions of inflow, but aggregate volume of inflow and outflow plus loses must be the same over a storage cycle. The outflow is artifically controlled but the inflow may be either entirely a natural affair or the consequence of artificial control from the reservoir upstream.

(*h*) **Pondage** is the term frequently used for the storage of only sufficient magnitude for daily regulation.• In Hydro-electric installations, the storage cycle is usually kept one day during which the inflow may be sensibly constant and the outflow varied in accordance with the daily load curve of the plant. Surplus waters are wasted in such cases.

(*i*) **Seasonal storage** is the term applied to reservoirs of sufficient volume to preserve water from the annual high water season to the period of low water. The storage cycle is usually one year but in some cases may be only a few months, if the water is wasted over the spill-way during a substantial portion of the time In such cases live storage capacity of reservoir with Hydro-electric Plant is only a percentage of the mean annual inflow.

(*j*) **Hold-over storage** is a term applied to those reservoirs with a sufficient capacity to supply a series of deficient years from storage held over from a preceding series of surplus years. Many seasonal reservoirs may also be in the hold-over class, provided the draw from them is not too great. The storage cycle may be any number of years in such cases.

3. Catchment Discharge.

(A) **Rain fall.**

The most important point to fix the size and height of the reservoir is the determination of the discharge from the catchment area of the river or streams feeding it above the site of the dam. The subject of maximum discharge from a catchment area is dealt with in Chapter I of Part IV.

(B) **Snow.**

In order to predict the spring runoff from melting snow, it is very essential to carry out the snow survey in the catchment areas feeding reservoir. A number of the snow courses, a half mile or so, are laid down in the representative and protected areas within the drainage basin. At interval of 50 to 100 ft. along, the snow is sampled and its water content determined usually in Fabruary, and two or three times thereafter, as the snow melting proceeds. A forecast is made following the February traverses and revised at each traverse that follows. In this manner high and low runoff volumes and to some extent the probable rate of runoff may be anticipated and the reservoirs gates manipulated accordingly. The practicability of this method has been abundantly demonstrated and such data has already become indispensable.

(C) **Flood flow from snow melting.**

(*a*) In many drainage basins the maximum runoff from snow-melting occurs during the spring or late winter storm season. In preparing maximum flood estimates for these basins, it is necessary to consider the possibility of snow-melt augmenting the runoff from heavy rainfall The rate and quantity of runoff from melting snow are determined primarily by the water equivalent of the snow cover, ground conditions affecting infiltration loses, and the rate of release by melting. The factors affecting runoff from a snow cover not only vary by geographical regions but also by seasons, and from period to period in the same area.

(*b*) **Ground conditions.** The infiltration capacity of the soil beneath a snow blanket has a very important influence on the quantity of runoff from snow-melt. If the ground is frozen, the infiltration loss may be very small, particularly if the soil is of reasonably fine texture and the field moisture content is high at the time of freezing. The infiltration capacity of a coarse-textured soil containing very little capillary water may not be appreciably reduced by freezing. As snow is a poor conductor of heat, very little additional freezing of moisture in the soil may be expected after a moderate depth of snow has been accumulated Therefore the optimum ground conditions for high volumes of runoff from snow-melt result when a severe freeze occurs prior to the occurrence of deep snow, and the capillary field moisture is high at the time of the freeze.

(c) **Water equivalent of snow cover.** The volume of runoff from a snow cover is limited to the amount of water contained in the form of snow and ice, plus the water stored in the snow structure. However, it should be observed that the water equivalent of a snow cover may substantially exceed the moisture equivalent of precipitation that occurs in the form of snow, by reason of the retention of rainfall in the voids of the snow structure, either by capillarity or by the formation of ice. Under average conditions, 10 inches of freshly fallen snow represent approximately 1 inch of water. However as the snow remains on the ground the density increases by reason of packing, by the possible addition of moisture from rainfall and condensation, and by an alternate thawing and freezing process. The density of the snow occasionally is as high as 50 to 60 percent, and it may be even higher in extreme cases. Densities of 20 to 30 percent are most common.

An appreciable amount of moisture may be retained in a snow cover in a "free" or unfrozen state. The amount of free water that can be retained in a snow mantle varies with the texture and general characteristics of the snow structure. A fairly comprehensive series of field studies have indicated that. under average conditions, a column of snow is capable of retaining free water equal to approximately 20 to 25 percent of the "dry" weight of the snow. A snow cover is said to "Ripen" as its wetness is increased by melting or by absorption of rainfall that is held in a liquid state in the snow structure. In order that the maximum probable rate of contribution of runoff from a snow cover may coincide with a major winter storm, a period of ripening must precede the heaviest rainfall. As major winter storms are frequently preceded by periods of light rainfall, it is usually reasonable to assume in estimating the maximum probable winter flood that the snow mentle contains on appreciable quantity of unforzen water which will be released as the snow melts, thus augmenting the runoff from snow actually melted by heat transfers during the rainfall period.

(d) **Rate of release by melting.** (1) **Heat transfers from air.** Studies made by the Hydrometeorological Section of the Weather Bureau in cooperation with the Corps of Engineers, U.S Army (Bib. 1) indicate that melting of snow during storm periods is primarily the result of the heat transferred from the air to the snow mantle in two ways (i) by direct heat exchange due to the temperature difference between the air and snow, and (ii) by the release of heat through condensation of moisture on the snow surface. This tranfer of heat results in conversion of the solid portion of the snow into liquid form at the rate of 1 gallon per 80 callories the heat of fusion of ice In as much as the heat of fusion of ice is only 80 callories, and the heat of vaporization of water is 600 callories per gallon, the moisture condensed on the snow surface melts 7·5 times its own weight in snow.

The thermodynamics involved in the melting of snow are discussed in detail by W.T. Wilson and Phillip Light. Wilson (Bib. 2) presents the following formulae to represent the depth of snow melted in a period of 6 hours

$$D_m = K_1 V(e - 6{\cdot}11) \qquad [1]$$

$$D_c = K_2 V(T - 32°) \qquad [2]$$

$$D_a = D_m + D_c$$
$$= K_1 V(e - 6{\cdot}11) + K_2 V(T - 32°) \qquad [3]$$

in which D_m is the water equivalent of snow melted in 6 hours by moisture condensation (plus the condensate) in inches depth ; D_c is the depth melted in 6 hours by direct heat exchange from the air by convection and conduction ; D_a is the total water equivalent of snow melted in 6 hours by the processes referred to above ; V is the wind velocity in miles per hour ; T is the dry-bulb temperature in F° ; e is the vapour-pressure in millibars ; K_1 is a coefficient involving the latent heat of ice, exposure of instruments, and conversion units, ; K_2 is a coefficient involving the latent heat of ice, exposure of instruments, conversion units air density, and certain considration involved in the theory of the turbulence. The coefficients K_1 and K_2 are related, K_1 being equal to approximately 3 to 4 times the value of K_2. For the most common arrangement of instrument installation, and stations less than a few thousand feet in elevation above sea level, Eq. 3 may be written in the following approximate from :—

$$D_a = V(0{\cdot}002T + 0\ 006e - 0{\cdot}100) \qquad [4]$$

There are a number of basin charactristics that determine the areal distribution and variations in depth of snow, the degree of ripening in various areas, and the rate of heat transfer during storm periods. The most important are range in elevation, topography, type of cover (forests etc.) and orientation of the basin with repect to the direction of movement of warm air masses. These factors can be evaluated approximately by rational analyses, but it is necessary to make certain arbitrary assumptions or utilize empirical factors. In order to obtain an estimate of the contribution of runoff from the snow cover in a drainage area of moderate size that takes into account the characteristics of the snow cover, meteorological influences in the region and conditions peculiar to the particular basin, Eq. 4 should be modified by a coefficient, K, to read as follows :—

$$D_a = KV(0{\cdot}002T + 0{\cdot}006e - 0{\cdot}100) \qquad [5]$$

The constant K is a basin characteristic that should be evaluated empirically. If records are avaliable for floods representing reasonably large quantities of runoff from snow melt in conjuction with major storms, the constant may be evaluted by sbstituting observed or deduced values of D_a, V, T, and e in Eq. 5 and solving for K. The empirical value of K would then be used in Eq. 5 to estimate snow melt quantities for other assumed conditions.

Estimates of runoff from snow melt during critical winter storm may be prepared adopting the following procedure :—

(*i*) For use in evaluating K in Eq. 5 select for analyses the most important winter floods of record that were characterized by large volumes of runoff from snow-melt in the drainge basin under study, or in similar basins in the region, and plot the hydrographs of observed runoff.

(*ii*) Compute rainfall quantities, mean temperatures, average wind velocities, and vapor pressures for each 6 hour period of the storms selected in step (*i*), and plot graphically, or tabulate the data above the observed runoff hydrograph in proper time relation.

(*iii*) Deduct the estimated base flow from the observed hydrographs to obtain hydrographs representing runoff from snow melt and rainfall combined.

(*iv*) Using unit hydrographs derived from the analysis of floods unaffected by snow melt, estimate by a trial-and-error procedure the hyetogroph of 6 hour values of rainfall-plus-snow melt required to reproduce the hydrographs computed in step (*iii*).

(*v*) Estimate infiltration losses for the flood periods being analysed, using as a basin information on infiltration capacities obtained by analyses of floods that were unaffected by snow-melt, and taking into considration all available data on the condition of the ground during the floods under study.

(*vi*) From the synthetic hyetograph computed in step (*iv*), subtract the observed rainfall values, determined in step (*ii*), and add the infiltration loss computed in step (*v*), to obtain estimates of water released from the snow cover during successive 6 hour periods of each storm.

(*vii*) Substitute computed or observed values of D_a, T, V, and e in Eq. 5 to determine values of K applicable to each 6 hour period of the record storms investigated, and average values for each storm period.

(*viii*) From available records of snow conditions in the region, estimate the critical quantity, distribution, and degree of ripeness of snow likely to exist immediately before the maximum probable storm of the snow season.

(*ix*) On the basis of data obtained by the study of major winter storms in the region, supplemented in so far as practicable by meteorological analysis, select values of T, V, and e to be used in estimating critical rates and volumes of snow-melt during the maximum probable winter storm, and compute theoretical values of snow-melt during the maximum probable winter storm, and then compute the critical values of snow-melt for successive 6 hour period by Eq. 4.

(*x*) Taking into considration the characteristics of snow mantles that prevailed during the record floods analysed in steps (*i*) to (*vii*), and conditions assumed to prevail during the maximum probable winter storm, select values of K to be used in estimating critical volumes of runoff from snow-melt during successive periods of the winter maximum flood.

(II) **Melting by Rainfall :**—Although the quantity of snow melted by rainfall is usually small in comparison with quantities melted by processess enumerated above, the amount may be significant in some basins during unusual storms. The depth may be computed by the following formula :

$$D_r = \frac{P(T-32)}{144} \qquad [6]$$

in which D_r is the depth of snow melt in inches, P is the precipitation in inches, and T is the wet-buld temperature of the air in F°.

(*e*) **Snow melt estimate by Degree-Day method** :—Several investigators have obtained approximate correlations between "degree-days" of temperature above approximately 32 F° and the runoff from snow-melt expressed in inches depth from the drainage area. In the majority of the studies, the maximum snow-melt runoff ranged from 0·04 inch to 0·09 inch per degree-day. However, in exceptional cases, the runoff from snow-melt has exceeded 0·22 inch per degree-day for at last 24 hours. Although the range in values given above is very large, it is not greater than might be expected in considration of the variables discussed. As the mean daily temperature of the air is only one of several factors that have major influences on the contribution of runoff from a snow cover, the degree day method is suitable only for approximate studies.

(*f*) **Snow-melt plus rainfall** :—The estimated snow-melt quantities corresponding to successive unit period of a design storm may be added directly to rainfall values for the respective periods to obtain the design-storm hyctograph Hypothetical hydrographs of runoff from the combined rainfull-snow-melt hyetograph may be computed in the same manner as followed in developing hydrographs of runoff from rainfall excess estimates alone.

4. Investigations and Surveys.

(A) **Preliminary Investigation** :—The preliminary investigation usually requires :—

(*a*) Approximate stadia site survey with the resulting topographic site map.

(*b*) Some investigation of the river bed deposite.

(*c*) A few borings, say from 6 to 50, according to the magnitude of the project and the character of the foundation.

(*d*) A preliminary geologic investigation and report.

(*e*) Investigation of available construction materials, such as earth and graval and concrete aggregates.

(*f*) The determination of public utilities which the dam might affect, such as roads, bridges, rail roads, telephone and telegraph lines, pipe lines, and power plants.

(*g*) For the proper layout allignment of the above facilities a fairly accurate topographic map of the basin is essential.

(*h*) Hydrologic studies.

(*i*) The checking of high water marks and their use in determining spillway capacity requirements.

The objective of the preliminary investigation is to obtain only sufficiently precise data to permit office studies and estimates of cost of sufficient accurancy to determine the most economical and suitable site among the several selected by the reconnaissance survey. A consideration which, for storage dams, affects the choice of general location is the quantity of silt carried by the stream. In some streams this is enormous and may in the course of a few years sufficiently fill the reservoir to destory its usefulness for storage.

(B) **Final Investigations** :—After the preliminary investigations at the several sites have been made and office studies and estimates for each of them completed, one of the several sites is selected for final, precise investigation. The site survey and the resulting topographic maps should be sufficiently accurate and precise to serve all the purposs of construction. All necessary borings, test pits, subsurface explorations, geologic studies, and tests on the materials in the foundation and in the proposed borrow pits will be made. As a result of the final investigation the engineer should have available all the relavent data to proceed with detail designs of the structures and the making of a control estimate of cost for construction. The line of demarcation between preliminary and final investigation of a site is not sharp. One often over laps the other. The point is that in the early stages of investigation where several sites are involved, the amount of investigation should be limited to that necessary to determine the relative merits of the sites ; thus avoiding the possibility of making a precise and costly investigation only to find that subsurface conditions are such that the site will have to be abandoned in favour of one of alternative sites The final investigations are usually supervised by the engineer who has conducted the preliminary examination or are at least conducted inaccordance with his recommendations. The principal items are :

(*a*) To determine the relative merits of two or more sites for the dam in question so that a final location can be adopted.

(*b*) To determine the type of dam to be used.

(*c*) To settle beyond doubt, by subsurface investigations, the nature of the foundation as affecting the safety and cost of the dam.

(*d*) To fix the limits of the lands to be controlled for flowage, for the sites of structures, and for other nacessary purposes.

(*e*) To determine the extent and character of the allignment of railroads and public highways necessary on account of raising the water surface.

(*f*) To ascertain the character of the Government regulations (Riparian laws) to be observed.

(*g*) To obtain sufficient information for an accurate estimate of cost.

(*h*) To determine the final location of the dam, construction equipment, camps, cofferdams, construction highways and railroads, as well as the probable source of materials of construction and all other information needful to the constructing engineer.

(*i*) To obtain all necessary information affecting the design of the dam.

(C) **Choice of site** :—The general location of the dams, is fixed by factors related to the purpose of the project. Bearing in mind the fact that the general location to be adopted is that which, at reasonable cost, will be best suited to the purposes for which the dam is intended. We have only to consider here those factors, which affect the cost and safety of the choice of its exact position.

The general location having been chosen, the exact position will be fixed after careful consideration of each of the following factors : –

(*a*) The character of the foundation

(*b*) The topography of the earth and rock surfaces at the side and its effect on the dimansions of the dam, quantity of material to be excavated, and other factors.

(*c*) Availability and character of materials for construction.

(*d*) The value of the necessary lands and water rights.

(*e*) Requirements as to cofferdams pumping, conduits, and other provisions necssary for unwatering the site.

(*f*) Transporation facilities and the accessibility of the site.

(*g*) Availability of suitable sites for construction equipment and camps.

(*h*) The safety of the structure.

The foundation is one of the most important factors in the final location. It should be suffictently impervious for the use intended or capable of being made so and have sufficient strength to properly sustain the weight of the dam and the pressure of the impounded water.

It is desirable that the valley should have a width which is adequate but not greater than that required for the dam, including its spillway, power house, navigation lock or other necessary structures. Except in an earth or a rock-fill dam, a part of the dam is usually designed to act as a spillway. However, for an earth or rock-fill dam, the spillway must be a separate structure and a site for it must be located.

5. Capacity of Reservoirs.

(A) **General Remarks.** (*i*) It is generally better to store the same amount of water in one than in several reservoirs, as this will usually reduce the cost of storage, the supervision of the works, and the loss by evaporation.

(*ii*) Reservoirs should, if possible, not be 'in series', that is, one below the other, as the failure of an upper one may lead to the destruction of the lower ones, one after an other.

(*iii*) Where reservoirs in series cannot be avoided, their size, should be regulated so that the lower reservoir is the greater in its capacity, in order that it may be able to absorb the flood below resulting from the failure of an upper one.

(*iv*) The Storage capacity of a reservoir should be proportional to and somewhat greater, say by 10 percent, than the average yield of its catchment; it will thus be full in all good years, will nearly fill in bad ones and will suffer proportionately less by its reduction by silting.

(*v*) The storage capacity should be sufficient for the irrigation of the irrigable area during the fair weather, but need not provide for the draw-off during the season of replenishment, if that consumption will be made good by subsequent in-flow.

(*vi*) The amount of storage required depends upon the duty, or the rate of consumption, on the extent of the irrigable area and on the loss by evaporation etc.

(*vii*) The culturable land in a reservoir basin should not exceed one-fifth of that which can be irrigated by the work, or the loss of revenue due to the submergence of the former will bear too high a ratio to that of the latter as incraesed by irrigation.

(B) **The computations of the storage capacity** are made as below :—

Let A_1 and A_2 be the contour area ; distance apart being H

$$\therefore V = \frac{H}{3}(A_1 + A_2 + \sqrt{A_1 A_2}) \qquad (1)$$

If there be 3 equidistant horizontal sections then

$$V = \frac{H}{3}(A_1 + 4A_2 + A_3) \qquad (2)$$

If there be any even number (n) equidistance horizontal sections A_1, A_2 etc , up to A_n at a common distance H

$$\therefore V_n = H(\tfrac{1}{2}A_1 + A_2 + A_3 + \ldots + A_{n-1} + A_n) \quad (3)$$

The volume of water which can be stored in a reservoir can thus be calculated. This will give the gross capacity of the reservoir (Bib. 3).

The gross available capacity of the reservoir is the volume of water which can be stored above the outlet level.

The net available capacity is the last one minus the absorption in the tank, percolation below the dam and the evaporation

The reservoir can usually be filled more than once The actual available capacity may be taken $1\frac{3}{4}$ times the net available. (Madras Practice, Bib. 4)

(C) **Graphical Method.** The simplest approximate method of calculations given above has now been replaced in the modern practice by the graphical method described below :—

(*i*) **Area Volume Curves** : The reservoir storage is sometimes measured in gallons, or in cubic feet The commonest unit is the acre foot, which is an acre's one foot deep or 43,560 cubic feet. Some time cusec day is used as a convenient unit ; one cusec day equals 86,560 cubic feet or almost 2 acre feet. Runoff is usually specified in inches of depth over the drainage area. One inch runoff from one square mile equals 53·33 acre ft., or 26·7 cusec days. 6·25 English gallons approximately equal one cubic ft. In an area volume curve, the total volume in acre ft., and total area in acres are plotted against reservoir elevation as shown is fig. 1. The topographic survey showing contours forms the basis of such curves.

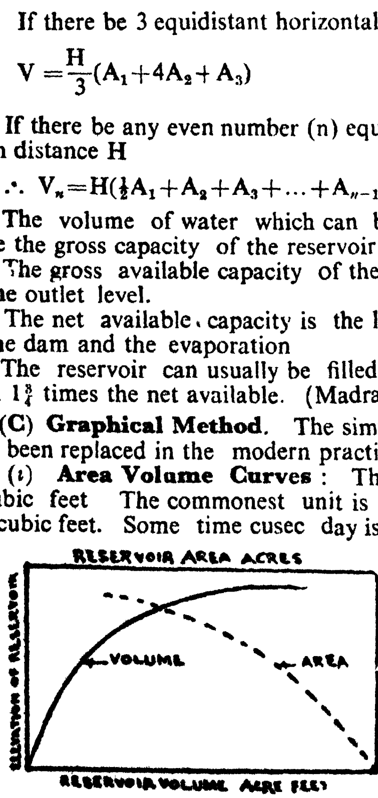


Fig 1 Typical reservoir area and volume curves

(*ii*) **Hydrographs** :—Hydrographs are plotted for the storage reservoirs by plotting inflow against time. In designing a reservoir conservation purposes, the critical period will be the minimum flow period during the driest period of year or the driest year of the record. In applying hydrographs, it is desirable to super-impose on the hydrograph of inflow the hydrograph of demand or use. Fig. 2 is inflow hydrograph with demand hydrograph super-imposed.

The flow deficiency during a period of full year must be made up by storage allowing for the driest years on record.

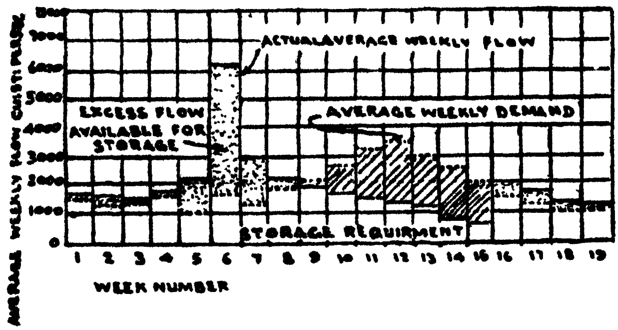


Fig 2—Flow and demand hydrograph

(*iii*) **Duration curves** :—Duration curves used to represent the relation between inflow and time. Fig. 3 curve 1, shows for a given period the percentage of the time that each flow is maintained. Where a duration curve represents a long period of time it is useful for estimating the amount of power that can be produced under average conditions. The effect of reservoir control is represented by a modified flow duration curve as curve 2. Fig. 3. Duration curves are useful in studying power, water supply, irrigation and navigation problems of reservoirs.

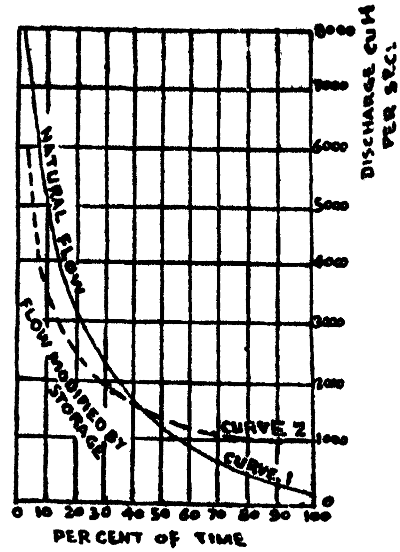


Fig 3—Flow duration curve.

(*iv*) **Mass Curves** :—Mass curves show cumulative flow and cumulative storage into the reservoirs. Stream inflow is corrected for evaporation and other losses are deducted. Similarly cumulative demand is the sum total of demands from the reservoir for irrigation, hydropower and slucing purposes. Fig. 4 shows a typical mass curve diagram OA is the inflow and OB is the demand. If a line parallel to OB is drawn tangential to point 'a', the begining of the longest dry period on record, the ordinate dc will represent the volume of storage required to maintain a rate of inflow not less than that represented by the slope of the line OB.

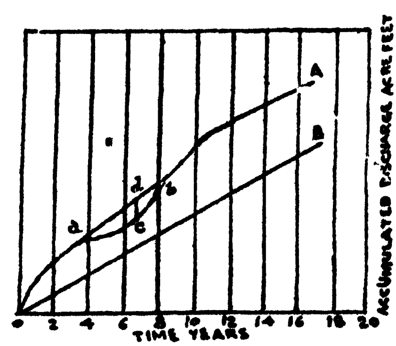


Fig 4—Typical mass curve

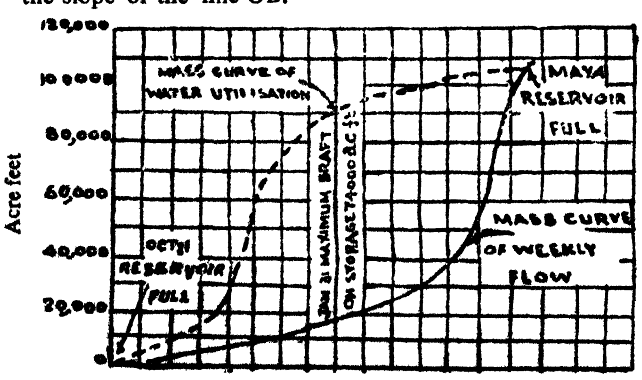


Fig. 5—Mass Curves of water utilisation and weekly flow

The mass curve of water utilization will not generally be as straight line. Fig. 5 show a curve of irregular demand for a typical dry period. The maximum vertical distance is the storage required to meet the needs of the project.

A modified form of mass curve is shown in Fig. 6 which is now widely used. The curve in a mass diagram on a horizontal basin about a Zero line representing the average inflow for the period of the record. This S curve is used in the same manner as ordinary mass diagram, Fig. 6, to determine storage requirements or rates of draw from the reservoir with the correction that to the slope of rate lines must be added the average flow to determine the true rate of use.

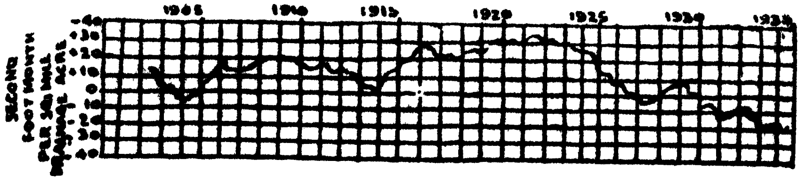


Fig. 6. Mass diagram of runoff

Effect of valley storage. The assumption that the hydrograph of inflow is the same as the hydrograph at the dam site is generally valid for normal flows and reservoirs of large capacity. In dealing with food-control reservoirs and in routing floods for spillway design, the problem may be complicated further by the effect of valley storage. Any stream in flowing through its normal or flood channel occupies a certain volume. To illustrate, if the average cross section of a stream is 1000 sq. ft. for 1 mile, the volume of water is 5,280,000 cu. ft. or about 120 acre-ft. Now the same stream in flood may use 2,000 acre ft. per mile in valley storage. With a reservoir 10 miles long, this might be 20,000 acre-ft, when the maximum discharge is at the dam site. Now in using the hydrograph at the dam site one would be making the automatic assumption that this 20,000 acre-ft. had not flowed into the reservoir, whereas an examination of Fig. 7 show that this water is already in storage in the reservoir, up to the reservoir water surface elevation.

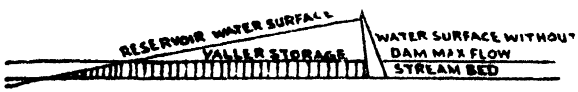


Fig 7. Valley storage diagram

It follows that at this particular time the mass curve of inflow is 20,000 acre ft. greater than the mass curve of flow at the dam site. It follows further that the hydrograph of inflow will have an earlier and greater maximum discharge.

(D) **Use of table.** When the demand is variable by months, the mass curve method is not so convenient as the tabular method. For either method, the runoff by days, months, or years is required according to the purposes required. For similar periods, the demand must be known, and it may become somewhat complex, being made up of such items as power, irrigation, releases for earlier rights, evaporation from the reservoir itself, all of which vary with the periods chosen. In some instances, a 10-day period is preferable. The total volume of supply are set up for the periods 1-10, 11-20, and 21-30 (—28 or —31 as the case may be) of each month. The demand for the corresponding periods are set up parallel. The difference give the defficiency (—) or surplus(+). The algebraic sum of these differences represents the the content of the reservoir. If its capacity is limited, the surplus must be spilled ; if the demand is too great, the reservoir may become empty and a lesser demand must be tried until a demand is found that is just equal to the supply, less spillages, over a representative storage cycle.

6. Irrigation Capacity of Tanks or Reservoirs.

(A) The irrigation capacity of tanks and the duty of water impounded varies naturally with the amount of absorption and evaporation. The following figures are supposed to give the usual Madras Practice as given by Bligh in his book "Design of Irrigation Works". The table below gives the quantity of water required in the tank for irrigation to bring the crops to maturity.

	cft.	acre ft.
1. Five months monsoon rice crops	2,16,000	5
2. Cold weather rice crop	1,75,000	4
3. No. 1 if stored for next year	5,40,000	12·4

This is rather excessive but includes all losses from the reservoirs to the fields. For other crops such as wheat, fodder crops, the storage capacity may be taken to be one half of item (2), that is, about 2 acre feet or 85,000 cft. nearly.

(B) In the modern practice supply available from the reservoir is expressed in cusec days available. The usual methods of canal irrigation as described in part II are followed to work out the areas which could be irrigated from the tank irrigation systems. Water allowance of 3·0 to 3·5 cs. per thousand acres area is allowed. Intensity of irrigation is from 75 to 80°/₀ with this water allowance in areas of the Punjab when full supply discharge is available in *Kharif* from 1st April to 30th September (snow melting and monsoon season) and about half the discharge in *Rabi* from 31st October to 31st March next year (dry season). The channels in *Rabi* could be run about 15 days in a month by rotation. Actual working of the canals under these conditions has shown that *Kharif Rabi* ratio of irrigation attained is 1 : 1. In actual practice the period of full supply is shifted by 15 days from 15/4 to 15/10 as there is no demand in the first fortnight of April.

7. Capacity of Tanks For Water Supply To Towns.

(*a*) **Binney's rules.** The maximum and minimum rainfall in the catchment area in any year must be known before fixing the capacity of a reservoir. The usual method applied is the one given by Alexander Binney sometime Chief Engineer, in the last century in Nagpur, Central Provinces, India.

Binney made a study of a long-time records of rainfall at 42 places all over the world and he deduced certain rules.

1. Observations of daily rainfall extending over 35 to 40 years will give a fairly accurate idea of rainfall likely to occur in future.

2. The maximum annual rainfall of any place is 1·51 times the average rainfall of a year, and the minimum rainfall of a year is 0·6 times the average rainfall of an average year, such a year is called the driest year.

(Normal or average rainfall is the mean of 35 years)

3. For two consecutive years, the maximum rainfall is $2\times1{\cdot}35\times$ average, and for a consecutive dry year the minimum rainfall is $2\times0{\cdot}69\times$ average.

4. For 3 consecutive years.

Maximum rainfall $=3\times1{\cdot}27\times$ average.

Minimum rainfall $=3\times0{\cdot}75\times$ average.

(*b*) The capacity of tanks for water supply for domestic purposes is calculated as illustrated in the example below :—

Mean annual rainfall in catchment area of 2000 acres is 27 inches. Find the day's capacity of the reservoir, making usual allowances for percolation and absorption in a severe tropical climate ?

Assuming daily requirement per head as 15 gallons, find the population it can serve ?

Capacity $=2000\times\frac{27}{12}\times\frac{1}{3}=1500$ acre ft. $=750$ cusec days.

Average for 3 consecutive dry years $=27\times0{\cdot}8=22''$, if one third lost, 14″ avaiable.

Capacity $=\frac{750\times12\times1}{14\times2}=321$ days in North India.

Gallons per day (available) $=\frac{2000\times4840\times9\times14\times6{\cdot}25}{12}$

Population served $=\frac{1000\times1210\times365\times14\times25}{12\times4}\times\frac{1\times1}{365\times15}=\frac{7000\times1210}{73}$

$=\frac{70\times121{,}000}{73}=116{,}000.$

8. Absorption And Evaporation Losses From Tanks.

The site for the reservoir should be such that the bed is not permeable. Absolute impervious sites are impossible because water can pass even through the pores of rocks and fissures between the lyers. It is usual to keep the outlet level higher than bed of the stream or the basin by about 10 feet so that in course of time silt deposits and staunches the bed of the reservoir. The losses by absorption and percolation can be considerably reduced in this way as compared with those in beginning.

There are three types of losses from tanks, (*a*) Evaporation (*b*) Absorption through the bed of the reservoir and (*c*) Percolation below the dam.

The evaporation in India will be maximum in hot months with strong winds in summer and minimum in winter with light winds. It is usually about 0·4 inch in summer months and 0·2 inch winter months. In Egypt it has been 0·38″ maximum.

In reservoirs, it is not, however, possible to evaluate the losses separately. Losses are supposed to exclude all the above-mentioned three factors which contribute towards them. In Bombay, the yearly average loss is taken to be the surface area multiplied by 4·0 ft. In a tropical climate like that of India, it will be best to make the estimates of water available allowing losses of 6 ft. of depth in a year which will be equal to a monthly loss of 3 inches in cold weather, 9 inches in hot weather and 6 inches during the monsoons, each season being assumed to last for four months.

9. Silting of Reservoirs.

(A) Reservoirs are liable to have their storage capacity reduced by the deposit in them of silt derived from their catchment in the worst cases the reduction is very considerable. The amount of silt depends upon the extent of the catchment area, the nature of surface soils climatic conditions and the slope of the country. Soils which disintegrate under the action of the weather or are soluble, are the most silt-producing. The climatic conditions leading to the production of silt are frost, intense heat, dryness, and violent downpour of rain scouring the ground. Every drop of water brought down by the floods contains some silt. To reduce the rate of silting up, it is, therefore, necessary that catchment area should not be excessive, *i. e.*, should be sufficient only to fill the reservoir to a slight excess in an average year, or conversely, that the storage capacity should be nearly equal to the average annual yield Also, the nature of the surface catchment should be such as will not produce silt. More silt is produced by steep slopes, bare or tilled surfaces and friable or soluble soils and less silt by gentle slopes, surfaces covered by vegetation, especially grass and trees, and hard or insoluble soils. In the tropics most of the silt is washed down by the first heavy storms which carry off materials loosened by a long period of draught with a high temperature.

(B) **Silt disposal devices.**

(*a*) **Scouring gallery.** Silt deposits can readily be removed in certain cases by means of a scouring gallery such as is known in Spain as a "*Desarenador*". Fig. 8 below shows a typical plan and section of a scouring gallery as used in the case of Alicante and Elche reservoirs. Every fourth year, about 30 feet of silt deposited is scoured out through the galleries. The deposits are, no doubt, compacted but once they are disturbed by poles, they are easily scoured out. These galleries are applicable only when the reservior can be completely emptied at more-or-less frequent intervals and that the bed of the reservoir must be steeper than would be deemed advantageous, if storage capacity alone were considered. Thus, rasérvoirs which can be cleared of silt by a scouring gallery are by no means satisfactory from other points of veiw, since the cost of the dam per cubic yard of water stored will obviously be comparatively high.

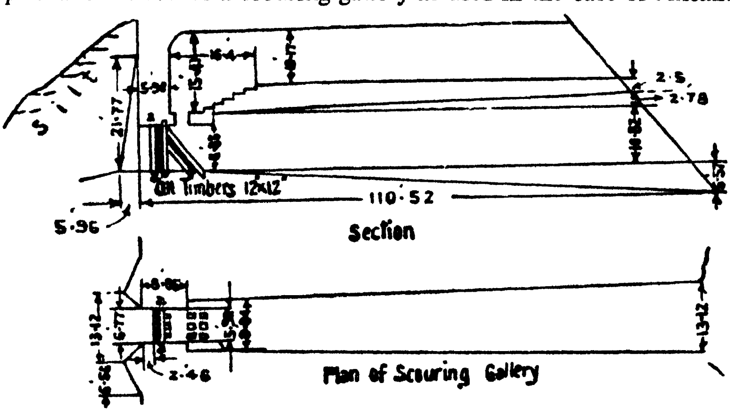


Fig 8.

(*b*) **Scouring sluices.** The Nile reservoir at Asswan is worked on the principal of rejecting the silted water of the rising flood, and retaining only clearer after-waters. The circumstances, however, are far more favourable than are usual in India. The Nile floods are so regular, and the system up-river gauge reports so excellent, that it is very rarely necessary for any silted water to be even temporarily retained in the reservoir. While in Indian reserviors owing to the uncertainty as to the future supply of water, it is frequently necessary to store heavily-silted water. Also the Asswan under-sluices can pass off the flood discharge of the river at a mean velocity of about 20 feet per second, so that hardly any ponding up of silted water (and consequent deposition of silt) takes place.

It would, therefore, appear that designs on similar lines will permit reservoirs to be kept clear of silt, but the paracitical difficulties of sacrificing all the high flood water in reservoirs, the capacity af which is a large fraction of the mean yield of the catchment area, are obvious.

It must also be noted that the Asswan dam is founded on hard granite. Even under these extremely favourable circumstances, extensive and costly repairs have been found necessary below the under-sluices and it is doubtful whether they will not have to be repeated at frequent intervals. See Fig 9.

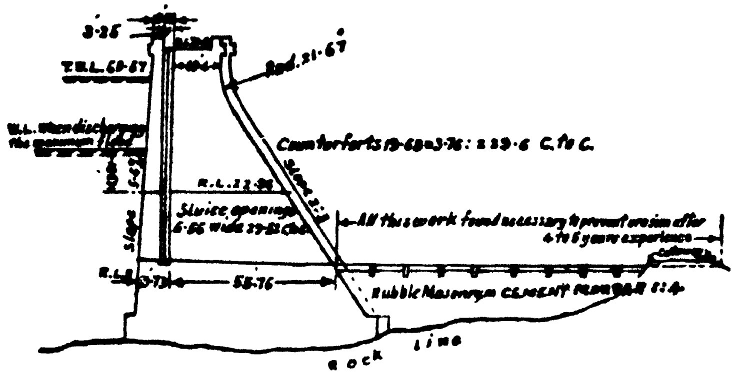


Fig. 9

(*c*) **Mechanical dredging.** The silt deposits in tanks can also be cleared by the mechanical removal by dredging, the requisite power being obtained from turbines worked by the stored up water and transmitted electrically. In case where this electrical dredging was proposed, the local conditions where somewhat peculiar. As a general rule, where such an installation proves necessary, a more economical from of power whould undoubtedly be steam or oil engines carried on the dredger itself.

(C) **Conclusions.**

When the reservoir volume is small in comparison with the mean aunual flow (or rather, with the yearly volume of silt carried by the river), the dam must be of the Asswan type, provided with powerful undersluices. These must be systematically employed during the high water season to pass off heavily silted water and to scour out deposite. Later on, the clearer water can be retained. In fact, we should endeavour to score what is mostly ground water flow, coming down after the flood season is finished.

The process is not difficult where the volume of the reservoir is 1/40th or 1/30th of the mean annual runoff of a fairly permanent river, which occurs in tolerably damp climates. Difficulties begin when the volume is one-tenth of the mean annual flow of a variable river. Even in a fairly moist continental climate, the minimum annual flow may be only about two fifth of the mean. In such a year, any error in judgment might entail the possibility of starting the dry season with the reservoir only partially filled ; and in a dry continental climate matters are even more difficult.

In all cases where the flow of the first driest year can be relied upon to fill the reservoir, silt deposits can be materially diminished, or can possibly be entirely prevented if the undersluices are sufficiently powerful. At present there is no experience to permit any definite rule being given. However, this is not so material, as a preliminary staunching of the reservoir bed by silt deposits is actually advantageous, since it will minimise leakage.

10. Auxiliary Reservoirs and Supply Channels.

(*a*) Occasionally the natural catchment is not sufficient to fill a reservoir, and has to be increased artificially by running supply or feed channels to lead part of the yield of the neighbouring drainage areas to the storage. In the simplest form these channels merely intercept part of the runoff of the rainfall from the ground upstream of them. The rate of this runoff is extermely variable being small during light and medium showers, which may occur frequently, and great in heavy downpours, which, although happening seldom, produce the bulk of the total annual yeild. If the cross section of the channel is designed to carry only the discharge of the former, it will be insufficient to convey much of that of the latter ; if, however, it is increased to pass down a large fraction of the latter, it will have to be considerable in size and expensive, and there will be much loss in transit during small flows. A compromise has, therefore, to be made and a section adopted between the two extrems. This might be made sufficient to deal with, say, one-eighth of an ordinary heavy runoff. It will thus intercept most of the lighter discharges, and might on the whole be estimated to pass one-quarter of the total yield into the reservoir, so that the "equivalent catchment" which it adds to the natural one of that storage might be taken as one quarter of its own drainage area. Such simple channels are liable to breach during heavy storms, and thus to lose much supply, and to silt extensively, and thus cost much to maintain.

(*b*) A supperior form consists in making auxiliary reservoirs where the line of the supply channel crosses water-courses as in Fig. 10. These will impound the bulk of the runoff of even heavy rainfall, and this can be passed to the main reservoir at an average rate in a channel of small size subject to less silting. Moreover, should the channel breach, the reservoir can be shut off until repairs are effected, and loss of yeild will thus be avoided. These advantages will usually have to be paid for by the increased cost of the system.

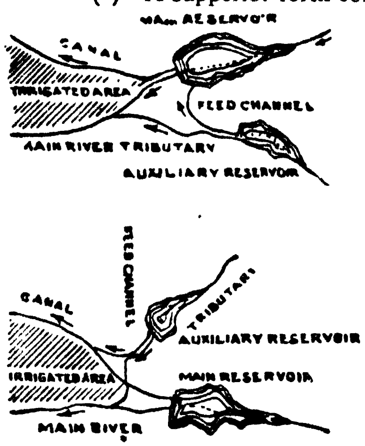


Fig. 10.

11. Flood Adsorptive Capacity of Tanks.

(A) The reservoirs are generally constructed to contain a small percentage of maximum annual runoff from the catchment. It is usually about 10 percent of the total runoff. The floods may be disposed of in the following manner :-

(*a*) Escapage through the undersluices.

(*b*) They may be escaped through waste weirs or spilway siphons when the reservoir is full.

(*c*) They may be absorbed to fill the reservoir.

It is, therefore, necessary to predict the rate of rise of water when the sluices or waste weirs are working or when it is being stored in the reservoir. A correct estimate of the flood absorptive capacity of the reservoir is very essential to predict the time when the scouring sluices should be closed so that the tank may be full in the remaining period of the monsoon season. In India, very little replenishment can be expected in the dry winter months.

(B) Flood absorptive capacity with sluices working.

Let Q be the discharge enterning the reservoir having an average cross sectional area A square feet between any two heights H_1 and H_2 and let q be the discharge of the under-sluices with area 'a' square ft.

Consider the reservoir at the instant when the water surface is at a height h above the sluices and let this height decrease or increase by dh in a short interval dt.

Amount of inflow $= Q.dt$;

Amount of out flow $= q.dt = Cd.a\sqrt{2gh}.\ dt = kh^{1/2}dt$, when $k = Cd\ a\sqrt{2g}$. The increase of water in the reservoir $= A\ dh$

$\therefore A.dh = Q.dt - q.dt = (Q - kh^{1/2})\ dt$, when $k = Cd.a$

$$\therefore dt = \frac{A}{Q - kh^{1/2}}\ dh \qquad (1)$$

Then intergrating this equation for variation H_1 to H_2 will give the time required for this change of water levels in the reservoir.

$$T = \frac{2A}{k^2} Q \log_e \frac{(Q - k\sqrt{H_2})}{(Q - k\sqrt{H_1})} + k(\sqrt{H_2} - \sqrt{H_1}) \qquad (2)$$

The value of co-efficient Cd may be taken as 0·8 for larger opening.

(C) Reservoirs with waste Weirs.

Let Q be the discharge entering the reservoir having an average cross sectional area A sq. ft. and let q be discharge of the waste weir B feet wide.

Consider the reservoir at the instant when the water surface is at height 'h' increased or decreased by dh in a short interval dt.

Amount of inflow $= Q.dt$
Amount of outflow $= q.dt$, where $q = C.Bh^{3/2}$
The value of C being 3·09 in long crest weirs
Amount of outflow $= kh^{3/2}.\ dt$, where $k = C.B$.
The increase of water in the reservoir $= A.dh$.
$A.dh = Qdt - qdt = Qdt - kh^{3/2}.dt = (Q - kh^{3/2})dt;$

$$\therefore dt = \frac{A}{Q - kh^{3/2}} dh \qquad (3)$$

Integrating this equation for variation from h to H, will give the time required for this hange of water levels in the reservoir.

$$T = \frac{2A}{3k^{2/3}Q^{1/3}} \left[\log_e \frac{\sqrt{1 + \sqrt{r} + r}}{1 - \sqrt{r}} - \sqrt{3}\left(\tan^{-1} \frac{2}{\sqrt{3}} (\tfrac{1}{2} + \sqrt{r}) - \frac{\pi}{2}\right)\right]$$

$$r = \frac{h}{H}$$

The limit when there will be no further rise is given when $Q = CBH^{3/2}$

$$\text{or } H = \left(\frac{Q}{CB}\right)^{2/3}$$

(D) **When water stored.** When the water stored in the reservoir for any change of level h with A as the average sectional area of the reservoir, the time required will be simply $\frac{AH}{Q}$ in seconds.

12. Reservoir Flood Routing.

Flood routing is a process of determining the outflow pattern from reservoirs from any given pattern of inflow and storage from river system. A general solution is given herein which permits a rapid adjustment for variations in discharge and correction for valley storage.

Inflow = storage + outflow

Inflow is obtained from the reservoir inflow hydrograph, if available. The storage measured in acre-feet is plotted against elevation as in curve 'a' Fig. 11.

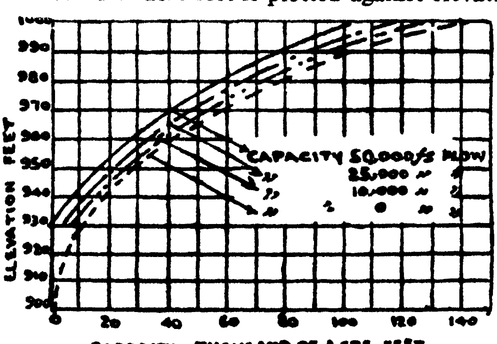


Fig. 11—Reservoir Capacity curves corrected for valley storage

Where the hydrograph is the one at the dam site, capacity curves corrected for valley storage should be used in which case there will be a family of, storage curves as in fig. 11. Outflow may be through the outlets or over the spillway, or both. For the outlets, the momentry discharge in cubic feet per second may be plotted against elevation as in curve 'b' (Fig. 12). Likewise, spillway discharge may be plotted against elevation. In Fig. 12, outlet plus spillway discharge is plotted in curve 'c'.

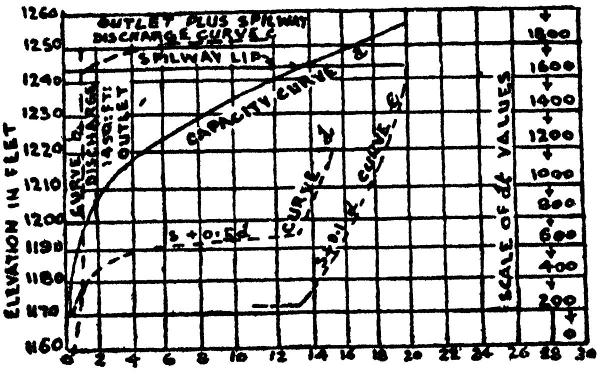
With reference to the basic relation given above and by using the following nomenclature,

I—inflow, acre-feet during any time internal T.
T—interval of time, days.
d_1—momentry discharge in cusecs at the beginning of the time interval
d_2—same at the end of time interval T.
S_1—storge, acre-feet in the reservoir at the beginning of the time interval T.
S_2—same at the end of time interval T. The following equations are derived :—

$$I = (d_1 + d_2)T + S_2 - S_1 \qquad (1)$$

$$I + S_1 - d_1T = S_2 + d_2T \qquad (2)$$

Note :—The average rate $(d_1 + d_2)/2$ for time interval T is not used because $(d_1 + d_2)$ T is in acre-feet and 1 ds. ft. = 2 acre-ft.



Capacity in thousands of acre feet, discharge in thousand of casecs, S + dT in thousand

Fig. 12. Typical reservoir flood-routing curves

In Eq. (2) the value of S_1 is obtained from curve 'a' Fig. 12, the reservoir water surface being known; d_1 is assumed in the case of an empty reservoir or is determined from curve 'b'. The algebraic sign of $(1 + S_1 - d_1T)$ may then be determined, which equals $(S_2 + d_2T)$. Since the value of S_2 and d_2T for one time interval equal S_1 and D_1T for the next time interval, it is necessary to determine S_2 and d_2T. Curves 'd' and 'e' give values of dT in terms of S + dT, and S is obtained by subtracting dT from S + dT.

Curves 'd' and 'e' are plotted for deginite values of T, which are taken in this case as 0·5 or one-half day for curve 'd' and 0·1 or one tenth day for curve 'e'. These two values are selected to permit a rapid computation by using curve 'd' in the reservoir filling stage and greater accuracy by using curve 'e' in the critical range above spillway crest. Of course any suitable values of T may be untilized.

To illustrate the construction of these curves, (Bib. 9), consider some reduced level say, R. L. 1,245. The storage in the reservoir at this elevation read from curve 'a' is 14,4000 acre-ft. The combined outlet and spillway discharge at the same elevation is 2,054 cusecs. Then for curve 'd' with T = 0·5 day, the ordinate against which the curve is plotted is dT, or 0 5 × 2,054 = 1,027. The abscissa is S + dT, or 14,400 + 1·027 = 15,427. Likewise for curve 'e' with T = 0·1, dT = 0·1 × 2,054 = 205·4 and S + dT = 14,605·4.

Table 1 :—*Typ'cal Flood-routing computation using Fig. 12.*

(1) Day	(2) Inflow acre-ft. I	(3) Storage acre-ft. S	(4) Discharge dT	(5) $1 + S_1 - d_1T$ $= S_2 + d_2T$	(6) Elevation of water in reservoir	(7) Surcharge over spillway ft.
0·0	....	0	50	250		
	300					
0·5	....	30	220	2,390		
	2,580					
1·0	....	2,005	385	11,480		
	9,860					
1·6	....	10,990	490	14,552		
	3,660		981			
1·6	....	14,352	200	17,312		
	3,160					
1·7	....	16,632	630	18,492	1,249 5	
	2,540					
1·8	....	17,552	940	18,812	1,251	
	2,220	....	......	....	(1,251 5)	
1·9	....	17,512	1,300	18,152	1,251	
Total	24,300	17,512	4,363			

Table 1 shows the routing computations for the reservoir of Fig. 12 with inflow as given in column 2 of the table. Column 1 is the time in days corresponding to the spilway design flood hydrograph. Column 2 is the inflow in acre-feet for each time interval as determined from the hydrograph. Column 3, 4, and 5 are determined as the routing proceeds. On the assumption of a base discharge of 50 cusecs, it is placed in column 4 opposit 0·0 for the initial value of dT Column 5 is the algebraic sum of columns 2, 3, and 4 *i.e.*, $(I+S_1-D_1T)$. Thus $300+0-50=250$, which is entered in coiumn 5. Since this is also S_2+D_2T, entering, curve 'd' the value of d_2T is found to be 220, which is entered in column 4. This is result being substracted from column 5, the value of storage S is found to be 30, which is entered in column 3. as the storage in acre-feet at end of the first half day. Likewise for the next time interval 2580 is the inflow, 30 is the value of S, and 220 is d_1T, so $I+S_1-d_1T=2{,}390$ which in turn is the new S_2+d_2T and is entered in column 5 The steps are continued until the storage column (3) begins to decrease. Care must be exercised at the time the water surface approaches the spilway crest, where a change in time interval is necessary. At this point of change, the dT for the longer interval is divided by the ratio of longer interval to the shorter interval to obtain the dT for the shorter interval. Storage column 3 may be converted into water-surface elevation Column (6 by the use of the curve 'a'.

Where the hydrograph is for the dam site, additional capacity curve or a curve should be drawn corresponding to net storage, as previously described. For each

Change of time interval from $T=0{\cdot}5$ to $T=0{\cdot}1$; $490\div\frac{0{\cdot}5}{0{\cdot}1}=98$

Check of computations

$(4{\cdot}363\times2)-(50+490+98+1300)=6788$ acre-ft. discharge, 17,512 acre-ft. storage
24,300 acre-ft. inflow.

additional 'a'curve, new 'd' and 'e' curves will be required. These having been established, the set of curves most nearly corresponding to the inflow should be used in each step.

13. Flood Control Reservoirs.

(a). **Retrding Reservoirs.** The simplest type of flood control reservoir is the retording basin with uncontrolled outlets. The outlets capacity is such that the unregulated discharge with the reservoirs full does not exceede permissible carrying capacity of the channel through the proctected areas Retarding basins have one major advantage in that the human element is eliminated from reservoir operatration, for no gates are provided. Another advange is that the expensive gate and gate-control installation cost is avoided. The further apparent anvantage of smaller conduit capacity is usually offset by the necessity for greater capacity during the construction period

A serious disadvantage in connection with retarding basins is that automatic regulation may cause the coincidence of flood crests further downstream The travel time of the tributary and main river flood waves should be given consideration in planning reservoirs of this type.

An ideal installation for a retarding basin is the case of a single reservoir located immediately above the city for which the protection is designed. A similar case would be that of reservoirs located above the confluence of two or more streams where the protected area is immediately below their confluence.

(b) **Detention basins.** Flood control reservoir provided with outlet control is termed as detention basin. The purpose gate cantrol is to achieve greater usefulness from the system than can be obtained with the retarding type. Generally, when the area under control increases in size and the protected area is more widespread, the advantages of this type outweigh its disadvantages. The disadvantages are the possibility of human error in operation and the cost of gate-contral installation.

The advantages of this type are several and must be weighed in each case to determine whether gate installation is advisable. With gate control, outlet capacity may be increased and a more uniform discharge being permitted throughout the flood period by gate manipulation.

Usually, the rainfall and runoff in a flood-producing storm vary in intensity over the watershed. With gate control, the outlet discharge in areas of low rainfall can be entirely eliminated, and where runoff is heavy, the gates may be opened to discharge a large volume than would flow from a retarding basin system. This is made possible since the closed reservoirs are contributing no water to take up part of the safe channel capacity. A third advantage on the tributaries of a major river is that the gates may be closed to retain large volumes of runoff until the flood situation on the main stream has cleared up. The advantages of the detention basin may be summed up as flexibility and greater use of the available storage, proper operation being assumed.

(*c*) **Retarding basins with auxiliary gate control.** This type has a system of uncontrolled outlets permitting normal operation as a retarding basin. Gated outlets are provided also to pirmit greater discharge in the early stages where such is disirable and more repid emptying of the reservoir when channel conditions permit. This type is particularly applicable where the storage capacity is large as compared with drainage area, with a correspondingly small conduit capacity.

(*d*) **Relation between reservoir and outlet capacity.** In designing a retarding basin, the permissible outlet size is assumed and the flood routed through the reservoir to determine the required capacity to spillway elevation. With detention basins, the outlet discharge hydrograph must be assumed and the same process carried out.

Where reservoir capacity is limited by some other circumstances, then the outlets for retarding must be designed by trial until the proper size is obtained. In designing outlets for detention reservoirs, the range of operation desired must be considered. Usually the minimum capacity will be that required to discharge the maximum safe capacity of the channel below with the reservoir storage capacity about 25 percent developed. In gate design, it is advisable to have sufficient gate area to develop full outlet capacity with one gate closed.

Capacities of flood-control reservoirs in the United States usually vary between 4 in. and 9 in. of runoff from the drainage area. Generally 6 to 7 in., is a desirable capacity for reservoir in the eastern states. Outlet capacities vary with reservoir capacity and discharge but usually fall between the limits of 20 and 40 cusec-month for detention reservoirs at maximum water surface.

14. Reservoir Operations.

(A) **Conservation reservoirs.** A reservoir operation plan is devised to achieve the greatest value or benefit from the storage capacity. The plan must be based (*i*) on a knowledge of the flow characteristics of the stream, *i. e.*, a history of its past performance. (*ii*) The purpose or purposes of the reservoir must be analyzed to determine how the hydrograph of flow shall be altered to produce the greatast benefits.

If the reservoir furnishes water to a city or regulates how water flows for navigation, it would be imperative to maintain a predetermined minimum flow, the aim being to utilize the available storage to increase the dependable flow. This might be termed the insurance method of operation. In carrying out the method, studies would be made to determine what flow could be maintained by the reservoir in question during the driest year of record only enough. water would be released from the reservoir during the dry period to maintain the predetermined minimum flow at the point of regulation; all other conservation storage would be held in reserve for the next dry period that might be expceted. From the water power point of view, there are several objections to this method. In all years except the driest, only partial benefits would be obtained from the reservoir, and in same year little or no water would be released from storage.

In contrast to the insurance method of operation, the utilization method aims to use practically all the available storage each year without special regard for maintaining a minimum regulated flow. This method provides for the release of water from the reservoir just as soon as the natural flow of the stream falls below the amount necessary to operate the plants at full capacities. The principal objection to this method is that the reservoir might be empted in extremely dry years before the end of the drought.

Modern practice in hydroelectric planning combines the best features of the insurance and utilization methods to provide a dependable minimum flow and at the same time use to the the best advantage the total volume of stored water each year. In carrying out this method, rule curves of operation are devised which indicate how the best results could have been obtained from the reservoir on the basis of past experience. These rule curves are then applied to future operations with the knowledge that the most efficient use will be made of the stored water in all except the extremely wet years, which are of infrequent occurrence.

If the stored water is to benefit a number of power plants located downstream. it is important to so operate the reservoir that the maximum benefits to the power developments in the stream as a whole will result. It is obvious that equal regulation cannot be obtained at all downstream plants; consequently, regulation must be planned for that point on the stream which will yield the maximum benefit to the whole system. Under ordinary conditions, the greatest benefit will result from locating the point of regulation as near as possible to the centre of gravity of power for the system. The centre of gravity of power may be based on either wheel capacities or developed head and drainage areas

(B) **Flood-control reservoirs.** Where no gate control is provided for the outlets, as in retarding basins, oparation is automatic. Sometimes, gated outlets are provided in addition to the automatic sluices. These are installed where the capacity is large, 7 inch of runoff in this case, and the uncontrolled outlets, therefore, are so small that the empting period is unduly long. The operation of these gates is quite simple; they are regulated to discharge as much water as can be carried without damage in the channels below the reservoir.

For the detention type of flood-control reservoirs, *i.e.*, reservoirs with controlled outlets, operation is primarily governed by the requirement that no water shall be released that will cause material flood demage below. It is easy enough to work out a plan for a uniform flood or a historic flood, but where the area is large, no future flood can surely be expected to have the characteristics of any past occurrence. The solution to the problem lies in analysis, information, and communications.

The operating personnel must study many possible flood combinations to determine an effective plan for each. An adequate system of rainfall and runoff measurement must be installed together with a system of communications that will get the information to the central office repidly. Likewise there must be a good system of communication with the operating personnel. In addition, a basic plan of operation must be installed at each reservoir. If the reservoir is near the flood damage centers, it can be operated so as to start storing at such a time as to be effective when damaging stages become imminent. Ordinarily, this will mean that the gates are open in the early part of the flood. In the case of remote reservoirs, all early runoff must be stored, and no releases are made until falling stages are imminent, or the reservoir capacity is exhausted or approaching exhaustion.

(G). **Multipurpose flood control reservoir.** Most storage reservoirs are constructed with the idea of impounding flood waters for the purpose of utitlzing them during periods of low flow In such reservoirs, there is no guarantee that will have sufficient available capacity at the incidence of a flood to provide effective flood control. Flood-control reservoirs are operated on the principal that they are never to be used except for the temporary storage of flood waters, which are subsequently released as rapidly as channel conditions will permit without demage. Sometimes these two types are combined in reserve storage reservoirs, where the lower levels are used for conservation storage and the higher levels reserved for flood-control purposes.

To be effective for combined purposes, a reservoir must be of large capacity. For any degree of positive operation, it must be tested against a long period of record. From this analysis, a rule curve of operation is developed. Similar curves are used in the operation of irrigation and power reservoirs without flood-control features.

15. Spillway Dischage Capacity of Reservoirs.

In most cases, it is necessary to provide sufficient spillway capacity to discharge an inflow as large as experience has shown can be expeced over to occur in that lacality. Where there is only a small storage capacity in the resevoir between the water surface level which will exist when the flood begins and the maximum level to which the reservoir may be allowed to fill, the spillway must have a discharge capacity practically equal to the peak rate of the flood flow. Where the storage capacity is relatively large, however, a considerable part of the flood may be temporarily stored in the reservoir and the spillway capacity reduced accordingly. A large number of methods have been devised to investigate the effect of this srorage on the capacity of spillway required, but no single one is best for all conditions. For approximate estimates, suitable for preliminary computations, probably the Woodward method is most efficient. For final computations, a more exect method, is desirable. The method most easily understood is a cut-and-try method, and where only a few computations are to be made it is perhaps the best to adopt. Where more then one or two computations are to be made, the

Posey slide rule method is very advantageous. After experience, one can work even a single computation more rapidly with it.

The Woodward method. The Woodward method (Bib. 5) is based on a mathematical solution involving four assumption :- (1) that the net inflow rate is constant, (2) that the reservoir area in which the storage takes place is constant, (3) that the spillway discharge varies as the three halves power of the head and (4) that the reservoir is filled to the spillway crest when the flood begins. The solution of this problem is presented in Fig. 13 which gives the ratios of the total net maximum spillway discharge to the net inflow volume the storage volume above the spillway for various ratios of inflow rate. The trem net inflow volume issued to indicate that the volume of flow which passes the dam through any other route than over the spillway (as, for example, through the power house) must be subtracted from the total inflow volume before using the diagram. A similar correction must be made to obtain the net inflow rate. To illustrate the use of this diagram, suppose a certain reservoir has a capacity between the spillway crest level and the maximum safe level to which it can be permitted to fill of 100,000 acre-ft. and that the reservoir is filled to the spillway level at the beginning of a flood of 200,000 acre-ft., which enters with an average flow rate of 50,000 cusecs. Suppose also that no discharge takes place except over the spillway. The ratio of the total net inflow volume to the storage volume above the spillway is therefore $\frac{200{,}000 \text{ acre-ft.}}{100{,}000 \text{ acre-ft.}} = 200$. Fig. 13 shows that for this condition the ratio of the maximum spillway discharge rate to the net inflow rate is 0·865. Since the net inflow rate is 50,000 cusecs, the maximum spillway discharge rate must be $50{,}000 \times 0{\cdot}865 = 43{,}250$ cusecs. Of course, the condition of an actual case never exactly fit the assumptions on which the solution is based; the inflow will probably not be constant, the area of storage in unlikely to be uniform throughout the height and the spillway discharge rarely varies exactly as the three halves power of the head. However by using the total volume of storage above the spillway and the average rate of inflow surprisingly close agreement is usually obtained. In computing the dimensions of the spillway required for a given discharge, a weighted average value of the spillway discharge coefficient should be used. Since most of the outflow takes place near the maximum stage, the coefficient should be less than but close to that which would exist at maximum discharge. Since the relation on Fig. 13 are nondimensional, any system of units may be used if care is taken that the two values used in obtaining any ratio are in the same units.

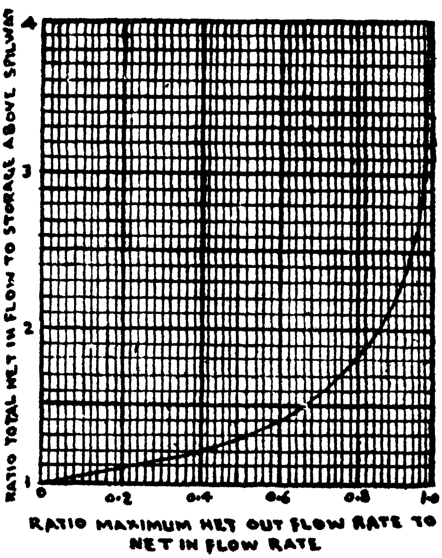


Fig. 13—Curve for estimating Spillway capacity required by Woodward method

The cut-and-try method. Final designs should always be checked by a more exact method. The easiest of these methods to understand is the cut-and-try or trial-and-error method. From the design of the spillway and the data on the reservoir and the flood considered three curves as follows should be prepared :- (1) a rating curve showing the total discharge of spillway and outlets plotted against reservoir elevation ; (2) a reservoir volume curve showing the volume available for the reduction of flood crests plotted against reservoir elevation and (3) an inflow hydrograph during the flood period. The spillway discharge throughout the flood can then be determined by the following procedure : the time period is first divided up into short increments, and the total inflow volume during each period is determind from curve 3. Begin with the first period, and make a trial estimate of the level in the reservoir at the end of the period. The discharge rate at the end of the period corresponding to this elevation is then determind from curve 1. The average discharge rate for the period is then computed by averaging the rates at the two ends of the period, and this multiplied by the duration of the period gives the total outflow volume. This volume subtracted from the inflow volume during the period should give the storage volume during the period. If the elevation at the end of the period is correctly assumed, this storage volume should equal that between the surface level at the beginning of the step and the assumed level at the end of the step, as indicated by curve (2). If it does not, the assumed elevation at the end of the step should be revised upward or downward, depending on whether the computed storage at the end of the step was greater or less than the storage indicated by curve (2), until agreement is secured. The process is then repeated by using the next step until the entire flood is routed through the reservoir.

Several variations of the cut-and-try process are possible, for several quantities can be assumed at the end of the step (for example, the outflow rate) and computations made to see if the value assumed fulfills the required conditions, but extensive experience has shown that the labour involved in all these methods is practically the same ; hence. only one is given.

The Posey slide-rule method. (Bib. 6) This method follows the procedure of the cut-and-try method in that short steps are used and that the spillway discharge throughout the flood is determined. The trial-and-error computations, however, are replaced by reading from single settings of a slide rule representing the particular reservoir and spillway. The slide rule may be quickly made with pencil graduations upon two pieces of wood about 2 ft. long. Operation of the slide rule in solving a step is shown in Fig. 14 in which a total inflow of 7,000 cusecs day* during a step changes the reservoir stage from 77·8 to 78·3. The slide rule solves the equation

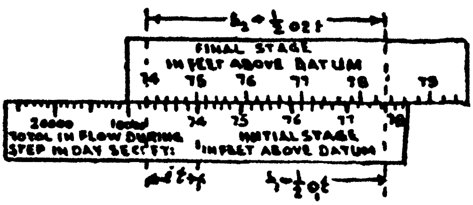


Fig. 14.

$$i.t+\left(S_1-\frac{1}{2O_1t}\right)=\left(S_2+\frac{1}{2O_2t}\right);$$

which is derived from the equation

$$S_1+i.t=S_2+\frac{O_1+O_2}{2}\times t$$

where S = Storage in the reservoir ; i = Inflow rate ;
O = Outflow rate ; t = Length of step.

Subscript 1 and 2 values at the beginning and end of step, respectively.

Distances on the left-hand lower scale represent values of the total inflow during a step of a uniform scale. Distances on the right-hand lower scale represent values of $S_1-\frac{1}{2O_1t}$ in the same units and plotted to the same scale as for 'i' and 't' corresponding to the various initial stages. Similarly, distances from the left end of the upper scale to the various final stages are corresponding values of $S_2+\frac{1}{2O_2t}$

Preliminary computations to be made before the slide rule can be graduated are the determination of values of $S_1-\frac{1}{2O_2t}$ and $S_2+\frac{1}{2O_1t}$ corresponding to various stages in the reservoir, the chosen length of step, and outlet capacity provided. The vertical interval between stages for which these values are computed will depend upon the accuracy desired. The contour interval of the reservoir survey will ordinarily be satisfactory, except immediately above the spillway crest where a smaller interval may be desirable. The graduations on the slide rule intermediate between the computed values may be equidistant, with only a very slight loss of accuracy in the results. The initial stage and final stage scales may be graduated in terms of intial outflow and final outflow, respectively. This graduation requires a little more work in the preliminary computions but saves time in the routing computations if successive values of outflow are required. The 'i' and 't' scale may be graduated in terms of average inflow if desired.

With this slide rule, it is not possible to compute the conditions for any time except multiples of the time interval chosen. This interval, howevere, can be chosen of a size such that intermediate values can be interpolated to any required degree of accuracy. A new rule

* Cusecs day The volume of (cusec flowing for) day=36,400 cu ft. the use of this unit of volume, or similar ones, such as hour-sec-ft =3,600 cu ft. will be found to be more convenient in routing computations than such units as acre-feet and cubic feet.

must be made for each different spillway, but with a little experience they can be made very rapidly.

16. Types of Spillways For Reservoirs.

There are usually six spillway types, (*i*) overfall spillway (waste weirs), (*ii*) trough or chute, (*iii*) side channel, (*iv*) shaft or glory hole, (*v*) syphon and (*vi*) gate type. The overfall type is by far the most common and is adapted to masonry dams that have sufficient crest length to provide the desired capacity and where the foundation will withstand or can be protected sufficiently to withstand the scour of the overfalling water. Trough or chute spillway are commonly used for earth dams. Side channel and shaft-spillway types are most frequently found in narrow canyons. The syphon spillway is usually used to provide automatically a nearly constant head water level under varying flow, and sometimes where the crest length of the dam is restricted. The gate type of spillway is used where it is desirable to remove the effects of the dam during high water to prevent excessive flooding. The trough or chute type is often combined with one of the other types, the trough sometimes taking the form of a tunnel through the abutment of the dam. Overfall and syphon spillways are usually located in the main dam and the trough, side-channel and shaft types near or in the abutments.

Fig. 15:—Spillways with special provision for erosion control.

(*i*) **Overfall spillway.** The usual form of overfall spillway has a rounded crest with an ogee face as shown in Fig. 15. The crest is formed to fit the shape that the over flowing water would take under conditions of maximum discharge, if the dam was replaced by a weir, with a sharp crest at the junction of upstream face of the dam with the rounded top surface. For example, the curve downstream would be shaped to fit the underside of the nappe of a weir with an upstream face of the same shape as that of the dam and the sharp crest at the top, when the weir is discharging the same quatity as the spillway under the maximum head that is expected to flow over it. If the upstream face of the dam is vertical, the crest shape will be that for a vertical-face weir, and if the upstream face of the dam is sloped, the crest will follow the shape of the nappe of a weir having a face of similar slope. The shape of the nappe from a sharp crested weir was first determined by Bazin. (Bib. 7)

If the form of nappe for a vertical-face dam is applied to the ordinary non-overfall cross section of a dam, it will be found that it is necessary to thicken the dam somewhat on downstream slope. (Bib. 11.)

Some dam designers have considered it desirable, in order to avoid the possible formation of vacuum beneath the nappe, to shape the spillway somewhat larger than would be indicated by the weir nappe shape, but experience by the U.S. Bureau of Reclamation indicate that this is unnecessary. Although small vacuum may form near the crest owing to the difficulty of accurately fitting the nappe shape, the area covered is small and does not extend much beyond the crest. Even for heads somewhat exceeding the design value. the vacuum is not severe over a large area.

Piers are frequently used on the crest of spillway dams to support the gates. To cause as little obstruction as possible, their upstream ends should be rounded or pointed like the bow of a ship.

The discharse coefficient for an overfall spillway of nappe shape varies with the head starting at about 3·09 and reaching approximately 3·98 at the design head. The coefficients are functions of the relation of the head to the design head.

(*ii*) **Trough or chuts spillways.** A trough or chute spillway consists of an open conduit conducting the water from the reservoir to the waterway downstream from the dam. Frequently there are gates at the upstream end to control the flow. Ogee or side channel spillways are often built with a chute to carry away the water. Tunnels through the abutment rock form a special class of chute spillways. Except where tunnels are used, the chute spillway are practically always straight, because of the difficulty of changing the direction of water owing at supercitical volocities.

Where chute spillways are formed on earth great care should be exercised to provide adequate subdrainage in order to prevent heaving due to frost or displacement due to water pressure.

The simplest form of trough spillway is of uniform width throughout. Changes of width or curved alignment lead to complications which can be solved only by means of laboratory experiments. In contracting or curved sections, stationary waves are apt to be set up which may strike and overtop the side walls. A contraction can sometimes be worked out in a spillway with gates at the upstream end by constructing a stilling pool just downstream from the gates with a narrower chutes leading away from it. If expanding sections are used the expansion should not be too abrupt, or the water will not follow the side walls.

(*ii*) **Side-channel spillways.** The side-channel spillway is commonly used in sites where the sides are steep and rise to a considerable height above the dam. In this form the water falls over the spillway crest, into a channel, in which the flow is parallel to the crest, which leads eventually to the stream bellow the dam. Space does not permit a discussion of the hydraulics of the flow in this channel. In the spillway of Boulder Dam, a cross weir was constructed at the downstream end of the overpour section to provide a considerable depth of water in the channel at all flows, in which the energy of the overflowing water could be dissipated, without causing excessive turbulence in the tunnel that carried the water back to the river. In this case, it was very desirable to avoid turbulence in order that the water might flow through the long tunnels without undesirable effects due to entrained air. The increase in the size of channel required with this weir was negligible.

(*iv*) **Glory-hole or shaft spillways** :--In Glory-hole, (Bib. 12) or shaft spillways Fig. 16, the water flows over the lip of a funnel shaped spillway and discharges down a shaft or a tunnel There are two general types, the first having a standard rounded crest and the other a flat crest. The usual details of both types are shown in Fig. 16. This from of spillway is adapted to narrow canyons where room for a spillway is restricted. A disadvantage of this type is that the discharge beyond a certain point increases only slightly with increased depth of overflow & therefore does not give so great a factor of safety against underestimation of flood discharge as do in most other forms.

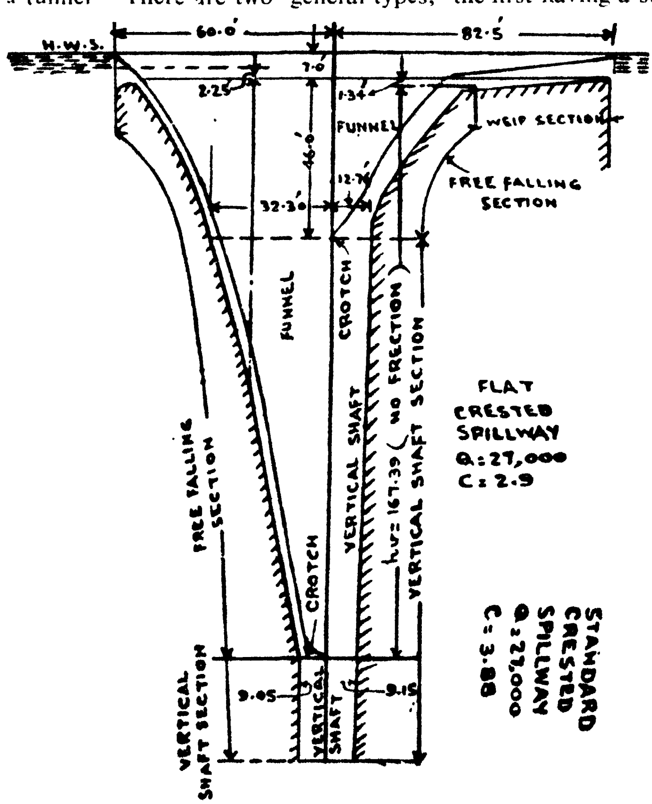


Fig. 16. Comparison of Standard with flat crested Shaft Spillway.

The glory-hole type has been tested extensively in models, but so far as known, it has not been subjected to high discharges on the actual structures. Because in ordinary model tests the air-entraining effects cannot be reproduced to scale, for the surrounding air pressure is not reduced, to a magnitude corresponding to the modle size, the degree of agreement of modle tests with the prototype action is uncertain.

Considerable turbulence occurs at the bend at the bottom of the vertical shaft from this form of spillway. For comparatively low heads this is probably not serious, but the action under high heads is uncertain. For high dams, it whould seem to be advantageous to being to incline the tunnel as short a distance as possible below the intak and provide ample access of air to the inclind section.

The from of the spillway is largely controlled by the discharge to be accomodated and the depth of overflow permitted, for the lenght of crest must be sufficient to provide for the required discharge at the maximum head permitted. Thus large discharges and small depths of overflow give rise to large diameters of the intake section. The size of the outlet tunnel is determined by the discharge and fall and is commonly constructed so that the tunnel will flow full throughout its length but not cause a backwater action on the spillway crest under conditions of maximum discharge.

(*v*) **Syphon Spillways.** Their working and design is descrided it detail in Part II Chapter XIII Syphon spillways and Hydratomats.

(*vi*) **Gate-or barriage-type spillway.** Gate-type spillway consist of a series of gates separated by masonry piers, with floors between the piers to prevent scour of the river bed, and cutoff walls extending into the river bottom to prevent undermining. The principal problems of such spillways are the design of the gates which is given in Part II chapter IV and the river bed protection which is described in Part II Chapter X.

17. Spill-Way Tunnels.

(*a*) Tunnels are frequently used as the outlet channels for various types of spillways. They are lined with concrete, and the cross sections are usually circular but not necessarily so. The ability of the concrete to withstand high velocities of flow seems to be amply demonstrated where the water is clear and flows parallel to smooth walls. If the water contains fine silt in suspension, it will probably not cause excessive scour unless the surfaces are rough or there are projections that cause eddies or whirls. which tend to project the abraiding silt particles against the concrete. Coarser silt will probobly cause greater cutting action than fine silt. When reservoirs behind dam become filled with material so that the bed material, sometime of heavy gravel or even boulders, passes over the spillway, severe abraiding of the concrete is apt to occur. If there are projections or sharp corners in the tunnels, vacuums tend to from behind them. and if the velocities are high enough, severe cavitation sometimes takes place.

The cross section of the tunnel generally should be designed to have ample air space above the water surface at all flows. The swift flowing water tends to drag the air above it along and if ample space is not present throughout the tunnal length, the air may collect at certain points and brust out at intervals. Under other conditions vacuum may form causing undesirable hammering. Models cannot be relied on to duplicate the prototype action in this respect.

Spillway tunnels are usually straight but may be curved if the radius of the bend is not too small. With high velocities and small radii, the water flowing swiftly along the bottom of a straight circular tunnel will rise on the outside of a curve and if the curve is not too abrupt return to the bottom again after passing it, with perhaps a tendency to swing from one side to the other beyond the bend. If the bend is abrupt, the water may rise on the side so rapidly that it is carried completely around the tunnel in a spiral movement. Such action would probably produce undesirable air conditions in the tunnel

(*b*) The tunnel end in the river bed downstream of the dam. Water spilled through usually spreads in the wide river bed. The required depth to from a proper hydraulic jump is not generally available according to the formula given below, where D_1 and V_1 are the depth and the velocity upstream of the jump and D_2 is the depth required downstream.

$$D_2 = -\frac{D_1}{2} \pm \sqrt{\frac{2V_1^2 D_1}{g} + \frac{D_1^2}{4}} \qquad \text{(Part VI, Chapter IV)}$$

A weir or Baffle wall is constructed as shown in Fig 15. to ensure depth of D_2 upstream of this weir so that a proper hydraulic jump is formed. Pick up weir is usually followed by cistern, apron and riprap, the dimensions of which are determined as discribed in part II, Chapter X.

18. Emergency Spillways.

Emergency spillway is that which will be called upon to operate infrequently that it is not considered necessary to protect the spillway control, the structure, its foundation, or its discharge channel from serious damage when it goes into action. However, further definitions of this type of spillway vary according to two points of view.

From the first point of view, the emergency spillway is an auxiliary spillway which would be called into action should a flood greater than the spillway design flood, occure. From that view point it is simply an added factor of safety.

From the second point of view, the emergency spillway is an auxiliary spillway which would be called into action should a flood occure whose magnitude was 60 to 80 percent of that of the spillway design flood.

These two points of view would coincide if it could be granted that from the first point of view the estimate of the spillway desing flood is inadequate, while from the second point of view it is completely adequate. Then, in both cases. the permanent spillway would be capable of taking only a percentage of the proper flood to be controlled and the emergency spillway the balance. That is, a permanent spillway would be built to accommodate 60 to 80 percent of the proper flood and an emergency spillway would be provided to take the balance with considerable damage to it but with safety to the dam.

The damage caused by the operation of an emergency spillway may vary from the loss, of the control apparatus to the complete washing out of the spillway structure and its foundation even to the extent of emptying the reservoir, but at such a low rate of discharge as to cause a downstream flood which would be insingificant as compared with that resulting from the loss of the dam.

The emergency spillway has its parallel in the emergency channels of the Mississippi River, where a section of levee is purposely left low or is ruptured in places where severe but relatively small damage would occur in order to prevent otherwise certian rupture of a levee system protecting a large city.

An emergency spillway is most easily obtained at a low divide in the reservoir rim. If the elevation of the divide is so low that a dike is required, the dike is left at such an elevation that it will be overtopped before the main dam is overtopped.

19. Biblography.

(1) A report on maximum possible precepitation over Ohio River Tributary Basins above Pittsburg, Pa., published report prepared by the Hydrometeorological section of the U.S. Weather Bureau in cooperation with U.S. Army Engineer Department.

(2) "An outline of the Thermodynamics of Snow-Melt" by W. T. Wilson in Trans. Am. Geog. phys. Union, 1941.

(3) Dams and Weirs by W.G Bligh, American Technical Society, Chacago, 1918.

(4) College of engineering manual, "Irrigation" by Col. W. M. Ellis, late Chief Engineer, Irrigation, Madras, 1931. (Government press Madras)

(5) Hydraulics of Miami flood control project, Miami conservancy district, Technical report Part VII by Sherman, M. woodward.

(6) Slide rule for Routing floods trough storage reservoirs or lakes. Engineering News Record, 114, 1935 by C. J. Posey.

(7) Recent experiments on the flow of water over weirs by M. Bazin, 1890.

(8) "Side channel spillways" by Hinds and Julian, Amercian Society of Civil Engineer 1926.

(9) Hand book of Applied Hydraulcis by C.V. Davis, 1942.

(10) Boulder Dam Bulleton-2, Bureau of Reeclamation U.S.A. 1941.

(11) "On the shape of weir nappes" by E. Seimemi (in Itallian) 1930

(12) "Hydraulic design of Shaft spillway", by F. Kurz, Trans. A.S.C.E., 1925.

20. Examination Questions.

(*i*) Mean annual rainfall in an area of 2000 acres is 27 inches. Find the day's capacity of the reservoir making usual allowances for percolation and absorption in severe tropical climate, assuming daily requirements per head as 15 gallons. Find the population it can serve ?

(*ii*) Describle the conditions which govern the capacity of storage reservoirs for irrigation purpose.

(*iii*) Describe briefly the conditions which govern the silt trouble in storage reservoirs giving with sketch the usual devices used to remove the silt deposits.

(*iv*) What are the chief sources of loss of water from reservoirs and channels and suggest remedies for removing the losses from reservoirs and state also what allownce has to be made for these in preparing a project. (Mysore 1940)

(*v*) A tank in Plain ~~*maidan*~~ part has an independent catchment of 20 sq. miles. What is the exact length of the waste weir required ? Draw typical cross section of a clear overfall weir to negotiate a fall of 5·0 ft. in the draft channel consisting of erodable soil.

Catchment area 40 sqaure miles.	coefficient,
3 to 10	700
20 to 25	550
25 to 30	500

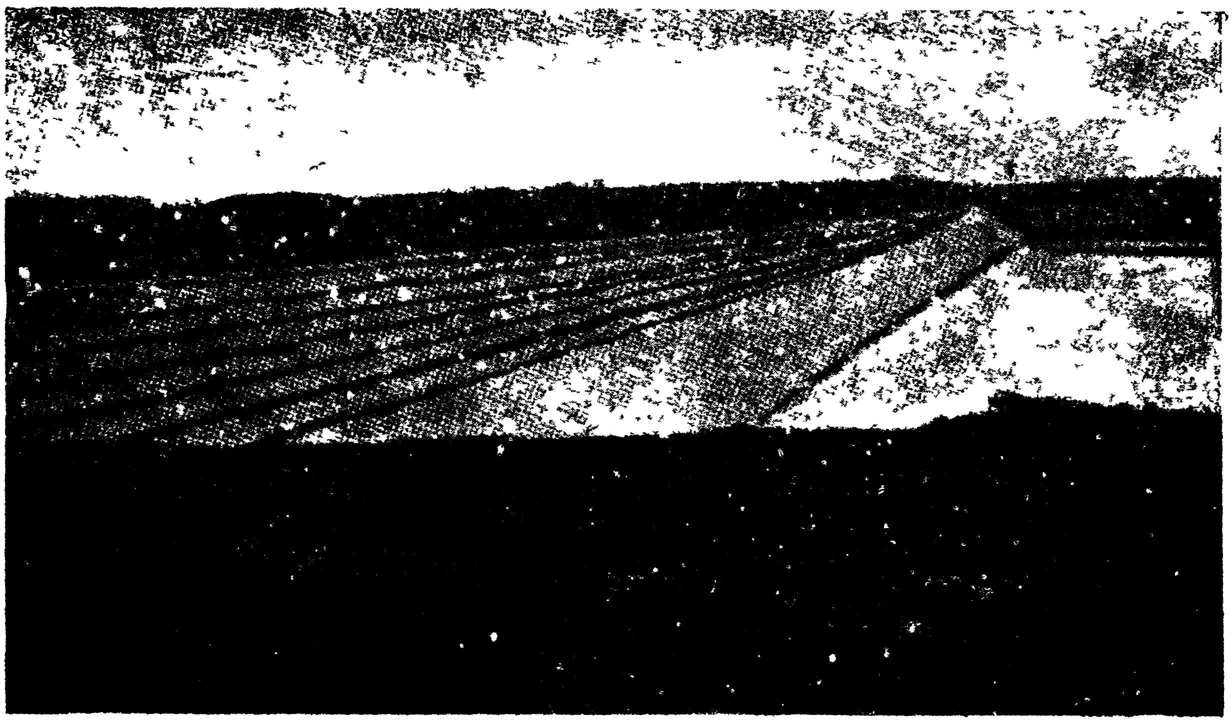
(*vi*) Explain the use of pick-up weir and waste weir in tank irrigation project. (*P. U.* 1944)

(*vii*) (i) Give the formula connecting rainfall and runoff for various types of catchments and explain the significance of the various symbols used.

(ii) Sketch the cross section of a high dam showing in particular (a) spillway. (b) spilling basin. (c) river outlets (d) penstacks (e) possible location of house. (*P.U.* 1952)

(*viii*) Distinguish between precipitation and run-off. (*P.U.* 1657)

Echo Dam, Completed in 1930, height _25'



Mckay Dam, completed in 1926, height 165′ Earth Dam.

PART III
TANK IRRIGATION
(Storages And Dams)
CHAPTER II
Earth Dams

1. Introduction.

The general principle which governs the design of construction of dam has been well understood by early engineers, who had given special attention to the subject even though they based their designs on the thumb rules based on the experience of previous constructions. The development of the science of soil mechanics has placed a valuable tool in the hands of the engineers, when which properply used enables them to design and build earth dams with greater assurance and economy than was previously the case.

Unfortunately the large and ever increasing number of failures of earth dams is the evidence of the inadequacy of the design methods in the common use in the past. Today an engineer experienced in the design and construction of earth dams, ulilizing the recent developments in soil mechanics may prepare a sound and well balanced design for almost every site. Dam engineers are rapidly becoming very scientific and are less rigidly bound for precedent provided that the proposed departure from precedent can be proved scientifically correct and also economically practicable. Precise and highly mathematical method for the design and analysis for earth dams are seldom justified because we know in advance that the assumptions, which must be made, are very seldom justifiable in actual practice. This chapter deals with the design and constructions of the earth dams, as practiced in the past and as now commonly adopted.

2. Foundation of Earth Dam.

(*a*) The Foundation of earth dams are often more or less recent alluvial deposits in the river bed which has not been consolidated under any material load. Coarse sands and gravels in the foundations earth dam give no trouble with regard to stability of the foundation, because even though they may not be consolidated, they will promptly consolidate as the load is applied. In case of very fine uniform sand in the dam foundations great caution is necessary. If the density observations of the soil samples from the foundations (Part VI, Chapter I) show the density less than the critical density of the saturated foundation material, then load will flow almost like a liquid. During the construction operations the disturbances like blasting or passage of material trains or earthquake activate the foundation soil resulting in clay sands. It is possible to consolidate such a foundation by the loads of the construction materials when it becomes more dense than the critical density and is thus no longer subject to flow on disturbances. A plastic clay foundation is the worst type of foundation for an earth dam. This requires the greatest amount of study and investigation in order to obtain unquestionable safety. Extremely flat slopes must be used for the earth dam build on such foundation in order to keep the stresses in the foundation sufficiently less than the strength of the meterial to provide a suitable factor of safety. The condition of the foundation is one of the importan factor in choosing a dam site. Other things being equal, we should choose the dam site with the best foundation conditions. The earth dams are constructed generally in Sub-Mountaineous regions where good earth is available near the site for the construction material. In the Sub-Mountaineous regions the foundations generally consist of grit sands and silt clay. Sometimes thematerials for the construction of earth dams are available higher up in the rocky hills wh the rivers have rocky and boulder beds.

(*b*) The foundation of earth dams may be classified in the following catagories.

(*i*) **Rock.** With a rock foundation a careful examination must be made to locate open seams, fissures, caverns through which leakage might endanger the structure. The presence of these should be determined by water pressure testing of the bore hole. Where any considerable leakage is detected, it should be sealed by grouting under pressure with water and cement or cement and some admixture of sand. Ordinarily in grouting, the pressure used should not exceed the safe load for the matter of the rock. Grouting pressure range from 5 to 10 lbs per sq. inch at the surface to 1000 lbs per sq. inch in deep holes. The usual thumb rule is that the pressure in lbs per sq. inch at any elevation should not exceed the depth in feet. The following equations are suggested as a rough guide only (Bib. 1.). Let p be the allowed pressure in lbs. per square inch at a depth h ft. below the surface, then

For massive rock, $p=h+1{\cdot}33h\left(\frac{h}{100}+\frac{3\sqrt{h}}{20}\right)$ (1)

For sound stratified rock, $p=h+1{\cdot}33h\left(\frac{h}{900}+\frac{\sqrt{h}}{20}\right)$ (2)

For sound stratified rock which has been grouted above the given elevation,

$$p=h+1{\cdot}33h\left(\frac{h}{400}+\frac{\sqrt{h}}{40}\right) \qquad (3)$$

Thumb rule can safely be exceeded in practice to the extreme calculated as above.

It is the common practice when an earth or rockful dam is located on rock foundations that key walls are constructed at suitable intervals under the dam. Usually a key wall of concrete lightly reinforced cut into the rock about 4 ft. wide, 2½ ft. deep projecting 5 ft. above the base and about 1½ ft. at the top will be sufficient.

(*ii*) **Pervious foundations :** – This term is applied to the foundation when they are more pervious than the materials of the dam. Pervious foundations generally consist of sand and gravel. Whether the pervious materials are relatively shallow and very deep say 100 ft. it is possible to design suitable stable dams on them. However, where the loss of water is an important economic factor, up stream earthen blankets, puddle, earth concrete or sheet pile cut off, should be used to reduce seepage. Sometimes the foundation material is also treated with cement clay or chemicals.

(*iii*) **Impervious deposits over rock** :—This type of foundation presents little difficulty from the point of view of seepage, but may be structurally unsafe. When there is considerable depth of fine silt or silt clay which could be easily converted into slush the site should be abondoned.

(*iv*) **Impervious over pervious** :—This type of foundation also presents serious difficuties. There is possibility of piping. In this case blankets upstream must be used and provision made for filter drains or rock toe on the downstream side extending to a depth close to the pervious strata.

3. Material of Construction of Earth Dam.

(A) In the interest of economy, the design of earth dam should be adopted to utilize the materials available at or near the site. The material of a concrete or a masonry may come from a distance as relatively smaller quantities are involved. But in the case of earth dam very much greater quantities of the material are involved, the material must come from the borrowpits near the site. Thus there will be a vast variety of the materials of construction of earth dams. This may be clay, loam, silt, sand, grit or gravel. It becomes the duty of the engineer to find out the properties of the materials available in the foundations and in the borrowpits as described in (Part VI Chapter I) and to base the design according to these tests.

Usual tests carried out in cases of coarse sand and gravel are as given below :—
(*i*) Mechanical Analysis (*ii*) Permeability (Transmission constant or permeability coefficient or Percolation intensity coefficient as used by the author, (Part V, Chapter IV) (*iii*) Void Ratio (*iv*) Presence of soluble material (*v*) Specific gravity of particles (*vi*) Dry density and (*vii*) Direct shear tests (not necessary with clear coarse sand and gravel as shear strength is usually high and the angle of internal friction is known more than 30°).

Usual tests carried out in case of clays, silts and fine sands are given below: -

(*i*) Mechanical analysis.
(*ii*) Hydrometer analysis of fraction passing finest sieve in above.
(*iii*) Permeability tests (not necessary in many cases).
(*iv*) Void ratio.
(*v*) Moisture content.
(*vi*) Plasticity limit.
(*vii*) Liquid limit.
(*viii*) Specific gravity of particles with Lichatilier Flask.
(*ix*) Density (dry weight per cubic foot).
(*x*) Percentage of soluble materials.
(*xi*) Direct shear tests.
(*xii*) Triaxial shear tests (occasionally, particularly where crictical density of very fine sands is in question).
(*xiii*) Consolidation tests.
(*xiv*) Optimum moisture content.
(*xv*) Expansion after consolidation.
(*xvi*) Shirnkage limit.

(B) In India, the dams so far constructed are really earthen in the true sence, the usual soil crust available is pretty deep. It contains fair proportion of clay about 25°/₀, fine silt and sand in proportions usually required to form an 'Ideal' soil for compaction (Part VI, Chapter I). The meterial characteristic are described below: --

(*i*) Earthwork gives way by slipping, or sliding of its parts. The resistance to this motion is due partly to the friction between the particles, and partly to their mutual adhesion or cohesion

The friction is measured by the angle of repose and constants for it for different soils have been determined. Friction is greatest for course and least for fine soils. On it depends the permanent stability of natural earthwork. A slight addition of moisture increases the co-efficient of friction, but an excess of water acts as an unguent in diminishing friction.

The adhesion or cohesion, may be measured by the depth to which an unsupported face of earthwork will stand before it is affected by the weather. It gives additional stability to earthwork. It is an extremely varying force, and is increased by a moderate amount of moisture, but is diminished by excessive wetness.

It is, therefore, evident that earthwork will be most stable when slightly damp, and least stable when charged with water. Hence its stability depends upon the facility and throughness with which it can be drained of superfluous and dangerous water.

(*ii*) Moreover, the stability of a soil depends upon the nature of the formantion and its stratification. There is very little research work done on this aspect of the problem but the practical engineers are fully aware of it. Earth consolidated in lyers, especially when the layers slope downwards away from a wall, causes very much reduced pressure against it. The angle of interal friction is not constant for any material but varies with the degree of consolidation of its particles. It is only possible to measure it when the soil is in the loose state and will, therefore, provide an ample factor of safety when the soil in practice is laid and consolidated in layers.

(*iii*) Moreover, proportions of clay, silt and sand play a great part in determining the cohesiveness and the stability of soil. Ideal soil for maximum compaction must contain definite proportions of these as explained in detail in (Part VI, Chapter I.)

(*iv*) In nealy all earthworks the practice is to treat the material as homogeneous from top to base, and to adopt uniform slopes throughout. The lower portions in a high dam must, however, be in a very different condition from that of, the upper ones, as they are far more highly compressed and are moisted. Probably the enormous superincumbent weight causes some stratification of the lower parts and also diminishes their cohesion. The increase in moisture at the base will diminish both the frictional resistance and cohesion. The variation in nature of the materials, and the differnce in their disposition and the methods of construction, introduce further elements of change. So there are numerous entirely hidden forces at work, of which the magnitude and resultant action can be determined only from experience of the work themselves. Well constructed dams up to 75 feet in height can be formed with the same slopes throughout ; for those of greater height it will be advisable to construct them of varing slopes, with strong toes of dry stone (packed with good grity and clayey soil to make them fairly water-tight, and to increase their resistance to slipping) or with berms at the base to buttress them.

The following table gives the general sections which may be adopted with safety and economy for all ordinarily good soils properly consolidated and resting on good foundations :—

1 Height of top of dam above ground level.	2 Height of top of dam above H F L.	3 Top width.	4 Upstream slope.	5 Downstream slope.	6 Width of dam at H F. L.
	ft.	ft.	Ratio of horizontal width to vertical height.		ft.
1 15 feet and under	4—5	6	2—1	1½—1	20—23½
2. 15 feet to 25 feet	5—6	6	2½—1	13/4—1	27½—31½
3. 25 feet to 50 feet	6	8	3—1	2—1	38
4. 50 feet to 75 feet	7	10	3—1	2—1	45

Above 75 feet in height special precautions have to be taken. They may consist in flattening the slopes considerably at the base, adding berms at the base, or constructing strong dry-stone toes. There are several instances of earthen dams having been raised to 100 feet in India and hydraulic-fill dams even up to 300 in America.

In a long dam three changes at the most will suffice, different sections being adopted for the gorge embarkment and the high and low parts of the flanks embarkments. To facilitate the work of setting out, such changes should be made in lengths of 100 or 200 feet, instead of being uniform throughout the whole dam.

The typical section as used India are in Fig. 1 to 4. Typical sections of some of the well-known dams of U. S. A. are given on folder pages 28*a* and *b*

(C) If near the site nothing is available but sand, then adopted design should utilize this sand for the bulk of the dam limiting the imported material of concrete, clay, or silt for providing impervious core to the minimum required. Figure 5 shows a design suitable where there is nothing available but sand gravel. The nearest impervious material is a sandy clay.

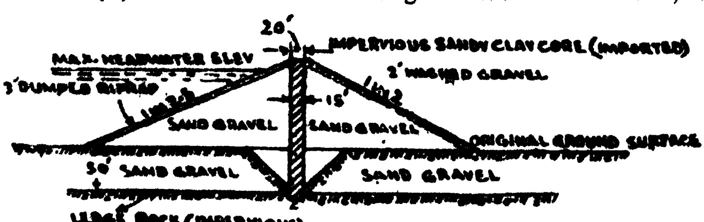


Fig. 5. Suitable design for a site where only sand gravel is available and foundation is pervious to depth of 50 ft.

A trench 1 to 1 slope is excavated to rock or impervious stratum below the foundations or to a vertical distance of at least 50 ft. Impervious stratum or rock is cut and about 15 ft. width clay section usual puddled bounded to it. The rest of the trench is then filled in dry and throughly compacted using sheep foot rollers. Above the base of the dam, the 15 ft. wide core is extended, being placed as a part of 8″ to 9″ layers in which the dam is being curved up and compacted. The drainage condition in such a dam would be satisfactory. The upstream slope is usually flatter.

Figure 6 shows a design suitable for a site where both clayey silt and coarse pervious sand are available in adequate quantities in borrow pits near the site. The impervious stratum 50 ft. below the base of the dam has adequate shear strength. The foundation cutoff leaves the pervious foundation downstream from the cutoff available as a drain for such water as gets through the core of cutoff.

IMPERVIOUS STRATUM

Fig 6. Suitable design for a site where both clayey silt and coarse sand are available and where foundation is pervious for depth of 50 ft.

Note :—Steel sheet piling might be substituted for foundation cutoff.

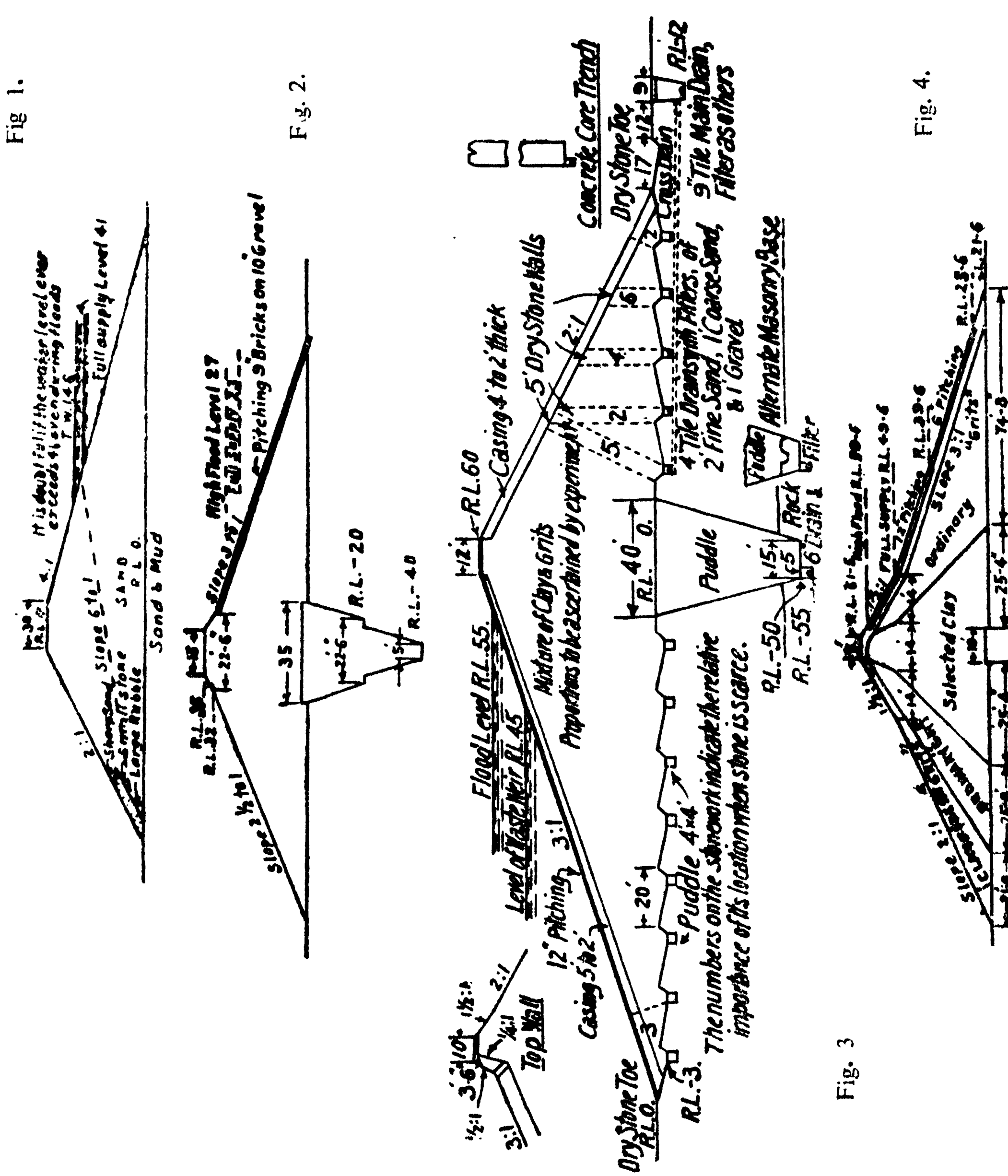


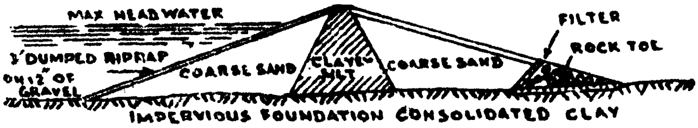
Fig 1.

Fig. 2.

Fig. 3

Fig. 4.

The design in Fig.7 is suitable for a site where both clayey silt and coarse sand are available in adequate quantities and where the foundation is impervious (either ledge rock or consolidated clay). In this case it is evident that such water as does get through the relatively impervious central section of the dam must appear at the downstream face or toe because it cannot enter the foundation. Hence to prepare for this condition and avoid the possibility of sloughing due to saturation a rock toe with a filter consisting of smaller stones with gravel and sand just ahead of it to protect it against impregnation is utilized. Even without going into the design from a quantitative standpoint, it is evident that seepage will be entirely insignificant.



IMPERVIOUS FOUNDATION

Fig. 7. Suitable design for a site where both clayey silt and coarse sand are available and where the foundation is impervious.

In Fig. 8 is shown a suitable design for a site where both sand gravel and clayey silt are available and where the foundation is highly pervious to a great depth. A blanket of clayey silt, which is very impervious as compared with the sand gravel of the foundation, is carried from the impervious core upstream under the upstream shell and extended for a distance frequently 10 or more times the head upstream from the upstream toe of the dam. Such blankets cut down the seepage materially by forcing the water to pass through several times the distance which it would have to pass through without the blanket.

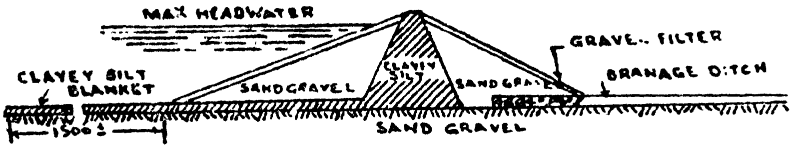


Fig. 8. Suitable design for a site where both sand gravel and clayey silt are available and foundation is highly pervious to a great depth.

Also under conditions in Fig. 8 the foundation will be full of seeping water and provision to take care of seepage is provided by a filter layer at the base which is even more pervious than the sand gravel of the foundation.

In Fig. 9 is shown an earth dam design which is suitable for a site where the only material available is a silty clay and where the foundation consists of a silty clay which is highly unconsolidated.

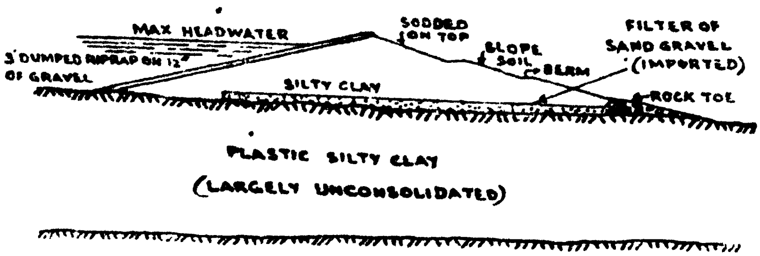


Fig. 9. Suitable design for a site where the only embankment material available is a silty clay and where the foundation consists of a silty clay which is largely unconsolidated.

In this case the upstream slope is flattened to take care of rapid drawdown, and also in many cases the flatness of both slopes is determined by the requirements for spreading the load so that the maximum unit stress induced in the foundation will be less than the shear strength of the plastic material in the foundation with a fair factor of safety. It will be noted that a filter layer is placed on the foundation under the base of the dam except near the upstream toe. The filter layer of sand gravel may be "Run of bank devoid of clay", as it will surely be vastly more pervious than the silty clay. This filter layer has two parlicular functions :—

(1) To provide drainage for the small amount of seepage and thus prevent any possibllity of saturation of the downstream face and (2) by providing drainage for water squeezed out of

the silty clay by the loading added during construction, it has a marked effect in accelerating consolidation and hence hastens an increase in shear strenght of the foundation material.

4. Practical Criteria for the Safety of Earth Dams.

(*i*) There should be no danger of over topping.

(*ii*) The seepage line should be well within the downstream face.

(*iii*) There should be no opportunity for the free passage of water from the upstream to the downstream face.

(*iv*) Water which passes through or under the dam when it reaches the discharge surface should have a pressure and velocity so small that it is incapable of moving the material of which the dam or its foundation is composed.

(*v*) The upstream face should be properly protected against wave action and the downstream face should be protected against the action of the rain.

(*vi*) The upstream face slope should be safe against sudden drawdown.

(*vii*) The upstream and downstream face slopes should be flat enough that with the material utilised in the dam, they should be stable with ample factor of safety.

(*viii*) The upstream and downstream slopes of earth dams should be flat enough so that the shear stress induced in the foundation is less than shear strenght of the material in the foundation with enough factor of safety.

5. Safety Against Overtopping.

There should be sufficient free board to guard against overtopping of a earth dam. Gross free board is the term usually used for the vertical distance from crest of spillways to the top of the dam. It is so sometimes also called 'Surcharge'.

Net free board is the vertical distance from the high flood - surface expected in the reservoir to the top of the bank. when simply free board is mentioned, it stands for net free board unless otherwise stated. The net free board should be the sum of the length of the tides, sieches, wind set up, and the height to which the waves will rise on the upstream face plus a margin of safety in fact based on judgment. Margin depends on the reliability of flood discharge calculations. The free board should atleast be equal to the same height above spillway crest required for maximum estimated spillway discharge plus an allowance for wave action.

The height and velocity of the waves is a function of the fetch or exposure over open water. The formula applicable to reservoirs is given by Stevenson as $h_w = 1.5\sqrt{f} + 2{\cdot}5 - \sqrt[4]{f}$ where h_w is the length of the wave in feet and f=fetch in nautical miles For value of f more than 10, $h_w = 1{\cdot}5\sqrt{f}$,

For wave height between 1 and 7 feet, the velocity feet per second is given approximately by the formula : $V = 5 + 2h_w$. This formula was developed empirially by Gaillard in 1935 reprinted by Engineer School, Ford eluoir, Va. The wind velocities normal to the dam, from the reservoir will not exceed 60 miles per hour. Free board for wave action in feet $= 0{\cdot}75 h_w + \frac{v^2}{2g}$ where h_w and v are the height and the velocity of the wave as culculated above.

Crown width. The crown or top width of an earth dam should not be less than 10 ft. for maintenance purposes. Where a highway is to use the top of the dam, road requirements may fix the minimum. Apart from these requirements, the crown should be wide enough to provide a margin of safety in case the wave-wash protection fails and wave erosion takes place during a flood stage in the reservoir. Common practice is fairly well represented by the formula.

$$C_w = 3\sqrt{H_d}$$

Where C_w-crown width.

H_d-height of dam.

With sandy materials the top width should be greater, and with rock or tough clay-type embankments the safe width may be less.

6. Seepage.

(A) Seepage takes place through and under all dams, both earth and concrete. The problem is to minimise and control seepage so that it will have no harmful effects. The character of the materials comprising the foundation and the embankment has a very important influence on seepage and its effects. For any dam of homogeneous material founded on an impervious base, the seepage will pass through the dam and will appear on the downstream face because there is no other place for it to go. This will happen regardless of the tightness of the embankment material. If the dam of homogeneous material is founded on a pervious foundation, seepage may still be expected to appear on the downstream face unless a cutoff has been constructed through the pervious foundation, thus permitting the downstream portion of pervious foundation to act as a drain. Of course, it is a simple matter to provide drainage so that the seepage does not reach the downstream face but will be taken care of and conducted to the downstream toe. In that case the dam would no longer be strictly speaking, composed of homogeneous material.

(B) Seepage line coincides with the phreatic surface (Chapter III, Part VI). It is always desirable to be able to predict at least approximately, the position of the line of seepage in the cross-section of a proposed earth dam. If this line is allowed to intersect the outside downstream face much above the toe more or less serious sloughing may take place and ultimate failure may result. Arthur Casagrade treatment of the subject (Bib. 2) is briefly stated below :—

For an earth dam composed of homogeneous material the seepage line will cut the downstream face above the base of the dam unless, of course, special drainage measures are adoped. The location of the seepage line in this case and the point at which it cuts the downstream face is dependent only on the cross-section of the dam. Its position is not influenced by permeability of the material composing the dam so long as that material is homogeneous. The seepage line under the assumed conditions has been shown to be fundamentally a parabola with departures therefrom due to the local conditions of ingress and egress as in Fig 10.

B_2 is a point on the seepage parabola extended to intersect the water surface.

A is the downstream toe of the dam. If the dam is composed of a relatively impervious core with a pervious shell, A is the downstream toe of the core.

C is the intersection of the seepage line with the downstream face of the dam (or core).

d is the horizontal distance from point B_2 to point A.

h is the verticle distance from pt. B_2 to pt. A and represents the head causing seepage.

a is the distance AC and represents the wetted portion of the downstream face.

α is the internal angle formed by the downstream discharge face and the horizontal base, as in Fig. 10 (A).

m is the horizontal projection of the wetted upstream slope.

k is the coefficient of permeability of the material comprising the dam (or core).

Calculations of seepage through soils are based on Darcy's law.

Casagrade has shown that the computed seepage line for all but the smallest angles of discharge slope approximate quite closely the 'base parabola' established by professor Kozeny (Bib 3). for the case when α is 180°.

If in Darcy's Eq. the area of cross-section, A, at any point along the base of the dam is represented by 'y' and the hydraulic gradient, 'i' at that point is represented by the slope of the seepage line $\frac{dy}{dx}$ (see Fig. 10), then the seepage flow through the dam would be given by

the equation $q=ky\frac{dy}{dx}$ (1)

Kozeny proved that for the case where $\alpha=180°$, the seepage line could be represented by the equation $X=\frac{y^2-y_o^2}{2y_o}$ (2)

Which is a parabola with the focus at A intersecting a perpendicular at A at a distance y_o from the base line (Fig. 10).

The parabola should be continued to intersect the water surface at point B_2 (Fig. 10), which has the coordinates $y=h$, $x=d$, where d equals the width of the base of the dam minus 0·7m. If these values of x and y are substituted in Eq. 2, the value of y_o becomes.

$$y_o=\sqrt{h^2+d^2}-d \qquad (3)$$

The value of y_o may be readily determind graphically since it is the difference between the slant distance and the horizontal projection of the line AB_2, Fig. 10.

The point C_o where the base parabola intersects of the downstream face is easily found from the polar equation of a parabola which is $r=p/(1-\cos\theta)$ (4)

where r is radial distance from focus to a point on parabola, p is intercept of parabola on normal to axis line through focus, θ is the angle of radial line with axis of parabola.

For the particular values of y, r, and o being considered $r=a+\triangle a$=slant distance from A to point of intersection of base parabola with downstream face, $p=y_o$=intercept of parabola on vertical line through focus, $\theta=\alpha$=angle of downstream face.

Thus it follows from Eq. 4 that $a+\triangle a=y_o/(1-\cos\alpha)$ (5)

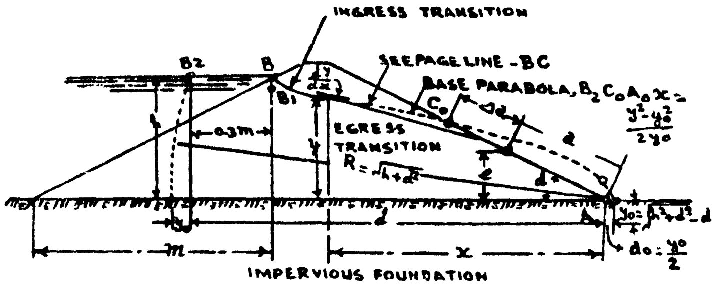
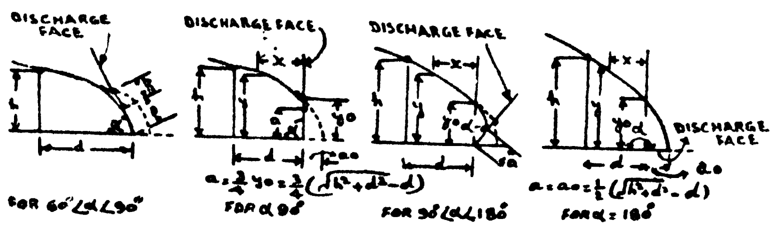


FIG 10 DETERMINATION OF SEEPAGE LINE (A)



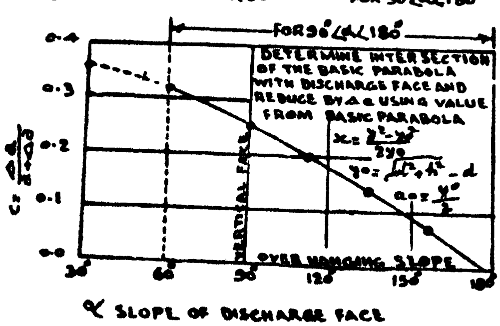


Fig. 10B. Graph to determine $\triangle\alpha$ and α

Reference to Fig. 10 shows that the intersection of the seepage line with the downstream face occurs at point C, a distance $\triangle a$ below the point of intersection for the base parabola C_o. Casagrade (Bib. 5) has shown that the distance $\triangle a$ varies with the slope angle α, becoming zero when $\alpha=180°$. Fig.10B gives the ratio of $\triangle a$ to $a+\triangle a$ as deterimined by his graphical studies by means of flow nets. The lower end of the seepage line is completed by drawing in a transition curve from C to the base parabola by eye, as indicated in Fig. 10.

The upstream end of the seepage line is also sketched in by eye, a short transition curve being drawn to connect point B with the base parabola as shown in Fig. 10. This transition curve should start normal to the upstream face at the point of intersection with the free water surface. Where the upstream face of the dam or core has a very steep slope, the transition may be a reverse curve.

There are several equations which may be used to calculate the seepage flow. One of the simplest of these is derived for the case where the seepage line is represented at its lower end by the base parabola ($\alpha=180°$). From Eq. 2 of the base parabola.

$$y=\sqrt{2xy_o+y_o^2} \quad (6) \quad \text{and} \quad \frac{dy}{dx}=\frac{y_o}{\sqrt{2xy_o+y_o^2}}$$

If the values of y and dy/dx are substituted in the Darcy equation as given in Eq. 1.

$$q=k\sqrt{2xy_o+y_o^2}\times\frac{y_o}{\sqrt{2xy_o+y_o^2}} \quad \therefore \quad q=ky_o \quad (7)$$

Eq. 7 gives the seepage flow for the case where $\alpha=180°$. Inspection of Fig. 10.B shows that the mean length of path and the cross-section area of the seepage flow is but slightly different for angles less than 180° but greater than 30°. If the values of y_o given in Eq. 3. is subtituted in Eq. 7

$$q=k\sqrt{h^2+d^2}-d \qquad (8)$$

For most cases encountered in earth dam design, the seepage may be calculated with sufficient precision by Eq. 8: where the angle of the discharge face, α is less than 30°, the following equation may be used.

$$q=ka\sin^2\alpha \qquad (9)$$

when $$a=\sqrt{h^2+d^2}-\sqrt{d^2-h^2\cot^2\alpha} \qquad (10)$$

Eq 9 gives somewhat smaller values for q than Eq. 8 in the range for which it is suitable.

(B) (*ii*) **Seepage line where vertical and horizontal permeability differ.** Soils deposited by water and soils placed in earth dam, in rolled dams or in hydraulic fill dams may show a wide difference between their vertical and horizontal permeability. For water deposited soils in nature, horizontal permeability may be 4 to 20 times the vertical permeability. In such cases where one desires to locate the seepage line and/or draw the flow net a transformed section may be utilized. To make the transformation, mlutiply the actual horizontal dimentions by $\sqrt{kv/kh}$ where kv equals permeability coefficient of the material in a vertical direction and kh equals permeability coefficinent of the same material in a horizontal direction. The line of seepage and flow net may thus be determined in the same manner as for a soil which is homogeneous and isotropic, and the dimentions including the seepage line and/or the flow net may be transfered back to the true section.

(B) (*iii*) **Seepage line in earth dam of composite cross-section.** An earth dam which consists of a central section of highly impervious material as silty clay, with shells of very material frequently a very desirable type of dam both from the standpoint of stability and watertightness provided that the dam is on a firm and relatively impervious foundation. The cross-section of such a dam is shown in Fig. 11.

The shells of sand gravel are doubtless several hundred times as pervious as the central portion of the dam consisting of clayey silt. The upstream pervious shell will have practically no effect on the position of the phreatic line, and the downstream pervious shells will act as a drain. Owing to the tremendous difference in permeability, the shell will have practically no influence on the position of the seepage line in the central section. Consequently the position of the seepage line may be determined of the central section by the methods discussed.

In the downstream pervious shell of the dam the seepage line will rise above tailwater only slightly, just enough to provide the necessary head for the slight amount of water which gets through the central impervious section to flow out through the downstream shell. The approximate position of the seepage line for such a section is shown in Fig. 11.

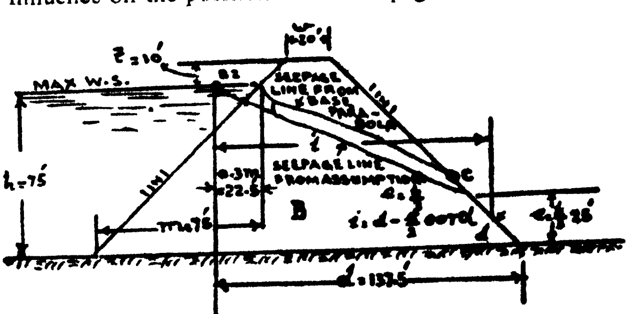


Fig. 11. Central portion (B) of fig. Showing comparison of rough method of plotting seepage line with seepage parabola.

7. (A) Calculaions of Seepage Flow.

All methods of computing seepage under or through earth dams are primarily based on Darcy's law $q=k.i.A$. Of the following formulae for seepege, three have already been developed in connection with the determination of the seepage line in the foregoing sections, but they are assembled here to present a resume of formulae for computing the seepage through earth dams.

In all these formulae the symbols have the following meaning :—

q discharge in cubic feet per minute per foot of width.

k peameability coefficient in feet per minute.

i hydraulic gradient.
h head causing seepage.
l length of path of seepage in which loss head h takes place; thus $i=\frac{h}{l}$.
d horizontal distance from the point where seepage tangent intersects the maximum headwater surface to the downstream toe of the section as in Fig. 10.
α angle which discharge face makes with the horizontal. (Fig. 10)
e vertical distance from base to point where the seepage line intersects the discharge face.
a ledgth of line on the discharge face from downstream toe up to point of intersection with seepage line (See Fig. 10).
z free Board.
w top Width.

$q=k\ a \sin\alpha$

Where $a=50-\sqrt{50^2-\frac{b^2}{\sin^2\alpha}}=\sqrt{b^2+d^2}-\sqrt{d^2-b^2\cot\alpha}$

$50\ \sqrt{b^2+d^2}$; Applicable where α is less than 30°

$q\ k(\sqrt{d^2+b^2}-d)$

Applicable for α between 30° and 180°

$q=\frac{4kh^2}{9l}$

Where $l=(1{\cdot}3b+2z-\frac{e}{2})\cot\alpha+w$

$=(1{\cdot}133b+2z)\cot\alpha+w$, since $e=\frac{b}{3}$

The above is an approximate equation, applicable for α less than 90°.

For a section where the horizontal and vertical permeabilily coefficients differ materially, the value of k used in the above equations is determined as follows :—

$k=\sqrt{k_v \times k_h}$

where k_v is the vertical permeability coefficient and k_h is the horizontal permeablity coefficient.

While usually in natural formations the horizontal permeability is from 4 to 10 or more times the vertical, there are exceptions. Thus loess in its natural undisturbed condition is sometimes 20 to 50 time as pervious in a vertical direction as in a horizontal direction. This is because of the verical passageways or tubercles which abound in loess formations.

(B) **Total seepage through and under the Dams.** If the base of the dam is pervious and if it is desired to determine the total seepage through and under the dam. Referring to Fig. 12.

$qr=qd+qf$

where qr is total discharge through the dam and foundation per foot of width, qd is discharge per foot of width through the dam and qf is the discharge below the base.

(C) **Piping.** (i) Piping occurs when seepage water issues from an embankment or ground surface under sufficient pressure and with sufficient velocity so that the particles comprising the material are carried away under identical conditions.

(ii) **Vertical piping.** Vertical piping sometimes occurs at the downstream toe of an earth dam or levee. Usally in such cases there is a layer of material in the foundation which is much more pervious than the strata at the surface of the foundation, and as the head increases the hydrostatic pressure in this layer increases until the water lifts the overlying strata, forming boils on the surface of the ground.

Boils in themselves are not necessarily disastrous. Sometimes boils at its downstream toe continue for several years without dangerous results. In each one of the involved cones which forms a boil the material is in a condition of floation or suspension ; individual particles rise up and fall back. The head increases a bit and than the particles in the cone rise higher and flow over the edge of the cone or boil and flow away. This is then a condition of incipient vertical piping.

With this condition, to save the dam it is necessary to quickly ring the boils with sand bags, put in a low auxiliary dam downstream, or adopt other means so that some back pressure will be exerted on the boils. If sufficiently prompt action is taken it is usually possible to establish a condition of stability. so that the boils will just go on boiling and the water will flow out of the boils, but it will not carry material with it.

To make a permanent repair, coarse sand and gravel is first placed over the area in which the boils occur and then covered with coarse stone until the added weight overcomes any tendency toward movement even at maximum head. Local vertical piping may occur even if the path of percolation is very long in relation to the head because all that is necessary is that the escape gradient should be very steep. Such local piping remote from the dam is generally not serious.

(*iii*) **Horizontal piping.** Horizontal piping occurs where the seepage water issue from the downstream face of an earth dam or natural bank with enough force and velocity to carry away particles of the materials of which the dam or the bank is composed. With horizontal piping the water sometimes almost gushes from the face and erodes the downstream face below the point of egress. The horizontal piping soon results in the formation of a small tunnel running back into the dam with the roof continually falling in and being carried away by the piping water. Once horizontal piping has started the only remedy is to dump rock grading from fine to coarse right into the downstream face where the horizontal piping is occuring so that an improvised drain and filter will be formed and the piping stopped. Both horizontal and vertical piping are serious and may lead to complete failure of the dam if not promptly corrected. The possibility of serious piping may be prevented by having the path of percolation sufficiently long in relation to the head, thus reducing the hydraulic gradient and by providing properly designed and costructed filters and drains so that a dangerous escape gradient will be avoided. Under special conditions where drainage and filter system are specially designed for given conditions lower ratios of l/h may be acceptable.

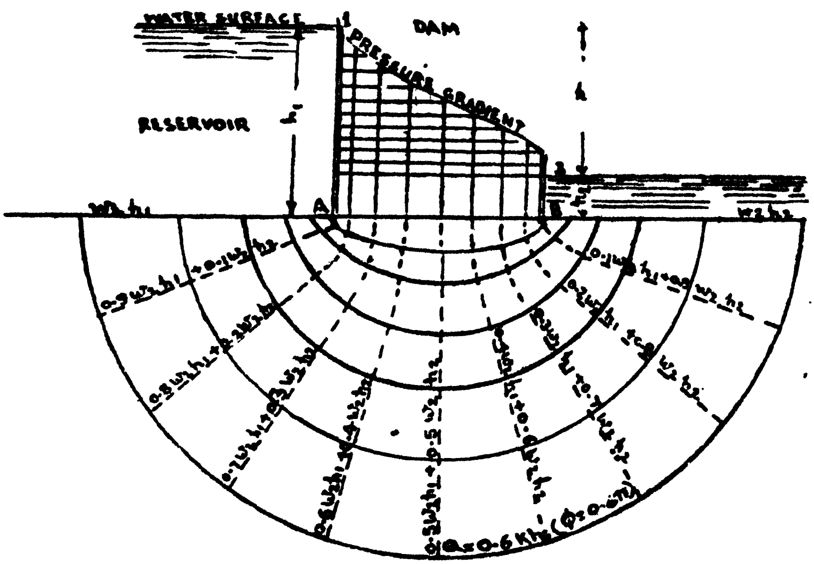


Fig. 12. Percolation of water under an impervious dam

8. Flow Net and Seepage Flow Under Dams.

The flow net is a diagrammatic representation of the lines of percolation and lines of equipotential in a porous medium, such as earth subject to a head of water. Fig. 12 shows a typical flow net for a homogeneous, isotropic foundation without drains or cutoffs. A-B is the length of the impervious elements of the dam. The solid lines are the lines of percolation, or flow lines, and the dotted lines are lines of equal potential. In the true flow net, all of the areas bounded by any pair of flow lines and any pair of equipotential lines are homologous *i.e.*, have the same ratio of width to length.

The potential at any point in the foundation is the elevation to which water would rise in a piezometer at that point as shown in the figure. For homogenous, isotropic materials, the flow lines are normal to the equipotential line. The friction loss of seepage along any flow line is between equal to the head, h, on the dam. The head lost in friction between any two lines of equal potential is equal to the difference in their potentials.

The longer the path required for the flow lines, the futher apart will be the lines of equal potential, the smaller the friction loss per linear foot, the slower the velocity and the less the seepage per square foot.

The hydrostatic pressure, in feet of water, at any point on the base of the dam can be determined from the flow net simply by subtracting the elevation of the base from the potential at that point as shown in Fig. One hundred percent of the hydrostatic pressure is always assumed for uplift for dams on earth.

Should the permeability at all points in the foundation be increased, the amount of seepage and velocity of flow would be increased in direct proportion but the flow net and the amount of uplift on the dam would not change.

The flow net can be determined analytically for simple conditions; but where there are one or more cutoffs, strata of variable permeability, lenses of different permeability, drains and other complications, it can be made most conveniently by the electrical analogy model test.

If the permeability of the foundation, K, is known, the amount of seepage past any length of dam can be computed from the flow net. Casagrade has shown that the seepage loss in cubic feet per second may be found by the following equation, if the flow net is formed of squars.

$$Q = LKHS/60$$

When Q is discharge in cu. ft. per sec;
L is length of the dam considered, in feet ;
K is coefficient of permeability of the material in cu. ft. per min. per sq. ft. of area ;
H is head on the dam in feet ;
S is ratio of the number of squares between any two neighbouring equipotential lines and the number of squares between any two neighbouring lines of percolation.

For example, the coefficient of permeability, K, is the discharge in cubic feet per minute per square foot of the foundation material under unity gradient *i.e.*, when the friction loss per foot of travel is unity.

9. Methods of Dealing with Excessive Seepage Through Earth dam.

(A) **Earth with puddle core wall.**

The arrangement is shown in Fig. 13. In England it is a general practice to make a puddle wall along the centre line of the dam and vertically over the puddle trench, so as to from with it a water-tight septum (extending throughout the embankment from below bed rock level to above high-flood level) in order to intercept any infilteration that may have penetrated so far. In this position the quantity of the wall is reduced to the minimum and the puddle is most effectually protected from the action of the reservoir and the weather and from perforation by vermin. It may, however be distorted or broken by the settlement of itself, or by unequal settlement of the embankment on each side. It cannot be inspected or repaired to any depth. The top of a puddle wall is usually from 4 to 10 feet and is normally kept as one third of the base.

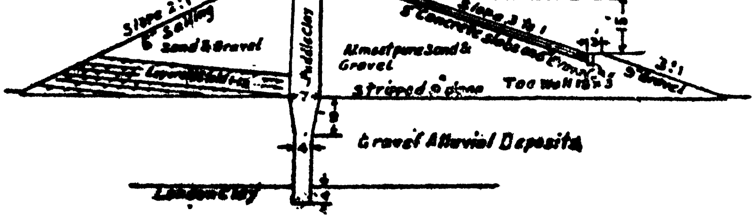


Fig. 13.

Earth work of the dam near the puddle needs to be specially selected and consolidated that the difference between the settlement of the puddle and earth work is made less abrupt.

A puddle lining to form the upstream slope has also been tried. Here it has the advantage that it will settle regularly with the embankment, can easily be inspected and repaired, and pervents excess infilteration into the whole mass of the work. It has, however, the disadvantages that it is not in direct connection with the puddle trench ; its volume, and thus its cost, are at a maximum. It is exposed directly to the reservoir water, the weather, and to burring rates and crabs, and it may not be able to stand at the ordinary slope of the dam.

In India puddle walls are not now built on account of there disadvantages. It is, therefore preferred to form the dam as one homogeneous mass, and recognized that the introduction of the puddle wall destroys that homogeneity. It would appear that in adopting the puddle wall English Engineers are influenced by the excellence of the clay at their disposal, and rely upon it to make good the permeability of the general mass of the dam due to its want of artificial consolidation or to the inferiority of its material.

Compared with a masonry core, a puddle wall of the same cost had the advantages of greater water-tightness and of flexibility, where by it can better conform to changes due to the unequal settlement of the earthwork on each side of it. Compared with each other, the central type of puddle wall appears to be better than the slope from.

(B) **Masonry and concrete core walls.**

(*I*) American Engineers prefer a masonry or concrete core wall instead of a puddle-wall Fig. 14 and 15. respectively. This should be founded on an unyielding, impervious stratum

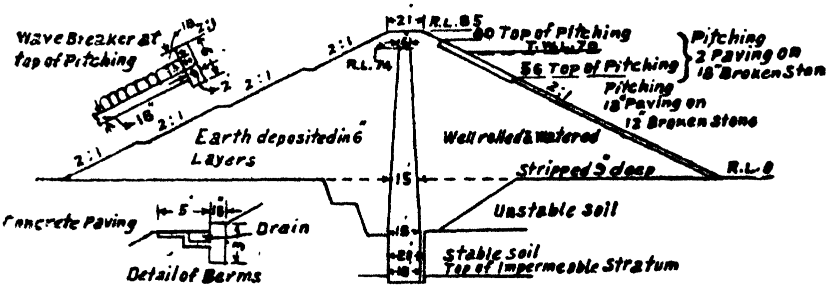


Fig. 14

should be thick enough to resist infilteration and pressure, due to the full head of water and rupture by unequal earth pressure, and should have its faces gently battered so that the earth-work of the dam during and after settlement may abut tightly against them.

The following advantages are claimed for the core wall :—

(*i*) It prevents water percolating through the upstream slope of the dam from passing into the downstream one.

(*ii*) It cannot be washed out, as puddle wall may be, should a leak through it be formed.

(*iii*) It separates the dam into two distinct portions, an upstream one, which should be made as water-tight as possible, and a downstream one, which should be formed as stable as possible. If the two portions abutted directly on each other, cracks might occur in the centre of the dam on account of differences in their settlement.

(*iv*) It enables the outlet culvert to be carried through the dam with perfect safety.

(*v*) It allows the outlet tower to be replaced by a "dry well" tower.

(*vi*) It gives an earth dam greater strength to resist the erosive action of water passing over its top.

(*II*) A modified form of core wall has recently been devised. This is built hollow, of reinforced concrete, with vertical cores walls at intervals so as to make it cellular. The longitudinal walls are spaced sufficiently for apart to permit of inspection of the interior, between them and are lined externally by a dry filtering layer, which collects the drainage of the earth-work of the dam and admits it through deep holes to the centre of the core wall, whence it is led out of the embankment by base drains on the downstream side.

This design has been recommended for adoption in hydraulic-fill dams,as it is considered that it will ensure the rapid drainage of the water deposited material and its early consolidation. Detail of typical expension joints used in concrete core wall is shown in Fig. 16.

(III) The section of a masonry or concrete wall is determined approximately as below (Fig.15).

Draw from the base of the core wall.

(*a*) On the water side a line ab inclined to the horizontal, at an angle ϕ equal to the angle of repose of the saturated earth *i.e.*, $\phi = 20$ to 23 degrees.

(*b*) On the downstream side a line cd, inclined at θ to the horizontal, where θ is the angle of repose of the rammed earth *i.e.*, $\theta = 45$ to 55 degrees.

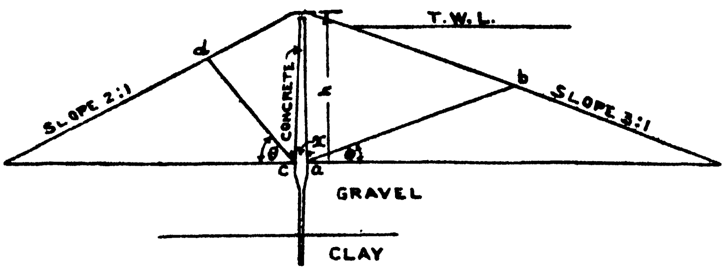


Fig. 15.

Calculate the area of the portions of the cross-section of the dam cutoff by these lines. Roughly they are with $\phi = 20$ degrees and a 3:1 slope $\frac{3}{4}h^2$, on the water side, and with $\theta = 45$ degrees, and a 2:1 slope, $\frac{h^2}{3}$, on the downstream side, where h is the height of the core wall. The weights of the saturated earth on either side may be assumed as 160, and 132 lbs per cubic foot so that the core wall has to sustain a thrust of $h^2 (160 \times \frac{3}{4} - 132 \times \frac{1}{3})$ lbs. per foot run or $76h^2$ lbs. say; and if the thickness of the wall is x feet its ultimate resistance to shear when composed of concrete is about 30,000 x lbs. per foot run.

Thus for strength only $x = \frac{fh^2}{400}$, where f is the factor of safety, say equal to 2, in view of the extremely adverse assumptions made. Unless h be great this will usually lead to smaller values of x than those indicated by practical experience, as requisite to stop percolation through the wall.

Herschell (P.I.C.E., Vol. 132, P. 255) gives the following rule :—

In first class work, 4 to 5 feet thick concrete at the bottom of the trench enlarging to 8 feet at the natural surface and tapering off to 4 feet at the top of the wall.

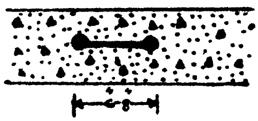
The masonry is relatively more pervious than concrete and therefore, the masonry core walls are relatively thicker as shown in Fig. 14.

10. Precuations to Guard Against Free Passage of Water Through Dams.

Free flow may be caused in a large number of ways as given below :—

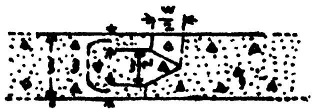
1. By water following the exterior surfaces of pipes or conduits through the embankment.
2. By Burrowing animals such as Muskrats.
3. By the placing of very pervious material containing large stones in an otherwise impervious embankment in such a manner as to make a blind drain from the upstream to the downstream face.
4. By failure to bond and compact the suceeding layers of the embankment properly.
5. By failure to bond the lower layers of earth dam properly to the foundation.
6. By water following the smooth surface of concrete abutments or other concrete structures, or passing through drains accidently created by loose deposites.

In case of pipes and conduits it is usual to provide concrete cutoff walls across them at suitable intervals. Pipes and conduits should never be placed in the dam. They should properly be located near the abutments away from the deep portion of the river bed in separately dug out trenches with concrete collars. It is usual practice to give in dug out trenches with puddle because good puddle work is seldom done on a big job it is perferable to fill the trenches with compacted soil.

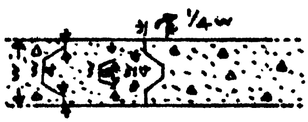


RUBBER JOINT

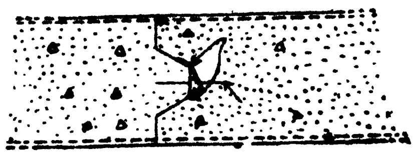
Rubber Expansion Joint applicable only to location where it will always bewet and dark.



WEDGE TYPE



THROUGH TYPE



REINFORCING COTINIOUS THROUGH JOINT

Strip of tar paper fastened on to concrete surface and metal strip with hot pitch.

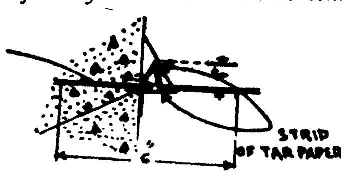
Copper or Aluminum strip No. 20 Gage.

WATER STOP TYPE

Suitable for any structure where absolute water tightness is essential.

No. 20 Gage copper or Aluminum strip.

This space stuffed with cotton waste



Enlarged section showing metal strip used in THE WATER STOP TYPE

Fig. 16. Type of expansion joints used for core walls in earth rock fill dams.

Burrowing hole by rats should not be disastrous in earth dams of massive sections but if there is any throuble on account of this, special treatment is shown in (Chapter VII, Part II). Proper bond should be ensured in succeeding layers earth fill. Foundation bond is ensured by removing loose soil, vegetation and large roots.

Spillway abutments power house walls or other concrete walls are provided with cutoff batresses projecting well into the embankment. Batresses are generally lightly reinforced usually 12″ cuter.

11. Remedies to be Applied to Reduce Excessive Flow Under the Dams.

(a) **Cutoff walls.** Cutoff walls are usually differentiated from core walls in that cutoff walls are in or on the foundation and just reach into the impervious section enough to make a bond with it, whereas core walls generally extend up into the body of the dam a substentical distance. Cutoff walls, like core walls, were much more used in previous years than they are today. As a matter of fact if proper care is used in bounding the impervious section of an earth dam to the foundation of ledge rock, hardpan, clayish material, etc., more resistance to seepage can be obtained than with the use of the usual concrete cutoff, for the simple reason that clayish silt is tighter than and also the length of contact is greater.

Cutoff walls are sometimes constructed from ledge rock or from any other relatively impervious stratum up through a pervious foundation stratum to make a bond with the relatively impervious section of the dam. As a rule, if the depth through the pervious foundation stratum to ledge rock or other impervious stratum is not more than about 30 ft., a trench refilled with relatively impervious soil, if available nearby, is usually the most economical way of obtaining a cutoff. If the depth materially exceeds 30 ft., it is usually more ecconomical to drive steel sheet piling to form a cutoff.

(B) **Sheet pile cutoff walls.** In many cases steel sheet piling is utilized for foundation cutoffs for the reason that when the depth exceeds about 30 to 50 ft. it is practically always cheaper to drive steel sheet piling than to dig a trench and refill it with compacted impervious soil. At Kingsley Dam, Nebraska, steel sheet piling 125 ft. long was driven to connect the core of the hydraulic fill dam to the impervious stratum below. With modern methods of driving and jetting it is feasible to obtain real assurance that the steel sheet piling reaches the recorded depth. By drilling holes each side of the piling it is possible to find out weather or not piling has curled or gone out at an angle. The kind of steel pipe section chosen is important. If there are many boulders a straight heavy section is desirable, with a strong interlock. It is not practicable to drive steel sheet piling so that it is absolutely tight. One should never expect it to be as tight as a cutoff trench filled with compacted impervious meterial. This if steel sheet piling were used instead of a cutoff of impervious material, one could with careful workmanship and the use of interlock compound reduce the seepage to 1/5 or 1/10 of what it would be without any cutoff. With poor workmanship, or if the piles cannot be seated in the impervious stratum, the reduction might be only 1/2 or less. There would be no chance at all of obtaining a reduction of seepage to 1/1000 of the seepage without any cutoff. Jetting is generally essential in reaching depth greater than 60′ or 70′ with steel sheet piling in sandy and gravelly material. In some cases in order to reach depths of 125 to 140 ft., it has been necessary to use jets discharging 200 to 300 gal. per min. at 300 ft. head and in addition to use compressed air with the water. Two or three jets on each side of the piles being driven are semetimes used in a yoke

Partial cutoff. A partial cutoff is one which extends down from the impervious section of a dam into the underlying strata, but does not reach an impervious stratum. In many cases it would be impracticable or extremely expensive to continue the cutoff to an impervious stratum, and accordingly, the engineer should consider the use of a partial cutoff. Owing to the fact that alluvial deposites are stratified and that, therefore, the horizontal permeability may be 10 to 50 times the vertical, the effect of such a partial cutoff in reducing seepage may be much greater than might at first appear probable. Frequently in driving a steel sheet piling cutoff which is intended to reach an impervious layer, it is found impracticable to get all of the piles fully driven and seated in the impervious layers. This may be due to nests of boulders encountered or other causes. The seepage which will occur through such openings in homogeneous, isotropic materials is much greater than one would expect and is somewhat analogous to the flow through a partly closed valve.

(C) **Upstream blanket.** Instead of using a cutoff under a dam on a pervious foundation, an impervious upstream blanket may sometime be advisedly used. The purpose of such a blanket is to increase the length of the path of percolation far seepage under the dam and thus decrease the velocity and quantity of the seepage. In comparing the efficiency of a blanket with a partial cutoff, consideration must always be given to the fact that horizontal permeability is usually much greater than vertical permeability.

In Fig. 17. is shown a dam with a pervious foundation and a relatively impervious natural or artificial upstream blanket. In the case of many dams, there is a netural blanket from which trees and other vegetation should be removed and then defective places patched and the entire surface rolled.

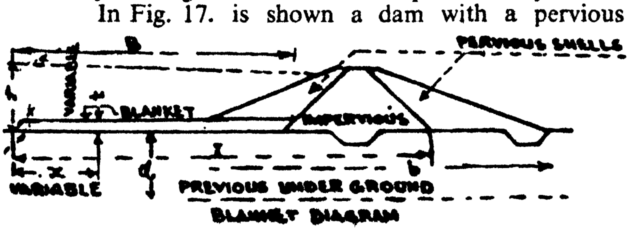


Fig. 17.

The stream channel has frequently been completely eroded down to sand and gravel so that here a complete blanket must be constructed. Even a fully natural blanket should be thoroughly rolled with a sheepfoot roller after removing the vegetation. The reason for this is that otherwise there may be numerous root holes or tubercles through the blanket to the pervious foundation. For usual conditions the upstream impervious blanket should be not less than 5 ft. thick and there seldom any good reason for requiring it to be over 10 ft. thick. The total seepage should not be greater than is economic for the given project. The path of percolation under the blanket and the impervious portion of the dam should never be less than 8 times the gross head and a ratio of 10 or more is desirable. The theoretical blowout or floatation gradient is 1 to 1. However local piping may occur with much flatter over all gradients. Properly graded and weighted filters are necessary to prevent escape gradients approaching 1 to 1 in all such cases, and the higher the seepage the more important are such details. Usually the material available for the blanket is so tight in relation to the pervious underground that it is not necessary to consider flow through the blanket in determining the desired length of the upstream blanket. Fig. 17.

$$l=\frac{khd-pQb}{pQ}$$

in which l is length of impervious upstream blanket in feet,

k is mean horizontal permeability coefficient of the pervious underground,

h is gross head in feet on impervious upstream blanket,

d is depth of pervious underground, in feet,

p is percentage (stated as a decimal) of flow under dam without a blanket to which it is desired to reduce the seepage by means of the blanket,

b is length of impervious portion of base of dam,

Q is flow under dam without a blanket per foot of dam $(khd)/b$.

Example. Assume the following values $k=0{\cdot}12$ ft. per min (medium sand gravel), $h=50'$ $d=80$ ft., $p=0{\cdot}20$, $b=200$ ft.

If there were no blanket at all the discharge under the dam would be $Q=\dfrac{0{\cdot}12\times50\times80}{200}$ $=2{\cdot}40$ cft. per min. per ft. width of dam. Note that if the dam were 1000 ft. long, the seepage under it would be 2400 cft. per min (40 cft. per sec.) which we will assume it is economically desirable to reduce to about 20 percent of this amount. Hence p is assumed as $0{\cdot}20$.

$$l=\frac{0{\cdot}12\times50\times80-0{\cdot}20\times2{\cdot}4\times200}{0{\cdot}20\times2{\cdot}4}=800\text{ ft.}\quad\text{and }\frac{b}{p}=\frac{200}{0{\cdot}20}=1000\text{ ft.}$$

and the seepage under dam and blanket is $khd/(x+b)=0{\cdot}48$ cft. per min per ft, of dam. Thus, on the above basis the desired length of upstream blanket to be added is 800 ft. The ratio of path of percolation to head is $1000/50=20$, which is more than that required.

If there is any doubt that seepage through the blanket might be enough to make it desirable to increase this length of blanket. It is a simple matter to compute approximately this seepage and if large enough to materially affect results increase the thickness of the blanket or extend if further upstream. Thus, in the above example, assume that the relatively impervious blanket is 5 ft. thick and is composed of medium silt which in place after rolling has a vertical permeability of $0{\cdot}00002$ ft. per min. Assume that the average net head on the blanket is $h/2$ (actually it is somewhat less).

Then the discharge through the blanket by the Darcy equation is,

$q_1=\dfrac{k_1h\,(h+b)}{2t}$ in which q_1 is discharge through blanket, k_1 is vertical permeability coefficient of blanket, t is the thickness of the blanket.

The thickness t of the blanket at any point may be expressed by the following equation :—

$$t=\frac{k_2}{k_1}\times x\times\frac{b}{d}$$

wherein, t is thickness of blanket at point under consideration.

k_1 is average permeability of the underground materials.

k_2 is permeability of the blanket.

b is width of blanket from upstream toe to impervious section.

x is distance from point under consideration to upstream toe of the blanket.

d is depth of pervious underground materials.

For practical reasons of construction, t should not be less than $2+\dfrac{x}{100}$ in feet.

Example :—$k_1=300\times10^{-4}$, $k_2=1\times10^{-4}$, $b=500$, $x=400$, $d=50$.

$$\therefore\ t=\frac{1\times10^{-4}}{300\times10^{-4}}\times400\times\frac{500}{50}=\frac{40}{3}=13{\cdot}3\text{ ft}$$

12. Drainage.

W. L. Strange, Chief Engineer, Sind, (Pakistan) had studied the problem and a brief summary of his method is given below :—(Bib. 11).

(a) "Drainage is necessary to remove water which may be met within the heart of the dam, due to percolation from the reservoir or the wetting of the downstream slope by tain, or along the junction with the ground of the base of the dam, or through the strata underlying that base. The last two are chiefly due to leakage from the reservoir.

To prevent the internal percolation from entering into and remaining in excess in the embankment, its upstream portion should be constructed of water tight material and its downstream one of permeable material, with more grit in it to render it self-draining, and both should be well consolidated".

(b) **Surface drainage.** To heal with the leakage along the base of the dam, the surface of the natural soil should be excavated in a series of longitudinal furrows, or 'foundation benches', parallel to the centre line, with the long slope next to the toe of the embankment, and small "foundation trenches" should be dug in their troughs, Fig. 18. On the upstream side the lat'er should be filled with water-tight material so as to make them into small puddle trenches, which will tend to resist the creep of water, to from an impermeable base for the dam, and to aid the main central puddle trench. On the downstream side the trenches should be filled with gravel, porous material, so as to convert them into small 'foundation drains,' which will receive the water creeping along the surface of the foundation and that derived from the percolation through the dam, which will be led uniformly in to them by the slopes of the benches. This drainage should be passed out of the dam by cross drains at intervals. To stop water from lodging downstream of the downstream toe of the embankment and soddening the ground, which might induce a slip, it is advisable to excavate a longitudinal 'surface drain' parallel to and near that toe.

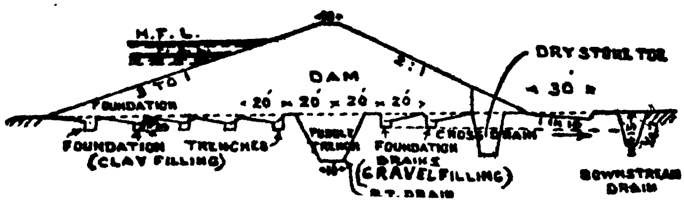


Fig. 18.

(c) **Subsoil drainage.** To cut off leakage through the strata underlying the dam a "puddle trench" is made. Some leakage, is likely however, not to be intercepted by it, and this will tend to soften the puddle and sodden the impermeable strata immediately downstream of the trench. To deal with this flow it is desirable to construct a longitudinal puddle-trench-drain with a continuous fall along the downstream toe of the trench, and to led the water out of the dam by cross drains. Still further to prevent the ground below and downstream of the dam from becoming sodden and incapable of resisting the weight of that heavy mass of earthwork, a 'downstream drain should be formed downstream of the surface drain' and into it should be led the discharge of the foundation, surface, and puddle-trench-drains. This should be passed out of it by cross drains, where practicable or into the river bed. These drainage arrangements are shown in Fig.18. and an alternative method of efficient drainage is given in Fig. 19. by having a complete rock-stone toe and heel.

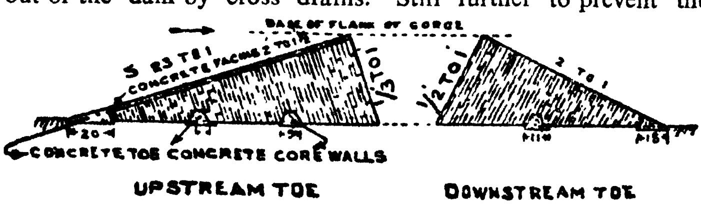


Fig. 19·

(d) **Drains.** All drains should be in discontinuous length terminating at the cross drains, and separated from each other by short lengths of undisturbed soil, so as to obviate long, continuous drainage lines under or near the dam. All sub-soil drains should be constructed on the principle of the inverted filter, *i. e.*, surrounded by fine porous material to prevent them from choking As long as the drains run clear and their discharge does not increase in the fair weather, it shows that damage is not being done to the embankment. Should, however, the flow become discoloured, it is a sign that a leak is forming, at this should be dealt with atonce by cutting out the unsound part of the dam and remaking it. A proof of the efficiency of the drainage arrangements will be afforded by the dry condition or the ground downstream If, however, swampy places exist there, they will show that the drains are not working properly, and the defective length should be put right. With good drainage a well-designed and well-constructed earthen dam is rendered quite safe, as there by the line of saturation is lowered and the foundation are secured and slips are thus prevented. Undrained clay dams are liable to failure due to percolation through them.

(*e*) **Filter layers and filter toe.** Filter layer is provided as shown in Fig. 20. instead of massive toes as in Fig. 20. The massive toes usually get choked in course of time. In hamogenous damp downstream face does become wet and sometimes slushy when the dam is founded on impervious foundations. The provision of a filter depresses seepage line. Typical design of a filter toe is sketched in Fig. 20. This is a very effective type of filter toe which serves to collect the seepage flow through the dam, as will as serves to control the floatation of the bed material in case of excessive exit gradiant of flow.

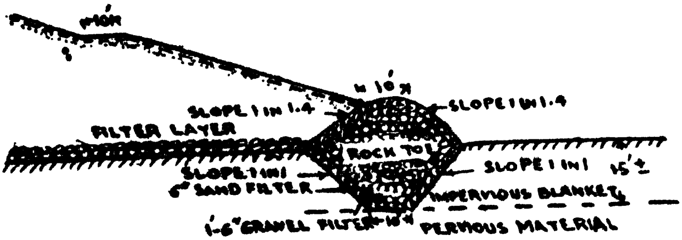


Fig. 20 Typical rocktoe with filter protection

13. The Stability of Earth Dams.

(A) Earth dams usually fail by development of excessive stresses in the material of the dam due to the load of the dam itself, water pressure and internal hychostitic or fore pressures. The dam embankments should be so proportioned so that the strength of material safely exceeds the stresses induced with ample factor of safety. The strength of an earthen material is a function of cohesion (shear stress with zero vertical load) and its internal friction depending on the observed values of the angle of internal friction. These should be determined for the undisturbed foundation material and for the prepared sample of proposed compacted embankment of the dam. Cohesion varies with the moisture content and the condition of placement but the apparent angle of internal friction varies directly with the applied load.

The formula for the shear strength of a material is given below :—

$$S_s = C \times P \tan \phi \qquad (1)$$

where S_s is shearing strength. C is Cohesion. P is Intensity of load. ϕ is angle of internal friction of material (under given conditions of loading)

(B) Shear calculation for the dams founded on impervious foundations are usually made under the following conditions :—

(*a*) Shear forces taking the dam as a whole.
(*b*) Horizontal shear in the downstream half of the dam.
(*c*) Horizontal shear in the upstream half against sudden draw-down.

In a general case of sudden draw-down the upstream portion of the dam would still be saturated on its completion and accordingly, one should utilize the saturated weight of the material for all material below the maximum seepage line when computing the shear. On the other hand, when it comes to computing the forces resisting shear, the unit weight utilized for all materials below the maximum seepage line will be the submerged (or buoyant) unit weight of the material except that clean rock and clean coarse gravell may be taken at its dry or moist unit weight both in computing shearing forces and in computing resisting forces. The position of the maximum unit shear stress may, without serious error, be taken at a point 40 percent of the horizontal distance from the top of slope to the point where the slope intersects the horizontal plan or base under consideration. If the material of the dam on the upstream part of the dam is relatively clear rock and gravel, it may be assumed that it will drain just as quickly as the reservoir can be drawn. The provision of loose stone protection on the upstream face about 3·0 ft. deep would not only make an earthen dam safe against drawdown but also ensure safety against wave action.

(C) Unit weights of material of the dam are calculated according to the conditions of the water content in the soil with the dam full.

(*i*) Dry unit weight as determined by porosity observations.

(*ii*) Saturated soil unit weight capillary portion after the seepage line in the body of the dam, pores filled with water.

(*iii*) Submerged a buoyant weight. Portion below the seepage line up to the base of the dam.

(*iv*) Equivalent liquid weight of soil. It can be calculated from the Rankins formula or taken from the graph in Fig. 21.

$$W_1 = w \tan^2 (45° - \tfrac{1}{2}\phi)$$

(D) The factor of safety should be 2 or otherwise the slopes should be flattened and the calculations repeated. The shear force resisting the horizontal thrust due to water pressure at any section of the dam is calculated on the basis of uniform shear resistance force developed in the whole length of the base, but in actual stresses up to about double the uniform shear stress intensity assumed in the calculation are likely to be developed. The position of the maximum unit shear is empirical as far as these calculations are concerned, but it is in substantial accord with photoelastic determinations and with other methods of calculation. The assumption that the maximum unit shear is twice the average is in accord merely with the simple elastic theory with triangular loading. Photoelastic studies indicate that this is a conservative assumption, as in such model studies the ratio.

$$\frac{\text{maximum unite shear}}{\text{average unit hear}}$$

is frequently 1 : 4.

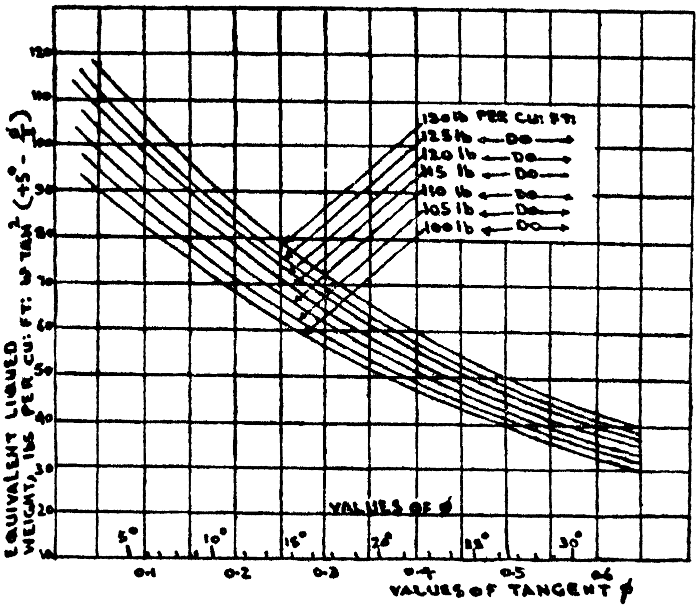


Fig 21 Curves of equivalent liquid weight for various angles of internal friction (φ) (from Rankin's formula. equivalent liguidweight $= w \tan^2 (45° - 1/2\ \phi)$ where w is weight of soil in lb per cft.

The foregoing rough method of analysing stability of slopes of earth dams is claimed to be simple and quick and to give results which agree quite well with results obtained by other suitable methods. It may be applied to any horizontal plane through the dam.

14. Stability of Rook Fill Dam.

In the design of the hydraulic or semi-hydraulic fill, the assumption is made that the core is a heavy liquid. It follows that the shell should be designed for stability subject to this pressure. Economy in design requires that the core be as narrow as possible. A recommended thickness of core at any elevation is equal to the height of dam above that elevation.

Prof. Gilboy has developed a method of calculating the stability of earth dams with liquid cores. Fig. 22 is a diagram showing this method. (Bib 13).

a is cotangent of angle of core slope with horizontal.
b is cotangent of angle of internal friction of shell material.
c is cotangent of angle of outer slope with horizontal
r is ratio of unit weight of core to unit weight of shell.

For a section on the point of failure ;

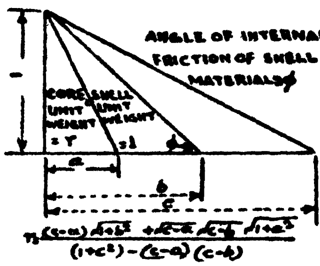


Fig. 22. Diagram illustrating Gilboy's solution for hydraulic-fill dams.

$$\sqrt{r}=\frac{(c-a)\sqrt{1+b^2}+\sqrt{c-a}\sqrt{a-b}\sqrt{1+a^2}}{(1+c^2)-(c-a)(c-b)}$$

Example : – Core slope $\frac{1}{2}$ to 1, $a=0{\cdot}5$, $b=\cot\phi=1{\cdot}2$, $r=1$ then $c=2{\cdot}25$.

That is, the required outer slope is $2\frac{1}{4}$ to 1. It should be noted that the slopes are independent of the height. That is, in an hydraulic fill dam there is no justification for flattening the slopes at the lower elevations unless required for foundation stability The solution given apparently has no excess factor of safety ; actually this is only true at the instant of placing of any section of the core, for consolidation tends to increase the internal strength and decrease the equivalent liquid pressure acting against the shell. In the example, a rather high angle of internal friction has been selected ; probably the value should be somewhat less in most cases.

Where the hydraulic fill is constructed of well-graded material with the shell section of coarse sand and gravel and the core width maintained nearly equal to the depth below the crown, outer slopes of 1 in $2\frac{1}{2}$ to 1 in 3 are recommended. With finer shell material or with a wider core the outer slopes should be flattened.

The same design principles apply with semihydraulic fill dams provided that a narrow core is maintained during consrtuction and the material is essentially cohesionless. Where the outer material in a semihydraulic fill is not water sorted, care must be taken to provide drains through this shell for the more pervious washed material between it and the core. Frequent construction failures in semihydraulic fill dams have resulted from wide pools and lack of drainage.

Factor of safety with Gilboy formula. In order that the Gilboy method may give directly a factor of safety against failure, an additional ratio will be introduced.

r_1=ratio of equvialent unit liquid weight of core material to the effective unit weight of shell mateaial. $r_1=\dfrac{W_1\tan^2(45°-\phi\ 1/2)}{W}$ in which

W_1 is unit weight of core material in actual wet or moist condition,

ϕ is angle of internal friction of core,

W is effective unit weight of shell material,

r_1 is ratio as defined by Gilboy,

$F_3=r/r_1$, in which F_3 is factor of safety of the shells against shear from the internal pressure of the core.

15. Foundations.

(A) The foundations under an embankment must be carefully studied when borings indicate the presence of a silt or clay-type material. Where tests indicate an angle of internal friction of less then 30 percent in the foundation material, a careful study of the foundations should be made to determine their stability.

Jurgenson (Bib. 15) derived a very simple formula for obtanining the approximate shear stress in a plastic layer in the foundation of an earth dam of triangular cross-section.

$$s=(Pa)/L$$

in which s is maximum unit shear,

p is the maximum unit pressure on the foundation at the plastic layer,

a is 1/2 the thickness of the plastic stratum,

L is 1/2 the base width of the structure.

The formula has been used a great deal because of its simplicity. It is directly applicable, of course only to the conditions which Jurgenson assumed.

Although the formula may in some cases give results which are quite for from the truth, it is believed to be frequently suitable for use in preliminary analyses. In the author's opinion the Jurgenson formula should not be applied to thick plastic layers and certainly not to layers thicker than 1/2L. For a through and final analysis of plastic foundation photoelastic methods are desirable.

(B) Rankins assumption that an earthen meterial has an equivalent liquid unit weight which would produce the same shear stress as the material itself, affords a convenient through approximate method of shear stress calculations in the plastic fouudations.

According to Rankin's hypothesis (Bib. 7), to total horizontal shear stress in the foundation can be written in the form

$$S = \frac{h_1^2 - h_2^2}{2} w \tan^2 (45^\circ - \tfrac{1}{2}\phi)$$

where S = total horizontal shear down to rigid boundary,

h_1 = vertical distance from top of dam down to the 'rigid boundary', such as ledge rock or sand gravel stratum, the strength of which is great as compared with that of the overlying materials,

h_2 = vertical distance from bases of dam (or original ground surface), down to the 'rigid boundary',

b = horizontal distance along base from top shoulder of slope to the toe of the dam,

w = effective weight per cubic foot of the material in its actual condition.

It is here assumed that unit weight of dam and foundation material are the same ; if they are different, use a mean (weighted in proportion to depth of each).

ϕ_1 = equivalent angle of internal friction determined as follows : –

$$\tan \phi_1 = \frac{c + wh_1 \tan \phi}{wh_1}$$

in which c is the determined cohesion or no load shear in per equare feet and ϕ is angle of internal friction as determined by test. Having determined $\tan \phi_1$, the value of ϕ_1 in degrees is readily determined from Fig. 19 or from any table of natural tangents.

As S above is the total horizontal foundation shear, the average unit shear is

$$S_a = \frac{S}{b}$$

in which S_a = average horizontal foundation shear per square foot,

S = total horizontal shear per square foot.

b = horizontal distance along base from top shoulder of slope to the toe of the dam.

The maximum unit shear may be found from the following relationship, which has been substantially checked by photoelastic analysis :—

$$S_{max} = 1{\cdot}4\, S_a$$

in which S_{max} is maximum shear per square foot in the foundation.

In locating the horizontal portion of the maximum unit shear, its location may be taken at a point 0·4b from the upper shoulder of the slope Photoelastic model studies indicate that this is a fair approximation for its position.

(C) **Elastic theory for determining shear in foundation.**

The elastic theory has been utilized by Terzaghi (Bib. 14) Jurgenson, (Bib. 15) Timoshenko, Tuissul, (Bib. 16) and other for the solution of problems involving stress in foundations. With the elastic theory there is a fairly definite relationship between stress and strain (movement), and when a soil is in the elastic state, the stress continues to increas with increasing strain. On the other hand, when a soil is in a plastic state shearing stresses have reached the shearing strength and the strain (movement) increases with the stress remaining constant.

In Fig. 23 is given the distribution of shearing stresses in a foundation under slope of an embankment. Jurgenson assumed homogeneous isotropic material of infinite and the maximum load of infinite depth in making the computations for this figure. The maximum load per square foot here extends for an indefinite distance, where as with an earth dam the maximum load per squre foot extends for a relatively short distance before it reaches the top of the opposise slope. The reason for using the so called terrace loading is that this loading gives more nearly the true horizontal position of the maximum unit shear stress than the triangular loading does. Thus for triangular loading maximum unit shear is at dam centre line, whereas for terrace loading is it at the midpoint of the slope (=1/2 horizontal distance shoulder to toe). Photoelastic test give a similar position (0 4 to 0·55). Thus it is believed that conditions in Fig. 23 are quite analogous to the conditions in the foundations of many earth dams.

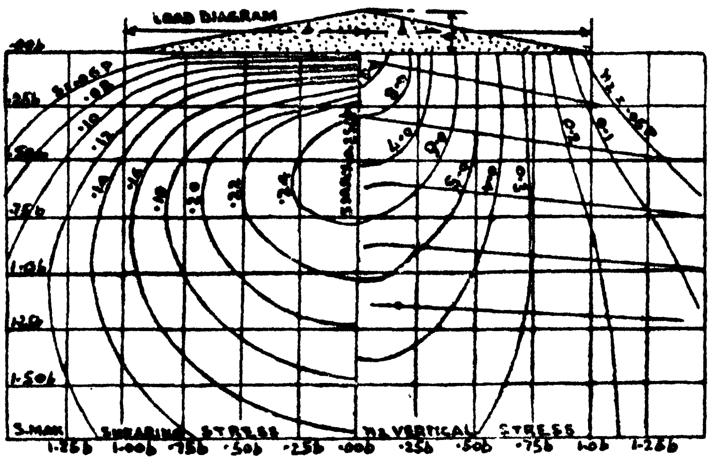


Fig. 23 a.

S_{max}-shearing stress

N_z-vertical stress

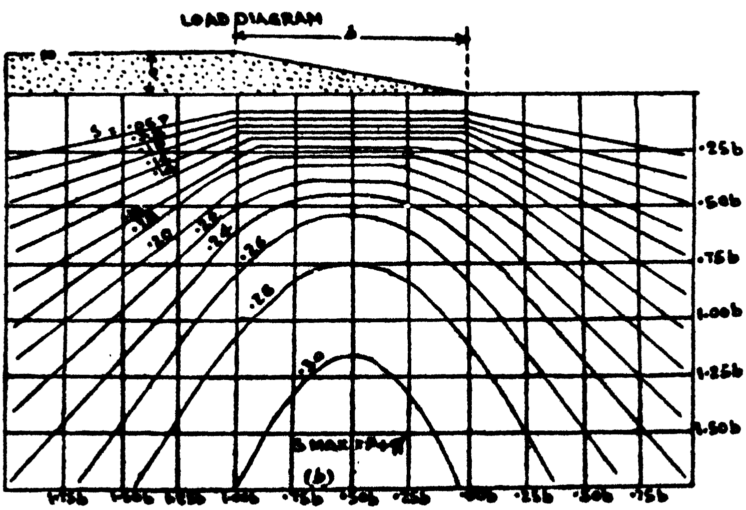


Fig. 23 b.

Distribution maximum shearing stresses in foundation of homogeneous isotrophic material of infinits extent.

17. Slip Circle Method of Testing Earthen Embankments.

(A) Swedish National Commission published a report in 1922 based on Mr. K E. Patterson studies that the line of failure of slider of earth dams roughly approched the circumference of a circle. The failure circle might pass above the tone, through it, or below it. This is generally accepted as approximately correct solution for determination of the factor of the safety of slopes of embankments and their foundations. Big investigating the strength along the arc of a large number of slip circles, it is possible to locate the circle which gives the lowest resistance to shear. The developments of the method by A.C. Gourtoney simplifies the method of finding the dangerous circle.

(B) The original method, as developed by Swedish Engineers consists in passing a circle through the slope and foundation, taking moments of all the forces about the centre acting along the circumference of the circle. As all the force have the radius of the circle for their lever arm, the radius cancels out of the computations. Consequently to get the factor of safety, all that in necessary is to add up all the resisting forces along the circumference of the circle and divide this by the sum of all the forces tending to produce movement along the circumference of the circle.

With the assumed slope of the upstream and downstream face of an earth dam, the values of α and β of the first trial are taken from Fig. 24. α is laid from the toe of the slope and β from the horizontal at the top of the slope as shown in Fig. 25, intersecting in some point O. With O as centre, inscribe a circle having AO as radius, cutting the embankment lines. The area intercepted between the circumference of the circle and the lines of the embankment is then divided up into a number of vertical slices as is indicated in Fig. 25. The vertical slices are preferably of equal width. The number of slices used should not be less than five and it is seldom necessary to use more then twelve. The area and effective weight of each of the vertical slices is determined, and a vertical line proportional to the total effective weight of the slices drawn from its centre of gravity. At the circumference this weight is resolved into its normal and tangential components. This process is followed for each of the vertical slices.

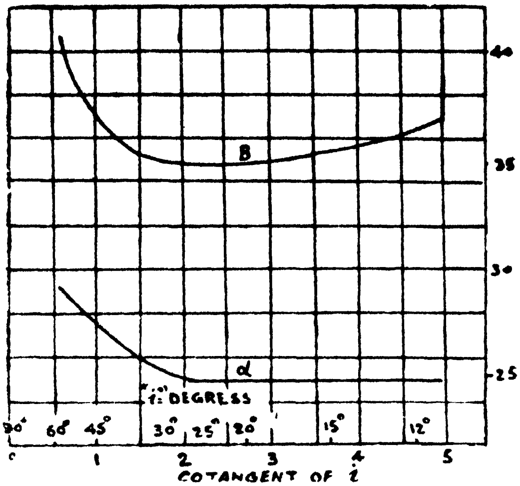


Fig 24. Angles α and β for different slopes to determine centre of most dangerous circle which passes through toe.

The next step is to add up all the tangential forces or T forces for the slices (with proper regard to sign). Also add up all the normal forces.

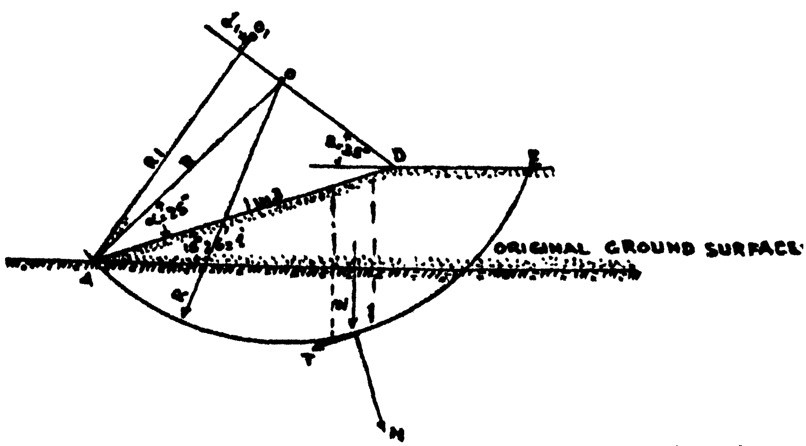


Fig. 25. Method of analysis when dangerous circle passes through toe.

ΣT=force tending to produce movement along the circumference (shear).

The force tending to prevent movement is $\Sigma N \tan \phi + Lc$ which is total shear strength of the material along the arc of the circumference AE. In the above

ΣT is summation of tangential forces tending to pruduce movement,
ΣN is summation of the normal forces for all of the slices,
ϕ is angle of internal friction of the material as determined by tests,
L is length of arc AE intersecting the embankment,
c is cohesion per square foot.

F_s is the factor of safety against shear failure along the arc of circle under investigation.

$$F_s = \frac{\Sigma N \tan \phi + Lc}{\Sigma T}$$

If a satisfactory factor of safety is shown it means merely that shear failure need not be expected along the assumed circle, but it may occur elsewhere along some other circle unless the one under investigation is already known to be the most dangerous. Consequently a number of circles must usually be drawn and analized

For the centre of additional trial circles through the toe of slope, proceed as follows. Back up the line OD, Fig. 25, and then up and away from it in a perpendicular direction in such a way that od_1 is not greater than $\frac{1}{2}OD$ and $o_1 d_1$ is approximately equal to $\frac{1}{3}\ od_1$. O_1 will then be the centre of a new trial dangerous circle with radius R_1.

This new circle is analized as before and its factor of safety determined. The procedure is continued until the circle is found which gives the lowest factor of safety.

(C) If the foundations and the dam material are homogenous, the earth dam may have the most dangerous slip circle passing below the toe. Fellenius found that the angle intersected in Fig. 26, at O is $133\frac{1}{2}°$ for a circle passing below the toe. If the slope of the face of the dam is as shown in Fig 26, erect a vertical at the mid-point of the slope. On this vertical will lie the centre O of the first trial dangerous circle. In locating trial circle use an angle of $133\frac{1}{2}°$ between the two radii at which the circle intersects the surface of the embankment and foundation. After the first trial dangerous circle has been drawn, the method of analysis is the same as described above in (B). After the dangerous circle has been analised the trial centre is moved somewhat to the left, the radius shortened and new trial circle drawn and analised. Finally after a number of circles, have been analised, the computed failures of factor of safety are tabulated and composed. The circle showing the lowest factor of safety will be the most dangerous circle.

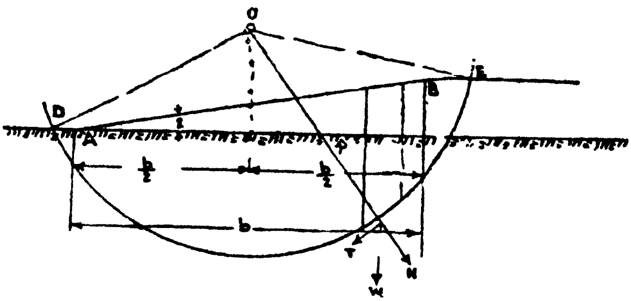


Fig. 26. Method of analysis when dangerous circle passes below toe

(D) (*i*) N. C. Courtney has developed an abbreviated method of graphical solution which requires only a small part of the time otherwise required for analysis by the method of slices and gives results just as precise.

Fig. 27. shows a section through the downstream slope of a dam and illustrates the application of this method. A dangerous circle has been drawn cutting the slope as indicated. Any vertical line from the slope of the dam to the dangerous circle represents the weight (W) of a strip infinitely small in width. The components N & T of one of these vertical lines represent the resolution of the weight W into forces normal and tangential to the dangerous circle. If we now plot these N & T components from a horizontal base for various points through out the section and join there extrimities we obtain curves, the

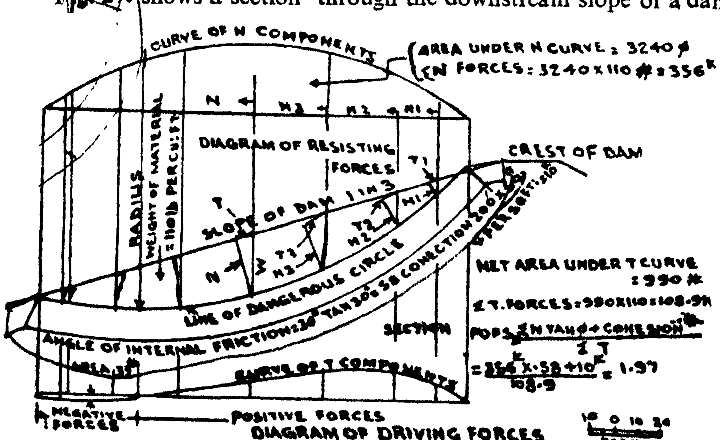


Fig. 27. Dangerous circle analysis abbreviated method

areas under which represent respectively the summation of all N and T components.

These areas, determined by planimetering or otherwise, multiplied by the effective unit weight of material give the total N and T forces acting on the particular circle The summation of the N forces when muitiplied by the tangent of the angle of friction gives the total resisting frictional force along the arc. Any cohesive value of the material along the length of the arc of the dangerous circle must be added to the friction value determined from the N forces to obtain the total force resisting shear along the arc. The summation of the T forces gives the total driving force tending to cause the material to shear along the arc.

The factcr of safety for any given dangerous circle is obtained, as in the slice method, from the formula $F_s = \frac{\Sigma N \tan \phi + Lc}{\Sigma T}$ in which F_s=factor of safety against shear along the arc of given circle,

ΣN (summation of normal forces)

—area under the N curves multiped by the unit effective weight of the material,

$\tan \phi$ - tangent of angle of internal friction of the material which the circle cuts,

L – length of arc of circle,

c - unit cohesion as pounds per square foot of the material cut by the circle,

ΣT (summation of tangential forces)

—area under T curve times unit effective weight of material.

This method is readily adapted to any complicated case, as the N and T curves can easily be plotted for various matrials, slopes and conditions. The application of the method to a simple case is thus shown in Fig. 27.

(D) (*ii*) When sudden drawdown takes place this pressure is removed the drawdown level. The shell material is ordinarily submerged below pond level, but immediately after drawdown the shell material is either moist, owing to being able to drain as rapidly as the pond level goes down, or it is saturated, owing to the inability of the shell to drain freely.

When determining the stability of an upstream dam slope under full head water conditions the resisting and driving forces acting on any dangerous circle are calculated on the basis of all the materials being submerged except those above the normal pond level.

When determining the stability of an upstream dam slope against drawdown, we must determine if the shell material is or is not sufficiently free draining. If the shell matarial is clean rock or coarse gravel and, therefore, will drain as fast as the pond can go down, then the resisting and driving forces within the drawdown range are calculated for the dry or moist weight of the rock material. If the shell material within the drawdown range will not drain as rapidly as the pond can be drawndown, the resisting forces are calculated for the submerged weight of the material below water surface and the dry weight above water surface but the driving forces are calculated for the saturated weight of the shell material below water surface plus dry weight above water surface. All marerials below the drawdown level are calculated on the basis of the submerged weight of the materials. The method of analysis is the same as described in D (*i*) above using the weight as calculated above.

(E) **Taylor's stability numbers.** If the slope angle, height of fill, effective weight of material per cubic foot, angle of internal friction, and unit cohesion are known, the factor of safety may be determined. In order to make unnecessary the more or less tedious stability determinations. Taylor conceived the idea of determining the stability of a large number of slopes througout a wide range of slope angles and angles of internal friction and representing the result by an abstract number which he calls "the stability number."

$m=c/(FwH)$, where m Taylor's stability number.

c cohesion in pounds per square foot,

F factor of safety,

w effective weight of material in pounds per cubic foot,

H height of slope in feet.

$$\therefore F=c/(mwH)$$

In order to illustrate the use which this equatian may be put to help to solve the problems of design, we will assume the following. The factor of safety of the downstreom slope is desired.

Slope, 1 in $2\frac{1}{2}$,
ϕ, angle of internal friction, 15°
c, conhesion, 750 lb. per sq ft.,
w, effective weight per cubic foot of material, 120 lb.
$H=140$ ft.
For the slope of 1 in $2\frac{1}{2}$ and $\phi=15°$, the Taylor's stability number is 0·028 (Bib. 17).

$$\therefore F=\frac{750}{0{\cdot}028\times 120\times 140}=1{\cdot}6$$

Which is a satisfactory factor of safety under most conditions. If the effective unit weight of the material had been only 60 lb. per cubic foot, as might be the case with a fully saturated downstream slope, it should be noted that the factor of safety would have been only 0·8 in the above case.

18. Construction of an Earth Dam.

(A) **Preparation of the dam site.**

(*a*) The whole base of the dam shall be excavated at least 6″ deep and as much deeper as may be directed in order to remove all grass, roots and vegetable matter, (and pervious soil if directed).

(*b*) All tree stumps and roots to be removed and the voids thus produced to be filled in with well-rammed material.

(*c*) All field drains crossing the site of the dam to be traced out, completely excavated and to be refilled with puddled clay.

(*d*) The base of the completed excavation to be hollowed in a direction parallel to the length of the dam, so as to promote a satisfactory union between the natural earth and the embankment.

(B) **Construction of earthwork.**

(*a*) The embankment to be formed in regular layers not exceeding 9 inches in depth at the heart of the embankment, and 18 inches at the outer parts and slopes when spread out. The layers are to slope from the outer sides down to the centre at an inclination of 1 in 12.

(*b*) The earth for embanking may be obtained from the excavation for the puddle trench, waste weir channel, etc., so for as it is available, provided that the material is in accordance with the specification. The remainder of the necessary earth must be taken from excavations (from inside the reservoir).

(*c*) There is to be no excavation within 100 feet of the toe of the embankment.

(*d*) The most clayey portion of the material is to be used for the heart of the embankment, and more especially on the water side of the puddle wall. The more stony, gravelled, and sandy portion is to be used in forming the outer parts and slopes, especially for the outer side of embankment.

(*e*) No grass, peat, moss, mud or vegetable matter is to be put into the (heart of the) embankment.

(*f*) Every layer must be carefully spread out to the proper thickness, and is to be rolled with a heavy grooved roller, at least 2 tons in weight, till quite consolidated, optimum moisture in the being added during the process. The perecentage of optimum moisture should be determined in the laboratory.

(C) **Coustruction of puddle wall or trench.**

(*a*) The puddle to be made from clay suitable for tenacity and low permeability.

(*b*) All stones to be removed and the clay to be left exposed in layers not more than 12 inches thick for at least 24 hours and to be watered once or a number of times as dircted.

(*c*) The scoured clay to be passed through an approved pug mill or to be otherwise reduced to a homogeneous mass.

(*d*) The broken clay to be deposited in layers and watered as directed and to be allowed to weather for at least a week.

(*e*) The puddled clay to be deposited in the trench or wall in layers not exceeding three inches in thickness and to be well cut up by an appropriate tool at least 6 inches long, so as to be incorporated with the lower layer.

(*f*) All clay surfaces in the puddle wall or trench to be coverd with bags, or to be otherwise protected against drying and the falling materials. The old surfaces to be well heeled over or to be otherwise cut up before new puddle is deposited. All puddle which has become dry, cracked or muddy or mixed with impurities, to be replaced by approved puddle.

(*g*) The filling of the puddle trench to commence at the deepest point and to be carried on right and left continuously but no portion of the trench bottom to be covered up until it has been inspected and passed by the engineer-in-charge.

(*h*) The filling of the puddle wall to be carried up simultaneously with the construction of the bank, and between properly supported timbers. The top of the puddle clay to be at least 3 to 6 inches above the earthwork.

(*i*) All timber stringers and wallings to be removed from the puddle trench, or wall, as the clay is deposited.

(D) **Junction of earthwork.**

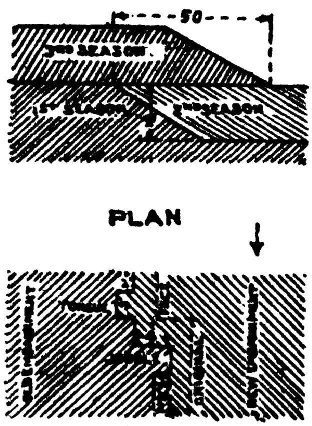


Fig. 28.

Junction of new to old earthwork should be avoided as much as possible, as the rates of settlement of the two will differ until practically final consolidation is attained, and until then there is a risk of separation taking place between them.

When such junctions are unavoidable, cross sectional ones should be made as illustrated in Fig. 28, and logitudinal ones as in Fig. 29, the former have to resist the tendency to the formation of leaks and the latter of slips.

(E) **Closure of a dam.**

On account of the magnitude of floods in the tropics it is not practicable to divert them outside the dam site during its construction by by-pass channels, and they have, therefore, to be passed through a gap left for the purpose. The closure of a high dam is thus a difficult work: as it ought, if possible, to be effected in a single season and well before the commencement of the monsoons. Originally it was effected as shown in Fig. 29, but there are many objections to this, and the closure should perferably be carried out as sketched in Fig. 31. In this latter arrangement the dam is raised in continuous level stages, commencing with the one at its lowest base at *A*, and working gradually upto that forming its final crest *B*. To provide for the passage of flood water a temporary waste weir is made at a suitable site in the longitudinal section so that it can discharge the maximum flood with safety.

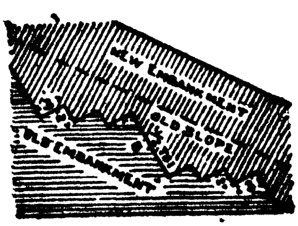


Fig 29.

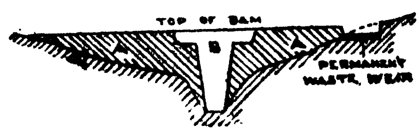


Fig. 30

Fig. 31.

19. Construction of Hydraulic Fill Dams.

(A) **General Remarks.** Hydraulic fill and semi-hydraulic fill dams must fulfill the same criteria of design as earth dams, which are built by depositing material in layers and rolling, but, by reason of the methods of construction used, they offer an entirely different construction problem.

The following definitions for hydraulic fill and semi-hydraulic fill dams are generally accepted.

Hydraulic fill dam. An earth dam in the construction of which the materials are transported on to the dam by water and distributed to their final position in the dam by water is a hydraulic fill dam.

Semi-hydraulic fill dam. An earth dam in the construction of which the materials are transported on to the dam and dumped within the section of the dam by some other means than water, but some of this material is moved to its final position in the dam by the action of water is a semi-hydraulic fill dam.

The semi-hydraulic fill method of construction has sometimes been the cheapest and most convenient for use at a given site. There are however, certain dangers generally inherent in this method of construction which should be appreciated and guarded against if this method is adopted. Dams built by the hydraulic fill method have been comparatively free from slides during or immediately after construction, whereas several dams constructed by semi-hydraulic fill method have had slides during construction.

A foundamental difference in stability during construction thus seems to be indicated. The hydraulic fill method deposits the material from flumes or pipes near the faces of the dam; the larger particles stay there and the finer ones move toward the centre, the finest of all going into the central pool and being deposited there. Thus the toe and faces of a dam produced by this method are more pervious, allowing water to drain out from the interior of the dam. Even if most of the drainage from the cores is by vertical crater action, as some engineers maintain such action takes place not only in the portions of the core underlying the central pool but also in those portions of the core covered by the pervious outer sections of the dam. Hence, whether the main drainage of the core is upward, downward, or sidewise, the importance of pervious outer sections is just as great.

In dams built by the semi-hydraulic fill method, the toes and faces usually have consisted of car dump fills. Material is washed away from these fills by jets of water from giants. The finer material goes into a central pool and is deposited, forming the core; the coarser particles are dropped near the car dump fill. In consequence of this action, the car dump fill at the face is often more dence and impervious than the material immediately adjoining it on the inside of the dam, for this latter material has had the fines washed out of it by the action of the monitors.

In some cases tests have shown that the material in the car-dumped fills is actually more dence and impervious than that immediately adjoining. Through the presence of the central pool and the sluicing operation, this comparatively pervious area is kept full of water which exerts considerable hydrostatic pressure on the relatively impervious car fill material at the face. Thus the car fills may from an element of weakness and by imprisoning the water may sometimes be the cause of slides even through the central core of fine material is so stable that it exerts no hydrostatic pressure.

(B) **Construction of hydraulic fill dams.** The hydraulic fill is best suited to sandy and gravelly materials such as glacial deposits where the whole material does not contain sufficient fines for relative impermeability, or where a well-graded material from gravel or sand to silt or clay is available but can be placed cheaper by the hydraulic method. Materials containing high percentages of impervious material are not suitable for hydraulic fills.

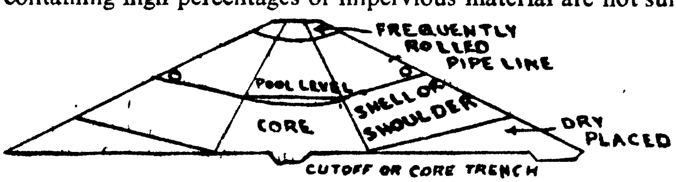


Fig. 32. Hydraulic-fill construction

The method of constructing hydraulic fills is illustrated in Fig. 32. In modern practice, the base is prepared by rolled-fill methods so that the impervious upstream blanket, where required, and the pervious downstream drainage section together with pervious shoulders up and downstream drainage toward the proposed pool, are first built as illustrated in Fig 32. The pipe line or sluiceway is carried along the shoulders near the outer slopes where the material is discharged to that the cour se material will settle at the outside and the fines will be carried into the pool. The width of the core is usually equal to the height of dam above the horizontal section under consideration. Where there is a large amount of clay in the material or where there is an aboundant

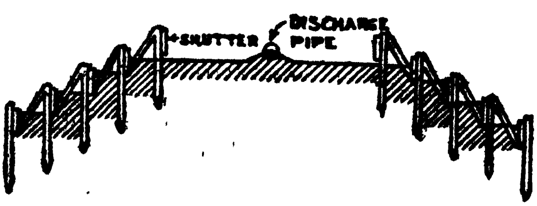


Fig. 33. Hydraulic fill construction

supply of water, the pool water is wasted through a spillway channel or pipe. When the supply of water is limited, the pool is reused. The material is sometimes excavated and transported hydraulically through pipes Fig. 32, or flumes and sometimes excavated and hauled by dry methods to a long box where it is mixed with water and pumped or allowed to flow by gravity through the distributing system.

(C) **Construction of semi-hydraulic fill dams.** In this type, the material is dumped in the outer sections and washed into the core. Although extensively used, this method has resulted in many construction failures. The dumped material is frequently more impervious than the washed material between it and the pool, which may result in the saturation and failure of the dumped shell. This type of construction is best suited to the case where a uniform gravelly material deficient in fines is the only material available. This method can be made safe by adequate underdrainage of the washed shell or by the use of rock shells, brought up with the fill, permitting all the other material to be washed into place. A frequent cause of trouble with this type is the maintenance of an excessively wide pool.

20. Observations and Studies to Watch Safety of Dams After Construction.

Failures in earth embankments or foundations are proceeded usually by movements of such magnitude as to indicate clearly the probability of failure. In cases of foundation failure a rapid increase in foundation settlement is evident when overstress takes place initially, and somewhat later the ground near the toe of the embankment commences to rise in the form of a mud wave. By the use of plates set on the foundation soil, the rate of foundation consolidation may be measured by bringing pipe sections up vertically with the fill. When this rate suddenly increases, danger is indicated. Likewise, hubs set on the ground surface at varying distances from the toes tell when failure is approaching, as they start to rise vertically. Embankment failures are indicated in advance by bulging of the slopes, evidenced by horizontal movement of hubs set in the slopes. Where there is any question as to the safety of the design, by observing the action of the embankment and foundation, failure can be prevented by ceasing the working of raising the embankment as soon as the first symptoms appear, adding berms or flattening the slopes to stabilize the tendency toward failure. Particular vigilance muse be maintained as the fill nears completion. At such times, the slightest movement must be observed and corrective measures undertaken despite the temptation to finish the job and take a chance on stabilization taking place. The chances are that failure will occur in such a case although it may take several days or weeks or monthes to develop. Immediate action will permit the utilization of this period to place the necessary reinforcement and prevent failure.

21. Waste Weirs.

(A) The object of the waste weir is to provide a passage for the discharge of floods so that the water in the reservoir may never rise higher than a safe height below the top of the dam. The *desiderata* for a waste weir are a sufficient length, a flat longitudinal and a gently sloping crossection, hard strata, the proper elevation, and a good outfall. The length is necessary to get discharging capacity; the flat sections and hard strata to diminish the cost of the works, and to obtain security for them ; the elevation of the weir crest depends upon considerations of storage ; and the good outfall is wanted so as to lessen damage by floods. The best site is generally on a long, somewhat narrow level ridge of hard material, with a gentle slope d/s.

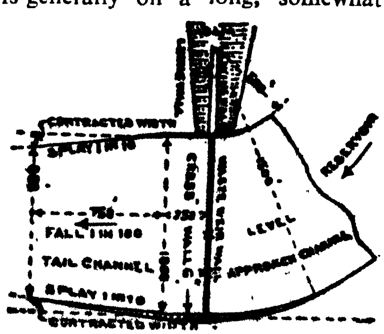


Fig. 34

(B) Waste weirs may be classified as below :—

(*i*) Flank weirs, at the immediate flank and in continuation of the dam embankment.

(*ii*) Saddle weirs, separated from the dam by high ground.

Flank weirs are the less safe form, as the discharge from them may outflank and injure the dam. To prevent this a wing wall is required on the upstream side, and a lining wall on the downstream side of the embankment Fig. 34.

Saddle weirs discharge clear away from the dam, which therefore, does not need protection, but to obviate the out-flanking of the weir itself, masonry flank on one or both sides of it may be necessary.

(C) A suitable design for a waste weir would be one with clear overfall. Drowned waste weirs and those with drowned channels should be avoided. A clear over fall weir shall need rocky foundations. If not available, the foundations should be deepened to the firm ground and depth increased. The weir should be provided with a capacious and unobstructed approach channel upstream.

(D) Discharge capacity of a waste weir is fixed from the following considerations :—

(*i*) It is essential that the waste weir should be able to discharge safely the maximum flood possible, and as it acts as a safety valve of the reservoir, that discharge must not be under-estimated.

(*ii*) Where flood observations have been recorded for twenty five years, an allowance of 10 percent over the highest known flood should be made to provide for an abnormal flood, unless such a flood has occurred in the period observed. Where the record is shorter, the allowance should be greater.

(*iii*) When a reservoir is downstream of another which may breach or is liable to be visited by a cyclone, extra flood provision should be made for the abnormal discharge which may thus reach the work. That provision may consist in lengthening the waste weir, or in forming high level safety flood-cuts in addition.

(*iv*) The discharging power of of a wasting weir depends upon the unobstructed normal width of its approach channel just downstream of its crest.

22. Tower and Outlet.

Principle types of outlets as used in practice are :—

(*i*) Culvert under the dam. (*ii*) Head wall in the centre line of the dam. (*iii*) The outlet Tower (or 'Dry-well') in the centre line of the dam. (*iv*) Tunnel round the, dam. (*v*) Head wall accross an open channel outside the dam. (*vi*) Siphon over the dam and (*vii*) Pipe outlet.

Of these Nos. (*i*) and (*ii*) are generally built in India. No. (*iii*) is practicable only for dam with masonry core wall ; it tends to admit water into the heart of the dam. No. (*iv*) is the safest form but is generally expensive ; it is quite independent of the dam, a condition which some engineers consider to be essential for safety. No. (*v*) combines the advantages of No. (*ii*) and (*iv*) but is usually costly. No. (*vi*) can be used only for small discharges On account of its liability to failure No. (*vii*) should never be used except for a very small and unimportant work. The pipes are apt to break or draw under the weight of the dam, and water may leak along them, or through their joints. It is also impossible to to inspect the pipes after the dam has been raised over them. If pipes are necessary for a large reservoir, say, for a town water-supply, they should be inserted in a culvert large enough to be traversed. The usual designs adopted are shown in Figs. 35 to 38.

23. Maintenance and Repairs.

(*a*) **The bund.** The top should be maintained at its designed height and any settlement that occurs should be made up before the next rainy season. The drainage of the dam should be continuously attended to. Rain water should not be allowed to concentrate when flowing down the slopes not to lodge anywhere near the base of the dam. All drains should be maintained free from obstructions, and their discharge recorded. The best test of the sufficiency the drainage is that the ground downstream of the dam is dry.

The dam should be kept clear of the long grass and shrubs, the latter being rooted out. The slopes should be maintained to an even surface so as to shed the rainfall regularly. Rat and crab holes should be dug out and the earthwork remade. Cracks should be filled by ramming into them a gritty and clayey mixture by means of chisel-pointed poles A permanent gang of some strength should be employed for the first two years in making up cracks, settlement and rain scoures and should there after be reduced as found necessary.

The pitching should be examined as the water level falls, and all loose stones, settlements, and other defects made good by a system of continuous repair. All shrubs should be rooted out.

(*b*) **The waste weir.** The masonry of this and the outlet should be pointed repaired and underpinned as may be necessary, and all plants growing in it rooted out. All scour channel in the tail channel near the weir should be prevented from cutting back towards it, and approach channel should be maintained free from obstructions to flow.

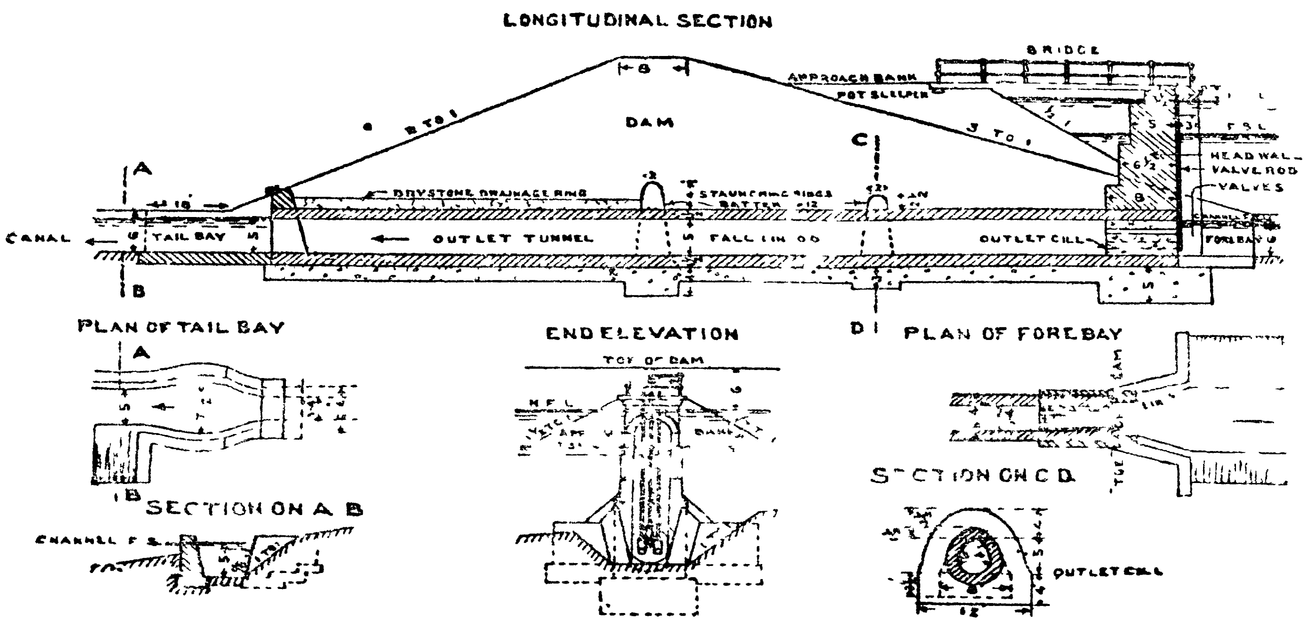

LONGITUDINAL SECTION
BRIDGE
APPROACH BANK
POT SLEEPER
DAM
3 TO 1
HEAD WALL
VALVE ROD
VALVES
DRYSTONE DRAINAGE RING
STAUNCHING RINGS
CANAL
TAIL BAY
OUTLET TUNNEL
OUTLET CILL
FOREBAY
PLAN OF TAIL BAY
END ELEVATION
TOP OF DAM
PLAN OF FOREBAY
SECTION ON A B
SECTION ON C D
OUTLET CILL

Fig. 35

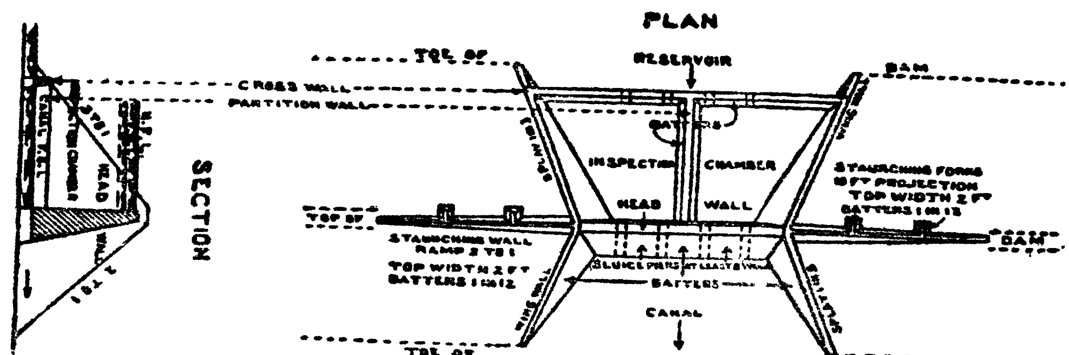

PLAN
SECTION
RESERVOIR
CROSS WALL
PARTITION WALL
INSPECTION CHAMBER
HEAD WALL
BATTERS
STAUNCHING WALL
TOP WIDTH 2 FT
BATTERS 1 IN 12
CANAL
TOE OF
DAM

Fig 36

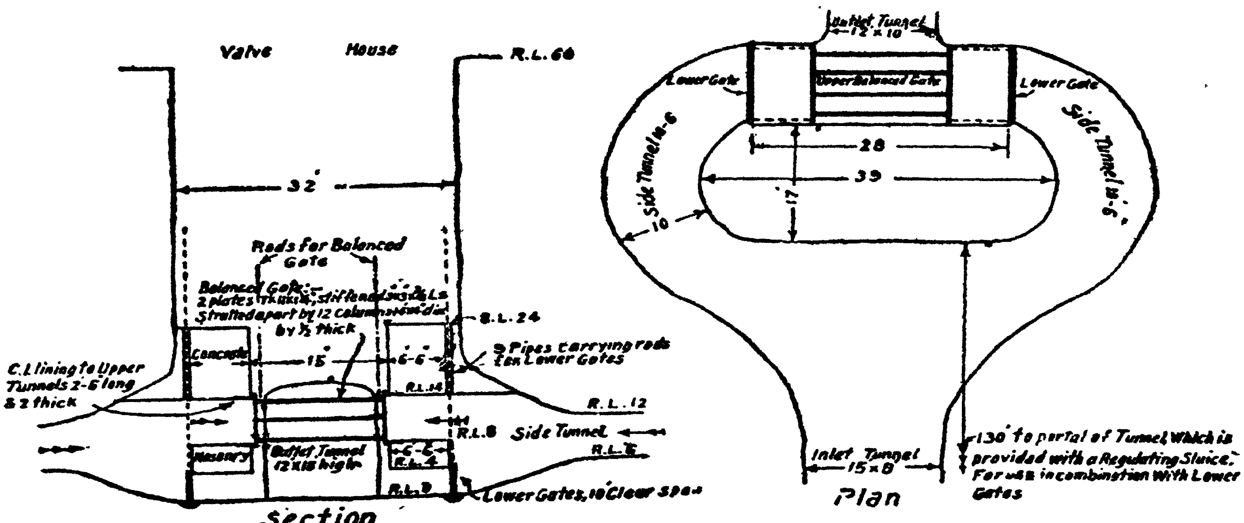

Valve House
R.L. 66
32'
Rods for Balanced Gate
R.L. 24
9 Pipes carrying rods for Lower Gates
C.L lining to Upper Tunnels 2-6 long & 2 thick
Concrete
15'
6-6
R.L. 12
R.L. 8
Side Tunnel
Masonry
Outlet Tunnel 12 x 18 high
Lower Gates, 10' Clear span
Section
Outlet Tunnel 12 x 10
Lower Gate
Upper Balanced Gate
Lower Gate
Side Tunnel 10-6
Side Tunnel 10-6
28
39
17
10
Inlet Tunnel 15 x 8
130' to portal of Tunnel which is provided with a Regulating Sluice. For use in combination with Lower Gates
Plan

Fig. 37

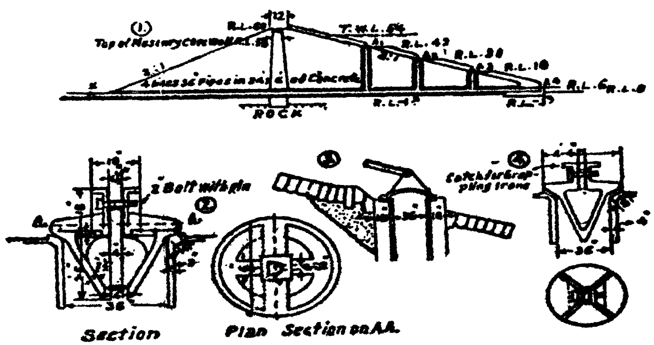

ROCK
2 Bolt with gib
Catch for Grappling irons
36
Section
Plan
Section on AA

Fig. 38

(c) **The outlet.**

The culvert should be examined twice a year and all repairs should then be carried out. The valves, their seatings, and their rods should be examined and attended to when the water level is at its lowest. All parts in working contact should be greased and others painted. The iron work of the approach bridge and headwall should be kept well-painted, and the woodwork tarred and any settlement of the approach bank should at once be made up.

24. Failures of Earth Dams.

The usual cause of a bad failure is the insufficient capacity of the waste weir, as was the case in the Johnstown dam.

The Holmfirth failure (P. I. C. E., Vol. 59, P. 51 and 57), where the dam had been allowed to settle until its crest was but little above the waste weir-sill, can only be regarded as a result of careless maintenance.

Among other disastrous failures the Dale Dyke may be mentioned. This was apparently caused by the bursting of an unprotected line of pipes laid throught the dam. Since, however, the earthwork is described as very loose and more like a quarry tip than a dam, it is difficult to allocate the responsibility accurately.

Small failures usually consist of slips or sloughing of the downstream slope; and if the water in the reservoir is rapidly lowered, similar, but usually more localised slips, may occur on the water face.

Earthen dams have very seldom failed due to percolation through the embankments and most notable cause has been the leakage along the sides of the culverts. A good water-tight connection between the masonry of the culvert and the earthwork should be ensured at the time of connection.

25. Examination Questions.

1. Explain the function of a puddle-trench in an Earthen Dam. Sketch a suitable puddle-trench for a dam with 60 ft. head of water.

2. What points would you keep in view as an Engineer-in-charge of the maintenance and repairs of an Earthen Dam ?

3. (a) Compare the advantages and disadvantages of using masonry or concrete core walls instead of puddle core walls in Earthen Dams.

(b) Sketch a suitable section of a concrete core wall for an Earthen Dam for 80 ft. head.

4. (a) How will you fix the capacity of a waste weir in an Earthen Dam ?

(b) Sketch a suitable design for a waste weir.

5. Explain the construction of a hydraulic-fill Earthen Dam and sketch a suitable section for 70 ft. depth of water.

6. What is the object of providing drains at the base of an Earth Dam? Sketch the usual arrangement.

7. What precautions are usually taken in the design to protect the sides against wash and rain water.

8. Sketch a typical cross-section of an earth dam commonly used in India giving approximate dimensions keeping high flood level at R. L. 790 and the bed of the reservoir 730. *P.U. 1941.*

9. Write a note on the flood absorption capacity of reserviors. Sketch an earthen Bund ment to impound 50 ft. depth of water. The foundations consist of impervious soil and the earth available is loam *T.C.E. 1934.*

10. (a) What are the main points to be considered in selection of a site for an earthen or masonry dam ?

(b) Describe briefly what information you would collect and the surveys you would conduct for preparation of a complete irrigation project. *Mysore U. 1939.*

11. (a) Describe briefly the relative merits of earth embankments and masonry dams for reserviors.

(b) Draw a typical cross section of an earthen reservoir across a perential stream on the river bed portion where 10 feet of sand is found to overlie a hard rocky bed. *Mysore U. 1941.*

12. (a) Describe briefly the causes that lead to the failure of earthen dams and the precautions to prevent them. *Mysore U. 1941.*

(b) (i) Describe the principle of Drainage in the body of an earthen dam and below it.

(ii) Describe the suitable methods of closure of earth dam construction.

13. Describe the construction of Rock-fill Dams. What advantnges do they have over ordinary earthen dams ?

14. What is the nature of damage expected when earth dam is over topped by a flash flood. What preventive measures are indicated *(P. U. 1953)*

15. Sketch cross section of on Earth dam to impound 100 ft. depth of water above the torrent bed, spillway discharge 50,000 cusecs, canal irrigation offtake discharge 2000 cusecs. The zones of various types of materials to be employed in the body of the dam should be specifically indicated *(P. U. 1953)*

16. Differentiate between storage and diversion dams. Name two exemples of each. What are essential requirements for proper design of an earth dam. Could an earthen dam be constructed at Bhakra. If not, why ? (*P. U. C. 1957*)

26. Biblography.

1. "Improving foundation rock for Dams" by J. B. Hays. Civil Engineering May 1939.
2. "Seepage through Dams" by Arther Casagrade. New Engineering water works association U. S. A. June 1937.
3. "Effect of Minor Geological details on the safety of Dams" by K. Terzaghi, American Institute Mining, Met. Engineers, No. 215, February 1929.
4. "Up lift and seepage under dams on sand" by L. F. Harza Trans. American society of Civil Engineers 1935.
5. Seepage through foundations and Embankments studied by glass models, " by H. M. Hill, Civil Engineer, Junuary 1934.
6. "Action of water under dams" by J. B. T. Coleman Trans. American society Civil Engineer. 1916
7. "Engineering for dams" by W. P. Creager, J. D. Justion and J. Hinds, 1944. John Wiley & sons, I. N. C. Newyork, Chapman and Hall, Ltd. London.
8. "Dams and Weirs" by W. C. Bligh, 1918, Chicago.
9. "The control of water" by P. A. M. Parker, 1925, London.
10. "Hand book of Applied Hydraulics" by C. V. Davis, Magraw. Hill Company, Newyork and London. 1942.
11. "Irrigation Practice" by W. L. Strange, Cheif Engineer, Sind, Pakistan.
12. "Theory of Earth presures" by W. J. Rankins Trans. Roy. Soc. London. Vol 147-1857.
13. "Mechanics of Hydraulic fill dams", by Glennon Gilboy, Boston society Civil Engineers, July 1934.
14. "The practical soil mechanics" by Karl Terazghi, John Wiley & sons, Newyork, 1943.
15. "Application of theory of Elasticity and plasticity to Foundation problems", by Leo Jurgenson, Boston Soc. Engrs. July 1934.
16. "Theory of Elasticity" by S Timochesrko, Magraw Book company, Newyork.
17. "Stability of Earth slopes" by Donald, W. Taylor, Boston Soc. Civil Engrs. July 1937.
18. United states Bureau of Reclamation and Reaction on Dams.

PART III
TANK IRRIGATION
(Storages and Dams)
CHAPTER III
Rock Fill Dams

1. Introduction.

Rock fill Dams are suitable in the out of way localities where the cost of cement for concrete dam would be high, where suitable material for a earth dam are not available and where suitable rock for a rock fill dam can be quarred near the dam site. Moreover the rock fill dams are especially suitable when the foundation is not hard and strong enough for a gravity dam (concrete or masonry) but consists of soft rock which could not however, be eroded by any percolation through a rock fill dam. The soft and plastic foundations which could be suitable for earth dams cannot be used for Rock fills. The foundation requirements of rock fill dams lie intermediate between those of Gravity and Earth Dams.

A large number of Rock fill dams exists in United States of America and some have been built in North Africa by the French Engineers. There is not a single example of a rock fill dam in India. Rock fill diversion weirs of the type as Okhla were tried but were soon discarded ; (See Photos). The advances in the art of quarrying and in the manufacture of loading and transportation equipment have made it possible for the rock fill dam to complete favourably with high dams of other types. Settlement and other data obtained in recent years have led to a more scientific knowledge concerning their design, construction, safety and behaviour. The modern practice in the rock fill dam design and construction is typified by the Salt Spring Dam in California.

2. Typical Cross-Sections of Rock Fill Dams.

Cross-sections of Rock fill dams are usually intermediatory between the gravity and earth dams and may generally be classified in the any of four types shown in Fig. 1.

(*i*) **Type section A.** This was the type section usually used in the earliest constructions of Rock fill dam in California, United States of America. There were some variations in the outer slopes and in thickness of the dry rubble sections. The interior fill generally consists of loose dumped rock which contains much dirt and small rock.

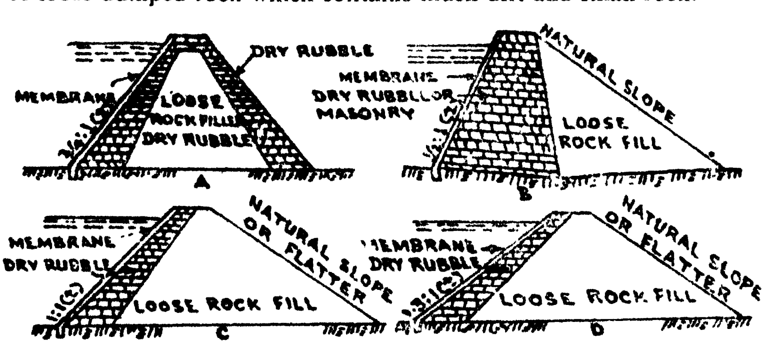


Fig. 1. Typical Sections of Rock fill dams.

(*ii*) **Type section B.** This consists of a massive upstream section of dry rubble and a supporting fill of loose rock.

(*iii*) **Type section C.** This contains a relatively thin section of dry rubble on the upstream face laid with a slope steeper than the natural angle of repose of loose rock and supported on a loose rock fill of major proportions. The dry rubble section acts as a retaining wall to support the loose fill and as a foundation for the impervious membrane.

(*iv*) **Type Section D.** This section has been very much in use recently. The main body is constructed of loose rock having a downstream slope equal or flatter than the natural slope of the fill and an upstream slope that is equal to the angle of repose and supports a dry rubble wall of suitable section. The impervious membrane is laid on the dry rubble face. The most notable dam of this type is Salt Spring Dam in California.

3. The Foundation Characteristics of a Rock Fill Dam.

The essential condition with regard to the foundation for a rock fill dam is that it shall not be subject to material settlement or to the erosion from such seepage, as may pass through or under it. To prevent such seepage, a concrete cutoff wall at the bottom of the facing is usually essential. It should be ponded into firm ledge rock or other suitable impervious material. If the foundation is anything other than sound ledge rock, the possibility of a foundation blowout must be carefully investigated and guarded against. The concrete cutoff wall should extend across and up the sides of the canyon. Grouting may be required in the ledge rock below the cutoff in order to seal the dam against seepage under the structure.

The connection between the impervious diaphragm on the upstream face of the dam and the concrete cutoff wall should be flexible so that a material amount of movement in the slab may take place without causing a rupture which would produce extensive leakage. The problem is particularly difficult at the junction of cutoff walls up the sides of the canyon with the flexible concrete facing and the design at this point should permit a large amount of movement without danger of fracturing the concrete facing. The cutoff wall should be able to take any thrust which may be transmitted to it from the impervious diaphragm membrane on the upstream face. There is usually little or no settlement at the cutoff, but a short distance away from it the settlement may be considerable.

4. Safety Against Sliding.

Rock fill dam with an impervious upstream face founded on very regid foundation must excessively have a relatively high factor of safety against sliding because a very large mass is involved. In the usual design practice, the ratio of height to base is from 1 : 2·25 to 1 : 3·0. The sliding factor ratio, *viz.*, ratio of weight of rock to water pressure, is 4·5 to 6·0. The ratios do not practically vary for all lengths of dams. Never-the-less the adequacy of foundation against sliding should in all cases be investigated.

5. Earthquakes.

The rock fill dam, properly designed and constructed with conservation upstream and downstream slopes, is one of the most satisfactory types of dam to resist earthquake shock. The flexible character of the impervious memberane and the supporting fill permits considerable movement without serious effect.

6. Separate Tower and spillway Work.

The safety of a rock fill dam requires adequate spillway facilities independently placed with respect to the rock fill structure. Under no circumstances should the spillway be embodied within or upon the fill. Nor should the waste channel be located as to permit spill water against or dangerously near the toe of the dam.

A separate outlet tunnel is advisable for high and important dams. Conduite may be sucessfully placed on a solid rock bench or within a deep trench beneath the fill of low or moderately high dams.

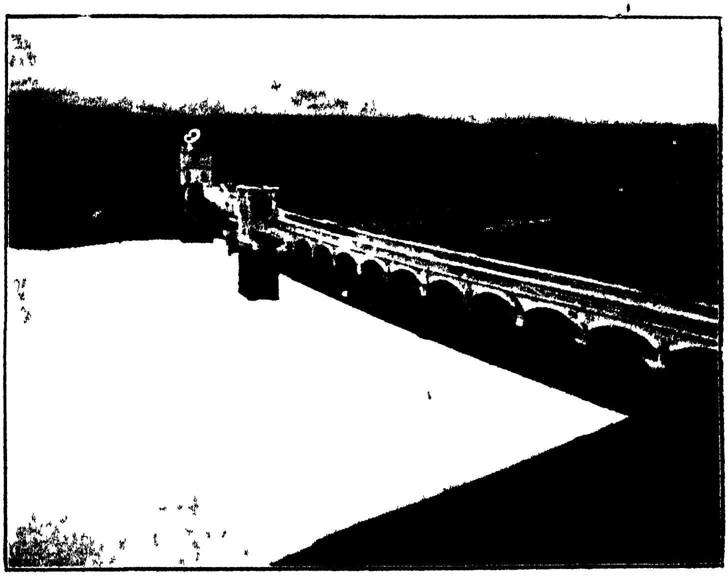
7. Design of Rock Fill Dams and its Parts.

The principal factors which govern the design of rock fill dams sections depend on the character of the fills. The method of construction and the permissible settlement.

(*i*) **Crest width.** Crest of rock fill dams range from 5 to 30 ft. In U.S.A., the crest widths are governed by statute regulations in different states. Minimum requirements of crest width are given below :—

Height of the dam in ft.	Crest width in ft.
Under 100	8
100-150	10
150-200	12
Over 200	15

Arch Dam showing the Crest and overhang



Laggan Dam with view from above the Dam

Tielon Dam, Completed in 1925, height 300 ft. Earthgraval
and Rock fill embankment, concrete corewall.

Crest width wider than otherwise necessary are sometimes used to accommodate roadways. In such cases, parapets or railings are also built on both sides. The section beneath crest is generally built entirely of dry rubble above the looses fill.

(*ii*) **Face slopes.** The steep slopes resulting in eventual slips and settlement are not considered economical in the long run The minimum allowable upstream slopes are ½ to 1, ¾ to 1, 1 to 1 and 1 3 to 1 for heights below 50 ft. 50-100, 100-150 and over 150 ft. respectively. Some sloughing of fills has actually occurred in practice with slopes steeper than 1 to 1. Unfilled horizontal keyways encourage this action. The upstream slope of Salt Springs Dams averages 1·3 to 1. A slope approximating the angle of repose for the loose fill will permit easy safe and rapid construction of all features of the dam proper. It allows use of high lifts for placement of loose fill well in advance of the dry rubble facing and impervious membrane.

The downstream slopes of loose fill of rock dams of all length are not allowed to be steeper than the angle of repose of the fill. The flatter slopes than the angle of repose are uneconomical and unnecessary except for dams situated in the earthquake regions. The natural slopes vary with the character of the fill but range from 1·2 : 1 to 1·4 : 1. The flatter downstream slopes are usually provided in lower levels of high dams.

(*iii*) **Rock fill.** The rock fill in the main part of the dam must be of sound rock which will not readily disintegrate, spilt, or crush. Thus, shales which slake in the presence of air are dangerous and should be rigidly excluded. Rock which, when short, shatters into very small pieces with a high percentage of chips and dust is unsuitable.

Method of placing rock fill vary with height of canyon walls and location of quarries. At sites with high canyon walls it is often possible to drive power drifts into the walls in such a way that a considerable portion of the rock fill required may be short down to, or near, its final positicn .n the dam.

The rock fill must usually be transported from the quarry by dump trucks, railroad, derricks, cranes or cableway. With rail transportation side-dump cars are often used, working out on a circular track from one or both abutments. Where the width of dam is too narrow for such a circular dump track a timber trestle along the line of the dam is employed to start the fill, wkich is widened by side dumping after reaching trestle level. The trestle timber is left in the fill. Successive lifts are made in this manner. An alternative to trestle method is to use end-dump cars, working out from the side walls of the canyon, to from a rock-fill bank from which side-dumping may be more efficiently carried on. End-dump trailers drawn by caterpillar tractors were effectively employe at Salt Springs.

With truck transporation the various lifts may be extended across the valley by both end and side dumping. Derricks, cranes, or cableways are used chiefly when the rock fill is obtainable directly at the abutment ends of the dam. In most recent large rock-fill dam construction railroad cars and trucks have transported the rock fill.

Considerable difference of opinion exists, as so the height of drop, or lift, which is advisable. At Salt Spring (Fig. 2), with granite, a maximum drop of 165 ft. was used, and the settlement, in the dam proper was relatively small The practical limit would be such that it would not seriously injure the rock or be dangerous to workmen.

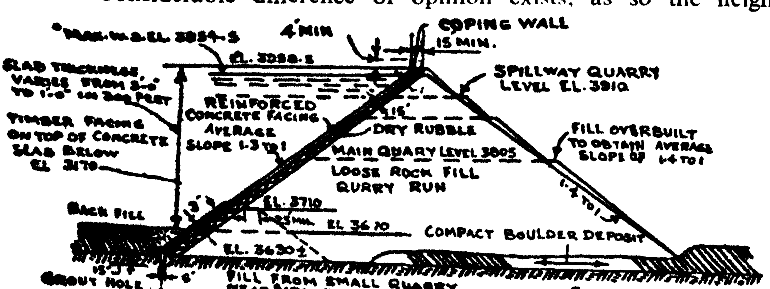


Fig. 2, Salt Spring Dam, Mohelumne River, California Maximum) cross section, (Courtesy Pacific Gas and Electric Co. San Francisco)

The size of the individual rocks may vary greatly. With ordinary dump trucks the maximum size is about 3 tons. Using air dumpted railroad cars or heavy trucks it may average 10 tons, reaching a maximum size in recent dams of about 25 tons. The sizes if placed by derricks. cranes or cableways depend upon the economical machine capacity. As for as practicable chips and dust should be excluded from the fill. Certainly, 10 perecent should be the top limit for such objectionable material under most conditions.

(*iv*) **Dry-rubble or masonry section.** The function of the upstream dry-rubble (or masonry) section is to support the face membrane, retain the loose fill if on a slope steeper than the angle of repose, and transfer the water load to the mass of loose fill in the dam. The character of material and workmanship is exteremely important. Sound Rock, selected as to size and shape, is essential. Well-bounded large rock should predominate. Rock-to-rock contact throughout with thorough bedding and chinking of voids is desirable to ensure a dense, compact mass.

The average percentage of voids in the dry rubble of the latest Algerian dams varies from 25 to 32%. Unchinked voids in the upstream face of the dry-rubble section permit deep bonding of the conerete face with the underlying rock, but may cause sagging of concrete and loss of mortar during construction of the membrance. This largely may be avoided by adoption of a reasonably low water cement ratio and through concrete placement methods. Sand grouled voids reduce bonding but reduce loss of concrete and mortar.

Thickness requirements depend upon the upstream slope, height of dam, character of adjacent loose fill, size and character of available rock and type of equipment used in placing the rubble. Slopes steeper than the angle of repose requires sufficient thickness through out to support and retain the loose fill with reservoir empty. Dry-rubble sections of most dams taper from a minimum thickness at the crest to a maximum thickness at the base. A thickness normal to the slope of 5 to 10 ft. at the crest, depending on the height of the dam, and increasing about 5 ft. for each 100 ft. depth of fill, is generally considered adequate. Advantages and reasons for adopting a tapered section are as follows :--

1. Settlement at or near the crest is not serious because water and rock loads are light.
2. Total settlement at or near the upstream toe of deep fills owing to water pressure is relatively small but critical. Large differential movement from zero at bedrock to substantial or moderate amount within short distances there from may cause membrane rupture.
3. Face rupture near the stream bed level permit leakage under heavy water pressure and high velocity. High velocity, though quickly disipated, may cause disintegration of the concrete and foundation material near points of rupture.
4 The bottom area of the face membrane, especially across the stream bed, generally is the most difficult of access for making repairs.
5. During construction the stability of the dry rubble walls may be maintained as the height of the fill increases. Dry rubble on downstream faces is necessary only if the slope is to be steeper than the angle of repose.

(*v*) **Impervious membrane.** The impervious facing a membrane is usually provided on the upstream slope in such a way that it has got watertight connection with same degree of flexibility to the cutoff wall at the upstream toe. The facing can be made of wood, steel or concrete (plain or reinforced). In a few cases, it has been of bituminous concrete or gunite. Generally speaking reinforced concrete is referred.

(A) **Concrete membrane.** This may be monolithic, sliding or laminated type. The monolithic type, most frequently used, consists of a single thickness of concrete poured against the rock. It may be of either rigid or flexible construction. Both have proved successful and are adaptable to all usual conditions of stress and exposure. The rigid type contains no expansion joints. Reinforcing steel is continuous in both directions. No very high dam has been constructed with rigid type face.

Most dams have been built with flexible monolithic slabs as used in the Spring Dam Fig. 2. Consisting of rectangular panels, formed by tight horizontal joints and vertically inclined expension joints. The typical expension joints used are shown in Fig. 3. Reinforcing steel does not pass the joints. A minimum of 0·5 percent of reinforcing steel in each direction is considered good practice. This amount was provided at Salt Springs. The flexible monolithic face at Malpaso Dam embodies the principle of the wing type. There are no horizontal joints above the cutoff level. Vertically inclined panels (40 ft. wide above the continuously submerged portion of the face) were poured. Construction of the joints, 1 inch in width, extend parallel to and midway between anchors and are provided with copper seals and soft wood fillers.

The sliding type membrane. The sliding type membrane usually consists of a thin, smooth, unreinforced sub-slab poured against the face rock with asphalt or oil coated top

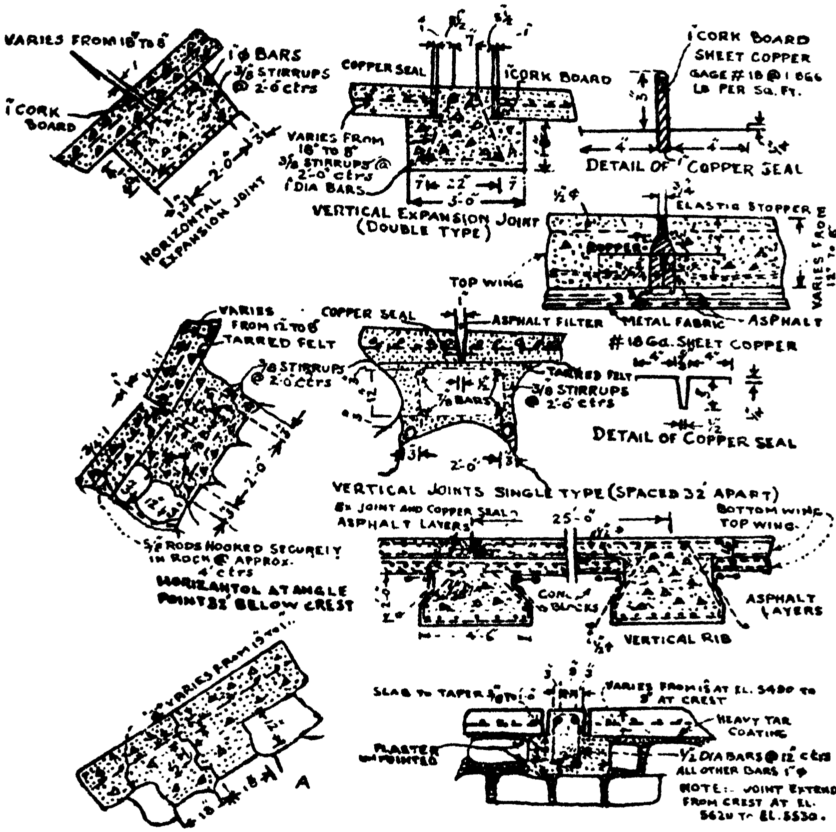


Fig. 3 (A) Monolithic flexible type Joints

Fig. 3. (B) Laminated Wing type joints.

surface against which is laid a heavier reinforced concrete top slab. The top slab is free to move independently from the body of the dam unless prevented by water pressure. Vertically inclined expansion joints are spaced. Horizontal joints are tight. Construction inequalites and settlement during construction may cause irregularities in the supporting face and defeat the purpose of the design. Theoretical forces acting parallel to a sliding-type face of uniform slope may be computed from the formula,

$$F_e = W(\sin\alpha + f\cos\alpha)$$

$$F_c = W(\sin\alpha - f\cos\alpha)$$

Where F_e force under expanding conditions.
F_c force under contracting conditions.
W Weight of slab.
α angle of face slope with horizontal.
f coefficient of friction.

It is believed that a slab of thickness af 1 percent of the head of the of water will provide ample thickness It is also believed that the slab should not be less than 12 inches thick at the top and preferably thicker where climatic conditions may lead to disintegration from frost. Minimum thickness may be applied to dams constructed wholly or largely of dry rubble, refacing of old dams, or replacement of temporary membranes when evidence indicates little future settlement. Sound concrete of not less then 3,000 lb., strength at 28 days and designed for maximum density is the best insurance against percolation and ultimate disintegration of concrete membranes. Concreat in many older membranes was of poor quality, especially at pour joints, and considerable repair expense has been incurred. Sound dence concrete. old and new, generally has been satisfactoy. Waterproofing or surface treatment, of concrete membranes is seldom done.

Gunite Face. Gunite is usually more dense and impervious than concrete. It is especially adaptable for the membrane on dams of low or moderate height where settlement is small for replacement of temporary membranes and for repair of concrete slabs. Minimum thickness requirements are generally less than those for concrete.

(B) **Timber upstream facing.** The older dams were usually faced with wood and its flexibility, which permits the rock dam to settle without significant damage, is quite an advantage It is used frequently in remote locations where timber is plentiful. Below the minimum water surface elevation timber facings are practically permanent. Above the water surface level, the life of timber membrane may be short and in some cases timber impregnated with creosote has been used which lengthens the life of the facing.

Timber facings are subject to a serious fire hazard at any time when exposed but this hazard may be largely obviated by the addition of a layer of gunite on where mesh attached to the timber face or by an efficient patrol system. Timber facings are often economical in remote locations and are particularly applicable where the variations in the elevation of the water surface is not great. These facings are usually from one to three layers of 2 by 12 inches or 3 by 12 inches planks laid parallel to the dam axis and spiked to sills, say 8 by 8 inch, embedded in anchored to the rubble cushion.

(C) **Masonry facing.** Masonry as impervous membrane has been used on few dams. It is seldom used on high dams because of slowness of construction and difficulty in making face repairs if cracks develop. Watertightness is more difficult to obtain than with concrete gunit, or steel membranes.

(D) **Steel facing.** The steel facing has been successfully used as impervious membrane on the upstream face of rock fill dams. The expansion joints of the *U* type are generally used and some means must be adopted for enchoring the plates to the rubble backing. The plate joints may be riveted and caulked or welded. Facing plates range from 1/4 to 1/2 inch in thickness and should be of copper bearing steel to reduce corrosion. Protection from the action of water and air is secured by painting all surfaces with bituminous paints. After 30 years of use, the Skaguay Dam, which was painted on the face only shows only very slight corrosion.

(E) **Upstream face drainage of rock fill dams.** A rock fill dam may be made impervious to harmful passage of water by building a suitable earthen fill or blanket on the upstream slope. Such dams are composit structures. Water from seepage, leakage, rain or sluicing may accumulate behind face membranes due to presence of fines in the fill or river-gravel foundations or if the face slab rests against a tight wall such as concrete or masonry. Uplift pressure thus created may cause rupture or failure of the membrane. Artificial drainage should be provided if conditions indicate this possibility and other corrective means are not feasible.

(*vi*) **Cutoff wall.** Some form of cutoff wall is required along the lower boundary of the impervious membrane to prevent harmful leakage at or below foundation level. This usually consists of a concrete filled trench excavated vertically into solid rock or other impervious foundation material. It may be placed at some other inclination along the abutment slopes. Trench width usually is governed by construction requirements. Trench depth depends upon the desired length of the path of percolation. Governing factors are reservoir pressure and character of foudation. Conservatism is desirable because the cutoff wall and face membrane are the only water barriers and because leaks here are difficult to locate and repair. A length for the path of percolation of 10 to 25 percent of the dam height generally should be sufficient in solid rock or impervious gravel foundations. Pervious gravels and foundations containing faults, open fissure or cavernous rock may require deeper cutoff walls and other special treatment.

8. Settlement of Rock Fill Dams.

Rock-fill dams of all types settle. Settlement results from compaction of fill and foundation under loads from weight of rock and resrvoir pressure. All loads are carried through the fill to the foundation or transferred there to through shearing resistance of the rock mass. Extent of total settlement is somewhat proportional to the amount of applied loads and the distance in the direction of the resultant forces through which they act vertical settlement results from weight of rock and vertical components of water pressure. Total vertical settlement in excess of 5 percent of the height has occured in some rock-fill dams and the horizontal displacement may be nearly as great. If, however, the loose rock dump fill is constructed in advance of the rubble cushion with a proper use of sluicing water, the initial settlement may be large ; but subsequent settlement, after the placing of the rubble cushion and the impervious facing should not exceed 2 percent of the height. Horizontal or downstream movement is principally caused by horizontal components of water pressure against the impervious membrane. Lateral movement is caused by settlement of rock fill from abutment foundations towards the deeper central sections of fill.

Small chips and dust, if present to any considerable extent, will lodge between the rocks and will later shift down into the interstices of the larger rock under the action of rain falling on the dam and passing down through it. This may cause a material settlement of the dam and may cause serious movement and cracking of the impervious upstream face. Accordingly the fines should be constantly washed into the rock mass with hose stream as the fill is being made. The quantity of water which should be used for this sluicing operation will vary with local conditions. Theoretically if there are no spalls or dust, no sluicing would be required, but practically it has been found desirable to use from 2 to 4 time the volume of the dam in sluicing water. Continuous sluicing during constructions will produce presettlement of the fill and thus imcrease stability. The elimination of lenses of small particles thus obtained will reduce local settlement when water pressure is applied on the dam. Generally speaking, the use of an adequate amount of sluicing water during construction is one of the most important factors in the proper construction of rock-fill dams.

It is customary and desirable to construct rock-fill dams with liberal allownces for vertical and downstream settlement. The crest usually crowned above theoretical level in amounts proportional to estimated vertical settlements or from zero at the abutments to some maximum over the deepest section of fill. Otherwise, additional fill, or a coping wall, may later be required to maintain intended freeboard.

Upstream camber (in plan) ensures lateral compression in the face membrane as a whole during all stages of downstream and vertical settlment. Otherwise, tension cracks may develop and vertical expansion joints may open beyond desirable limits.

The face of Salt Springs was constructed concave to reservoir water in vertical planes to prevent buckling due to outward components of downslope thrust. Settlement records show that face concavity would have resulted had the face been constructed along a straight line. Concavity does not prevent face rupture due to either local convexities or eccentric loading within the face occasioned by differential settlements.

Face membranes, subjected to heavy water pressure, must follow the settlement of and generally remain in contact with, the underlying fill. Stresses occasioned through settlement and temperature change must be provided for if serious face rupture is to be prevented. Hazards of rupture usually is reduced through division of the membrane into panels bounded by joints along which movement may take place. Maximum joint spacing is limited by temperature requirements and in general, may be adopted for that large area which is subjected to reasanably small and uniform increments of differential settlements. Ample strength and flexibility here usually are obtainable with any reasonable type of membrane and joint arrangement.

9. Core Wall Type of Rock-Fill Dams.

Rock-fill dams are sometimes built with a core wall of concrete or reinforced concrete. A steel diaphragm, with a thin protecting layer of concrete on each side, has also been used as a core wall in such a dam. The core wall is usually placed in about the centre of the dam and the loose rock fill dumped on each side of it. The upstream half of the rock dam is thus submerged, which decreases the stability of the dam, requiring additional material downstream to take the full horizotal thrust of the water pressure. The centre diaphragm cannot be repaired in case of settlement or cracking. A type design is shown in Fig.4. A dam of this type is practically an earth dam of excessively pervious material, depending absolutely on the imperviousness of the core wall. In turn, the stability and safety of the core wall depends on the supporting power of the loose rock fill behind. It is essential that there should not be much earth or debris in the rock fills, as this leads to undue settlement, which may unbalance the pressures on the core wall to such an extent as to cause serious cracks and distortion and in an extreme case, bring about failure.

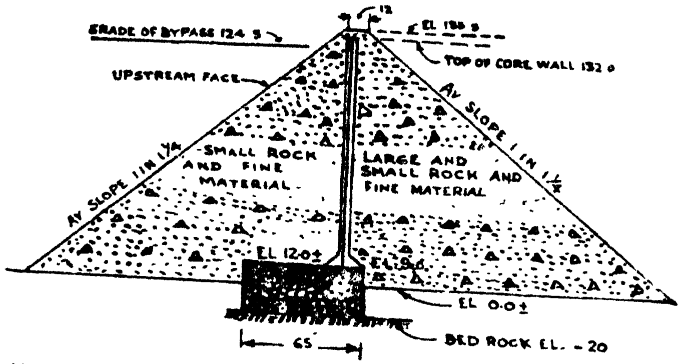


Fig. 4. Typical cross-section of Lower Otay Dam (Courtesy George Cromwell, Chief Engineer San Diego Country Water Co., Los Angeles, Calif.)

10. Composite Type of Rock-Fill Dam.

This type consists of a rock fill on the down stream side of the dam and an earth fill on the upstream side. Such a dam, when properly constructed, produces a very stable and satisfactory structure. The earth fill furnishes the watertight portion of the dam, an intermediate section provides a filter and rock fill forming the downstream portion provides ready drainage for seepage water and adds a greater degree of stability to the structure than would generally be provided by an equivalent amount of earth. A type design of this type is shown in Fig. 5.

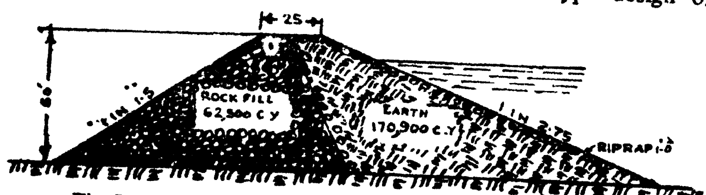


Fig. 5. Composite type Rock fill Dam.

In constructing such dams, the rock fill should be carried up ahead of the earth. Where the earth fill and the rock fill portions of the dam join, the voids between the big stones should be chinked up with smaller stones and spalls and then several heavy layers of graded crushed stone or gravel should be placed on the upstream surface of the rock fill before the earth fill is placed against it. The proper use of these layers of crushed stone or gravel is a matter of the greatest importance to the permanent safety of such a dam, as there must be no penetration of finer material into coarser material due to water pressure or seepage.

A suitable gradation for such a composite dam of moderate height would be the following :—

1. On the chinked-in surface of the dumped rock place a layer 3 ft. thick of screened gravel or crushed stone having a minimum size of ½ inch.
2. On the above layer place another 2 ft. thick, grading between ¼ inch and ½ inch.
3. On the above layer place one 18 inch thick of coarse sand which with all pass a ¼ inch sieve. On top of this third layer place the earth fill, placing the most pervious portion of the material near the downstream limits and most impervious near the upstream face. If the dam is over 100 ft. high greater thickness of filter may by desirable in its lower portion.

11. Earth Core Type of Rock-Fill Dams.

A conventional type of rock fill dam having an impervious element consisting of an earth core in the centre of the dam is indicated in Fig. 6. This type of dam is particularly adapted to locations where there is a plentiful supply of good rock fill and at the same time, sufficient earth for the core. In addition, the earth core possesses certain advantages over the concrete slab or other conventional facings frequantly used for rock fill dams. It can yield readily to settlement of the fill without damage ; it is self-healing with respect to any cracks which might be formed. Because of this feature dams of this type often have less seepage than rock-fill dams with a concrete face. As shown in Fig. 6, the core is protected

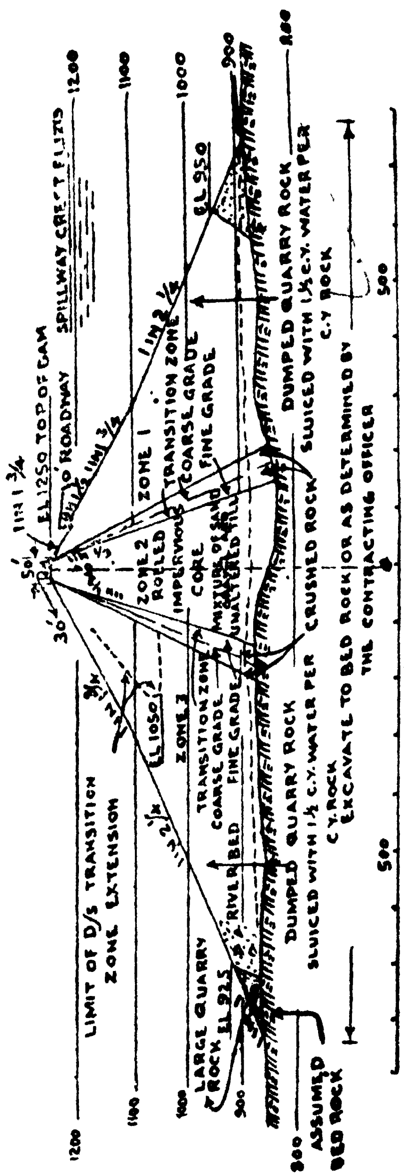


Fig. 6 Earth Core Rock Fill Dams

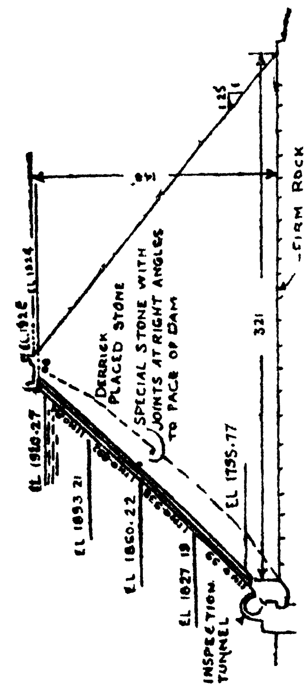


Fig. 7. Hand Packed Rock Fill Dams.

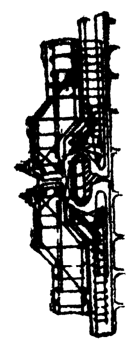


Fig 7 A. Detail of join at Inspection Tunnel.

on both side by filter layers. The d ownstream filters revents loss of core material by piping due to pressure of reservoir water, and the upstream filter affords similar protection against reverse flow when the reservoir is drawn down.

The required thickness of the core depends on the permeability of the material. When highly impervious fill is available, a very thin blanket would, theoretically, suffice. However there are practical considerations which also govern, such as adequate space for spreading and rolling and the provision of an ample mass to allow for deformation due to settlement of the rock fill.

12. Hand Packed Rock Fill Dams.

French Engineers have designed and constructed a number of rock fill dams in Algeria in North Africa. A section of Bakhadda Dam in Algeria is shown in Fig. 7. Recent rock fill dams in Algeria have been constructed entirely of derrick hand placed stone. Essentially they are dry rubble dams with special care taken in the use of selected rock near the upstream face. Voids are thus reduced to 26 to 32 percent of the total mass as contrasted with 35 to 45 percent in American rock fill dams.

Upstream slopes are usually somewhat steeper than 1 in 1 and downstream slopes may be as steep as 1 in $1\frac{1}{4}$. Selected carefully placed stone is used at both faces.

The impervious diaphragm is usually located at the upstream face, and refinements in design, unknown in America, are used to insure watertightness and accessibility for inspection and repair. It appears to usual practices to locate an inspection gallery in the cutoff wall at the point where it connects to the impervious upstream face.

In cases where seepage downstream from the cutoff walls is anticipated, extensive precuations are some times taken to drain away the seepage water at velocities which will not erode the foundation. The Bakhadda Dam (Fig. 7) in Algeria was built entirely of derrick placed stone and was designed to have two concrete facings. The lower facing was built and the dam was placed in operation in order to allow the settlement under water load to take place. The leakage was found to be practically nonexistent with only the lower facing installed. However, on account of possible earthquake disturbance, the original plan was carried through and the upper facing was poured after the two facings were thoroughly separated by bituminous paint. A system of drainage by semicircular conduits was installed and connected with the observation gallery.

13. Examinatoin Questions.

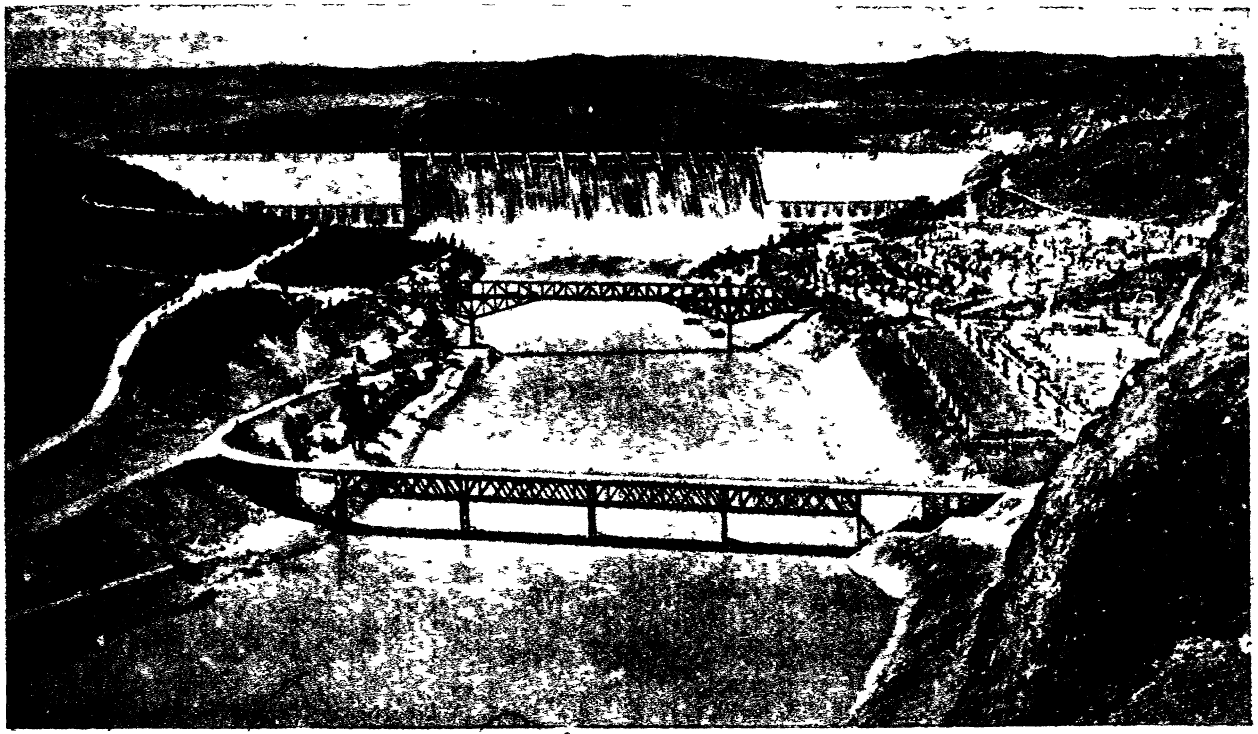
1. What are different types of impervious cut offs used in rock fill dams.
2. What section would you use for a 150 feet high rock fill dam and specify the materials for the various zones. *(P. U. 1955)*

13. Biblography.

1. "The Design of Rockfill Dams", by J. D. Galloway. Trans. Am. Soc. Civil Engrs. 1927.
2. "Rock fill Dam with Steel Face Plates", in O. E. Howvey, Steel Dams, American Institute of Steel Construction, New York, 1935.
3. I. C. Steele, "Salt Springs Dam, California", Eng. News–Record Construction Methods, June 30; I. C. Steele and Walter Dreyer Trans. Am. Soc Civil Engrs. 1939; O. W. Peterson, Trans. Am. Soc. Civil Engrs, 39.
4. Fred A. Noetzli, "Oued Kebir, Tunis", Eng. News Record, L. F. Harza, Eng. News–Record.
5. Herbert M. Wilson. "Mannual of Irrigation Engineering".
6. French Lake Dam, California, (1) Galloway. 1455.
7. Joel D. Justin, Julian Hinds and William P. Creager "Engineering for Dams" New York.

Easton Dam, completed in 1929, height 70   Concrete Gravity Section

Grand Coulee Dam and Power Plant with Government town at the right,
Concrete Gravity Section, height 350′.



PART III

TANK IRRIGATION

(Storage And Dams)

CHAPTER IV

Gravity Dams (Masonry or Concrete)

1. Introduction.

It has been shown in chapter II of this part that earthen Embankments or dams can safely be constructed upto limited height. There have been more accidents to them than to masonry dams properly designed and built. When the height exceeds such limit the earthen dams are out of question, because failures cannot be ricked. A masonry dam is peculiarly adapted to a site having a gorge with steep side slopes and high flanks with good rocky foundations. In a hilly countary, stone masonry or concrete will be cheap as the earth suitable for a dam is not generally available there.

Any dam which depends on its gravity action to resist the forces imposed on it is termed a gravity dam. The term is, however, restricted to solid masonry or concrete dams of roughly triangular section which have a straight face at right angle to the stream. The dams of this type depend for there stability almost entirely upon their weight and inspite of there impressive bulk have a small factor of safety.

2. (*a*) Advantages of masonry Dams.

(*i*) They can safely be constructed across gorges with very steep side slopes, whereas such slopes would tend to make an earthen embankment slip.

(*ii*) They can be built at sites where waste weirs cannot be formed ; in such cases a flood can be discharge over a lowered part of the crest or through under-sluices.

(*iii*) It is easy and comparatively cheap to have in them numerous and large under-sluices to pass of the earlier heavily silt-laden floods and thus to diminish the rate of silting of the reservoir.

(*iv*) Masonry Dams can be built with a greater height than earthen ones.

(*v*) Masonry dams are more suitable than earthen ones in places where the rainfall is excessive.

(*vi*) The maintenance cost of a massonry dam is lower as compared to an earthen one.

(*b*) **Disadvantages of masonry dams.**

(*i*) Only the sound rock foundations are permissible ; any defect therein is likely to cause the dam to fail.

(*ii*) The expenses of the construction of a masonry dam generally are higher than those of an earthen one. This will be the case especially where the foundations are deep.

(*iii*) The construction of a masonry dam takes a long time.

(*iv*) A masonry dam requires a large number of skilled labourers and it is, therefore, not so suitable as an earthen one for providing work for famine relief purposes.

3. (*a*) **Examination of Site.**

The first step should be the preparation of an accurate topographic map of each possible site. The scale of the maps should be large enough for layout purposes and the accuracy and detail should be sufficient to permit close visual indentification on the ground of topographic features shown on the maps. The second step should be a field examination by a stractural geologist. This examination should be directed primarily toward a determination of condition that might render unuseable one or more of the possible sites, such as the presence of faults or the absence of rock of adequate strength. At the conclusion of the preliminary geologic examination , tentative paper locations should be made and the outlines of the dam should then be flagged out on the ground.

At this point in the selection of the site, it becomes necessary to ascertain the actual conditions that will be encountered in the construction of dam.

(*b*) **Foundation exploration.** Rarely will conditions be so favourable as to warrant the design and construction of a gravity dam without exploring the foundation. Where the structural geology is simple and clearly indicated by surface exposure, diamond drilling is generally sufficient Where conditions are less definite, the drilling of large diameter holes or the sinking of shafts and the driving of tunnels becomes advisable.

It is significant that the actual amount of exavation for the foundations of gravity dams has almost always exceeded the estimate of what would be required, owing primarily to insufficient data on existing conditions at the site. Experience thus indicates the advisability of much more exploratory work than is customary.

In any case, a sufficient number of holes should be drilled to determine the probable depth of excavation to sound rock. Some of the holes should be drilled deep enough to disclose any weekness in the under lying rock structure, especially in sedimentary formations and near intrusions. Slanting holes should be drilled to interest seams are faults that would not be disclosed by vertical holes Shafts should be sunk near the upstream face of the dam on the axis of all faults. In locations where there is considerable overburden, trenches should be excavated along the entire length of the proposed dam to expose the surface of the rock.

(*c*) **Geology of site.** Much of this exploratory work would naturally be carried on under the direction of an experienced geologist, who should furnish the engineer with a detailed geologic report covering all factors that would have material bearing upon the final location and design of the dam to be constructed. The location of all faults, contacts and other structural features should be accurately indicated on the topographic map of the site. The characteristics of the foundation materials and their limitations should be clearly and accurately defined. The probable required depth of axcavation at all points should be set forth.

In brief, axploration and geologic study, there should be disclosed to the designer the foundation upon which the gravity dam must stand and by its own weight, resist all forces upon it.

(*d*) **Topographic position.** The next step preliminary to actual design of a gravity dam should be the preparation of a topographic map of the sound rock surface, *i.e*, the surface of the foundation on which the dam is to be built. Paper locations should then be made on this topographic map as a base, some typical cross section being used for comparison of quantites.

In making a paper location, it is generally advisable to select the best position for each block of the dam, without regard to the desirable position of other blocks and keep in mind at all times that the toe of the dam should be as high or higher than the heel. In order to bring these individual block into a consistent pattern some must be shifted upstream and others downstream.

Usually, the best location for a gravity dam is just far enough upstream from the narrowest section of the valley or gorge so that the trace of the downstream toe is on the axis of the ridges that form the narrowest section. The upstream face of the dam may be straight in plan or curved depending upon the topographic from of the sound rock surface. A dam straight in plan is to be preferred from the standpoint of design, unless the curvature is great enough to ensure effective arch action. A dam slightly curved in plan is some times desirable for coustruction reasons, as this tends to bring the centres on gravity of all the block into a straight line.

4. Forces Acting on a Gravity Dam.

The external forces acting on a gravity Dam are shown in Fig. 1.

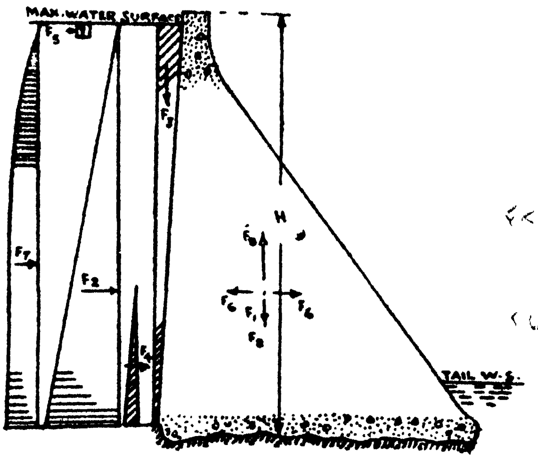


Fig. 1. External forces acting on gravity dam.

F_1 Gravity acting upon the concrete.
F_2 Hydrostatic pressure of water on the upstream face.
F_3 Gravity acting upon the water on the upstream face.
F_4 Excess fluid pressure on the upstream face due to silt.
F_5 Pressure of ice on upstream face.
F_6 Inertia force of the concrete due to horizontal earthquake acceleration.
F_7 Inertia force of water against upstream face due to earthquake acceleration to left.
F_8 Inertia force of the concrete due to vertical earthquake acceleration.
U Uplift acting on base of dam due to hydrostatic pressure.
R Equilibrant of foundation reaction upon the base of the dam.

(A) Student is in a position to calculate the forces F_1, F_2 & F_3 from his elementary knowledge of hydraulics and from field results of the tests made about the weight units mass of the material of the Dam as shown paragraph 6.

(B) **Silt pressure** Silt pressure against the dam is calculated usually according to

Rankine's Formula $$F_4=\frac{W_s\, h^2}{2}\left(\frac{1-\sin\alpha}{1+\sin\alpha}\right) \qquad (1)$$

Where W_s weight of unit volume of submerged silt;
h silt depth in feet;
α angle of internal friction;

F_4 is assumed to act horizontally at depth $\frac{2h}{3}$ below the surface of silt deposit. Recent experiments indicate that the value of α is not materialy changed by submergence and is generally taken as 30° for sand, gravel clay or silt. The submerged weight of unit volume of silt is calculated as below:

W'_s is the dry weight of the earth in a cubic foot of silt, and e is the percentage of voids, then in 1 cu. ft. of the fill there will be $(1-e)$ cu. ft. of solids, weighing W'_s lbs. The weight of water displace when this cubic foot of fill is submerged will be $w\ (1-e)$ lbs. Therefore, weight, W_s of the submerged fill will be w as weight of unit volume of water.

$$W_s=W'_s-w(1-e) \qquad (2)$$

If the specific gravity of the solid particles of the fill is ρ the submerged weight may be expressed thus.

$$W_s=W'_s\left(\frac{\rho-1}{\rho}\right) \qquad (3)$$

The dry weight W'_s of the solid material in a cubic foot of fill, as it will be laid down in the reservoir, may be estimated from deposits of silt in neighbouring reservoirs. A simple means for finding the dry density of soil in place has been developed by Proctor (Bib. 1) Values commonly used for the constants in Eqs. 2 and 3 are as follows :—

W'_s=dry weight =100 lbs. per cft.
e=voids percentage=40 percent.
ρ=specific gravity =2·67.

(C) **Ice pressure.** Ice expands and contracts with changes in temperature like other materials. In a reservoir completely frozen over, a decided drop in the temperature of the air may cause the opening up of cracks which subsequently fill with water and freeze solid. When the next rise in temperature occurs, the ice expands and if restrained it exerts pressure on the dam. Little is known of the magnitude of this pressure and allowances made in the past have been variable.

The magnitude of ice pressure naturally varies greatly, depending upon the thickness of the ice that will form on the reservoir and upon other factors, including the slope of the banks of the reservoir and the shape of the upstream face of the dam itself. The magnitude of ice pressure has been variously estimated from 5,000 to 30,000 lbs. per sq ft. of contact with the vertical face of a dam. It is believed that an allowance of 10,000 lbs. per sq. ft. would be ample under any ordinary circumstances.

(D) **Earthquake forces.**

(*a*) **Horizontal acceleration.** The intensity of the inertia force depends on the acceleration, *i. e.*, on the rate of change in the velocity of motion. This acceleration is usually designated by its ratio to g the acceleration of gravity. According to Dewell, (Bib. 2) accelerations ranging from 0·0037 g to g have been observed or estimated in earthquakes. An intensity of 0·4 g, or more than 12 ft./sec^2, is not uncommon in great shocks; but this, as well as the highest intensity just noted, is of such local occurence that from the standpoint of general structural design it need not be considered.

An acceleration of 0·1g has been used in the design of a number of recent dams. This value may be said to be tentatively standard for dams in seismically active regions. For sites close to known active faults, larger values should be adopted. In favorable locations particularly the seismographical history of the region should be refered to.

$$F_6 = Ma = \frac{W}{g}\,\delta g = \delta W \qquad (4)$$

where, M—Mass; W - weight; a—acceleration; δ—ratio of a to g,

However, earthquake movements are reversed and repeated. Also, an elastic structure, given an impulse, tends to oscillate should the periods of oscillation for the structure and vibration period of the earth tremor coincide, a dangerous cumulative effect may be produced. Viberation periods for important earth shocks are generally of the order of 1 sec., or more and the motions are in constant in magnitude, period and direction. Dewell's studies indicate that "long continued resonance between the structure and the earthquake wave, so feared by many writers, is not probable, but that any structure whose natural period of elastic viberation is in excess of one second, when subjected to an earthquake of major intensity, may suffer the effects of resonance for a few viberations". Westergard (Bib. 3) finds the time of vibration for a concrete gravity dam of triangular section, reservoir empty with a modulus of elasticity of 2,000,000 lb. per sq. inch to be ;

$$t_s = \frac{h^2}{2000\,l} \qquad (5)$$

where t_s is the time of viberation in second, h is the height of the dam in feet and l is the base length in feet. For reasonable ratios of h to l, t_s approaches 1 sec., only for heights in excess of 1000 ft. The possibility of resonance for dams of usual heights need not be considered.

(*b*) It has been usual to consider only the horizontal forces produced by the inertia of the dam (F_6) and the momentarily increased pressure of the water (F_7) as the foundation shifts laterally. A close approximation of the force F_7 is given by the following equation developed by Von Karman (Bib. 3 Discussion)

$$F_7 = 0{\cdot}555awh^2; \text{ acting at a point } (4/3\pi)h \text{ above the base.} \qquad (6)$$

(*c*) Seismic forces, however, may and do act in all directions. Vertical accelerations of 3 to 6 ft./sec^2. are to be expected during a severe earthquake; far greater acceleration has been reported. Any upward acceleration (F_8) opposes the acceleration of gravity, the effective weight of the dam upon which its stability depends being thereby momentarily reduced.

(*d*) **Most unfavourable direction of earthquake movement.**

An earthquake movement may take place in any direction. For a gravity dam, reservoir full, the most unfavourable direction is upstream normal to the axis. The corresponding force acts downstream. For reservoir empty, a downstream acceleration is more unfavourable. A vertical acceleration changes the weight of the masonry and the water in the same ratio. Considering these elements alone, the resultant is not displaced from the position it would occupy if there were no earthquake. However, the stresses are changed. If the acceleration is upward the stress is equal to the no-earthquake stress multiplied by $(1+\alpha)$, which is generally less than the stress for an equal horizontal acceleration. If the acceleration is downword, the multiplier is $(1-\alpha)$. For a direction of acceleration intermediate between horizontal and vertical, the situation is more complex. For small deviations from the horizontal, the maximum stress may be slightly greater than for a horizontal acceleration of equal value but the difference is smaller than the uncertainties in the value of α ; hence, deviation from the horizontal may be ignored for dams with straight vertical faces.

Uplift is usually assumed to be unaffected by earthquake. For arch dams with appreciable rise, maximum stresses may be caused cross-channel acceleration. The forces due to the inertia of the masonry are easily deduced and act unsymmetrically (as to direction) on the two halves of the arch. The increased water pressure is assumed to act normally but with varying intensity.

(E) **Uplift.**

(*a*) Dams are subjected to water pressure, not only on exposed faces but also on their bases and within the masonry itself. These internal pressures produce uplift. Uplift is the upward pressure of water as it seeps or flows through the dam or its foundations. It causes a reduction in the effective weight of the structure above it. Water causing uplife pressures may enter through pores or imperfections in the foundation, through imperfectly bonded foundation or construction joints or through pores in the structure itself.

The existence of these forces has been recognized for about half a century and has been the subject of much debate. There is perhaps one other factor in the design of masonry dams about which so little is known and about which there are so many differences of opinion. Uplift on a unit area at any point in a horizontal section may be considered to have two elements : (1) the hydrostatic pressure of the seeping water at that point ; and (2) the percentage, c, of the unit area on which the hydrostatic pressure acts.

(*b*) The total uplift force is obitaned by applying this pressure-intensity diagram to that portion of the area of the base of the dam on which it is assumed to act. For a dam slice of uniform thickness, this total may be computed from the equation.

$$U=cw\,[b^2+\tfrac{1}{2}\,\zeta\,(b_1-b_2)]\,A \qquad (7)$$

where A is the area of the base, c is the proportion of that area on which the hydrostatic pressure acts and ζ is the proportion of the net head, (b_1-b_2) remaining to be dissipated below the grout curtain, w is the weight of one cft. of water. The line of application passes through the center of gravity of the trapezoid 1-4-3-2 in Fig. 2.

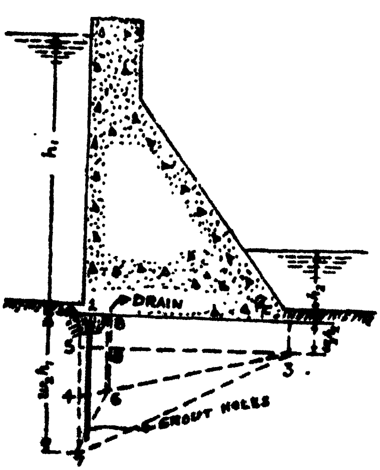


Fig. 2. Practical uplift pressure intensity diagrams.

Uplift determined in this manner can never be exact. Closer approximation can be attained by continued experimentation and the further observations on constructed dams but the the need for careful judgment on the part of the design cannot be escaped.

(*c*) **Values of ζ and c for use in design.** The following values of uplift constants are usually used. (Bib. 4)

1. For all conditions, $c=1{\cdot}0$.
2. For earth foundations, ζ should be determined by a flow net or special analysis.
3. For rock foundations :—(table on the next page).

"Moderate" represents dams up to about 200 ft. "High" represents dams above about 200 ft.

The ζ could be determined from the depth of the cutoff under upstream toe by the method described in Part II, Chapter III.

Height of Dam	Type of Rock Foundation.	Grouting and Drainage	C
Moderate	Horizontally stratified	None	1·00
Do	Fair, horizontally stratified	Yes	0·67
High	–do–	–do–	0·75
Moderate	Good, Horizontally stratified.	–do–	0·50
High	–do–	–do–	0·67
Moderate	Fair, massive	None	0·67
Do	–do–	Yes	0·50
High	–do–	–do–	0·67
Moderate	Good, massive	None	0·50
Do	–do–	Yes	0 50
High	–do–	–do–	0 50

(F) **Wind pressure.** Wind pressure is seldom a factor in the design of a dam. Such structures are usally in sheltered locations. Even in exposed locations, the wind has access to only the downstream face of a loaded dam. The maximum possible pressures are small when compared to the loads for which the dam is designed and it acts against the water load. An unloaded masonry dam is not subject to damage by wind. The superstructure of dams carrying very large sluice gates may need to be proportioned to resist wind loads of 20 to 30 lbs. per sq. ft. Transverse wind pressure on the exposed buttresses of hollow dams may require consideration under some circumstances.

(G) **Wave pressure.** The upper portions of dams are subject to the impact of waves. The dimensions and force of waves depend on the extent of the water surface, the velocity of the wind and other factors.

Knowledge of wave heights is important if overtopping by wave-splash is to be avoided. Wave pressure against massive dams of appreciable height is usually of little consequence. Wave forces on sea wall and breakwaters are important, but such structures do not come within the scope of this book.

Formulas for wave heights proposed by Stevenson have been widely used.

Molitor (Bib. 5) proposes modifications of the Stevenson formulas to include the wind velocity, as follows,

$$h_w=0{\cdot}17\sqrt{VF}+2{\cdot}5-\sqrt[4]{F} \qquad (8)$$

where h_w is the height of the wave from trough to crest in feet, V is the wind velocity in miles per hour and F is the "fetch" or straight length of water subject to wind action in statute miles. For F greater than 20 miles, this equation may be simplified thus :—

$$h_w=0{\cdot}17\sqrt{VF} \qquad (9)$$

Molitor develops empirical formulas for wave lengths, velocities, height of rise above still water level, height of rise against obstructions, force of impact and other wave functions suited particulary to the design of sea walls and breakwaters, having wave resistance as a primary function.

The maximum unit pressure occurs about 0·125 h_w above the still water level and is approximately.

$$P_w=2{\cdot}4\ h_w=150\ h_w \qquad (10)$$

The wave-pressure diagram is of a curvilinear form.

(*b*) **Sides, set-up and seiches.** Tide movements are imperceptible in inland waters, However, an appreciable piling up of water on one shore of a lake or reservoir may be caused by wind action, particulary in shallow water. The height of rise above the undisturbed lake level is called "set-up". For deep water and small areas, this effect is small and may be considered to be included in the freeboard allowance. For wide, shallow reservoirs a special study may be required. The Zuider Zee formula is the best available means for estimating "Set-up". It is as follows :—

$$S=\frac{V^2F}{1400\ D}\cos\alpha \qquad (11)$$

where S is set-up in feet above still pool level, V is wind velocity in miles per hour ; F is fetch in miles, D is average depth of water in feet and α is angle of wind and fetch.

It is unfortunate that upto the present no formula has been devised that checks accurately the set-ups experienced at different localities. Therefore, conservatiom is necessary in this respect.

Periodic undulations, called seiches, also occur. Seiches may be set in motion by intermittent wind action, variations in atmospheric perssures, earthquakes, or irregular inflow or overflow. They come and go at regular, periods, which may vary from a few minutes. › several hours. After the generating influence is removed, the oscillations gradually subside.

5. Requisites for Stability of Gravity Dams.

(*i*) **Location of resultant.** Tension shall not exist in any joint of the dam, under any condition of loading. For dams with rectangular joints, this requirement is met if the resultant of all forces including uplift, acting on the dam above any horizontal joint, for full or empty reservoir, intersects the joint, within the middle third. For irregular joints neither for reservior full, nor for reservior empty shall be any tension. This ensures factor of safety against overturning as 2 [paragaph 6 (a)].

(*ii*) **Resistance to sliding (shear neglected).** The tangent of θ, the angle between the vertical and the resultant of all forces, including uplift, acting on the dam above any horizontal plan, shall be less than the allowable coefficient of friction at that plane.

If f′ represents the coefficient of static friction of the materials above and below the joint; then $f'\Sigma W$ will be the frictional resistance to sliding.

For equilibrim, neglecting shear, $f' \Sigma (W)$ must be equal to or greater than ΣP. This may be expressed thus :—

Where $\Sigma P = F_2 + F_4 + F_5 + F_7 \pm F_6$; $\Sigma W = F_1 + F_2 \pm F_8 - U$; $f' \Sigma (W) \geqslant \Sigma (P)$

$$\text{or} \quad \frac{\Sigma (P)}{\Sigma (W)} = \tan \theta \;\not> f' \qquad \text{(A)}$$

where θ is the angle between the vertical and the resultant.

The coefficicnt, f′, in carefully constructed dams on reck foundatoins, with particular attention paid to obtaining rough surfaces at the base and at construction joints, is usually considered to be at least twice as great as indicated by experiments on well-dressed specimens of the same materials. Therefore, if tan θ is made equal to or less than the coefficient of friction, as indicated by such tests, a factor of safety in this respect of at least two will be provided; and the neglect of the adhesion or shearing resistance at the joints and foundation will serve to increase further the factor of safety. Therefore, for horizontal joints and rock foundations and neglecting shear. Eq. A may be modified for safe design thus :—

$$\frac{\Sigma (P)}{\Sigma (W)} = \tan \theta \leqslant f \qquad \text{(B)}$$

where f is the coefficient of friction of the materials on each side of the joint or at the base, as indicated by tests on well-dressed specimens of the same materials. Values of f for masonry on masonry and masonry on good rock foundations have been assumed variously between 0·6 and 0·75 In general and for careful work, a value of 0·75 is not excessive. Proper allowance, however should always be made where the rock foundations is poor or where it contains nearly horizontal seams close to the finished surface of the foundation. Such seams are particularly dangerous if they contain clay or other unstable material. Rock otherwise satisfactory may have to be removed to eleminato an objectionable seam below it.

(*iii*) **Resistance to sliding, shear included.** The total frictional resistance to sliding on any joint; plus the ultimate shearing strength of the joint, must exceed the total horizontal force above the joint for all conditions of loading, by a safe margin.

This relationship may be stated algebarically thus :—

$$\Sigma (P) \leqslant \frac{f\Sigma (W) + r\, S_a\, A}{S_{s-f}} \qquad \text{(C)}$$

where S_a is the unit shearing strength of the material, S_{s-f} is the shear friction factor of safety, A is the area of the joint, r is the ratio of the average to the maximum shearing stress on the joint and other symbols are as already defined. The friction factor is that for well-dressed specimens. The value of r may be actually determined but for practical purposes, with the relatively large factors of safety hereinafter recommended, a value of 0·5 may be assumed. The value of S_{s-f} should be 4·0 as used in normal structural computations.

(*iv*) **Compressive stresses.** The unit inclined compressive stresses in the dam and the foundation shall not exceed certain prescribed values.

The maximum vertical compressive stresses are not the maximum stresses which occur in the structure. The maximum stresses occur at the ends of joints, on inclined planes, normal to the face of the dam.

Where there are no external forces acting against the faces, these maximum stresses are given by the equations worked in paragraph 6 (*c*) of this chapter. Baker (Bib. 6) recommends allowable stresses as follows provided each is the best of its class. Rubble masonry 20,000 to 30,000 lbs per sq. ft., squared stone masonry, 30,000 to 40,000 lbs. per sq. ft., Limestone Ashlar masonry, 40,000 to 50,000 lbs. per sq. ft. and Granite Ashlar masonry, 50,000 to 60,000 lbs. per sq. ft. This allows usual factor of safety.

In concrete dams, the allowablé compressive stress varies from 700 to 1000 lbs. per sq. inch after allowing a factor of safety of 4, according to the density of the concrete executed. Standard tables are available in all text books on concrete plain or reinforced giving the variation of the allowable compressive stress with density.

(*v*) **Tension on inclined or vertical planes.** The Dam shall be designed and constructed in such a manner as to avoid or adequately provide for tension on interior planes, inclined vertical or horizontal.

The application of this rule is complicated and can not be readily worked out in the step by step applications of the other rules for the design of a dam, ordinarily it is not a determning factor in design. Hence the normal procedure is to complete the design without regard to this rule and then to test for internal tension. Method of computation of internal tensile stresses is shown in worked example paragraph 11 of this chapter. Allowable safe tensile stress with factor of safety 4 is 200 lbs. per sq. inch in concrete and 100 to 150 lbs. per sq. inch in masonry.

(*vi*) **Factor of safety.** All design factors contributing to the permanent safety of a dam should be chosen with care and should be conservative. A careful estimate of the weight of the dam should not vary more than 1 to 2 percent from the actual weight and the weight and pressure of the water are closely known. The maximum depth of water should include liberal allownce for the highest possible flood and waves and seiches should be provided for if require, in order that the assumed water load surely shall not be exceeded Allownces for uplift, earthquake forces, silt and ice pressures must be adequate. The assumed safe sliding factor, foundation strength and concrete or masonry strength must be conservative. If all these factors are carefully chosen, the dam, if properly designed and constructed, will be safe. If the foundation is rock, there is an additional element of safety because of the adhesion of the concrete to the foundation. To this feature alone can be attributed to the continued existence of a number of poorly designed dams. These considrations lead to following rule :—

All assumptions of forces acting on the dam shall be unquestionably on the safe side, all units stresses adopted in design shall provide an ample margin against rupture and the adopted safe sliding factor or shear friction safety factor shall be conservative.

(*vii*) **Details of design and construction.** The shape of the section of the dam having been determined in accordance with established rules, careful attention must be given to the details of the design and the methods of construction, in order that the structure may be satisfactory in every respect.

The location and extent of vertical building joints, passageways and other planes of weakness must be within proper limits, in order that the stresses used in the design will not be seriously increased. Such features as drains and cutoffs, on which the assumption of uplift is based must be carefully designed and other matters of importance attended to. The masonry in the structure must be of a quality to withstand safely the working stresses adopted in the design, practically watertight and durable. Outlet works and spillway must be designed to avoid overtopping or damage from overfalling water. Freeboard must be provided to prevent overtopping by floods or waves. These considrations lead to following generalised rule.

All details shall support and conform to the assumptions used in the design ; the masonry or concrete shall be of a quality suited to the working stresses adopted and shall be practically watertight and durable; protection against overflowing water shall be ample.

6. Design of Low Gravity Dams.

(A) **Distribution of pressures at a joint.** In the theory of stability which relates to the distribution of pressures along the line of any joint of dams and other masonry structures, it is assumed that the change of intensity of pressure along any horizontal joint is uniform and this implies that the curve of pressure distribution along a joint is a straight line. The mean intensity of pressure along a joint of unit which is the total of the whole pressure divided by the length of the joint supporting it and under the assumption made, the maximum or minimum intensity of pressure must be at the extremities of the joint. Under such conditions it can be proved that ;

if S_a = mean or average stress $=\frac{W}{b}$; S = stress at any point.

$$S=S_a\left(1-\frac{6e}{b}\right) \qquad (1)$$

where b is the length of the joint or base width and e is the accentricity or distance of the point where the resultant cuts the joint or the base from the middle of the base.

$$\therefore\ S_{max}=S_a\ (1+\frac{6e}{b});\ \text{where } e=\frac{b}{6};\ S_{max}=S_a\ (1+\frac{6b}{6b})=2\ S_a \text{ and}$$

$$S_{min}=S_a\ (1-\frac{6b}{6b})=0.$$

The maximum eccentricity so that S_{min} is not negative is 1/6b which means that the resultant should cut the base not more than 1/6b away from middle of the base. This gives us the middle third rule. If the base be divided into three equal parts, the resultant must not fall outside the middle third. The stress distribution diagram is graphically obtained as in Fig. 3 *a*, *b*, *c* and *d*.

Draw AB to represent the base or length of the joint Fig. 3 (*a*). Let it be divided into three equal parts AC=CD=DB and E represents the midpoint of AB. Let the resultant R cut the base at L. Let W be the vertical component of R. Plot EF equal to W/b to a certain scale to represent S_a. Join DF and CF. Draw normal from L to cut DF at H′ and CF in G′. The harizontal projection G to G′ and H′ to H gives the pressure intensity AG and BH as shown to the same scale as EF.

Following deductions are evident from the pressure diagrams :—

(*a*) That when the centre of pressure passes through one extremity of the middle third

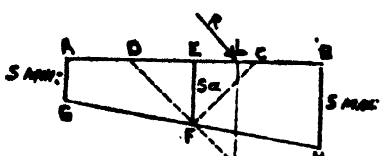


Fig. 3 *a*

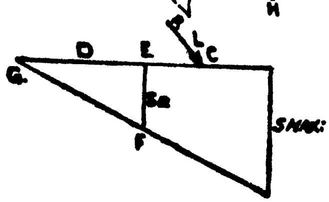


Fig. 3 *b*

Fig. 3 *c*

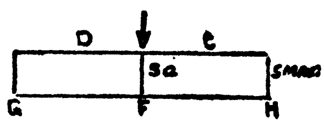
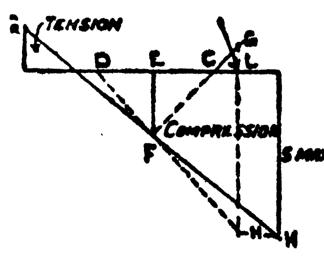


Fig 3 *d*



of the base or other horizontal joint of a dam, the stresses vary uniformly from zero at that end of the joint most remote from the centre of pressure, to a maximum, at the other end, which is equal to twice the meam stress, Fig 3 (*b*).

(*b*) That if the centre of pressure is outside the middle third tension is exerted from some distance along the end of the joint most remote from the centre of pressure, Fig. 3 *d*).

(*c*) That if the centre of pressure falls within the middle third, there is compressive stress throughout the joint, which stress is smallest at the end remote from the centre of pressure and greatest at the other end Fig. 3 (*a*); the nearer the centre of pressure is to the centre of the joint the joint the nearer the maximum stress is to the mean stress, that is, for the same total pressure, the smaller is the maximum stress. It is by designing so as to induce this condition that the pressures developed in high dams are kept within the toto limits.

(*d*) If R meets the middle of base Fig 3 (*c*) the pressure is uniform and equal to the mean average.

(B) **Profile of a low dam (elementary profile, neglecting all forces except F_1, F_2 and F_3)**

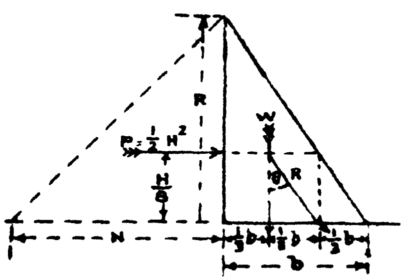


Fig. 4

Let b be the length of the base (Fig. 4).

ρ Specific gravity of material, (2 for brick masonry and 2·25 to 2·4 for stone masonry). P total water pressure. W weight of the dam. w weight of one cft. of water = 62·5 lbs., say 1/36 ton. R represents the line of resultant pressure which cuts the base at the downstream extremity of the middle third. $P=F_1$, $W=F_2$ and $F_3=0$

$W=\frac{wHb\rho}{2}$ or $H=\frac{2W}{wb\rho}$; but $P=\frac{wH^2}{2}$ as the resultant passes through the outer middle point

$$\frac{P}{W}=\frac{b/3}{H/3}=\frac{b}{H} \text{ or } \frac{b}{H}=\frac{\frac{1}{2}wH^2}{(wHb\rho)/2}$$

Hence $b^2\rho=H^2$; and $b=H/\sqrt{\rho}$ (2) if $\rho=2.25$ then $b=2H/3$ (2A)

(C) (*i*) **Maximum stress due to resultant pressure.**

The maximum unit stress in the interior of a dam is not identical with equation (1) the maximum vertical unit reaction at the base, but is a function of S_a. In Fig. 5, a representive

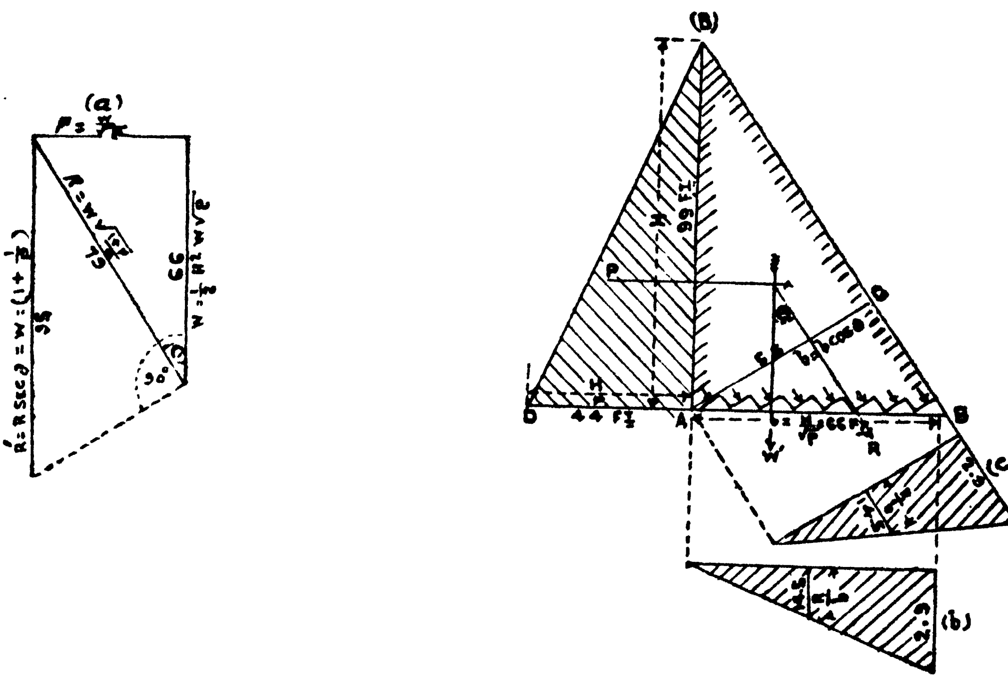


Fig. 5

triangle of forces is shown composed of N the vertical force (W), P the horizontal water pressure and R the resultant of N and P, therefore $R=\sqrt{N^2P^2}$ also $=N\sec\theta$. If the back were vertical, N and W would be the same and then $R=\sqrt{W^2+P^2}$. Various views have been current regarding the maximum internal stress in a dam. The hitherto most prevalent theory is based on the assumption, see Fig. 5, that the maximum unit stress

$$C=\frac{mR}{b}=m\frac{\sqrt{N^2\times P^2}}{b}=\frac{mN}{b}\sec\theta \qquad 3(A)$$

where $m=(1+(6e)/b)$ and C is the maximum compressive stress in masonry. Another theory, which still finds acceptance in Europe and in the East, assumes that the maximum stress is developed on a plane normal to the direction of the resultant forces as illustrated by the stress lines on the base of Fig. 5 According to this, the mean stress due to R would not be R/b but $\frac{R}{b'}$ and the maximum stress will be $\frac{mR}{b'}$. But $\frac{R}{b'}=\frac{R\sec\theta}{b}$ and $R=N\sec\theta$ consequently the maximum unit stress would be

$$C=\frac{mN}{b}\sec^2\theta=\frac{mN}{b}(1+\tan^2\theta) \qquad 3(B)$$

Recent experiments on models have resulted in the formula for maximum internal unit stress being recast on an entirely different principle from the preceding one. The forces in action are the maximum vertical unit force or reaction S combined with a horizontal shearing unit stress $S_s=P/b$. The shearing force is the horizontal water pressure. or $(H^2w)/2\rho$ symbolized by P, which is assumed to be equally resisted by each unit in the base of the dam ; the unit shearing stress will thus be P/b. These forces being at right angles to each other, the status is that of a bar or column subject to compression in the direction of its length and also to a shear normal to its length. The combination of shear with compression produces an increased compressive stress and also a tension in the material. The formula recently adopted for maximum unit compression is as follows :—

$C=\frac{1}{2}S+\sqrt{\frac{1}{4}S^2+S_s^2}$; In this $S=mS_a=\frac{mN}{b}$; As before $S_s=\frac{P}{b}$ substituting we have ;

$$C=\frac{mN}{2b}+\sqrt{\frac{(mN)^2}{4b^2}+\frac{P^2}{b^2}}=\frac{mN+\sqrt{(mN)^2+4P^2}}{2b}$$

When $m=2$, as is the case when the incidence of R is exactly at the outer boundary of the middle third,

$$C=\frac{N+\sqrt{N^2+P^2}}{b}=\frac{N}{b}(1+\sec\theta) \qquad 3(C)$$

Application of the above formulae is shown in the example below :—

Example.

Let H in elementary profile = 150 feet, $\rho=2{\cdot}4$, $\rho=\frac{3}{40}$ ton.

Whence, according to (3C) ; $C=\frac{150\times3}{2\times40}(1+1{\cdot}187)=12{\cdot}3$ tons per sft.

Taking up formula (3A) ; $C=\frac{mW\sec\theta}{b}=\frac{2W\sec\theta}{b}$

as above $\frac{W}{b}=\frac{Hw\rho}{2}$; $C=Hw\rho\sqrt{\frac{\rho+1}{\rho}}=Hw\sqrt{\rho}.\sqrt{\rho+1}$

Example with conditions as before : $C=\frac{150\times1\times1{\cdot}55\sqrt{3{\cdot}4}}{32}=7{\cdot}26\times1{\cdot}84=13{\cdot}3$ tons.

With formula (3B) $C=2.\frac{W\sec^2\theta}{b}$, or in terms of H, $C=Hw\rho\left(\frac{\rho+1}{\rho}\right)=Hw(\rho+1)$;

therefore with values as above $C=\frac{150\times1\times3{\cdot}4}{32}=15{\cdot}9$ tons.

From the above it is evident that formula (3B) gives a very high value to C. Tested by this formula, high American dams appear to have maximum compressive unit stresses equal to 20 tons per square foot, where, as the actual value according to formula (3A) is more like 14 tons. However, the stresses in the Asswan dam, the Periyar and other Indian dams, as also French dams have been worked out from formula (3B) which is still in use.

(*ii*) **Equations giving relations between pressures and stresses.**

$$P=\frac{wH^2}{2}=\frac{W}{\sqrt{\rho}} \quad (4); \qquad W=\tfrac{1}{2}H\times\frac{H}{\sqrt{\rho}}\,w\rho=\frac{w\sqrt{\rho}.\,H^2}{2} \quad (5)$$

$$R^2P^2+W=\frac{w(1+\rho)}{2}H^2=\left(1+\frac{1}{\rho}\right)W \quad (6)$$

Maximum pressure on base reservoir empty.

$$S_e=2\frac{W}{b}=\frac{w\sqrt{\rho}.\ \sqrt{\rho}.\ H^2}{H}=\rho wH \quad (7)$$

maximum stress reservoir full.

$$S_f=\frac{2R}{b\cos\theta}=W(1+\rho H) \quad (8); \qquad \cos\theta=\frac{\sqrt{\rho}}{\rho+1} \quad (9)$$

Serial No	N A M E	Notation	Formula	Value when $\rho=2{\cdot}25$	Value when $\rho=2{\cdot}4$	Remarks
1	2	3	4	5	6	7
1	Base width	b	$\frac{H}{\sqrt{\rho}}$	·67H ft.	·64H ft.	
2	Water Pressure	P	$\frac{W}{\sqrt{\rho}}=\frac{wH^2}{2}$	$\frac{H^2}{72}$ tons.	$\frac{H^2}{72}$ tons.	
3	Weight of masonry.	W	$\frac{w\sqrt{\rho}.\ H^2}{2}$	$\frac{H^2}{48}$ tons.	$\frac{H^2}{46}$ tons.	
4	Resultant pressure reservoir full	R	$\frac{w\sqrt{1+\rho}}{\rho}$	$\frac{H^2}{40}$ tons.	$\frac{H^2}{\varepsilon 6}$ tons.	
5	Maximum unit stress reservoir empty.	S_e	$w\rho H$	$\frac{H}{16}$ tons. per sft.	$\frac{H}{15}$ tons per sft.	
6	Maximum unit stress reservoir full.	S_f	$w(\rho+1)H$	$\frac{H}{11{\cdot}1}$ tons per sft.	$\frac{H}{10{\cdot}6}$ tons per sft.	

(*iii*) **Limiting height.** Let λ be the permissible stress in masonry in tons per square foot the limiting height of the dam will be determined from equation (8).

$$S_f=\frac{H}{11{\cdot}1}; \qquad \therefore\ H=11{\cdot}1\lambda \quad (9a)$$

Serial No.	Stresses. tons/sq. ft.	Stresses. lbs/sq. ins.	H in feet.	
1	10	156	111	Value of stress usually ranges from 10 to 18 tons per sq. ft. according the stone and mortar used.
2	16	250	177·6	
3	18	280	200	

The dams of the elementary profile, the height of which is limited to the above values of H according to the premissible stress, are called low gravity dams and when the height exceeds these figures, the dams are said to be high ones. The procedure of design of low dams as given above is lucidly described by W. G. Bligh in his book on "Dams and Weirs." Bib. 7.

(D) **Internal shear and tension.**

We have seen that the combination of compressive and shearing stresses in a dam produces an increased unit compression. It further develops an increase in the shearing stress and also tensile stress.

Compression as before

$$C = \tfrac{1}{2}S + \sqrt{\frac{S^2}{4} + S_s^2} \text{ or } \frac{mN + \sqrt{(mN)^2 + 4P^2}}{2b} \qquad (10)$$

$$\text{Tension } t = \tfrac{1}{2}S - \sqrt{\frac{S^2}{4} + S_s^2} \text{ or } \frac{mN - \sqrt{(mN)^2 + 4P^2}}{2b} \qquad (11)$$

$$\text{Shear } S_h = \sqrt{\frac{S^2}{4} + S_s^2} \text{ or } \frac{\sqrt{(mN)^2 + 4P^2}}{2b} \qquad (12)$$

The tension and shearing stresses are not of sufficient moment to require any special provision in the case of a gravity dam. The tension is greatest at the heel, diminishing toward the toe. This fact suggests that a projection of the heel backward would be of advantage. The direction of C to the vertical is not that of R but is as follows :—

$$\tan 2\alpha = \frac{2S_s}{s} = \frac{2P}{b} \div \frac{mN}{b} = \frac{2P}{mN}$$

when $m=2$, $\tan 2\alpha = \frac{P}{N}$. In Fig. 5, $P=555$ and $N=855$

$$\therefore \tan 2\alpha = \frac{555}{855} = 0{\cdot}649\text{ ; whence } 2\alpha = 33^\circ\text{-}00 \text{ and } \alpha = 16^\circ\text{-}30'$$

The inclination of R to the vertical or θ is 33°-50′, *i. e.*, twice as large as that of C. The direction of t is at right angles to that of C, while that of S_h, the shear, lies at 45° from the directions of either C or t.

(E) **Profile of a low dam in practice.**

In actual practice it is, of course, impossible to rigidly adhere to the elementary profile as a dam must have a certain crest width, while it also requires some 'free-board' above the maximum water level.

The crest width (*a*) of a dam is fixed from considerations of the special requirement of each case such as the nature of traffic required to pass over the top of the dam, the nature of the crest shutters, if any and the space required for them, etc.

Bligh suggests the following empirical relation between H and A which is generally suitable :—

$A = \sqrt{H}$; where A is top width in feet.

The free-board of a dam, that is, the height of the crest above water level, is generally fixed from consideration of the height of the waves likely to be raised at the dam site and this depends on the fetch, that is, the longest straight length from the dam face of water surface exposed to wind.

Stephenson's rule for finding the height (*h* feet) of wave with a 'fetch' of *f* miles is given in Paragraph 5, Chapter II of this part.

Generally the top of the dam would be raised above M. W. L. to a height somewhat greater than *h* and the free board would seldom be less than 4 feet.

The water level for purposes of stability should be taken either (1) as the top of the wave, that is. to *h* feet above M. W. L. or (2) as the level of the crest of the dam.

The necessity for having a crest of a certain width and height above water level; alters the position, on the lines of resultant pressure from those of the elementary profile, bringing that with the reservoir full (R) further towards the centre of the base, and thus increasing stability but throwing that with the reservoir empty (W) a little outside the middle third.

The adjustment required to rectify this is to give a slight projection outside the elementary profile to the lower part of the upstream face.

The adjustment required is worked out by Tudsbery and Brightmore (Principles of Water Works Engineering) and the resulting profile is shown in Fig 6, in which the parts of the adjusted profile lying outside the elementary triangle are shaded.

The adjustment required to the upstream face to balance the top thickness is only 1/16 of the latter and in many cases may be omitted as unnecessary.

When such balancing masonry upstream of the vertical face of the elementary profile is introduced, it is frequently given as a slight and continuous batter of 1/50 starting from top.

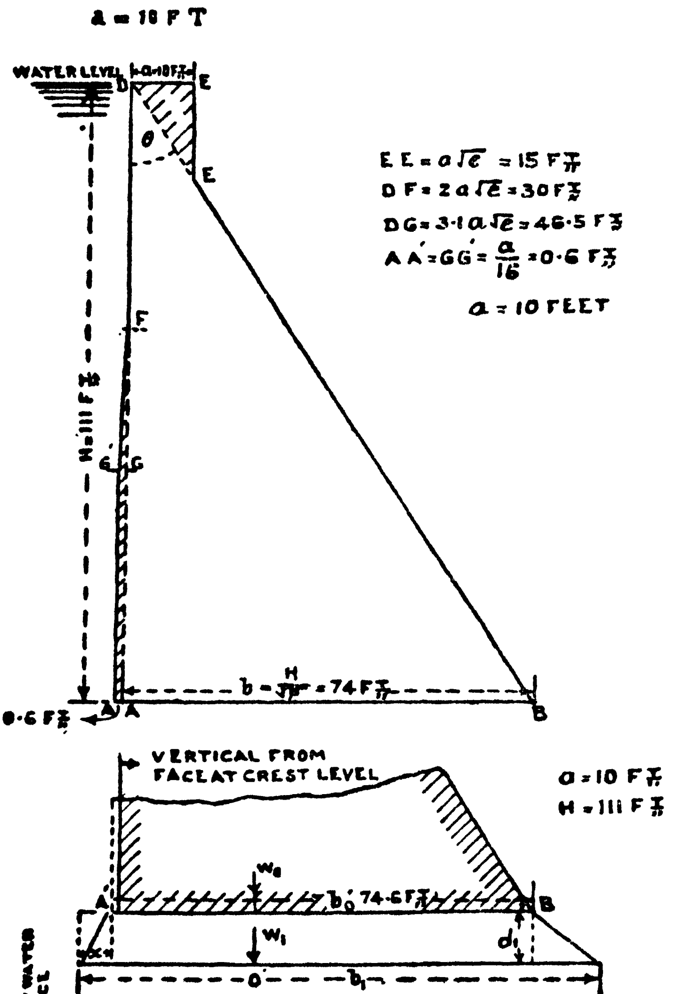

WATER LEVEL
EE = a√c = 15 FT
DF = 2a√c = 30 FT
DG = 3·1a√c = 46·5 FT
A A' = G G' = a/16 = 0·6 FT
a = 10 FEET
VERTICAL FROM FACE AT CREST LEVEL
a = 10 FT
H = 111 FT

Fig.6 (a)

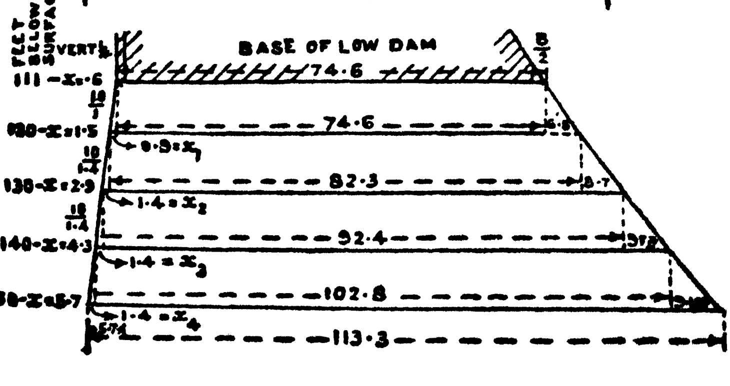

BASE OF LOW DAM
74·6
82·3
92·4
102·8
113·3

Fig.6 (b)

6. Example of a Low Dam.

Design a low dam with $\rho=2{\cdot}25$ and permissible stress=10 tons per square foot, with top width at crest=10′ ; Height=111 feet ; The design is sketched in Fig. 6; Maximum stress developed= $\frac{111}{11{\cdot}1}$ =10 tons/sq. foot

7. High Dams.

(A) **Design of a high dam.** The design of a high dam is far more complex than that of a low dam and the distribution of materials in the profile, while following the same rules as those necessary for low dams, is governed by limitations in respect of maximum stresses in masonry. In addition to this it is most undesirable to increase the width of base in the lowest layers of a dam profile by flat slopes as this weakens the toe or heel of the dam. It is not desirable to have any slope flatter than 1 to 1 on either face.

Such limitation introduces new factors and in practice in designing a high dam after arriving by calculation at a theoretical profile conforming to the standard rules, it is usual to fix a final profile which is practical modification of the former and to test this profile for stability and stress limit by calculation and stress diagrams. In the following paragraphs is described a method of computation for arriving at the theoretical profile and the methods of testing the derived practical profile, both by arithmetical computation and by diagram, are also given.

In important cases both methods may be employed as each forms a check on the other.

(B) **Formulae applicable to design of high dams.** The following method of designing the theoretical profile of a high dam below the limiting depth of water suitable for a low dam is due to Messers Tudsbery and Brightmore ("Principles of Water Works Engineering"). The design of the dam from water surface to the low dam limit is assumed to be that shown in Fig. 6, the form of which is also due to the same authors.

The method depends on the following propositions :—

If u is the distance of the centre of pressure from that end of the horizontal base at which the stress is greater, than by equation, the following can be deduced :—

$$u=\frac{b}{3}\left(2-\frac{S}{2S_a}\right) \qquad \text{(A)}$$

Let γ=the limit stress in masonry in tons per square foot ; H=the depth of water ; w=the weight of water per cubic foot, in tons; W=the critical force due to weight of masonry and of any water lying over the sloping section of the upstream face ; b=the width of the base.

Then from the above equation (A) the following relation can be derived (the proof of this is given in "Water Works Engineering").

$$b=\sqrt{\frac{wH^3}{\gamma}\left(1+\frac{w^2H^4}{4W^2}\right)} \qquad \text{(B)}$$

If W be known even approximate by the value of b which will keep maximum stress within the limit, γ can be deduced with close accuracy.

Having found the length of b, from equation (B) it has to be ascertained how it is to be placed under the masonry dam, that is, how much (x) of its length is to project upstream of the vertical from the upstream edge of the crest of the dam.

Referring to figure 6 (a), if O′ is the point where W acts on this base the length of which is termed b_1 then the conditions require that O′ shall not be closer to the upstream face than $b_1/3$

The base b_1 should be so placed that O′ is at a distance $b_1/3$ from its upstream extremity.

If moments be taken round O′ of the vertical forces on either side of it, the sum of such moments must be equal to zero.

From this, the following equation is derived ;

$$\frac{Pwd_1}{24}\left\{3b_0^2-b_1^2+6x_1(b_0+b_1)+2b_0b_1\right\}-\frac{wx_1}{12}(H+H_1)(2b_1-3x_1)-W_0\left(\frac{b_1-b_0}{3}-x_1\right)=0 \qquad \text{(C)}$$

where H_1=depth of water at base $b_1=H+d_1$=vertical depth of new strip added below the base b_0; x_1=the projection of the end of the base b_1, above the base b_0, that is, upstream of A'.

Thus referring to Fig. 6 and the known base A'B of the low dam, the value of W_0 is the resultant vertical pressure due to W, the weight of masonry plus the vertical component of water pressure and this is known.

To fined the length b, of the base at a depth d, below A'B (the base of the low dam). the value W_1 can be approximately ascertained by continuing the profile with the same side slopes for a vertical distance d, below the base A'B and computing the additional weight.

The method of procedure is demonstrated in the following example :—

(C) **Example of high dam.** As an example, a case of design is taken from "Water Works Engineering" and suitable actual dimensions assigned to the low dam Fig. 6 and lower layers will be designed o ta depth of 150 feet below the crest level, which it is assumed, is also the maximum water level.

The top width $a=10$ feet ; $\rho=2\frac{1}{4}$ and limiting pressure=10 tons. The limit depth of the low dam is $11\cdot1=111$ feet.

Also $A'B=b_0=\frac{111}{\sqrt{\rho}}+\frac{10}{16}=74\cdot6$ feet.

It is required to design the lower portions of the dam down to 150 feet below water level.

To do this vertical space from 111 feet to 150 feet will be divided into four horizontal laminae, numbers 1, 2, 3 and 4 with base of length b_1, b_2, b_3, b_4 at levels below water.

H_1-120 feet ; H_2-130 feet ; H_3-140 feet ; H_4-150 feet.

Correspondingly the vertical depth would be denoted as d_1, d_2, d_3, d_4, of each lamina, and the projection of each base upstream of the one above it x_1, x_2, x_3, x_4, and the vertical component of weight of masonry and water above each base W_1, W_2, W_3, W_4.

First Lamina. $W_0=265$ tons, $H=111$ feet, $b_0=74\cdot6$, $x_0=0\cdot6$ ft. Approximate weight of lamina produced from 111 to 120 feet below water in tons.

$$=\frac{9\times74\cdot6\left(1+\frac{120}{111}\right)}{2\times16}=40 \text{ tons.}$$

Then as a first approximation $W_1=265+44=309$ tons. Substituting values in equation (B);

$$b_1=\sqrt{\frac{(120)^2}{36\times10}\left(1+\frac{(120)^4}{4\times(309)^2\times(36)^2}\right)}=82\cdot5 \text{ feet}$$

Next step is to find x_1 from equation (C)

$$\frac{9}{16\times24}\left[3\times(74\cdot6)^2-(82\cdot5)^2+6\times157\cdot1\times x_1+2\times74\cdot6\times82\cdot5\right]$$

$$-\frac{x_1}{36\times12}\left[231(165-3x_1)-265\left(\frac{7\cdot9}{3}-x_1\right)\right]=0$$

$$\therefore 1\cdot6x_1^2+199x_1-177=0$$

$$\therefore x_1=\frac{-199+\sqrt{39\cdot601+1113}}{3\cdot2}=\frac{3}{3\cdot2}=0\cdot9 \text{ feet}$$

From the approximate values of b_1 and x_1 thus found make a second calculation of the weight of the lamina plus that of the water on its upstream face (*viz* : $44\cdot2+2\cdot08$) and W_1 will be found to be $265+47=312$ tons. Substitution of this value in equation (B) gives a revised value $b_1=82\cdot3$.

The difference is too small to affect the value of x_1 which thus remains 0·9 feet.

The results are therefore :—

$W_1=312$ tons, $b_1=82\cdot3$, $x_1=0\cdot9$ foot ; $x=x_0+x_1=0\cdot6+0\cdot9=1\cdot5$ feet.

Second lamina. $H_2=130$ feet.

The weight of the lamina produced from profile above (first approximation) plus overlying column of water$=\frac{10}{16}\left[82\cdot3+\frac{10}{9}-\frac{1}{2}(82\cdot3-74\cdot6)\right]+\frac{120+130}{2\times36}\times1\cdot0=58$ tons

W_2 (first approximation)$=312+58=370$ tons. Applying this to equation (B) we get $b_2=\sqrt{\frac{(130)^3}{360}\left(1+\frac{(130)^4}{5184\times 370}\right)}=92{\cdot}5$ feet

Applying this value of b_2 to equation (C) ; we get $x_2=1{\cdot}4$ feet

From these first approximations of b_2 and x_2 recompute the weight of second lamina plus water ; $W_2=312+59=371$ tons.

Recalculation of equation (B) with this value of W_2 gives ; $b_2=92{\cdot}4$ feet

The values at base of second lamina are, therefore,

$W_2=371$ tons ; $b_2=92{\cdot}4$; $x_2=1{\cdot}4$; $\therefore\ x=0{\cdot}6+0{\cdot}9+1\ 4=2\ 9$

Third lamina. $W_3=371+67-438$ tons ; $b_3=102\ 8$ feet ; $x_3=1{\cdot}4$

The above are first approximations

The assumed values from prolonging slopes of the trapezoid above are so nearly correct that no second approximation need be worked.

For third lamina the base dimensions are ; $W_3=438$ tons ; b_3 - $102{\cdot}8$ $x_3=1{\cdot}4$ feet ; $x=2{\cdot}9+1{\cdot}4=4{\cdot}3$ feet.

Fourth lamina. Similarly the fourth lamina gives ; $W_4=513$ tons ; $b_4=113{\cdot}3'$ $x_4=1{\cdot}4'$; $x=4{\cdot}3+1\ 4=5\ 7$ feet.

The lower laminae fulfil the following conditions for 'reservoir full' :—

(*a*) Weight of masonry plus water lying on upstream face 'reservoir full' acts at the upstream extremity of the center third.

(*b*) Maximum stress is limited to 10 tons per square foot.

It does not, however, necessarily follow that the maximum stress 'reservoir empty', might not exceed 10 tons this will be examined by the method of moments in paragraph (8) below.

(D) **Practical profile based on the theoretical profile.**

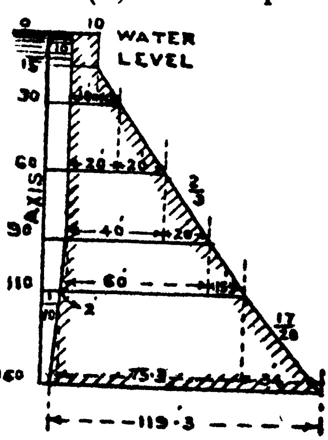


Fig. 7

From the profile of the laminae as drawn in Fig. 7, it will be seen that the sides of the trapezoids have slightly different slope and in drawing a practical section the profile would be simplified so as to have as few changes of slope as economically possible.

For the dam in the above example, the practical constructional profile may be as in Fig. 7.

The practical profile must now be examined to see if it fulfils the required conditions *viz.*

(1) Lines of resistance within the middle third.

(2) Maximum stress of masonry not to exceed 10 tons per square foot.

(E) **Essential divisions and stress distribution.** Before proceeding further the distribution of the different essential parts of each horizontal joint in a dam will be examined. If 'b' be the length of the horizontal under examination (Fig. 8) this may be divided into three parts, by the centres of pressure 'reservior full,' such that $b=n'+v+u$. Let H be the depth of water above the joint ; let M be the moment ; let P be the total horizontal water thrust above the joint and ; let W' be the weight of the dam plus weight of water on the slope (if any) of the upstream face.

Take moments round the intersection of R with the base ;

$1/3\ PH=M-W'v$; $\therefore\ v=M/W'$ (D)

$b=n'+v+u$; $\therefore\ u=b-(n'+v)$

Again if the center of pressure is within the middle third of the base and distant 'u' from the end of the maximum stress,

we have from equation (1)

$$S=S_a\left(1+\frac{6e}{b}\right)\quad \text{(a)}\quad \text{also } e=\left(\frac{b_1}{2}-u\right)\ ;\ \text{and } S_a=\frac{W'}{b}$$

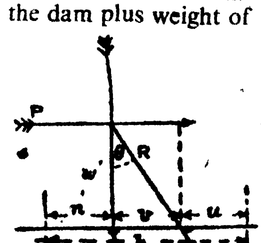


Fig. 8

Substituting in (a) for e and S_a the maximum stress,

$$S=\frac{W'}{b}\left(1+\frac{3b-6u}{b}\right) \text{ or } S=\frac{2W'}{b}\left(2-\frac{3u}{b}\right) \qquad (E)$$

This stress is the maximum vertical stress on the horizontal base, but if the resultant R is inclined at an angle θ to the vertical the resulting maximum stress S_a (as already explained in paragraph above) becomes ;

$$S=S_a \sec\theta=\frac{2W'}{b}\left(2-\frac{3u}{b}\right)\sec\theta \qquad (F)$$

8. Method of Moments for Testing Stability.

This method can be applied to weirs, regulators, and other structures, as well as to dams.

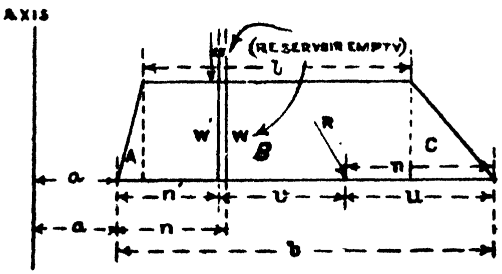


Fig. 9

Divide the dam up into a suitable number of horizontal laminae.

Consider one of these as in Fig. 9

Take moments round a vertical axis at any convenient distance from the upstream face. It is assumed that the weight (W) and its moment about the axis "reservoir empty" is known from the working out of the upper laminae.

The weight of the parts A, B, C and their individual moments about the axis can be computed.

Reservoir empty. The sum of the moments of W, A, B and C round the axis gives the moment of the whole weight (W) on the base b about the same axis and this moment divided by W will give the length $(a+n)$ of the lever arm 'reservoir empty'.

Thus n (reservoir empty) becomes known.

Reservoir full. If the upstream of the dam from the water surface down to the joint is vertical, the weight (W') "reservoir full will be the same as reservoir empty" (W). If, however, the upstream face is sloped, separate moments be taken for the condition 'reservoir full' and the values found of :—

N' 'reservoir full' as well as of n 'reservoir empty'

From equation (D), [paragraph 7 (E) above.] ; $v=M/W'$

From this u will be found by equation.

$$b=n'+v+u \;; \quad \therefore\; u=b-(n'+v) \qquad (G)$$

Example of test by the method of moments :—The practical profile, Fig. 7 will now be tested throughout the whole height of the dam for middle third and limit of stress (10 tons per square foot) by taking moments round an axis 10 feet from the vertical upstream face of the dam. In this example the unit of weight and pressure is taken as the weight of one cubic foot masonry. The dam will be divided into laminae above bases at levels below water surface 30, 60, 90, 110, 150 feet.

(1) Reservoir empty and full.	Weigh	Lever arm	Moment
Rectangular portion above level 30	300	15	4500
Triangular „ „	75	23·33	1750
Total	375	...	6250

$$n=n'=\frac{6250}{375}-10=16{\cdot}66-10=6{\cdot}66$$

$$\text{Moment of water thrust}=\frac{4\times30\times30\times30}{9\times2\times3}=2{,}000$$

$$v=\frac{M}{W}=\frac{2000}{375}=5{\cdot}33\;;\;(1/3)\,b=6{\cdot}66\;;\;n=n'=6{\cdot}66\text{ ft.}\;;\;b=20{\cdot}00\;;\;u=8{\cdot}01\text{ ft.}$$

(2) Section to 60 feet below water, (b = 40 feet).

Reservoir empty and full.	Weight	Lever arm	Moment
Portion of dam above level 30	375	...	6250
Rectangular portion between level 30 and 60	600	20	12400
Triangular ,, ,,	300	36·66	10998
Total	1275	...	29248

$n = n' = (29248/1275) - 10 = 22{\cdot}9 - 10 = 12{\cdot}2$

Moment of water thurst $= (2 \times 60 \times 60 \times 60)/27 = 16{,}000$

$v = 16000/1275 = 12{\cdot}55$; $(1/3)\ b = 13{\cdot}33$; $n = n' = 12{\cdot}9$; $b = 40{\cdot}00$; $u = 14{\cdot}55$

(3) Section to 90 feet below water level (b = 60 feet).

Reservoir empty and full.	Weight	Lever arm	Moment
Portion of dam above level 60	1275	...	29,249
Rectangular portion between level 60 and 90	1200	30	36,000
Triangular ,, ,,	300	56·66	16,998
Total	2757	..	82,247

$n = n' = (82247/2775) - 10 = 29{\cdot}6 - 10 = 19{\cdot}6$

Moment of water thurst $= (2 \times 90 \times 90 \times 90)/27 = 54{,}000$

$v = 54000/2775 = 19{\cdot}46$; $(1/3)\ b = 20$; $n = n' = 19{\cdot}6$; $u = 20{\cdot}94$; $b = 60{\cdot}00$

(4) Section to 110 feet below water level (b = 75·3 feet).

(a) Reservoir empty.	Weight	Lever arm	Moment
Portion of dam above level 90	2775	...	82247
Dam between levels 90 and 110.			
Triangular portion upstream of vertical face	20	9·33	186·6
Rectangular portion downstream	1200	40	48000
Triangular portion ,, ,,	133	74·44	9900·5
Total	4128	...	140334·1

$v = 54000/2775 = 19{\cdot}46$; $(1/3)\ b = 20$; $n = n' = 19{\cdot}6$; $b = 60{\cdot}00$ and $u = 20{\cdot}94$

$n = (140334/4128) - 8 = 33 - 8 = 25$ ft.

(b) Reservoir full.	Weight	Lever arm	Moment
Reservoir empty as above	4128	...	140334
Whole water on slant upstream face	88	8 9	783
Total	4,216	...	141,117

$n' = (141117/4216) - 8 = 33{\cdot}5 - 8 = 25{\cdot}5$

Moment of water thurst $= (2 \times 110 \times 110 \times 110)/27 = 98{,}592$

$v = 98592/4216 = 23\ 4$; $(1/3)\ b = 25{\cdot}1$; $n' = 25{\cdot}5$; $b = 75{\cdot}3$; $u = 26{\cdot}4$.

(5) Section to 150 feet below water level (b = 113·3 feet).

(a) Reservoir empty.	Weight	Level arm	Moment
Portion of dam above level 110	4128	...	140,334
Triangular portion upstream of vetical face between levels 110 and 150	80	6·66	532
Rectangular portion downstream	3012	45·6	139·147
Triangular portion ,,	680	94 6	64328
Total	7,900	...	344,341

$n' = (344341/7900) - 4 = 43{\cdot}6 - 4 = 39\ 6$

(b) Reservoir full	Weight	Lever arm	Moment
Weight of dam and water on the slant face above level 110	4216	...	141,117
Masonry only between levels 110 and 150	3772	...	204009
Water on slant face between levels 110 and 150	231	5·95	1374
Total	8219	...	346,500

$n'=(346500/8219)-4=42{\cdot}2-4=38{\cdot}2$

Moment of water thurst $=(2\times150\times150\times150)/27=250{,}000$

$v=250000/8219=30{\cdot}4$; $(1/3)\ b=37{\cdot}7$; $n'=38{\cdot}2$; $b=113{\cdot}3$; $u=447$

Maximum stress on masonry in tons.

Reservoir empty vide equation (E).

$$\text{Maximum stress}=\frac{2\times7900}{16\times113{\cdot}3}\left(2-\frac{3+39{\cdot}6}{113{\cdot}3}\right)=\frac{7900}{906{\cdot}4}\times0{\cdot}95=\frac{7505}{906{\cdot}4}=8{\cdot}3 \text{ tons}$$

Reservoir full.

$$\text{Maximum vertical stress}=\frac{2\times8219}{16\times113{\cdot}3}\left(2-\frac{13\times44{\cdot}7}{113{\cdot}3}\right)=\frac{8219}{906{\cdot}4}\times{\cdot}82=\frac{6739{\cdot}6}{906{\cdot}4}=7{\cdot}4 \text{ tons}$$

$$\tan\theta=\frac{\text{Water thrust}}{\text{Weight reservoir full}}=\frac{150\times150\times4}{2\times9\times8219}=0{\cdot}609\ ;\ \therefore\ \theta=31^\circ\ 21',$$

$\sec\theta=1{\cdot}17$; Maximum vertical stress $\frac{mN}{b}\sec^2\theta=10{\cdot}06$ tons.

The actual vertical pressure reservoir empty (W) and reservoir full (W′) are 7900/16=493 tons and 8219/16=513 tons showing that the weight of water resting on the upstream sloping face when the reservoir is full is 20 tons.

9. Graphical Method of Testing Stability.

The practical profile in Fig 10 will be taken as a concrete example for examination of stability and stress intensity of a dam by graphical method. The procedure is as follows :—

Draw to a suitable scale the dam profile DEE″FBA Fig. 10. From A draw AA′ at right angles to AC and make AA′ equal to $\frac{H}{P}=\frac{4}{9}\times(150)=66$. Draw A′J parallel to CA. Produce AC to meet the water line at K, join JK and A′K. Draw CM and CC′ parallel respectively to AJ and AA′ to meet JK and A′K in M and C respectively. From CM cut off $CS=\frac{90}{P}=40$ and join SD. The area of the triangle DSC gives the water pressure on DC and that of the trapezoid MCAJ the water pressure on CA.

Divide (H) the whole depth of the dam into 5 laminae of equal depth horizontal base lines distant $H/5=30$ feet apart. The weights, centres of pressure and stresses on base lines are to be delineated by graphic statics.

From what has already been stated it will be clear that the pressures P_1, P_2, P_3, P_4, P_5 on the upstrsam faces of the laminae are proportionate to and may be represented by, the mean horizontal widths of the shaded pressure triangle of trapezoids opposite to them and will act at right angle to the face of the dam through the centres of pressure which are the points of intersection of horizontals through the centres of gravity of the shaded pressure areas and the face of the dam.

Similarly the weights W_1, W_2, W_3, W_4 of each lamina may be represented by the mean horizontal width of the laminae of the profile, each of heights H/5 while the pressure due to the weights of each of laminae will act vertically through their centres of gravity in the vertical lines g_1, g_2, g_3, g_4, g_5.

A diagram of forces Fig 10, may now be constructed by drawing a vertical line from any convenient point along which the mean widths ab_1, b_1c_1, c_1d_1, d_1e_1, e_1f_1 of successive laminae of the dam are set off. For convenience the scale is half that of dam profile.

Again from the point 'a' in a horizontal direction set off the mean width ab_2, b_2c_2, c_2d_2, of the areas of figures representing P_1, P_2 and P_3. From the extremity (d_2) of this line, draw a line, d_2f_2 parallel to and in direction of P_4 and P_5 and set off along it d_2e_2 and e_2f_2 the mean widths, of the areas representing these water pressures.

Draw R_1, R_2, R_3, R_4, R_5 which represent the resultant pressures of the masonry weights and water pressures of each lamina.

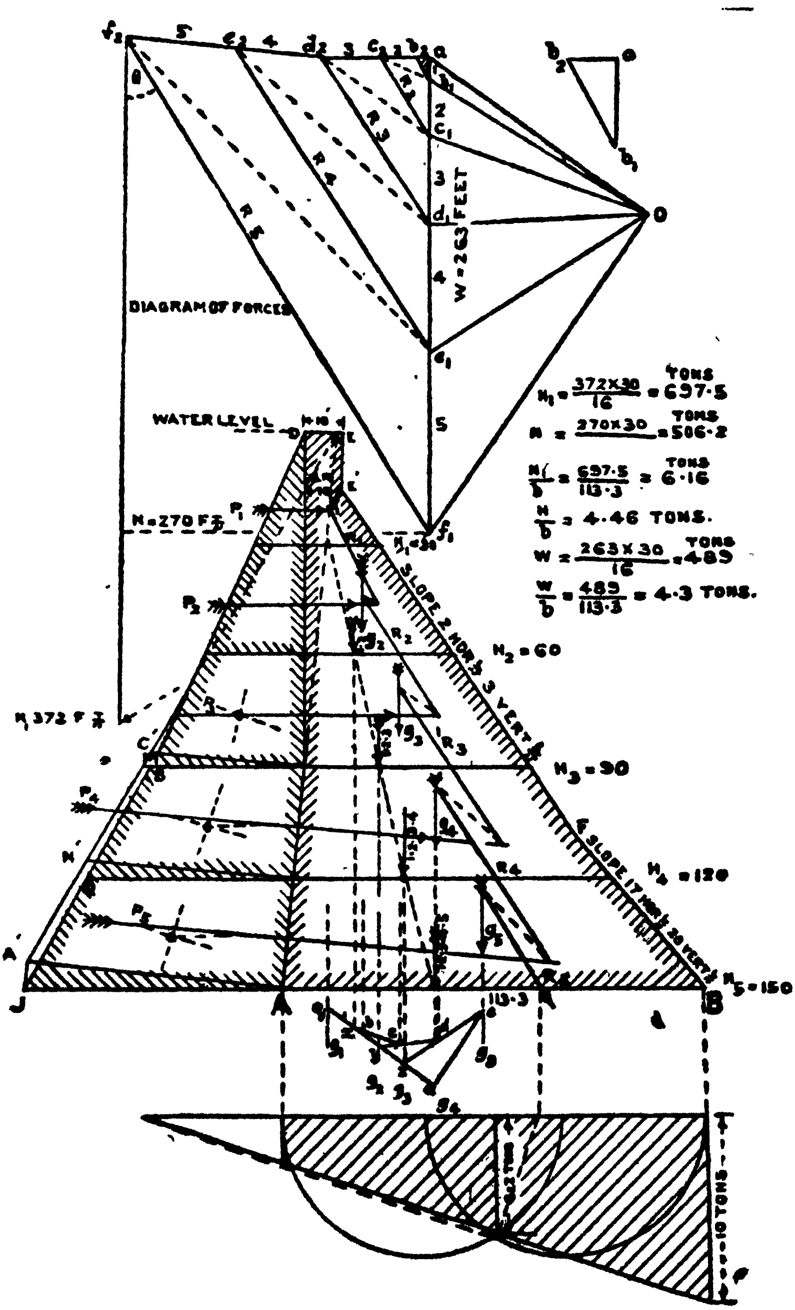


Fig. 10

Draw the four dotted cross lines b_1c_2, c_1d_2, d_1e_2, e_1f_2, joining the lower end of one resultant with the upper end of the next.

Take any convenient point o as a pole and draw the diverging rays from o to each of the divisions of the vertical line af_1.

Take any convenient point 'q' Fig. 10, in the vertical g_1 which passes through the centre of gravity of the uppermost lamina of the dam and produce the verticals g_2, g_3, g_4 which pass through the centres of gravity of the other laminae.

(1) Through 'q' draw qa and qb parallel to the rays oa, ob_1 the latter cutting g_2 in b
(2) Draw bc parallel to the ray oc_1 cutting qa in x
(3) ,, cd ,, ,, od_1 ,, qa in y
(4) ,, de ,, ,, oe_1 ,, qa in z
(5) ,, ea ,, ,, of_1 ,, qa in a

Then a vertical through x passes through the the centre of gravity of W_1 and W_2.
A vertical through y passes through the centre of gravity of W_1, W_2, W_3.
A vertical through z passes through the centre of gravity of W_1, W_2, W_3, W_4.
A vertical through a passes through the centre of gravity of W_1, W_2, W_3, W_4, W_5.

The intersections of these verticals with the bases of the laminae give the intersections of these bases of the resultant pressure 'reservoir empty'.

The lines of resultant pressure 'reservoir full' R_1, R_2, R_3, etc., are drawn in on the dam profile parallel to the similar line in the diagram of forces. R_1 is drawn from the intersection of P_1 with g_1 and is produced to intersect P_2.

From this intersection draw a line parallel to the dotted line c_2b_1 on the diagram of forces being the resultant of P_1P_2 with W_1. From the point where this intersect g_2 draw R_2 on profile line parallel to R_2 on the diagram of forces.

Similarly in succession R_3, R_4, R_5 are drawn on the profile and where these points cut the base lines of the laminae, are points of centre of pressure 'reservoir full'.

The curve of pressure 'reservoir full' it drawn by joining up the points where R_1, R_2 etc., cut the bases of the corresponding laminae while the curve of pressure 'reservoir empty' is obtained by joining up the points where the vertical through the centre of gravity of all the masonry above the base of each lamina cuts that base.

The resultant total stress R_5 is acting on a horizontal base. If the stress on a plane at right angles to this is required, this may be found from the intersection at N_1 in the diagram of force of a vertieal line from the upper extremity of R_5 and a line at right angles to R_5 from the other extremity. This gives the value of $N_1 = R_5 \sec \theta$.

To reduce N_1 to tons the length in feet has to be multiplied by the common height factor $H/5 = 30$ ft. and by $w\rho = 1/16$. Scaling from the diagram

$$N_1 = 372 \text{ ft.} ; \; N_1 = 372 \times 30 \times 1/16 = 697{\cdot}2 \text{ tons.}$$

Mean Pressure $\frac{697{\cdot}5}{113{\cdot}3} = 6{\cdot}2$. A pressure diagram, below the base of the dam profile is constructed as shown in Fig. 10.

Setting off the mean pressure of 6·2 tons and constructing the diagram the maximum pressure resulting, is 10 tons 'reservoir full'.

Analysis of results of the above example. Analysing the result it will be observed that the curve of resultant pressure 'reservoir empty' falls slightly outside the middle third from 30 feet to about 100 feet below water level. This was to be expected from tne fact that the batter of the water face only commence at a depth of 90 feet below water level.

The tensional pressures developed would, however, be so small that the profile need not on this account be rejected.

In all other respects the profile is suitable and the maximum pressure just reaches the limit 10 tons which is the basis of the design.

It will be seen that the graphical results correspond as closely as can be required to those given by arithmetical computation. This method of examining the distribution of stresses has the advantage of more clearly depicting the forces than the more exact arithmetical method, while at the same time it involves less labour.

10. Another Graphical Method of Testing Stability (Haessler).

A simple method of applying the force diagram to the dam profile is adopted in figure 11 which gives a profile of the Wigwam dam (United States).

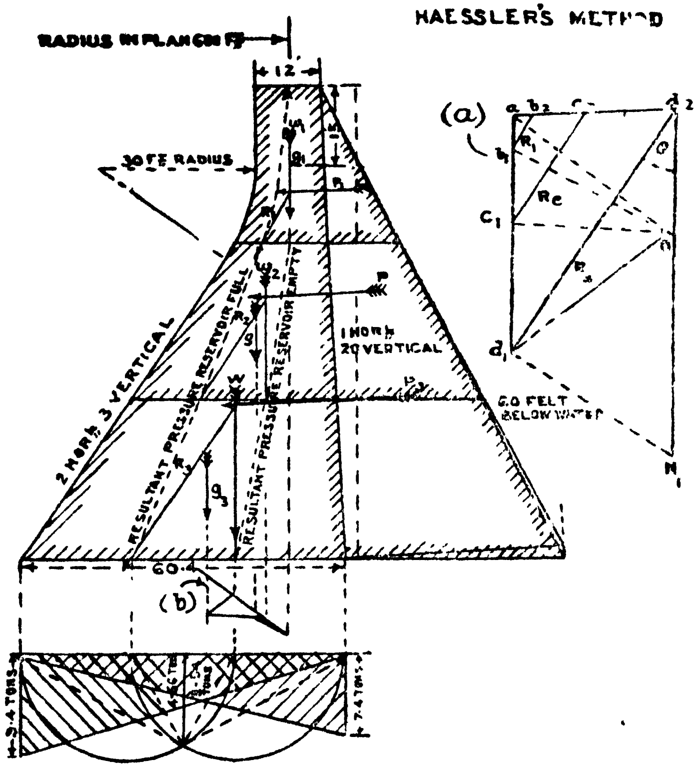


Fig. 11

The dam was designed in 1893 for a water depth of 90 ft. but was only built to a height of 75 feet, the proposal being to raise it subsequently when more water was required.

The diagram is drawn for the ultimate height with a top with of 12 feet and water level reaching the top of the dam. It will be seen by reference to Fig. 11 that the dam has been divided for purposes of examination into 3 laminae and in the method here adopted, instead of computing the pressure on each lamina separately as in the previous case, this has only been done for the top lamina giving a resultant R_1. Then W_2; the sum of the weights of Nos. 1 and 2 laminae acting vertically through their combined centre of gravity, is taken, the line of action being ascertained by graphical construction as in the last case, Fig. 11. The intersection of W_2 with P_2 the line of resultant water pressure on the faces of the two upper laminae, gives the point from which R_2 the resultant, is drawn parallel to R_2 in the force diagram. Again P_3 is the total water pressure on the face of the dam acting at 1/3 its height and the intersection of this with the vertical W through the centre of gravity of the whole gives the point through which the resultant R_3 should be drawn.

As before, the pressure N_1 which is $R_3 \sec \theta$ may be found by construction and reduced to pressure in tons by multiplying by $H/3=30$ feet and by $w\rho=1/16$.

The line d_1N_1 scales 130 feet. Therefore $N=130\times30\times1/16=245$ tons (roughly). The mean stress on the base at maximum loading is, therefore, $245/60{\cdot}4=4{\cdot}56$ tons. The maximum stress 'reservoir full' is found graphically as before (Fig. 11) and equals to 9·4 tons.

The pressure on the base 'reservoir empty' in the force diagram is ad=99 feet and $99 \times 30 \times 1/16 = 185{\cdot}6$ tons. Mean stress on base$=185{\cdot}6/60{\cdot}4=3{\cdot}54$ tons and the maximum stress 'reservoir empty' is found graphically and is 7·4 tons (Fig. 11).

Analysing the result it will be seen that if the water level is raised to a height of 90 feet, there will be tension of the upper part from about 15 to 45 feet depth. Below this level the dam is suitable as a gravity dam.

It should, however, be mentioned that the dam, although designed as a gravity dam, is curved in plan to a radius of 600 feet and the designer possibly relied on the support which arch action would give to the upper part of the dam, if the water level were raised to the top of the masonry. The nature of this support will be described in dealing with arched dams It appears, however, more probable that surplusing arrangements would make it impossible for the water to rise over to the top of the dam and the maximum water level would be 5 or 6 feet below the top.

The method employed in this diagram is superior to the previous one in the fact that each resultant is independently drawn for the whole mass above the base of each lamina, so that errors are not cumulative and each result is independent of the others. The mothod is, however, only applicable when the water face is of one slope throughout.

11. Vertcial Shear and the Ellipes of Stresses in the Body of the Dam.

In high dams when base is splayed more than the elementary theroretical section of a dam, it is necessary to test the section in vertical shear stresses. The method is illustrated in an example below :—

Data. Height of dam	110 ft.
Depth of fore-ends	20 ft.
Flood level above top of dam	8 ft.
Top width	10 ft.
Maximum compressive stresses	10 tons per sq. ft.
Specific gravity of dam material	2·4
Batter upstream face	1 in 20
Maximum sheat stress	7 tons/sq. ft.

Foundations from rock can stand pressure 10 tons/sq. ft.

(A) **Base Width.**

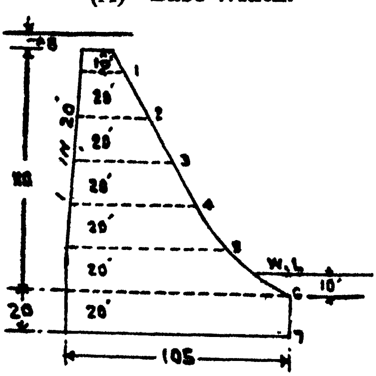


Fig. 12

Total depth$=110+8=118$ ft.

$$\text{Base width} = \frac{H}{\sqrt{\rho - 1}} = \frac{118}{\sqrt{2{\cdot}4-1}} = 100 \text{ ft. say 103 feet.}$$

Generally if the foundations are rocky and if uplift pressure is destroyed by providing cut-off near the bed, we need only allow uplife pressure not more than 40 percent.

$$\text{Base width} = \frac{118}{\sqrt{2{\cdot}4-0{\cdot}4}} = 83{\cdot}5 \text{ ft.}$$

(*i*) Divide the dam in sections as shown in Fig. 12.

(*ii*) Find the weight of the sections and locate the centre of gravity in each case.

(*iii*) Draw the middle third lines.

(*iv*) A line joining points where the verticals from the centre of gravity of section cut the base of the section gives the resultant pressure line when the dam is empty.

(*v*) Draw the pressure diagram above base of each section and than find the resultant of the water pressure and the weights and produce it to cut the base. The line joining all such points at the base of the respective sections, gives the resulting pressure line when the dam is full.

Read and tabulate the eccentricities of the resultant line from the mid-point at the base of each section.

Stress variation, $S = \frac{W}{b}\left(1 \pm \frac{6e}{b}\right)$; where W is Total downward load per ft. width of the dam b is Base width and e is eccentricity.

Table.

No.	Depth of sections	Weight in tons	Water pressure in tons
1	10	9·04	3·62
2	20	41·7	19·4
3	20	93·7	64·4
4	20	165·0	85·0
5	20	256·0	134·0
6	20	365·0	193·0
7	20	505·6	265·0

(B) **Stresses at the base.**

$S = \frac{W}{b}\left(1 \pm \frac{6e}{b}\right)$ where W = 505 tons ; b = 103 ft. ; e = 14·5 ft

$S_{max} = \frac{505}{103}\left(1 + 6 \times \frac{14{\cdot}5}{103}\right) = 9$ tons per sq. ft.

$S_{min} = \frac{505}{103}\left(1 - \frac{6 \times 14{\cdot}5}{103}\right) = 0{\cdot}795$ tons/sq. ft.

(C) **Vertical shear calculations, 30 ft. above base.**

Eccentricity = 7·5 ft. ; Base = 82 ft. Weight above the section = 4600 M. U. (Masonry Units). Considering value of $\rho = 2{\cdot}0$, allowing reduction of 0·4 due to uplift pressure, weight above this section $= \frac{4600 \times 2}{2{\cdot}4} = 3833$ M. U.

$S = \frac{3833}{82}\left(1 \pm \frac{6 \times 7{\cdot}5}{82}\right) = 73$ or 21·5 M. U.

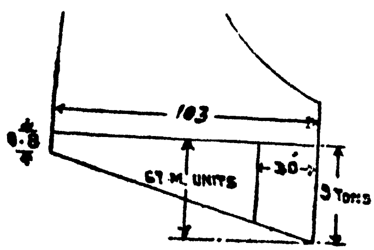


Fig. 13.

The stress distribution is shown in Fig. 13. Take a section at a height of 5′ above the first one

Base, b = 78·8 ft. ; e = 7·15.

Masonry load = 4229 M. U.

Allowing uplift 40% = 3520 M. U.

$S = \frac{3520}{78{\cdot}8}\left(1 \pm \frac{6 \times 7{\cdot}15}{78{\cdot}8}\right) = 67{\cdot}5$ and 19·9 M. U.

Draw the stress distribution diagram Fig. 14. Divide the base line of section 30 ft above base into four equal parts. Find the area of the shaded parts starting from the left and this gives the vertical shear at each point for a depth of 5·0 feet.

Section 1.

Vertical shear at first section $= \frac{3{\cdot}4 + 3}{2}$ 20·5 = 65·6 say 66

Vertical shear per ft. depth = 66/5 = 13·2 M. U.

Section 2. $\frac{3{\cdot}4 + 2{\cdot}5}{2} \times 41 = 122{\cdot}5$

Vertical shear per foot depth $= \frac{122{\cdot}5}{5} = 24{\cdot}5$ M.U.

Section 3. $\frac{3{\cdot}4 + 2}{2} \times 61{\cdot}5 = 167$

Vertical shear per foot width = 167/5 = 33·4 M. U.

Section 4. $\frac{3{\cdot}4 + 1{\cdot}5}{2} \times 78{\cdot}8 = 197$

Vertical shear per ft. depth = 197/5 = 39·5 M. U.

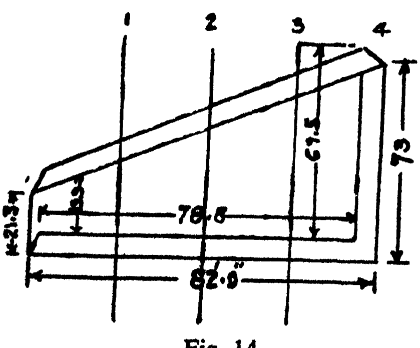


Fig. 14.

i e., $\frac{150\times 39\cdot 5}{2240}=2\cdot 63$ tons/sq. ft. which is safe for masonry

(D) **Horizontal shear calculation for the base.** Masonry load=7540 M. U. ; Effective weight allowing value of ρ to reduce from 2·4 to 2·0 due to uplift pressure $=\frac{7540}{2\cdot 4}\times 2=6283$ M U.

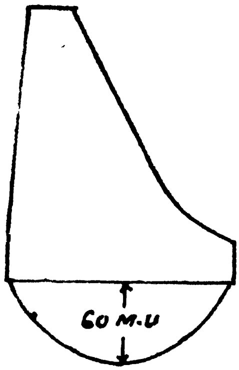


Fig. 15

$$S=\frac{6283}{103}\left(1\pm\frac{14\cdot 5\times 6}{103}\right)=95\cdot 5 \text{ or } 23\cdot 4 \text{ M. U.}$$

Horizontal shear at the base = Horizontal thrust due to water $=\frac{H^2}{2\rho}-\frac{h^2}{2\rho}$

where H=136 ft. ; h = 30 ft. $=3940-188=3752$ M. U.

The unit shear stress is maximum at the neutral axis and zero at the extremes (bottom and top) in a beam section. The shape of shear stress diagram is parabolic. The area of a parabola $=2/3\times x\times b$ approximately where x is the mid-ordinate and b the base.

$\therefore$ equal to $2/3\times x\times 103=3752$

$\therefore$ $x=54\cdot 4$ M. U., say 60 M. U.=4 tons per sq. ft. Draw the parabola at the base with mid-ordinate 60 M. U., to represent the horizontal shear diagram. Fig. 15.

(E) **Test of vertical section.** (30 ft. from toe)

Area of the horizontal shear diagram under the toe 30′ from extreme end = 800 M. U. Water pressure on the right 188 M U. The net horizontal thrust at the vertical section HD. F = 188 + 800 = 988 M. U. Erring on the safe side full uplift pressure is taken as effective and represented by rectangle ABCD. Weight of masonry= HECD. The resultant of uplift and downward weight is shown in masonry units by ABEH equal to 1710 units acting at centre of gravity of the (Figure 16) 17 ft. from R. B. The weight of water above the toe is 58 M. U. ; 7·3 ft. away from BC.

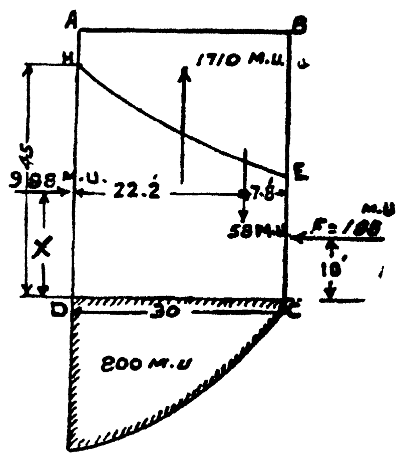


Fig. 16

Taking moment about D of all Forces

$188\times 10+1710\times 17=988x+58\times 22\cdot 2$

$x=31\cdot 3$ ft ; $e=31\cdot 3-45/2=8\cdot 8$ ft. ;

$$S=\frac{988}{45}\left(1\pm\frac{6\times 8\cdot 8}{45}\right)=48 \text{ M. U. (compressive) and}$$

3·8 M. U. (Tensile). The tension is only 0·25 tons/sq. ft. which is neglible.

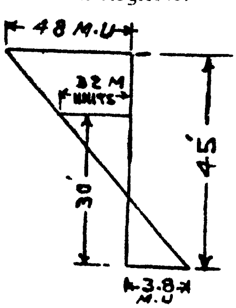


Fig. 17

(F) **Ellipse of stresses at 30′ above base along AD.** $P_y=67$ M. U. from reaction diagram (B) above 30′ from toe $P_x=32$ M. U. (Hoaizontal reaction 30′ above base (E) above) q = 36·5 M. U. (ordinate of shear diagram below base 30′ from toe). Let f be the principal stress $(f-P_y)(f-P_x)=q^2$; $(f-67)(f-32)=36\cdot 5f^2-99f+825=0$; $f=90\cdot 6$ M. U.

$$\tan\theta=\frac{q}{f-P_y}=\frac{36\cdot 5}{90\cdot 6-67}=1\cdot 54\ ;\ \therefore\ \theta=57^\circ$$

θ is the angle which the principal stress makes with the horizontal. The ellipse of stresses will be as sketched in Fig 18.

Similarly other vertical sections may be tested for the horizontal reactions and the principal stresses.

12. Masonry Work of Dam.

The body of the dams must be water-tight ; horizontal cracks or planes must not occur in it, otherwise, water will permeate through the work and les on its weight and stability. Brick work is, therefore, out of question for dams of any height and coarsed masonry or *ashlar* for dams exceeding a moderate height ; for large dams the masonry should be uncursed and should break jo nt in all directions and each stone should have clean irregular surfaces and should be solidly built in and bedded on stiff mortar without grouting. In order to resist vertical and horizontal, pressures, shear building stones drived from stratified rock or in good blocks should, near the face of the dam, be built with their beds alternately parallel to and at right angles to that.

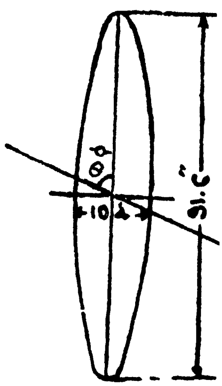


Fig. 18

For the sake of appearance, dams are often faced with heavy coarsed masonry, which increase their strength there, but interferes with the homogeneity of the structure. Such facing should consist of large stones so to reduce the number of joints and should be carefully bounded with the hearting to prevent separation. On the upstream side it should be pointed or plastered with cement to make it as water-tight as possible. This cyclopean method of construction in blocks of two as large as can be handled and set in good mortar has many advantages over smaller stone or cut masonry. It appears quite unnecessary to spend extra money on especially cut stones for face work. A good dam is independent of outside prettiness.

Concrete can be used in the upper parts of the dam, so as to effect economy in cost without loss of stability and water tightness. To ensure these qualities it must have its ingradients carefully proportioned and mixed and must be consolidated free from all vacuities. To increase the weight of the concerete and to bind its layers together rough stones known as "plums," as large as can be handled, are laid in the work ; near the faces they should be placed with their longer dimension roughly parallel there to; else where they should be laid to break joint in every direction. They should be bedded full in mortar, should be malleted down on to their beds and their tops should project into the upper course ; the concrete wriggeld tightly on their sides.

13. Conrete Construction in Concrete Gravity Dams

(A) Mass Concrete. Concrete is a graded mixture of either natural or crushed rock particles embedded in a slowly hardening paste composed of cement and water. A limited amount of water is necessary for a hardening of the cement and total water content is necessary to serve as vehicle for movements of rock particles during placement. Because of the heat-generating qualities of cement during the setting process and the resulting change in volume as this excess heat is dissipated, it is not an ideal structural material. Its cofficient of volume change varies through a wide range of values, depending on the minerals of which the rocks are formed and upon the raw materials from which the cement is manufactured. Compared to brick and building tile, which have a much lower coefficient of expansion, this one of the most objectionable features in concrete as a structural material and one which necessitates the use of joints at every critical section if future troubles relative to expansion and contraction are to be avoided. The student should refer to any standard book on concrete and reinforced concrete construction for detailed specifications of concrete work and its components parts cement, fine aggregate and coarse aggregate and the various mixture with suitable water content For many year in dam construction plum stones or derrick stones were embedded in the concrete as it was placed so as to effect greater economy in the use of cement, but this practice has now been almost abandoned in favour of the more economical and workable method of putting all ingradients through the mixer and thus simplifying placing costs and procedure.

The small smount of mortar in mass concrete is the source of many of its special advantages for massive work These are mainly lower cost and lower generation of heat and temperature rise due to the lower cement content. The corresponding lower water content reduce shrinkage on drying and improves durability. However, to obtain the full measure of these benefits; it is necessary to keep sand content at a minimum The percentage of sand required is surprisingly low, especially with entrained air Tests should be made to determine whether the practical minimum percent of sand is being used, by observing when further decrease produces concrete that cannot be worked satisfactorily.

The practicality of maintaining low sand content depends largely on the uniformity with which the concrete can be produced, handled and placed. Factors that contribute toward this end are: (1) finish screening at the batch plant; (2) good drainage of sand; (3) rock ladders; (4) self-cleaning bins frequently emptied; (5) accurate batching; (6) good mixing; (7) handling without appreciable separation of slump loss; and (8) strong, effective vibration. With these requirements satisfied, and with ample vibration, only narrow margins in sand and mortar content are needed to off set variations in workability.

A problem that requires carefull attention in the construction of large dams in localities where freezing occurs is the placement of concrete somewhat richer in cement in the upstream and downstream faces of the dam. Where this has been done on several Bureau and TVA dams the extra cement, as compared to that in the concrete in the interior of the dams, ranged from 0·20 to 0·40 barels per cubic yard and the thickness of the concrete facing ranged from 4 to 12 feet. A 4 feet facing is regarded as ample to provide the required protection against weathering, but the thickness actually obtained is usually a matter of construction expediency influenced by the character of the placing and dispatching facilities, the size of buckets used, and whether the buckets can dump a part of a batch. An alternative to the use of richer concrete may be the treatment of the surfaces with the vacuum process, which may lower the water-cement ratio an average of as much as 20 percent in the 6 inches of surface concrete. Experience has demonstrated that best results are obtained from the vacuum treatment when (1) the mix contains the practical minimum of fines. (2) newly placed concrete fully covers the vacuum panel so that the vacuum can be applied promptly while the concrete is still plastic, and (3) the concrete near the panel is vibrated during the first few minutes of the vacuum treatment.

Mass concrete is placed in lifts approximately 5 feet deep Each lift is usually made up of four 15 inch layers or three 20 inch layers. It is required that these layers be placed without cold joints. During hot weather it is some times required that the exposed area of mass concrete be maintained at a practical minimum by first building up the concrete in successive approximately horizontal layers to the full width of the block and to the full height of the lift over a restricted area at the downstream end of the block and then continuing upstream in similar progressive stages to the full area of the block.

Since mass concrete is placed with a relatively dry consistency of 1 to 2 inch slump, it is important that it be adequately and thoroughly vibrated into place. Mass concrete should be so vibrated that there will be no doubt as to its through consolidation, batch by batch, as the concrete is placed. Usually it is customary to revibrate the concrete against the forms around the perimeter of each block; and as a result, few rock pockets are found when the forms are removed. However, the absence of rock pockets on the face is not a proof that full consolidation is obtained within the block This can be determined only by watchful inspection as the concrete is placed and vibrated, and by establishing procedures for this vibration that will eliminate the possibility of incomplete consolidasion.

The portions that some times may be suspected of being poorly consolidated are the outward edges of the batches as deposited which have to wait for some time before concrete will be placed against them. Sometimes these edges are left unvibrated until a bucketful of concrete is placed against the during the next advance in the placing operations. Often by that time the unvibrated concrete at the edges becomes too hard to consolidate properly, or there is lack of the systematic vibration that is required to assure thorough consolidation at the junction between the batches. It is best to vibrate fully all parts of each bucketful when it is dumped, sloping the forward edge about 4 : 1, or flater as necessary, to avoid flow on the slopes and overrunning the lower slopes. Thus, no matter what the delay, all concrete in place is fully consolidated.

Delays in placement may occur which result in cold joints within a lift. When placement is resumed while the concrete is so green (and therefore capable of ready bonding) that it can readily be dug out with a hand pick, the usual construction joint treatment will not be required if (1) the surfaces are kept moist by a light spray of water (if necessary), and (2) if the concrete placed against the surfaces is thoroughly and systematically vibrated over the entire area adjacent to the older concrete. As compared with the mortar joint treatment, this procedure involves less interference with normal placing operations, and also enables a saving in cement.

Before the top surface of each lift sets, it should be gone over by a man wearing wooden "snowshoes" which give him two or three times the area of support provided by his ordinary boots or shoes and prevent footprint depressions. Using a small immersion-type vibrator as he steps on protruding large pieces of rock, he can embed them to the level of the surface of the lift. Clean-up is marerially aided if the surface of a lift is reasonably free of protruding boulders, footprint and other irregularities.

(B) **Mixing concrete.** Constant pressure for more speed and better concrete in recent years has resulted in radical improvements in batching and mixing equipment. The measurement of all materials by weight instead of by volume is one of the principal factors contributing to the present high degree of uniformity. In the batching and mixing plants now is general use for dam construction, the aggregate is delivered to the top of the storage bins by belt conveyor and the cement is pumped through pipes The storage bins are above the batchers and the mixers below them. All materials descend through the plant and into the buckets by gravity. Batching equipment is usually air-operated and electrically controlled from a single board. In the latest plants the tilting mixers are charged from a common central collecting cone and discharged through a central common hopper into the bottom dump buckets. The tilting mixer is the only type that will satisfactorily handle mixes containing lumps. The shape of the mixer drum and the shape and location of the blades within the drum, as well as the method and sequence of charging have a marked effect upon the uniformity of the batch. All mixers should be so located and arranged as to permit the operator to view the mixing operation during its progress rather that to judge the qualities of the mix after it is dumped. The proper mixing time for any batch depends on the speed of rotation of the mixer and the other factors mentioned above ; it varies from 1 min. for small mixers to 3 min. for 4-yd. mixers.

(C) **Placing of concrete.** The demand for an increase in uniformity and the elimination of segregation as experienced in the long-used chuting system brought into use the modern appliances such as the cableway, the trestle, the concrete pump and the belt conveyor. The belt conveyor, however, is not permitted on many jobs because of the tendency for segregation and erratic loss of moisture in transit and the consquent effect on the placing qualities of the concrete. The trestle and the cableway are the most common systems of distributing concrete structure. If a cableway is used, bottom dump buckets are transported from the mixer to the cableway on cars ; and the cableway, in order to serve any point on the dam, is equipped with a movable tower at one or both ends, depending on the plan layout of the dam. If a trestle is used, the car transport the buckets from the mixer along the trestle to the point of deposit, where a crane picks the bucket off the car and throws it in the forms.

Concrete should be transported from the mixer as a unit mixture, deposited as near as practicable in its final position and consolidated by vibration with as little segregation as possible. The vibrator, by permitting the satisfactory placement of concrete (too dry to be placed by hand), has done more than any other agency to promote the benefits of the water-cement ratio law ; but even so, vibration has its objectionable features, one of which is the tendency among too many operators to cause objectionable lateral flow within the mass and its attendant segregation, rather than consolidation. There is a definite trend in recent years toward higher speeds in vibrators, but as yet the relative merits of speed versus amplitude have not been well established. In massive concrete dams, the exterior shell generally contains more cement than the interior and a common stipulation relative to the control and placing of this face mix is as follows. "The contrete to a depth of 5 ft. normal to the face shall contain more cement proportion than the concrete in the interior of the dam ; it shall be placed as simultaneously with the adjacent interior mix as plant operations will permit, so that the two mixtures will unite in their plastic state to form an integral mass". The last clause contemplates an overlapping and devetailing of the batches as they are dumped and cosolidated by vibration. The concrete face-mix placing layers are shown in Fig. 19. In order to control the

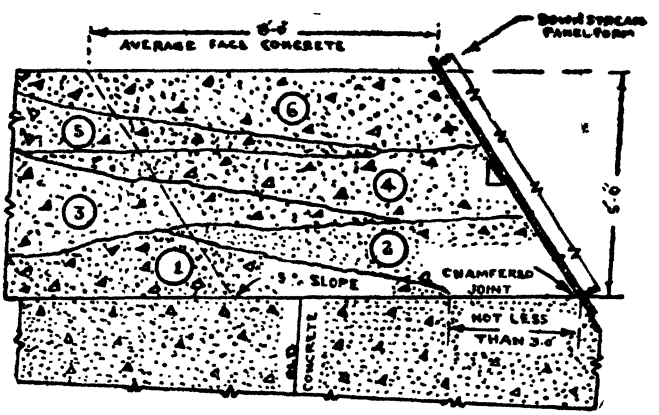


Fig. 19 Sequence of pours (circled numerals) for concrete on downstream face of dam (O. Zaurgaard in Proc. Am. Soc. C. E. March 1941)

maximum temperature in mass concrete it is generally specified that it shall be poured in 5 ft. lifts but multiple-arch dams, slab and buttresses dams and retaining walls are ordinarily carried up in 10 ft -12 ft. or even higher lifts since the dissipation of excess heat from such structures and the prevention of cracks are more readily accomplished. Lifts higher than 5 ft. have been used in mass concrete, but the form work difficulties increase rapidly with height. One of the objections to high lifts, especially in the arches and face slabs of dams, is the tendency toward the accumulation of water and cement at the top of the lift, thus producing a thin band of high water-cement ratio, porous, nondurable concrete. This condition in high lifts can be avoided and statisfactory concrete secured, but to do so requires eternal vigilance on the part of the inspection staff.

(D) **Curing.**

(*a*) **Moist curing.** The water content of fresh concrete is considerably more than enough for hydration of the cement. However, an appreciable loss of this water, by evaporation or otherwise, after initial set has taken place will delay or prevent complete hydration. The object of curing is to prevent or replenish the loss of necessary moisture during the early, relatively rapid stage of hydration. Usual procedure for accomplishing this is to keep the exposed surface continuously moist by spraying or ponding, or by covering with earth, sand, or burlap maintained in a moist condition. Precast concrete and concrete placed in cold weather are often kept moist by steam released within enclosures. These procedures are known as "moist curing". Early drying must be prevented or the concrete will not reach its full potential quality. In warm, dry, windy weather corners, edges, and surfaces become dry more readily. If these portions are prevented from drying, and fully develop their hardness and quality, interior portions of the concrete will have been adequately cured.

United States Bureau of Reclamation specifications require that the concrete be kept moist for a minimum period of 14 days when sulfate-resisting cement is used, and 21 days when low-heat cement, or a combination of modified cement and pozzolan is used. Tests indicate that a period of drying after completion of the moist curing considerably enhances the resistance of concrete to sulfate attack, probably as a result of carbonation.

Bureau specifications further require that concrete to be moist-cured be protected from the direct rays of the sun during at least the first 3 days of the curing period. Wet burlap in contact with the concrete is excellent for this purpose: it not only shades the concrete but also provides effective moist curing. Wood forms left in place furnish good protection from the sun, but should be loosened and wetted thoroughly at frequent intervals so that the water floods the space between the forms and the concrete. Merely leaving the forms in place will not keep the concrete sufficiently moist for proper curing. There is no better curing and protection than that provided by well-moistened backfill. Ponding of floors, pavement, and other slabs is effective in reducing crazing, cracking, and wear.

Water from spray-pipe systems may pick up enough dissolved iron to cause unsightly rust stains on the concrete. This trouble can be avoided if galvanized or alloy pipe is used. Not only are rust stains avoided, but a highly satisfactory and flexible means of curing is provided by use of soil-soaker canvas hose placed advantageously along the higher levels of the work; for instance, along tops of walls, piers and curbs.

Vertical surfaces can be cured very satisfactorily by arranging a system of pipes with spray nozzles at such intervals that the entire surface is covered with a fine spray. For horizontal surfaces the most positive means of securing continuous moist curing is a blanket of saturated sand applied immediately after placement is completed. This method of curing results in a surface that is as near ideal for starting the next lift as it is possible to produce and is equally good for finished surfaces.

(b) **Curing with sealing Compounds.** Under certain conditions it is desirable to cure concrete by applying to the exposed surfaces a sealing compound designed to restrict evaporation of the mixing water. An effective compound, properly applied, will, under most conditions, retain enough moisture for adequate curing.

The present practice on united states Bureau of Reclamation projects is to use sealing compound curing for all concrete in canal linings and related structures. A white-pigmented compound is specified. This compound consistes of a finely ground, white pigment of high hiding quality, dispersed in a vehicle of oils, waxes, or resins, and a solvent. It is furnished ready-mixed under Bureau specifications and should not be thinned or otherwise altered in composition on the job.

When applied at the normally specified coverage of 150 square feet per gallon, a coating of whitepigmented compound presents a uniformly white appearance and effectively conceals the natural colour of the concrete. The white coating considerably decreases the heat which would be absorbed from direct sunlight if the concrete were untreated or if it were coated with the clear or black compounds formerly used. In hot weather this decrease may be as much as 40°. The resulting lower and more uniform temperature materially reduces cracking caused by thermal expansion and contraction. Test data from the Denver laboratories and from the field show that the use of white-pigmented compounds approximates the effect of shading in maintaining low concrete temperatures.

A white-pigmented compound facilities inspection. Uneven distribution of the compound and thinly coated area are readily revealed by a nonuniform or grayish appearance. For this reason the white compound is required in preference to the clear or the black for interior surface of tunnels and siphon barrels where a heat-reflective coating is unnecessary.

Sealing compounds are not considered best suited for curing the concrete in large dams. There is usually plenty of water available at dam sites for moist curing and the prolonged curing period that results from excess curing water flowing down the faces of concrete in lower lifts provides curing which is superior to that obtainable with sealing compouds.

White-pigmented compound must be of such composition that it will remain intact as a sealing coat for at least 28 days. The coverage necessary to insure effective curing varies considerably with conditions It is obvious that the heaviness of the coating (effectiveness of seal) required in the very hot dry climate and would be greater than that needed in the cooler and more humid atmosphere. A smoothly troweled surface requires less compound for a given area than, say, the surface of pneumatically applied mortar. However it is impracticable to set up different coverage rates to apply to all variations in weather conditions and concrete structures, even for one project. Consequently, the Bureau has established a general coverage requirement of 150 square feet per gallon for all conditions except membrane curing of pneumatically applied mortar. Tests and field observations have shown that this coverage, on a reasonably smooth surface, will provide the equivalent of 14 days of continuous moist curing, even in hot, dry climates.

Under rate circumstances it may be necessary to augment sealing compound curing with preliminary moist curing. As an illustration, on one canal lining project considerable horizontal checking was occurring on the side slopes of certain sections. By employing 24 hours of moist curing prior to the application of the sealing compound, this checking was largely eliminated.

Application of sealing compound is usually adequate for proper curing, but if circumstances are such that there is reason to question the adequacy of the curing membrance, preliminary moist curing should be required as an invaluable adjunct and precaution.

The sealing compound is applied in one coat by spraying. To insure thorough and complete coverage, approximately one-half of the compound for a given area should be applied by moving the spray gun back and forth in one direction, and the remainder at right angles to this direction. Spraying equipment should be trapped to prevent moisture or oil from getting into the compound. Ordinary orchard-type, hand-spray outfits are not suitable. Brush application may be used for formed surfaces if desired, but should not be used on unformed concrete, because the compound is applied while the concrete is still soft enough to be marred by the brush. Such marring opens the surface to excessive penetration of the compound and thus breaks the continuity of the film. It is important that formed concrete be well saturated at the time the sealing compound is applied. Promptly after the forms are removed the surfaces should be moistened with a light spreay of water and the moistening continued until the surface will not readily absorb more water. The compound should be applied as soon as the moisture film has disappeared and there is an approach to surface dryness. Special care should be taken with formed concrete to see that edges, corners and rough spots are well covered with compound. The compound should be applied before any patching is done.

Continuity of the coating must be maintained for at least 28 days after its application. Whenever the curing membrane will be subject to damage from traffic or other cause, it should be protected, after drying for 24 hours, by a layer of sand or earth not less than 1 inch in thickness, or by other suitable and effective means. Any damage to the coating within the 28 days period must be repaired without delay by a liberal application of compound.

Sealing compound should be mixed thoroughly in the containers before it is withdrawn for sampling or for use in the work. If not adequately mixed, the sample will not be representative of the contents, nor will the compound give satisfactory results in service.

Shipments of sealing compound are sampled by the Government and the samples tested in Denver for approval prior to use. Whenever practicable, sampling is done at the shipping point to avoid possible delay in operations after receipt of the material at the project.

Sealing compounds manufactured in accordance with Bureau specifications are, under average temperature conditions, of the proper consistency for application. In cool weather the compound may become chilled and too viscous for satisfactory application. In this case it is permissible to warm the compound to reduce its consistency, but it must not be thinned. Heating should be done by placing the container in a hot water bath, never over an open flame. The container should be vented, and only about three fourths full, to allow for expansion. The compound should be heated enough to allow a reasonable margin for subsequent cooling in the feed tank, but not over 100° F.

(C) **Steam curing.** Use of steam curing is particularly advantageous under certain conditions, chiefly beeause of the higher curing temperature and the fact that moisture conditions are ideal. This type of curing is used by the Bureau in the manufacture of precast pipe. Its benefits are also realized in connectoin with the use of live steam for cold-weather protection of concrete. Steam-cured precast units attain strength so rapidly that the form can be removed and reused very soon after concrete placing.

Greatest acceleration in strength gain and minimum loss in ultimate strength are obtained at temperatures between 130° and 165° F. Higher temperatures produce greater strengths at very early ages, but there are severe losses in strength at ages greater than 2 days. Precast concrete pipe is usually cured at temperatures ranging from 100° to 150° F. Under such conditions the loss in ultimate strength is relatively small. Use of steam curing in winter to enable early form removal rarely involves temperature in excess of 100° F.

A delay of 2 to 6 hours prior to steam curing will result in higher strength at 24 hours than would be obtained if steam curing were commenced immediately after filling of the forms. If the temperature is between 100° and 165° F. a delay of 2 to 4 hours will give good results : for higher temperatures, the delay should be greater.

It is desirable that the insides and outsides of pipe sections be simiutaneously exposed to the steam curing (and on both sides of other concrete sections), especially in cold weather, in order to avoid stress-producing temperature differences in the concrete.

The necessary duration of steam curing depends on the concrete mix, the temperature, and the desired results. Pipe is commonly stripped at 12 hours, tipped off the base rings at 36 hours, and considered fully cured at 72 hours.

(E) **Joints horizontal and vertical.** Joints in dams are a necessary evil. They are necessary to permit systematic and economical construction and to prevent the formation of haphazard and ragged cracks. The proper treatment of horizontal lift joints is one of the most questions in mass concrete construction. If the concrete immediately below the joint surface is of normal consistency and there is no accumulation of water (water gain) and fluffy inert cement as the lift is completed, there does not appear to be logical reson for any treatment of the surface prior to starting another lift except to wash it off If, however, there is an accumulation of water and cement immediately below the surface of the lift this should be removed to such depth as is necessary to expose concrete of a high velocity jet of water and air applied at the proper time during the setting period. Once the concrete is hardened, clean-up is best accomplished by the use of the wet-sand blasting process, which will remove any undesirable material effectively and economically. Regardless of what kind of treatment the joint receives before an other lift is started. 1/2 inch of mortar should be applied immediately before concrete placing begins, to permit the proper bedding of the aggregate in the fresh concrete and the proper bond of old and new concrete.

(F) **Temperature control** There are millions of volume-change cracks due to temperature ; and hence temperature is fast becoming the most important problem to deal with in the construction of a dam. Deep cracking is all too prevalent, but even so, it does no deserve as much attention as surface cracking and checking because these are the entering wedges of wholesale disintegration. Deep cracking is caused by high interior temperatures, which create steep temperature gradients between the interior and the surface for long periods of time, while surface cracking and checking is due to high daily differentials in temperature between the surface and the adjoining surface areas. Surface cracks, once started, may progress into the interior or clear through the structure, depending on conditions.

The following are some of the various operations that may be incorporated into the construction programme for a dam as a practical solution of the elimination of cracks : (1) starting off all rock foundations or concrete surfaces that have set for several weeks with two or three low lifts (2½ ft. lifts) and 5 days between lifts ; (2) limiting the height of all other lifts to 5 ft. and 5 days between lifts ; (3) sprinkling the coarse aggregate and blowing compressed air through it in summer to standardize the moisture content and reduce the temperature ; (4) refrigeration of the mixing water, including use of ice if it can be properly batched and discharged into the mixer ; (5) use of low-heat cement ; (6) use of low cement content for interior and relatively higher for exterior or surface shell ; (7) circulating cold water or ice water through pipes embeded in the concrete of each lifts as soon as the pipes are covered and until the temperature of the mass has been reduced to mean annual temperature for that locality ; (8) control of form removal so that high differentials in temperature between the surface and near surface areas do not take place The size of the dam, the amount of concrete and the form in which the concrete is placed will be important factors in determining how many of these operations are applicable.

(G) **Use of pozzolans.** Pozzolans are siliceous materials, natural or artificial, processed or unprocessed, which, though not cementitious in themselves, contain censtituents that will, at ordinary temperatures, combine with lime in the presence of water to form compounds which have a low solubility and possess cementing properties. When pozzolans are interground with portland cement clinker, the cement is a portland-pozzolan cement. Use of pozzolans as replacements for part of the portland cement in concrete has come into prominence in recent years and is rapidly increasing. The Bureau is giving serious consideration to all opportunities to realize substantial benefits by requirement of the use of pozzolan. Pumicite was used in Friant and Altus Dams, and calcined shale is an ingradient of the concrete being placed in Davis Dam. Specifications for Hungry Horse and Canyon Ferry Dams provide for use of pozzolan Although the examples cited involve are major concrete dams, the general practice will be to use pozzolan wherever suitable material is economically available and substantial benefits can be realized.

Essentially, the advantages of using pozzolan are improvement in the properties and saving in the cost of concrete. Use of suitable pozzolan results in concrete having better workability, less heat generation, maximum rate of heat development at an earlier age, less water gain and segregation of solid ingredients, lower permeability, greater resistance to aggressive soils and waters, and with certain pozzolans, reduced expansion from chemical reaction between alkalies in the portland cement and reactive components of the aggregates. The amount of savings in concrete cost depends on costs of producing and shipping the pozzolan, which vary widely with the material and the locality from which it is obtained.

Substitution of pozzolan for a considerable portion of portland cement not only results in a low cement content which in itself, is highly desirable, but also compensates for any undesirable characteristics of the concrete that would be introduced if the cement content were similarly reduced without addition of the pozzolan. Caution must be used in selection and use pozzolan and portland-pozzolan cements because their properties are widely variable and some introduce adverse qualities into the concrete, such as excessive drying shrinkage and reduced strength and durability.

Pozzolans can be classified on the basis of their mineralogic and petrographic composition, as follows : –

(1) Volcanic ashes and tuffs (including pumicite) of rhyolitic, dactitic, or andestic composition.

(2) Siliceous sedimentary rocks, such as diatomaceous earth and opaline shales and cherts.

(3) Burnt clays and shales.

(4) Industrial by products, such as blast-furnace slag, fly ash, and ground brick.

Except for rare occurrences, such as the pumicite near Friat, Calif ; all natural pozzolans must be ground before use. The clayey pozzolanic materials, including altered volcanic ashes and tuffs as well as shales, must be calcined at temperatures between 1000° and 1700° F., to activate the clay constituent and decrease water requirement of the pozzolan.

The pozzolans known to effectively control alkali-aggregate reaction even where reactive aggregate and high-alkali cements are used, include :—

(1) Highly opaline material, such as diatomaceous earth and opaline chert.

(2) Certain volcanic glasses.

(3) Certain calcined clays.

(4) Certain highly siliceous industrial products, such as pyrex glass and silica fume, a by-product of magnesiurr metal production.

A calcined shale is being used as a replacement for 20 percent, by weight, of the cement at Davis Dam. Use of this shale was primarily occasioned by the unusually severe chemical ieation between the constituents of the aggregates and the alkalies in the cement. In this case the usual limitation of 0·6 percent for cement alkali content was considered inadequate as a remedy. Cement having substantially lower alkali content was unobtainable, but a calcined shale pozzolan highly effective in reducing expansion from the chemical reaction, was available and is being used.

The pozzolan for Hungry Horse Dam is to be a "fly ash", which will replace about 30 percent, by weight, of the cement. In this instance, alkali-aggregate reaction was not a factor, the outstanding reasons for requirement of pozzolan being the saving in cost and the reduction of heat generation in the concrete.

(H) **Design of concrete mixes.**

(*a*) **General comments.** The design of concrete mixes is the determination of the most economical and practicable combination of available aggregate, cement, water, and in some cases admixtures and or pozzolan, that will produce a mixture having the required degree of workability and will develop the required qualities on hardening. Requirements for concrete of good quality should not be relaxed because the structure to be built is small. A case in point is the small canal structure, the thin sections of which are in many cases subjected to exposures of extreme severity.

The most practical procedure for determining the final mix proportions is actual trial and adjustment on the job. The problem thus becomes a matter of selecting a trial mix, for initial use, that will require least adjustment. Design of trial mixes can be accomplished most effectively by laboratory investigation, whereby the concrete-making properties of the materials to be used are determined. For work of considerable magnitude, for unusual conditions or materials, or where strength is a controlling factor laboratory tests are particularly desirable. The minimum laboratory programme which will permit effective trial mix designs, includes tests to determine the grading, specific gravity, absorption, and moisture content of the aggregates. The specific gravity of portland cement may be taken as 3·15 without appreciable sacrifice in accuracy.

The specific gravity of pozzolans is usually about 2·50 but may vary considerably from this value.

(*b*) **Properties of concrete determined by trial mixes.** Laboratory tests of concrete, made to establish important properties and relationships, include.

(1) Strength, elasticity, and durability developed with variations in W/C and mix proportions.

(2) Workability characteristics for various combinations of the ingradients.

(3) Influence of aggregate gradings on cement and water requirements and, if pozzolan is being considered for use, the effects of types of appropriate pozzolanic materials on the properties of the concrete.

(4) Heat of hydration, temperature rise, valume change under various conditions, and thermal properties of mass concrete, if massive sections are involved.

In addition to the design of trial mixes, laboratory tests provide information that is useful in the design of the structures and in establishing construction procedures, also for evaluating economically the different combinations of materials. When it is impracticable to conduct such tests, trial mixes for starting concrete operations can be selected by judicious application of relationships established by tests and experience.

(*c*) **Procedure for the design of concrete mixes.** The seven steps involved in the determination of a trail mix for initial field use are :

1. Select the water-cement ratio from test data, experience, or established relationships to meet the specified requirements for durability and strength.

2. Select the limits of slump that will permit proper handling and consolidation of a concrete under job conditions involved.

3 Determine the largest size of available aggregate suitable for use under job conditions.

4. Estimate from test data experience, or established relationships, the minimum percentage of sand that will provide the proper degree of workability.

5. Estimate the amount of water per cubic yard of concrete that will be required to fulfill the conditions of steps (2), (3), and (4).

6. Select the amount and kind of air-entraining agent that will be required to fulfill the conditions of steps (1) to (5).

7. Compute the trial mix proportions that conform with the factors determined in steps (1) to (6). Make such adjustments in the trial mix on the job as necessary.

(*d*) **Selection of water-cement ratio.** Water-cement ratio should be chosen on the basis of the required durability and strength.

The water-cement ratio for the required strength should be determined by laboratory tests. When it is not practicable to make such tests, the water-cement ratio for the required compressive strength may be taken directly from standared table used on similar jobs which gives conservative values derived from a large number of tests with typical materials. Where strength is the controlling factor and it is desired to use pozzolans, air-entraining agents or other admixtures, it is particularly desirable to determine compressive strength and durability by actual laboratory tests. The lower of the two water-cement ratios (a) that required for durability, or (b) that required for strength as determined by tests, should always be used.

14. Failures of Masonry Dams.

Owing probably to the fact that masonry dams are rarely constructed without careful technical consideration, actual failures are but few. Nearly always a very long masonry dam which is straight in plan, has cracked and it may be laid down as a general rule that for this reason, at least, if for no other, the plan of a dam should be slightly curved.

Of actual failures, the most instructive are the Puentes in Spain, the Bouzey in France and the Austin dam in Texas.

Puentes Dam Failure. The failure of the Puentes dam is more interesting from the point of view of the properties of permeable strata, than from that of actual dam design. The following description and quotation are taken from Parker's 'The Control of water' page 389.

Sketch No. 20 shows the dam to be of unscientific design which is not surprising in view of the fact that it was constructed in 1784—1791. The design, nevertheless, is a safe one, since no tension occurs and the maximum compression is 16,250 lbs. per square foot.

The dam is founded on piles 22 feet long, driven into a bed of sand and gravel, at least 25 feet deep. The original design contemplated reaching solid rock, but this was found to be impossible.

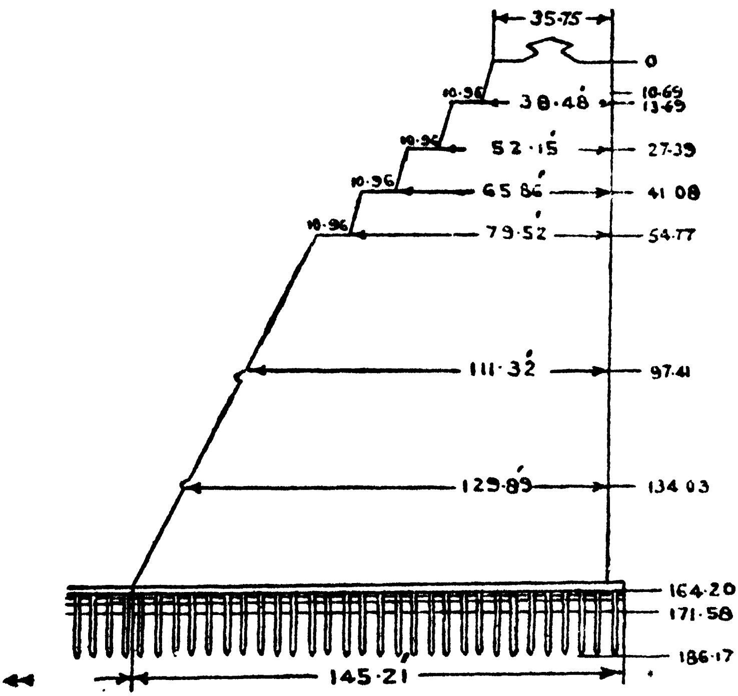


Fig. 20.

The piling and the slab of masonry 7·4 feet in thickness were continued for 131 feet (40 metres) below the dam, thus securing an impermeable coating 283 feet in breadth.

For 11 years the water never rose more than 82 feet above the top of this apron and the dam showed no signs of failure, the apron evidently being sfficiently broad to prevent deleterious precolation under this pressure.

In 1802, however, the water rose to 154 feet above the level 164·2 (*i. e.,* to the full supply level) and as stated by an eyewitness ;

"On the downstream side of the dam towards the apron, water of an exceedingly red colour was boiling up in great quantities." "Half an hour later, this boiling up had increased to an enormous mass of water and a definite passage was formed."

The dam still remains (1864) like a bridge, the opening being about 56 feet broad by 108 feet high.

I think that it is impossible to describe a failure due to percolation more clearly. The only doubt is as to whether the apron, shown as 131 feet broad, was blown up or whether there was some portion of the sandy pocket which was not covered by the apron.

In any case, it appears that the dam failed owing to the percolation being so excessive as to lift up the sand actually. The example is unusually interesting, because, so far as I am aware, most percolation failures can be traced to the foundations being too narrow. Where as in this case, the foundntions were evidently too shallow. Their effective depth, according to my rules being about 23 feet, under the assumption that the impermeable core wall is 7 feet deep at 131 feet from the centre, of the combined dam and apron.

As no Engineer is likely to be so reckless as to construct a stone dam on permeable foundations any further comment is needless.

Bouzey dam failure. The bouzey failure presents several interesting features. The detailed section taken from *Langlois Rutpure du Barrage de Bouzey*, is as per Fig. 21. So far as can be gathered from the figure and reports, the law of the middle third was unknown to the designer, who appears to have been satisfied, provided that the line of the resultant pressure fell within the section of the dam and that the maximum pressure did not exceed the working compressive stress of the masonry. Consequently the masonry seems to have been considered as capable of resisting tension and certain limiting values, both for the tensile and the compressive stresses, appear to have been laid down. From certain statements, these seem to have been 3075 lbs. per square foot (1·5 kilogram per square centimetre) in tension, and 20,500 lbs., per sq foot (10 kilos. per sq. centrimetre) in compression. At any rate, some limits were assigned and were evidently determined by experiments previous to construction. The tensile stress assumed to be permissible appears very great, but there is no doubt that the dam did actually sustain tensions exceeding 2000 lbs. per square foot.

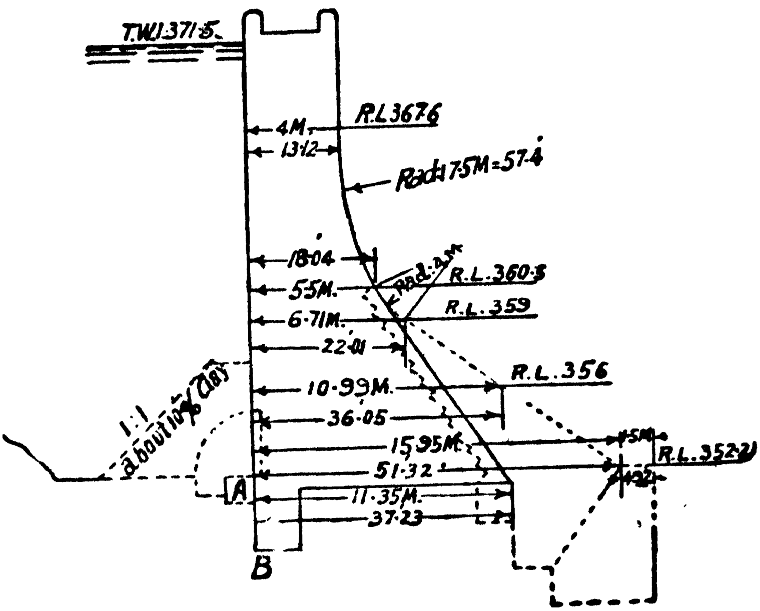


Fig. 21.

The dam was founded on a layer of micaceous sandstone, travesred by seams and fissures of clay The guard wall AB was intended to be carried down through this fissured rock into a compact and impermeable stratum. So far as can be ascertained, this was not effected in many places, owing to lack of pumping power and in certain spots where an impermeable stratum was reached this was only a layer and a permeable rock was known to exist at a still greater depth.

Under such circumstance it is not surprising that in 1884 (when the water level reached the 368·80) the dam cracked, moving bodily forward in places. Later, the reservoir was emptied and it was found that the dam had separated from the guard wall, over a length of 453 ft. and had moved backward as much as 14 inches at certain points.

The gravity of the situation does not appear to have been fully realized. The cracks were closed with grout and the repairs, as sketched in dotted lines, were carried out. These can hardly be supposed to have checked percolation along the crack in the guard wall and the consequent uplifting pressure.

The reservoir was again filled and all seems to have gone well, except for certain leaks, until 1895. Then the water level rose to the originally designed full supply level 371·5 and the dam cracked horizontally at the levels 361·5 and 358 and failed through the upper portion being swept away over a length of 600 feet. This length was quite distinct from the 345 feet that had previously failed.

It would, therefore, appear that a dam designed with only one half the factor of safety usually existing, fails, when besides heavy tensile stresses, percolation occurs. It seems unnecessary to add that both the stone employed and the mortar used, are stated to have possessed only about two-thirds of the usual strength.

Asutin dam failure. This failure is discussed by Gillete (Engineering News, May 30, 1901). The dam was founded on weak limestone, stratified in nearly horizontal layers. This was known to be leaky and water under pressure was found by boring into the foundations during construction.

The leaks which developed after construction were staunched by careful treatment with clay and in view of the large quantity of silt deposited in the reservoir, during its seven years of life, there is little doubt that the leakage and the permeable strata that probably existed beneath the foundation of the dam had little influence on the ultimate failure; although, no doubt, they could hardly be conducive to sound sleep on the part of the responsible engineer.

The circumtances existing at the time of the failure are shown in Fig. 22 (except for the unknown depth of the silt deposit upstream of the dam).

The horizontal pressure of the water on the upstream face of the dam was equal to 181,500 lbs. per lineal foot of the dam. The back pressure of 37 ft. depth of water below the dam is equivalent to 42,800 lbs. per lineal foot ; so that the net force producing sliding is 138,700 lbs. per lineal foot.

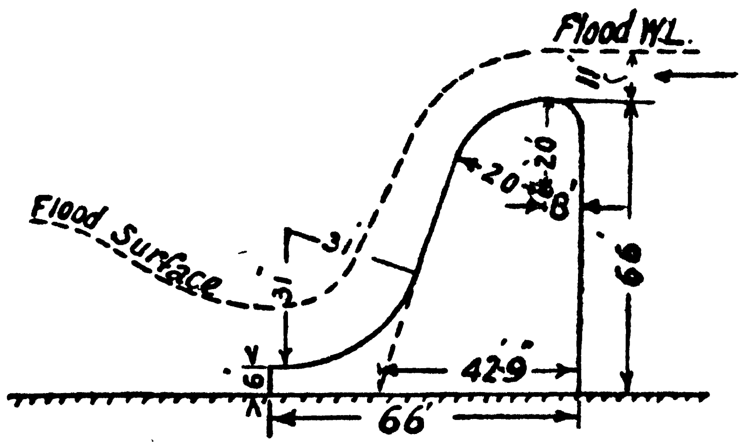


Fig. 22.

Assuming 145 lbs. per cubic foot as the weight of the masonry, the weight of the dam was 320,000 lbs. per lineal foot.

Weight of water, 11 ft. × 18 ft. in the crest of dam = 1300 lbs. per lineal foot.
Weight of water, 11 ft. × 40 ft. on slope of dam = 2700 lbs. per lineal ft.
Weight of water, 20 ft. × 30 ft. on curved toe = 37000 lbs. per lineal ft.
Total 361,000 lbs. per lineal ft.

Thus even with this large over-estimate of the water loads on the dam, we obtain the ratio against sliding as 0·38 and if we allow for a back pressure of only 26 feet depth of tail water, the ratio is 0·44.

Now, we know that there was a fault in the strata under the base of the dam, so that it can hardly be supposed that the limestone rock had any cohesive strength near this fault; and if not faulted, such a rock has little cohesive strength along its bedding planes.

Morin gives the co-efficient of friction for limestone on limestone as 0·38 and for stone on wet clay as 0·33 so that it must be acknowledged that there was very little, if any, margin against sliding, unless the limestone had some cohesion.

The reports on the failure of the dam, (which was seen and photographed in a very complete manner), seem to show that a length of about 500 feet did actually break away, and moved bodily downstream.

15. Temperature Stresses in Dams.

The question of the influence of temperature on the stability of a dam is puzzling. The coefficient of expansion of concrete is about 0·0000076 per 1 degree Fahr., while the value of Young's modulus ranges between 1,400,000 and 2,800,000 lbs. per square inch. Thus a fall of 20 degrees Fahr., below the temperature at which the concrete was deposited should theoreti cally produce tensile stresses ranging from 210 to 420 lbs. per square inch. Consequently variations of temperature occuring in any but the most equable climate (if they really penetrate into the substance of the dam) should cause cracks of an appreciable width (*e. g.*, 3/16 inch wide for each 100 feet length of the dam, if a fall of 2° Fahr., occurs and causes rupture).

So far it is the only case where cracks of a width equal to that indicated by the above calculation have actually been observed.

The facts may be explained as follows. The interior body of the dam probably does not experience an alteration of more than 20 degrees in temperature during the whole of its existance. The outer portions, especially those near the downstream face which experience greater ranges of temperature and are, therefore, called upon to sustain severer stresses, are prevented from cracking noticeable by the support of the main body.

Consquenty the shortning produced by the change in temperature is largely obsorbed in producing a slight flattening of the curve of the dam ; and the cracks are probably due more to differences in the expansion and contraction of the various portions of the dam, than to a more or less uniform contraction of the whole length of the dam. It should be remarked that Mouche Dam is straight in plan.

If the above reasoning is correct, we may deduce the two following principles.

(*i*) Dam must be curved in plan.

Arch and buttress dams, although theoretically economical must be regarded as more liable to temperature cracks than the usual type aud arming at the haunches of the arches with rods or beams should be held to be essential.

All the measurements of temperature deflections which have been made, appear to confirm this statement, although it is incorrect to consider the deflections produced by bending loads (such as the water pressure) as absolutely comparable with those resulting from extensions or compressions arrising from changes in the temperature.

As examples, De Burgh (P. I. C. E., Vol. 178, P. 64) reports that in the very thin Barren Jack Dam, a change in the temperature from 57 degrees to 100 degrees Fahr., produce an inward deflection of 0·14 inch (the reservoir being empty). The water load is stated to have produced an outward deflection of 0·47 inch. So also in the case of the far thicker Vyrnwy Dam (P. I. C. E., Vol. 115, P. 117) the deflection due to temperature does not exceed 0·366 mm ; and that caused by the load produced by the upper 13 feet of water retained, is not more than 0·868 mm.

In the Remschied Dam Inze reports temperature deflections $\frac{1}{8}$ inch and load deflections equal to $1\frac{1}{18}$ inch.

The ideas collected by Gower and others on the effect of temperature changes in American Dams, are very complete. Nevertheless, they cannot be considered as universally applicable since the climate of the United States in such that the difference between the temperature during the period of the deposition of the concrete, or masonry (*i. e.*, May to Nov.) and the lower temperature ever experienced, is far greater than that which occurs in other localities even when the same annual range of temperature is experienced. In most of these localities the working season is usually the colder portion of the year ; whereas in America it is the hotter portion. Thus American circumstances are unusually favourable to the production contraction cracks in masonary or concrete.

The general results are as follows :—

(*i*) The actual expansion on the surface of the concrete or masonry is about 0·6 of that calculated from the range of temperature experienced and from the co-efficient of expansion of similar concrete or masonry in small specimens.

These values are :—

For concrete, 0·0000054 to 0·0000081 ; For concrete exposed to alterations in humidity also 0·0000044 ; for masonry, 0·0000050 to 0·0000060.

The co-efficient of expansion actually observed by Gower in thick granite masonry ranged from 0·0000052 to 0·00000264 with a mean value of 0·00000307 per degree Fahr., and Dana finds that for granite the co-efficient of expansion ranges from 0·00000440 to 0·00000480 per degree Fahr. The differences are possibly largely explained by alterations in the humidity of the masonry.

(*ii*) The actual visible cracks which occur, rarely account for more than 0·6 of the theoretical contraction. (In the Asswan Dam the ratio is only 0·2 and it is highly probable that tensile stresses of 300 to 400 lbs. per sq. inch exist on cold days).

(*iii*) The actual temperature measurements in the interior of the Boonton Dam indicate that the annual variation of temperature at a distance D feet from the face of the dam, is represented by $x = \frac{R}{3\sqrt[3]{D}}$ where R is the annual variation in the surface temperature of the dam.

It wood also appear that unless x exceeds 60 degrees Fahr., cracks either do not occur or are not sufficiently wide to permit water to pass through them.

Cardew (P. I. C. E. Vol. 152, P. 239) attempted to provide for temperature stresses in the Burrage Dam, by burying iron rails in the top. The principle appears to be correct, as the rusting of the iron cannot influence the stability of the dam as a whole. If cracks are prevented in a thin top, where temperature changes are most marked, there is little likelihood of there starting in the interior of the dam.

16. Examination Question.

1. Sketch a suitable section of a gravity masonry dam 60ft. high on pervious foundations. Find the maximum compressive stresses when the reservior is empty and when full up to top level and the diagram below the base in each case.
2. Describe briefly the advantages and disadvantages of masonry dams as compared with earth dams.
3. Describe the conditions governing the selection of site and alignment of masonry dams.
4. (*a*) What kinds of masonry dams are usually built ?
   (*b*) What conditions govern the height of low gravity dams ?
5. Explain the requirements and conditions which must be satisfied for stability in the case of gravity dams.
6. Explain by giving sketches how a masonry dam be designed safe against the following in the case of pervious foundations.
   (*a*) Uplift pressures. (*b*) Full gradients.
7. Sketch a suitable section for a gravity masonry dam 60 ft. high on impervious foundations. Find the maximum compressive stresses developed in the masonry when the reservoir is empty and full to the top and draw the diagram below the base in each case ? (*P. U. 1941*)
8. Discuss the relative advantages and disadvantages of :—
   (*a*) Tanks in series and isolated tanks.
   (*b*) Regulation of supply to dam sluices by plug and rod arrangement and by sluices operated by gear rods.
   (*c*) Gravity and arched dam.
   (*d*) Perennial and Inundation system of irrigation from dams. (*Mysore 1941*)
9. Sketch a suitable section of a gravity masonry dam, 60 ft. high, on impervious foundations. Find the maximum compressive stresses developed in the masonry when the reservoir is (*i*) empty (*ii*) full. Draw the stress diagram below the base in each case. Take specific gravity of masonry as 2·25.
   (*b*) What conditions govern the height of low gravity dam. (*P. U. 1951*)
10. (*a*) What is "elementary profile of a masonry dam".
    (*b*) Prove that the base width of an elementary profile is $b = H\sqrt{P}$ where H is the height of the dam and P the specific gravity of its masonry.
    (*c*) If the sp. gravity of the masonry is $2\frac{1}{4}$, design the profile you would use for a dam of height 80 feet. Sketch a dimensioned sketch. (*P. U. 1949*)
11. What considerations should guide an Engineer in his choice between (*a*) A storage dam and a river weir (*b*) A masonry dam and an earthen dam (*c*) An aqueduct and a superpassage. (*P. U. 1949*)

12. What conditions must be satisfied in order that a high dam (680 feet height) may be constructed in a river gorge having a flood discharge of 3 5 lacs cusecs.

What would be the type of such a dam and what will be its principal features to permit of 25000 cusec irrigation flow and 200,000 kilowatts of power.

13. Give the relative advantages and disadvantages of masonry dams versus concrete dams and give your opinion with reasons as to which type is better in the height range of 300 ft. *(P. U. 1953)*

14. Describe and sketch the plant layout proposed to be used for the construction of Bhakra Dam *(P. U C. 1954)*

15. (*a*) Enumerate different types of dams and state what factors govern the selection of types of dams.

(*b*) Describe the procedure being followed for concereting Bhakra dam. *(P. U. 1956)*

16. What steps will you take to improve the foundations of a dam which are raby and which have weahers bands. *(P. U 1951)*

17. Bibliography.

1. "Design and construction of Rolled Earth Dams," by R. R. Proctor. Eng. News Record August & September 1933.
2. "Earthquake resistant Construction," by H. D Dewell Engineering News Record, April, 1928
3. "Water pressure on Dams during Earthquakes," by H. M. Westergard Trans Am. Soc. Civil Engineers 1933.
4. "Engineering for Dams", by W. P. Creager ; J. D. Justen and J. Hinds, Chapman & Hall Ltd. London, 1944.
5. "Wave Pressures on Sea Walls and Break Water", by D. A. Molitor, Trans. Am. Soc. Civil Engineers 1935.
6. "A Treatise on Masonry Construction", by I. O. Baker, John Weley & Sons 1910.
7. "Dams and Weirs" by W. C. Bligh, American Technial Society Chicago 1918
8. "Irrigation Manual" by W. M. Ellis, College of Engineering Manual Madras, India 1931.
9. "Stresses in Masonry Dams", by W. Cain, Trans., A. S. C. E , 1909.
10. "Flow of Heat in Dams", by R. E. Glover, American Concrete institute Journal, December 1934.
11. "Uplift pressure in Gravity Dams," and "Temperature variations in Concrete Dams" by I.E. Houk, Western Construction News U. S. A. 1930.
12. "Security from under seepage, Masonry Dams on Earth Foundation" by E. W. Lane. Trans. A. S C. E., U. S A. 1935.
13. "Dams stresses and strains studied by Slice models" by J. L. Savage Engineering, News Record U S. A. 1934.
14. "Special cements for Mass concrete" by J. L. Savage, bulletion 2 (d) International Commission on Large dams, world power conference 1936.
15. "Structural model Testing of Dams", and "Dams control works" by E. D Smith, Bureau of Reclamation U. S. A. 1938.
16. Concrete Manual issued by United State Bureau of Reclamation.

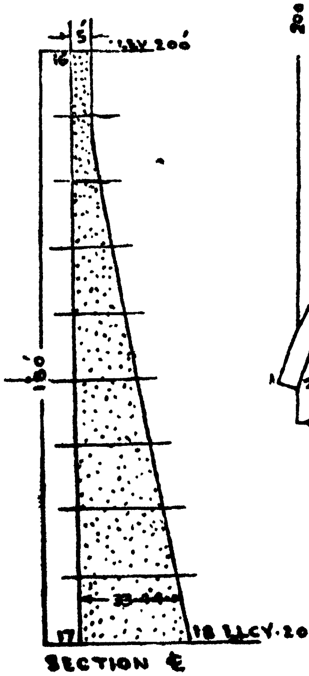
PART III
TANK IRRIGATION
(Storages And Dams)
CHAPTER V
Arch And Multiple Arch Dams

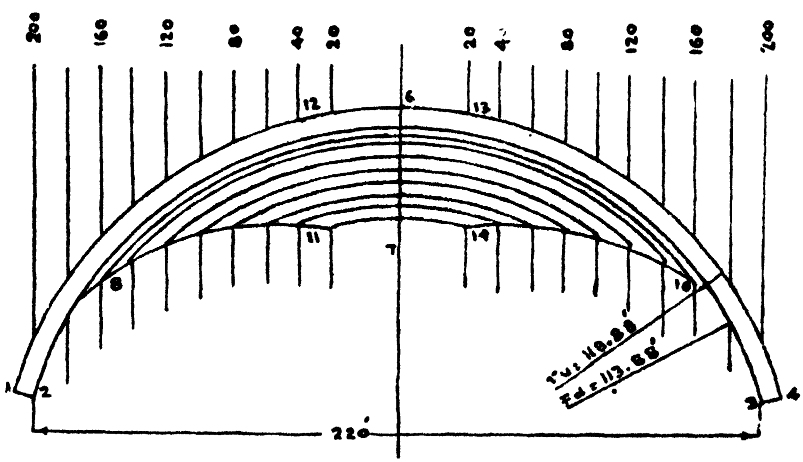
1. Introduction.

An Arch Dam is a solid concrete or masonry Dam, curved upstream in plan, which, in addition to resisting part of the pressure of the reservoir by its own weight, obtains a large measure of stability by transmiting the remainder of the water pressure or load by arch action into the canyon walls. Successful arch action is dependent on a unified solid structure, and special care must be taken in the construction of an Arch Dam to ensure that no structural discontinuities, such as open joints or cracks, exist at the time the structre takes up its water load. Certain words and phrases used in discussing the design of arch dams are defined below.

2. Arch Dam Types.

(A) Arch Dams may be classified as arch-gravity or thin-arch, although gravity action is present in all arch dams regardless of thickness. Under these two main classifications, more specific subtypes may be named such as constant angle, constant radius constant-centre, variable-radius, variable-centre, etc. The sub-classifications "constant-centre" and "variable-centre" have been selected for use in the discussion on layout procedure because of the distinct meanings implied by their names. Actually, either a constant-centre or a variable-centre type may have variable or constant radii.





Angle subtended by the arch at centre $=2\alpha=150°$

PLAN

Fig. 1. Constant Radius Arch Dam.

(*a*) **Constant radius dams.** A constant radius arch dam generally has a vertical upstream face. However, and unusually high structure, such as Boulder Dam, may have extrados curves of gradually increasing radii in the lower part of the canyon, to provide a vertical batter near the base of the higher cross sections. Intrados curves may be concentric or nonconcentric with reference to extrados curves. They usually have decreasing radii as the depth below the crest increases, to provide the increased thickness needed for the higher reservoir pressures. Constant radius arch types are particularly adapted to U-shaped canyons, where relatively large proportions of the water load at the lower elevations are carried by cantilever action Fig 1.

(*b*) **Variable-radius dams.** A variable-radius arch dam, also known as a constant angle arch dam, usually has extrados and intrados curves, of gradually decreasing radii as the depth below the crest increases Fig. 2. This is to keep the central angle as large and as nearly constant as possible, so as to secure maximum arch efficiency at all elevations. Variable-radius arch dams often have vertical or even overhanging, faces at the upstream side near the abutment and at the downstream side near the crown. Face slopes may be relatively flat at the upstream side near the crown and usually are relatively flat at the downstream side near the abutments. The variable radius type of arch dam is frequently adapted to narrow V-shaped canyons.

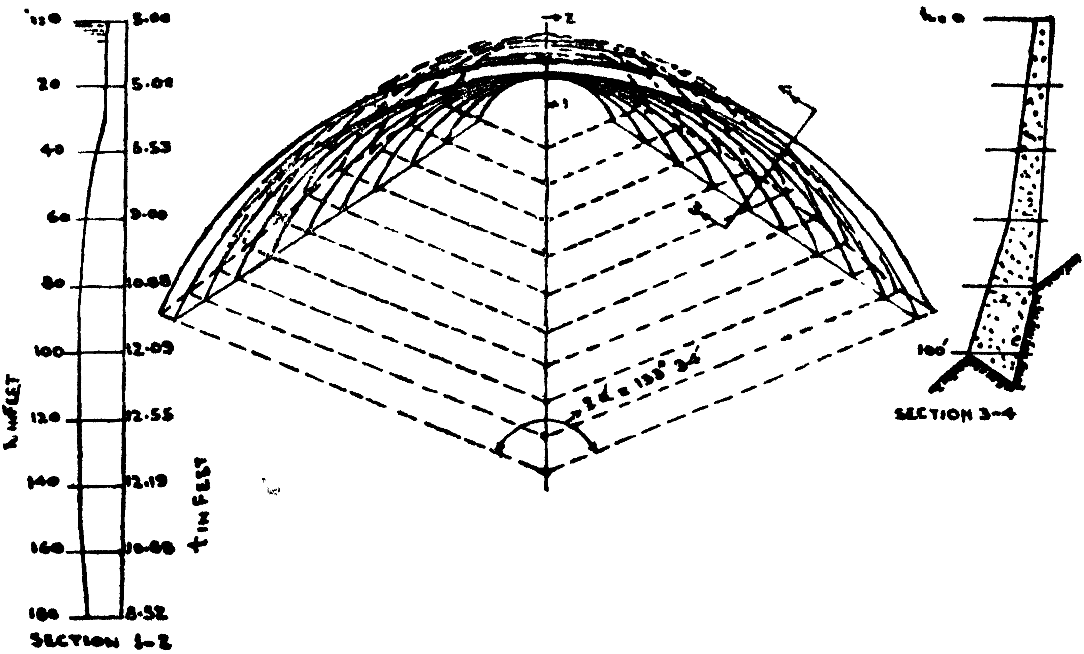


Fig. 2. Constant Angle Arch Dam, Example

(B) Arch dams may be also divided according the thoery of design used in the computation of stresses : (1) Cylinder theory arch Dams. (2) Elastic theory Arch Dams (3) Trial-load Arch Dams.

3. Design Procedure.

The design of an arch dam is a progressive procedure involving comparative trial layouts, and stress analyses and cost estimates there of. The progression is carried out only to the extent commensurate with the purpose of the design, that is, whether the design is to meet rough preliminary, preliminary, or specification design requirements. Rough preliminary designs, as distinguished from preliminary designs, are made in conjunction with field investigations for purposes of project planning to select the most favourable location and types of the proposed dam. Rough preliminary designs may be based on previous designs which are similar in height and shape of profile and for which the designs which are similar in height and shape of profile and for which the stresses are satisfactory. Preliminery designs are used for selection of the final location and final design, and as a basis for a request for funds for construction. Preliminary designs are made ingreater detail than rough preliminary designs where a closer approximation to final design is required. If time permits and enough funds are available, it is advisable to make rough stress analyses for a comparison of approximate stress distributions with comparable designs. A complete analysis by trial-load methods should be made of the adopted structure for the normal and the most severe loading conditions expected in actual service. The best of the alternative designs will be have stresses within allowable limits and distributed as uniformly as possible, with a minimum total expenditure.

4. Layout.

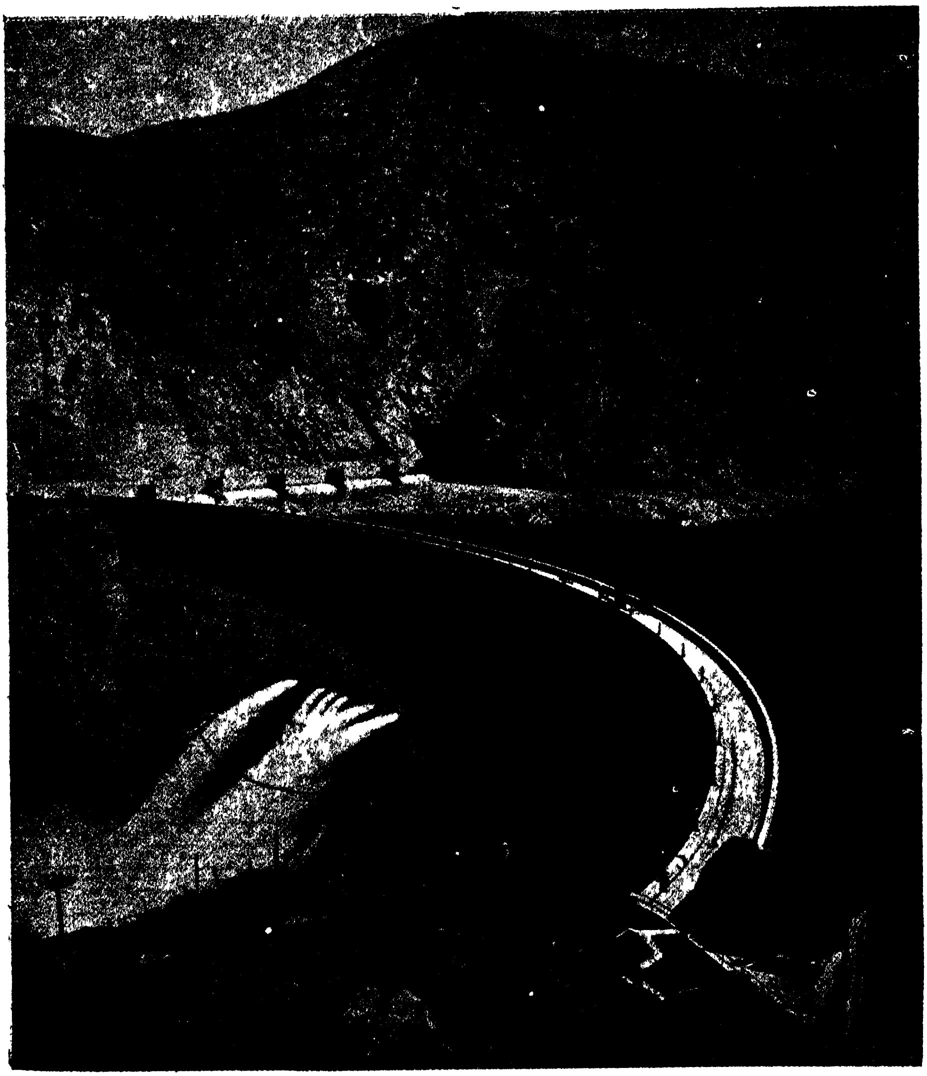
The primary objective in making a layout for an arch dam at a praticular site is to obtain the arch which will fit the topographical and geological conditions most advantageously, provide for the installation of adequate facilities for reservoir operation, and distribute the load with the most economical use of materials within allowable stress limitations. The load distribution, and the arch and cantilever stresses resulting from such distribution, depend largely on the shape of the canyon, length and height of dam; type of dam. thickness and shape of arch and cantilever sections, and the loading conditions. The following discussion is limited to general procedure and consideration of the factors which tend in influeuce the proper proportions of a dam for a given location and loading. For the determination of the initial preliminary design, it is recommended that an examination be made of all available designs of arch dams similar in length, height, and shape of profile to that which is proposed, to supplement the information given therein. A selection of the layout which best satisfies the primary objectives for the design of the proposed dam is made by a comparison of several alternative preliminary layouts, with their stress analysis and cost estimates.

(A) **Design data required.** The principal fundamental data which should be on hand before attempting to prepare a layout are : a topographic map of the proposed location ; geographical data from which the depth of overburden and the locations of faults, jointing etc. can be obtained ; reservoir water surface and tail-water elevations ; probable sediment accrual in the reservoir ; the required spillway and outlet capacities ; and power generation requirements. These data should be supplemented by (1) climatological records for special studies on temperature variations within the dam and (2) by laboratory tests for determining the strength and elastic properties of the rock and concrete.

(B) **Layout procedure.** The first step in making a layout is to determine the shape of the crown cantilever, and whether uniform or variable thickness arches will be used. The axis of the dam and the line of centres are then drawn in plan on transparent paper and shifted in position or orientated to determine the optimum location. A comparison of alternate trial axis curvatures is advisable before selection of the central angle of the top arch. The crown cantilever, profiles of centres of the upstream and downstream radii, and the plan, showing the arches at elevations convenient for stress analyses and quantity estimating, are then drawn in such a manner that the arches will have the required shape and the required abutment stability. Profiles along the axis and downstream edge of the base are advantageous for locating the arch abutments and base elevation for minimum excavation. It is general Bureau practice to make leyout drawings with the direction of streamflow toward the tope or the right-hand side of the sheet, and to record stresses on a profile developed looking downstream.

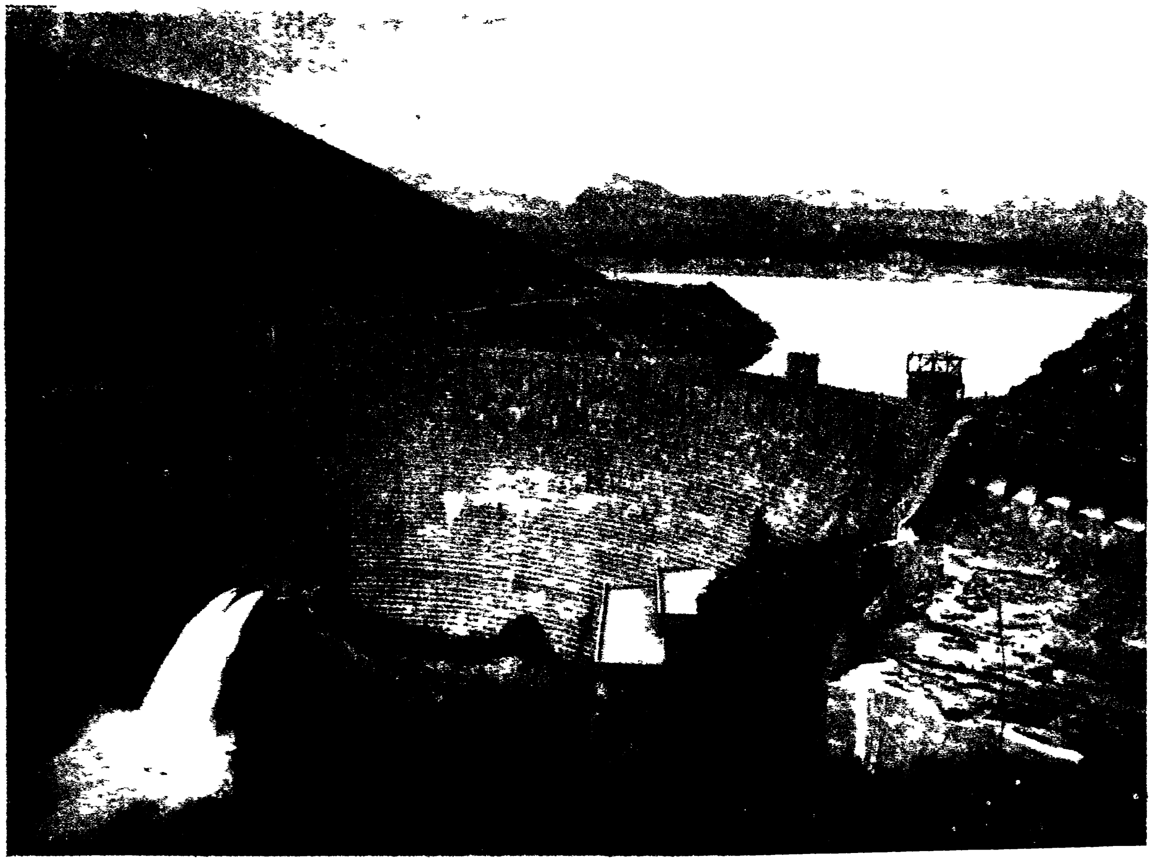
5. Analysis.

Since analysis is a part of the design procedure, the process is carried on in conjunction with making the layout. Prior to the development of the trial-load method of analysing arch dams many dams were designed on the assumption that the entire water load would be carried to the foundation by gravity action in both straight and arch-gravity dams; and horizontally to the abutments by arch action in thin-arch dams. Having assumed that the arches of thin-arch dams carried the total water load, they were analysed by application of the thin-cylinder formula, or by the elastic theory. The used of both of these methods of analysis resulted in structures in which defiections and stresses were not accurately determined and in which there was an inefficient distribution of material. The use of the trial-load method now makes it possible to analyse load distributions. deflections, and stresses in curved concrete or masonry dam of all sizes and shapes, whether of the massive arched-gravity type, or of the relatively thin-arch type. This method of analysis, although more complex than those mentioned above, provides a means of determining stresses and movements within the dam that are consistent with measurements made on existing structures or models. The trial-load method assumes that the water load is divided between arch and cantilever elements ; that the division may or may not be constant



Arrowrock Dam with six of the ten upper regulating outlets in operation.
Concrete Arch Section, completed in 1916, height 350′

Roosevelt Dam, Completed in 1911   Masonry Arch Section height 248



from abutment to abutment at each horizontal element ; and that the true division of load is the one which causes equal arch and cantilever deflections at all points in all arches and cantilevers in tangential and rotational directions as well as in radial directions. Since the required agreement of arch and cantilever deflections can only be obtained by assuming different distributions of load and calculating resulting arch and cantilever movements until the specified criterion is fulfilled, the procedure is logically called the "Trial-Load" Method.

Rough stress analyses are frequently advantageous for a comparison of the approximate stresses of alternate designs. For this purpose, so called crown adjustments have been found helpful. As the name implies, a crown adjustment obtains agreement of radial deflections between the arches and the crown cantilever only. The arches are assumed to be uniformly loaded. The adjustment may be made either by trial or by equating simultaneous equations to obtain unknown load factors, using unit-load deflections. For preliminary design, more reinforcement and accuracy is desirable than can be secured by a crown adjustment, and a full radial adjustment generally serves this purpose. A full radial adjustment permits the trial distribution of horizontal water and earthquake loads by the agreement in radial deflections of conjugate points on several arches and cantilevers representing the structure. For the complete analyses, radial, tangential, and twist adjustments are made to obtain an accurate divison of load between the arch and cantilever systems. For this division of load, stresses are computed to indicate the adequacy of the final design.

6. Factors Affecting Layout.

Although not an absolute necessity, a symmetrical or nearly symmetrical profile is desirable from the standpoint of stress distribution. A region of stress concentration is likely to exist in an arch dam having a nonsymmetrical profile, a condition tending toward an uneconomical section compared with that of a symmetrical dam. In addition, but of less importance. the amount of labour involved in the stress analysis is greatly increased over that for a symmetrical dam. In some cases improvement of a nonsymmetrical layout by one or a combination of the following methods may be warrranted : by excavating deeper in appropriate places, by constructing an artificial abutment, or by reorienting the dam.

The magnitude of the central angle of the top arch is a controlling value which influences the curvature of the entire dam Objectionable tensile stresses will develop in arches of insufficient curvature, such a condition being apt to occur in the lower elevations of a dam having a V-shape profile. The largest central angle practicable should be used, and consideration given to the fact that the bedrock topography may be inaccurate mapped and the arch abutments may need to be extended to points of somewhat deeper excavation than originally planned. Owing to limitations imposed by topographic conditions and foundation requirements, it will be found that, for most layouts, the largest practicable central angle for the top arch varies from 95 degree to 110 degree.

(*a*) **Length-height ratio.** The length-height ratios of dams may be used as a basis for comparison of proposed designs with existing designs. Such comparison should be made in conjunction with the relative effects of other controlling factors such as central angle, shape of profile, and type of layout. The length-height ratio also gives a rough indication of the economic limit of an arch dam as compared with a dam of gravity design. Generally, the economic limit of an arch dam occure for a maximum ratio between 5 to 1 and 6 to 1, depending somewhat on the height of dam and local conditions. Even if the length-height ratio for an arch dam falls within the economic range, the combined cost of dam and spilllway may be such that another type of dam would be the more economical.

(*b*) **Arch and cantilever shapes.** Arches and cantilevers should be proportioned for good stress distribution, with particular emphasis placed on the design of the system of elements that carry the greater portion of the load.

(*c*) **U-shaped canyons.** In dams constructed in U-shape canyons, of which Owyhee and Pathfinder Dams are examples, the lower arches have roughly the same chord length as those near the top. In such cases, use of a constant-centre type layout normally gives the arches sufficient curvature, thus leaving more latitude in the design of the cantilevers for an advantageous distribution of dead load.

(*d*) **V-shaped canyons.** In dams having narrow V-shape profiles, the lower arches are relatively short, and the greater portion of the load is carried by arch action. From the standpoint of avoiding excessive tensile stresses in the arch, a type of layout should be used which will provide as much curvature as possible in the arches. This may be accomplished in relatively low dams, of which Cat Creek Dam is an example, by a constant-angle type layout or a layout which approaches this type as closely as possible without producing excessive tensile stresses at the upstream face or excessive compressive stresses at the downstream face of the cantilever. The higher the dam the more it will be necessary to diverge from a constant-angle type to keep the cantilever stresses within allowable limits. Hoover, Buffalo Bill, and Cat Creek Dams are examples of dams with V-shape profiles having relatively small length-height ratios and covering a wide range of height.

(*e*) **Wide canyons.** Assuming for camparison that factors such as central angle, height of dam, and shape of profile are equal, the arches of dame designed for wider canyons would be more flexible in relation to centilever stiffness than those of dams in narrow canyons, and a proportionately larger part of the load would de carried by gravity action. In dams for wide canyons in which there is a tendency for cantilever stresses to be greater than arch stresses, it is desirable to obtain the maximum possible advantage from dead weight by using a cross section having a vertical upstream face, and large central angles. However, in high dams, a sloped upstream face may be advantageous for better distribution of stresses near the base under conditions of minimum resevoir drawdnown and earthquake shock. The layout may be a constant-centre type, of which Arrowrock, Roosevelt, and Deadwood Dams are examples, or especially where the profile is non-symmetrical, it may be a variable-centre type with variable thickness arches as in Gibson Dam.

(*f*) **Uniform-thickness arches.** From a consideration of arch stresses only, without regard to cantilevers, a uniform-thickness arch with fillets, rather than a variable thickness arch, is the economical type. This is because the critical stresses within an arch usually encountered at the crown and at the abutments, with forces of the greatest magnitude accuring at the abutments. However, there are cases where the cantilevers at other locations with the dam require greater thicknesses than at the crown. The type of arch required for a given layout is often difficult to predict, and in many layouts can be determined only by a comparison of stresses in alternative designs.

(*g*) **Variable-thickness arches**. Variable-thickness arches may be advantageous : in constant-angle designs, to furnish adequate thickness toward the base of the cantilever sections in the vicinity of the midheight of the dam where the greatest cantilever stresses are likely to occur ; in designs for dams having nonsymmetrical profiles ; and in very narrow canyons where the arches are too short for the addition of fillets without producing excessive curvature at the abutments.

(*h*) **Arch fillets.** In the most efficient and economical arch design, the stresses approach uniform values close to the established allowable values. For arches in which the abutment stresses are greater than allowable values, short-radius fillets should be added along the abutments at the downstream side. Fillet radii preferably should be of constant length at each side of the dam, but the length of radii used at one side need not be equal to that used at the other side. Also, the fillet centres at each side of the dam should fall on smooth curves in plan, to avoid irregularly warped surfaces. The locus of points of tangency between the intrados of the arches and the fillets at each side of the dam, called the trace of beginning of fillets, likewise requires a smooth curve. Fillet radii should have enough length to insure that the resultants of arch forces are directed safely into the abutment rock, and that curvatures at the downstream face of both the arch and cantilever elements are not so great as to produce excessive stresses parallel with the face of the dam. For this reason, the angle of intersection between fillet an abutment should be greater than 45 degrees. Fillets should be laid out, as a general rule, so that the traces of beginning at fillets will intersect the top arch at its abutments and intersect approximately the three-fourths points of the arches in the region of greatest arch abutment stress, that is, at about one-half to three-fourths of the height of dam above the base.

7. Masonry Arch Dams.

(A) Masonry dams are frequently built arched in plan, the abutments of the horizontal arch, which the dam constitutes, being the rock forming the sides of the gorge which is spanned by the structure. Such structures are designed to sustain by arch action the water pressures brought on them, the pressure being transmitted to the abutments through each horizontal arch ring

(B) The pressure of water acts normally on the surface in contact and therefore, the pressure on the extrados of an arch dams is radial to the arch and the line of pressure through the arch corresponds to its curvature.

The whole pressure transmitted through a horizontal arch ring exposed to water pressure over the extrados, which is the water face, is $R \times p$; where p is the water pressure per square foot and R the radius to the centre of the arch ring.

R is, in practice, frequently taken to the extrados (the water face of the arch ring) which is an inaccuracy on the side of safety.

(C) **Formulae giving stress in arched dams.** Let S and S_a be the maximum and mean average stresses in a horizontal arch ring, then from the above are derived the simple equations :—

$S_a = RHw/b$ (A) ; $b = RHw/S_a$ (B) ; where H is the depth of water, w the weight per unit and b the width of the lamina or arch ring ; where b is small in comparison with R, the average stress S_a differs but little from the S_{max}.

Where this is not so, if R and R_1 be the radii of the up and downstream faces of the arch ring, the relation between mean and maximum stress is given by the equation ;

$S_{max} = S_a \times \frac{2R}{R+R_1} = \frac{RHw}{b} \times \frac{2R}{R+R_1}$ (C) and $R_1 = R - b$; therefore, if the limit stress is ζ then ;

$$\zeta - S = \frac{2HwR^2}{2bR - b^2} ; \zeta - S = \frac{2Hw}{b/R\,(2 - b/R)} \qquad \text{(D)}$$

also $b = R\,(1 - \sqrt{1 - 2H/\zeta})w$ (E)

(D) **Theoretic profile, a triangle.** As in the case of the gravity dam the theoretic economic profile of an arched dam would be a triangle with the apex at water level. Also the upstream face should preferably be vertical, unless the distribution of pressures on the base 'reservoir empty' renders it necessary to give this a batter in order to bring the maximum stress within a suitable limit.

(E) **Crest width of arched dam.** Practically the crest of the arched dam must be given a certain thickness determined from general or local considerations. This would seldom be less than 3½ feet but is generally considerably less that the top width given to a gravity dam.

Bligh proposes a top width (a) ; $a = \sqrt{H}/2$. (F)
being half the width he suggests for a gravity dam.

(F) **Maximum stresses allowed in arch dams.** It is usual to allow higher stresess for an arched dam than for a gravity dam and this has been justified by successful practice.

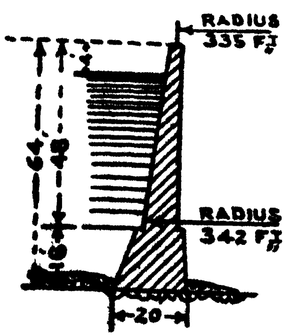


Fig. 3

The weight of the masonry of the arch although not taken into consideration in calculating stability of the dam does assist towards this and tends to reduce the computed arch pressure and for this reason it is desirable to keep the water face either vertical or with as small a batter as conditions permit.

(G) **The Bear valley dam.** The most remarkable example of such a structure is the Bear Valley dam Fig. 3 constructed in California in 1885. Its top length is 300 feet, top thickness 2½ to 3 feet and thickness, 48 feet below the crest, is 8½ feet. At the level it rests on a curved base 13 feet wide but of a slightly different radius so that the offsets on either side of the 8½ feet lamina are varying.

The full reservoir level is 4 feet below the crest or 44 feet above the base offsets, while the radius to the water face at this level is 342 feet. From this ; $S = RHw/b = 342 \times 44 \times 2/17 \times 36 = 49$ tons nearly.

The dam is constructed of uncut granite laid coursed in cement morter with a hearting of rough rubble. The work is regarded as one of the engineering wonders of the world and while it cannot be recommended for imitation, its success justifies the greater pressures usually allowed in arches, as compared with gravity dam.

It may here be remarked that Parker computes a reduction of arch stress in this dam, owing to the assistance given by the weight of the masonry acting as a gravity dam, as equal to about 17 percent (Control of Water, Page 408).

(H) **Australian arched dams.** Several arched masonry dams designed on the base of equation (B) have been completed in Australia in recent years and their designs have proved successful and economical. Examples among these are :—

The Barossa Dam, Fig. 4 and the Mudgee Dam, Fig. 5. The maximum arch stress on the former is $95 \times 200/18 \times 36$=about $15\frac{1}{2}$ tons and on the latter $50 \times 253/18 \times 36 = 19\frac{1}{2}$ tons nearly.

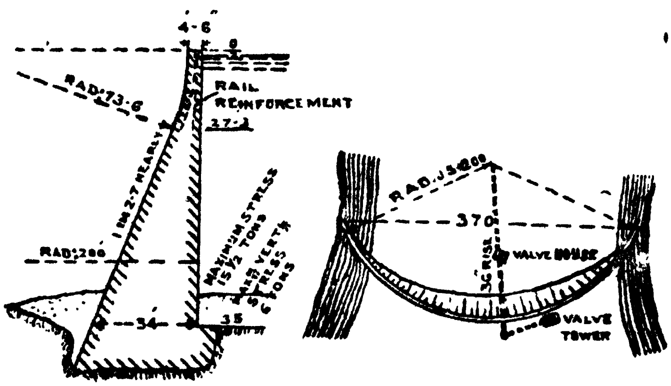


Fig. 4.

(1) **Arched Dams only applicable to narrow gorges.** It is clear from the stress formula for pressures that there is no economy in constructing an arched dam with a long radius and therefore, it is only across narrow gorges that such dams can suitably be constructed. With a limit of pressure of 10 tons per square foot and a radius of 270 feet, the volumes of masonry required for an arched dam and for a gravity dam are approximately equal. If the pressure is raised to 20 tons, the radius which brings about a similar state, is 500 ft.

It has been shown by Captain Garret, R. E., that the most economical curve for a given span is one with a central angle of about 120 degrees (Government of India, Technical Paper No. 170 on Arched Masonry Dams) ; when working with small spans (say under 150 feet) it is desirable to design for a smaller central angle as a certain thickness is desirable to prevent percolation. For this reason the arches of buttress dams are generally given smaller central angles.

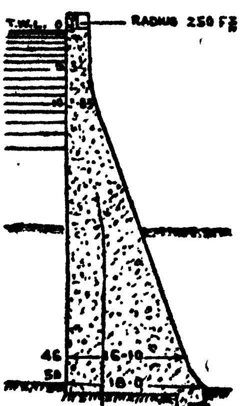


Fig. 5 (b)

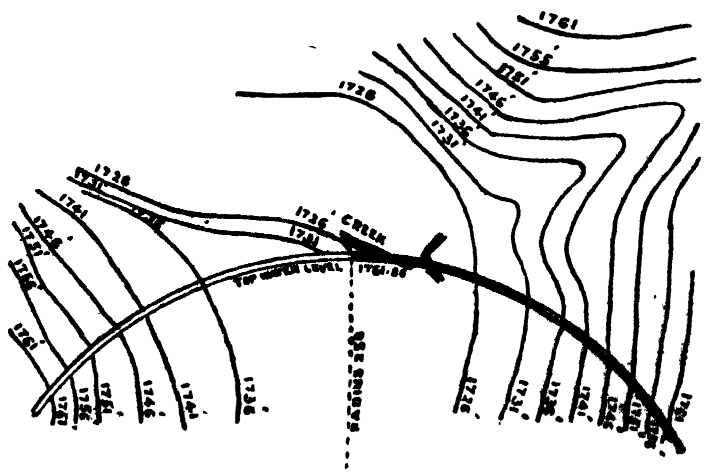


Fig. 5. (a)

8. Elastic Theory of Arch Dam Design.

(*a*) **Elastic deformation**. Because an arch slice from a dam is not a complete ring stresses computed in accordance with the cylinder theory are only approximate. A complete ring under a uniform external load is shortened. Because the shortening is uniform, the shape of every portion of the ring must remain constant in a segmental arch, such as any of the arches of Fig. 10 (a) & (b) the arch length is shortened by the load but the span is constant. Thus the loaded dam is deformed and moments and shears are introduced in adition to the normal-arch loads. Stresses produced in this manner are called rib-shortening stresses. If the arch is long and thin, with a large central angle, these stresses are small ; but in thick, small-angle arches they are important.

(*b*) **Temperature change and shrinkage.** A drop in temperature causes a shortening of the arch length. Shrinkage also results from drying out of the concrete. These effects produce moments which are aditional to those caused by elastic deformations. A rise in temperature has an opposite effect. These influences are small in flexible arches but are important in flat, thick ones Both temperature and shrinkage effects may be variable throughout the thickness of the arch ring.

(*c*) **Abutment yielding.** In discussing elastic deformation, the abutment span was assumed fixed. Actually, the abutments are elastic and are slightly spread apart by the thrust of the arch. Such spreading adds to the rib-shortening effect. Also, if there are moments at the ends of the arch, the abutment faces will rotate slightly, which introduces further elastic forces.

(*d*) **Variable loads and arch forms.** The cylinder theory can be applied only to a simple concentric circular ring subject to a uniform radial loading. These restrictions do not apply to an elastic arch. The elastic theory is essential where inclined or irregularly shaped arches, earthquake loading, variable silt pressure and other loading irregularities must be considered.

(*e*) The theory of elastic arches is amply covered in treaties on mechanics, masonry construction and continuous structures. Reference should be made to a good text on mechanics for a complete discussion of fundamental principles. The poineer work in developing the elastic theory of arch design was done by William Cain (Bib. 1 and Bib. 2). Fowler produced the graphical solution of Cain's formulae (Bib 3) and paragraph 16 of this chapter. Gravetr improved upon the Cain's elastic theory of arches by introducing the effect of abutments yeild and their deflection, (Bib. 4). Houk developed the subject further by introducing the effect of deflections caused by temperature stresses; solved examples are available in (Bib. 8, 10 and 13). The student should refer to the above mentioned pulication for a detailed study of the problem.

9. Trial and Load Analysis of Arch Dams.

(A) The cylinder theory may be used for the design of slenaer, simple arches, but it is a poor indication of stresses in thick arch dams, particularly where temperature variations and special loadings are involved. The elastic theory gives a better idea of actual stresses and permits allowance for temperature change, foundation yielding, earthquake forces and irreguler arch forms This theory applies to separate arches, each acting independently of its neighbours. Actually, the flow of stress in an arch dam is complex and adjacent arches restrain each other.

An exact mathematical analysis is not practicable, but a method known as the "trial load method" gives a reasonably satisfactory approximation. The dam is assumed to be made up of two systems of elements-horizontal arches and vertical beams or cantilevers. Each system occupies the whole dam structure and the loading is assumed to be divided between them in such manner that rhe computed deflection for any point in the dam, considered as a point in the arch system, will be identical with its computed deflection, considered as a point in the cantilever system. The division of loading which will cause coincident deflections at all points is found by a succession of trials.

(B) **Preparation of preliminary plan.** The trial load analysis is applied to a structure of predetermined dimensions, hence the first requisite is a preliminary plan Such a plan may be based on judgment. comparison with a similar structure, the cylinder theory or an elastic analysis. Errors in the trial design are revealed as the analysis proceeds.

(C) **Horizontal elements of dam.** The dam structure is assumed divided by harizontal planes, a unit distance apart into a continuous series of arches. Representative slices are selected for analysis.

(D) **Vertical elements.** The dam is likewise assumed to be divided into a continuous series of vertical elements and representative units are chosen for analysis.

In a constant-radius dam, like that shown at Fig. 6, (a) the upstream face may be considered as divided into unit chords as 1-2-3; etc., and vertical elements formed by passing vertical radial planes through these points. The units are simple with a constant trapezoidal cross section at any given elevation. Unless the ratio of thickness to radius of curvature is small, the thinning of the cantilevers at the downstream face should be considered.

In a variable-radius dam, vertical radial planes are not possible as the direction of the radius on a given vertical varies with elevation; hence some twisted cantilever form must be adopted which will permit the establishment of a set of essentially similar or systematically changing units exactly filling the full volume of the dam. In one of many possible systems, the centre line of the top arch or any other selected axis, is divided into unit lengths and warped radial planes are passed through vertical lines dropped from these divisions, as illustrated in Fig. 6 (b).

(E) **Interaction of elements.** In Fig. 7, let 1-2-3-4 represent one of the arches to be investigated and let 5-6 represent one of the cantilevers, these two elements being assumed to intersect in a common prism a-b. Some portion of the load on the face of this prism will be borne by the arch, the remainder going to the cantilever. Similar load divisions occur at other points. The result is a reduced and variable load on the faces of both the arch and the cantilever The arch is deflected to some new position as 1′-2′-3′-4′. The intersecting prism is moved from a-b to a′-b′. This movement generally has radial, tangential and angular components. The new position of the prism of intersection in the deformed cantilever must coincide with its position in the deformed arch Simple radial loads on the cantilever will not produce the required tangential and angular displacements. These movements are produced by forces internal to the dam as a whole but treated as external loads on individual arches and cantilevers. These forces are found by trial. A system of lateral forces is

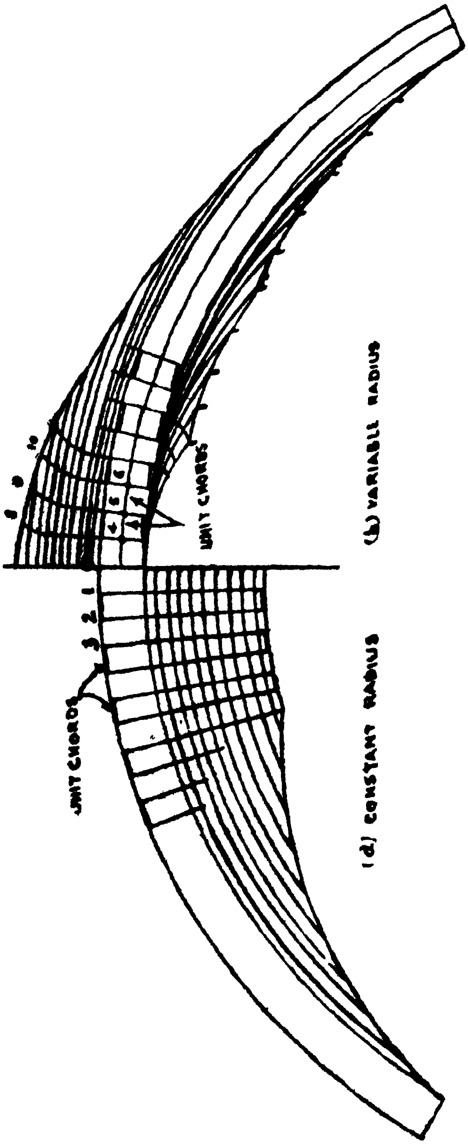


Fig. 6 Typical Vertical Elements. Trial Load Dams.

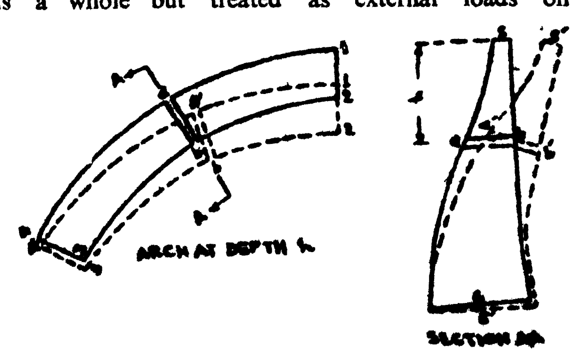


Fig. 7 Deflections of Arches and Cantilevers.

applied to the cantilever to make lateral shear deflections at all elevations identical with corresponding tangential arch deflections. The tangential continuity of the structure prevents measurable lateral bending of cantilevers ; hence only shear deflections need be considered. A system of tangential forces, equal and opposite to corresponding lateral forces on the cantilever must be included in the loads used for computing arch deflections.

A system of twist loads (moments in horizontal planes) must be applied to the cantilevers to produce angular deflections identical with those for the arches. Equal and opposite moments must be included with the loads and forces used in computing the arch deflections The radial cantilever deflections have angular components in vertical planes. A system of twist loads (moments in vertical radial planes) must be applied to the arches to cause them to conform. Equal and opposite moments must be included with the loads used for computing the cantilever deflections. It must not be concluded that these reactions give a complete picture of the internal forces in an arch dam. The actual situation is far more complex. However, experience indicates that a proper balancing of these elements gives results reasonably close to the truth. In simple structures, one or more of the enumerated influences may be negligible.

(F) **Factors influencing division of loads.** Not only the division of external loads between arch and cantilever but also all of the interactions discussed above. The most important influences are radial loads and radial deflections. It is usual to bring these elements into at least approximate agreement before introducing tangential shear and twist. The first tail division of external loads is made arbitrarily with only general experience on comparable dams as a guide. The division will vary from point to point. Frequently, one system carries a negative load, the other being subject to more than the total local loading. This condition is usual at the top of the dam where the freeboard arches are under stress from loads transferred to them by the vertical elements. Similar conditions may be found near irregularities in the foundation profile. The first trial division of loads is not likely to be accurate, but a skill computer usually can arrive at a sufficiently close approximation with a moderate number of trials. Typical load and deflection curves from the trial load analysis of a nearly symmetrical arch of simple cross-section are plotted. Radial, tangential and twist data are shown for three out of nine analysed cantilevers. Earthquake forces resulting from the inertia of the concrete and increased water-pressure are shown on the water-pressure force diagram.

(G) Trial load method of design of arches was first started in Europe 1922, (Bib. 12) and first taken up in U. S. A. 1923 by the Bureau of Reclamation. Detailed exposition of the subject is available in (Bib. 13) with solved examples. Simpheations of the method have been divised by Westergard Bib. 16. Student should refer to original publications for a detailed study of the subject and for solved examples to (Bib. 8) and (Bib. 13).

10. Design of Arch Dams.

(*a*) **Preliminary plans.** In preliminary plans for an arch dam, the engineer should study design adopted for similar sites, where dimensions and curvature were accurately determined by trial-load analysis. In order to avoid high-tension stresses at the reservoir face and to secure maximum arch efficiency, central angles should be as large as possible. Theoretical considerations, based on the thin-cylinder formula, show that a central angle of 133°34′ is most advantageous from the viewpoint of economy. However, practical considerations, together with topographical conditions, usually prevent the adoption of such angles for the lower arch elements.

The extrados and intrados curves should be located so that the ends of the arches converge in a downstream driection. Otherwise radial buttresses at the abutments may be needed to carry the loads transferred horizontally by radial shear. Such buttresses are often necessary where arch elements abut agains gravity tangents. Radial arch ends are most desirable ; but smaller amounts convergence usually suffice where radial construction requires excessive excavation, as in large thick dams.

Top widths of arch dams are usually made constant from abutment to abutment. Arch thicknesses at lower elevations may be constant or may increase toward the abutments. depending on stress conditions. Thickening toward abutments may begin at the crown sections, at the quarter points or may be secured by providing long radius fillets at the ends of the intrados curves. Abutment thickening should be warped between adjacent arch elements, so as to avoid undesirable appearances at the downstream face.

The ratio of length to thickness at the top of the dam should not exceed about 60. Usually the ratio will be smaller, owing to the desirability of stiffening the upper part of the structure or the necessity for providing a roadway along the top. Considerations of slenderness ratio are not important at the lower arches. The additional thicknesses needed from the stress viewpoint, together with the reduced widths of the canyon, will reduce the ratio to satisfactory values Furthermore, the restraining effect of the cantilevers on the bending of the arch elements increases as the depth below the top increase.

(b **Foundations.** Depths of required excavation must be estimated in determining dimensions for preliminary analysis. Sometimes humps in rock profiles, which may cause stress concentrations, can be removed in preparing rock surfaces. Sometimes deep holes or relatively narrow gorges, can be plugged with concrete and treated as part of the foundation instead of parts of the dam. Excavated surface should be gradually warped between adjacent elevations, pronounced stepping along abutment planes being avoided. Adequate grouting and draining should always be specified Geological conditions at the dam site should be approvd by competent foundation experts before proceeding with detailed designs.

Allowable stresses. Stresses in arch dams analysed by trial-load methods, on the assumption of 3,000 lb. concrete, a straight line distribution of stress and no loads carried by tension, should not exceed 600 per square inch compression or 300 per square inch shear, during maximum reservoir loads Increases of 10 to 15 percent may be permissible, momentarily, during intense earthquake shocks However, decreases of 25 to 35 percent should be made if the dam is analysed by approximate methods, such as placing the flull water load on the arch elements or bringing the arch and cantilever deflections into agreement at the crown section only.

Ordinarily, vertical tension at the upstream face may be as high as 100 per square inch without analysing secondary cantilevers, when the corresponding compression at the downstream face does not exceed 500 per square inch. Horizontal tension at the upstream face may be such as much as one third the corresponding compression at the downstream face without analysing secondary arches,, when the sum of the tension and compression does not exceed 600 per square inch. In an unusually high massive structure, provided with an upstream batter, compresssive stresses somewhat higher than 600 per square inch may be permitted along the upstream face near the base of the crown cantilever, owing to the triaxial conditions of compression occuring at such locations.

(*d*) **Maximum stresses.** Table 1 gives maximum arch, cantilever and principal stresses in some arch dams recently designed or analysed by trial-load methods Effects of tangential shear and twist action were included in all cases except Gibson Dam. Effects of rock deformations were considered in all cases.

(*e*) **Constants needed in analysis.** Table-2 gives general values of constants needed in analysing arch dams. These values may be used in preliminary studies where more accurate information is not available. They should be replaced by data based on field and laboratory measurements before adopting final designs. Tabulated values of modulus of elasticity are for sustained load conditions. Great accuracy in determining elastic properties of canyon rock is not necessary since effects of foundation and abutment movements are of a secondary nature. The modulus of elasticity for direct stress may be assumed to be the same for tension and compression, for both rock and concrete materials. The modulus for shear can be computed by the formula ;

$$E_s = E/2\ (1+U)$$

where U is Poisson's ratio, E the modulus for direct stress and E_s the modulus for shear.

Table 2 – Constants Needed in Analysing Arch Dams.

Constant	Material	Values	Units.
Weight, saturated	Concrete	150	Lb. per cu. ft.
Weight, saturated	Silt	110–120	Lb. per cu. ft.
Weight, saturated	Sand	110—120	Lb. per cu. ft.
Temperature coefficient	Concrete	0·0000040–60	ft. per ft. per deg. F.
Poisson's ratio	Concrete	0·15–0·22	
Poisson's ratio	Rock	0·10–0·30	
Modulus of elasticity	Concrete	2–2·5 million lbs. sq. inch	
Modulus of elasticity	Limestone	1–2 million lbs. sq. inch	
Modulus of elasticity	Granite	2–4 million lbs. sq. inch	
Modulus of elasticity	Sandstone	1–1·5 million lbs. sq. inch	

TABLE

Maximum Stresses in Arch Dams Determined By Trial-Load Analysis.

Serial No.	Name of Dam	Type	Max. Height Feet	Cantilever Stress Comp.	Cantilever Stress Tens	Cantilever Stress Shear	Arch Stresses Comp.	Arch Stresses Tens	Arch Stresses Shear	Principal Stresses Comp	Principal Stresses Tens.	Principal Stresses Shear	Leading Condition	U. S. Radius	Thickness Top	Thickness Base	Top length in ft. arch	Top length in ft. total	REMARKS
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20
1	Boulder	C. R.	731	565	None	154	231	31	120	565	16	160	Full Reserveir, 5 Deg. Subcooling	500	45	659	1220	1220	Inclined shaft spillway in abutment.
2	Owyhee	C. R.	421	358	6	86	294	242	175	413	344	143	W. S. At top of Dam?	500	30	255	528	829	Gravity section at ends, shaft, spillway at right end.
3	Arrow rock	C. R.	356	466	39	164	305	314	128	496	314	207	Full Reservoir with Earthquack	672	15.5	223	1100	1100	Side channel spillway at right end.
4	Parker	C. R.	335	289	C5	65	542	1	76	451	C	95	Full Reservoir with Earthquack 7 Deg. Subcooling	315	39	105	820	820	Central overflow 50ft × 50ft. stoneygates.
5	Arial	V. R	313	808	C	–	560	107	–	–	–	–	W. S. At top of Parapet	T 397, B 247	19.5	93	728	1250	Overflow section at right end.
6	Horse Mesa	V. R.	305	956	C	250	1061	C	273	1074	C	297	W. S. At top of Parapet	251 82	8	43	520	784	Ov.rflow section at ends.
7	Seminde	C. R.	261	303	8	100	429	193	103	485	193	134	Full Reservoir with Earthquack 5 Deg Subcooling	290	15	85	563	563	Inclined Shaft spillway at right end.
8	Mormon Flat	V. R.	229	633	C	74	893	C	181	1095	C	322	W S. At top of Parapet	T 187, B 100	8	20	360	623	Spillway at left end
9	Stewart Mountain	V. R.	212	862	C	56	625	C	282	990	C	331	W. S. At top of Parapet	273 170	8	30	580	1060	Gravity section at ends, spillway at left end
10	Gibson	C. R.	199	605	C	–	364	66	–	–	–	–	W. S. At top of Parapet	405	15	87	960	960	Shaft spillway at left abutment.
11	Dead wood	C. R.	168	472	C	–	360	184	–		–	–	W. S. At top of Parapet	290	9	62	501	749	Centre spillway, thrust block and gravity at right end.
12	Cat Creek	V. R.	113	413	C	–	277	114	–		–	–	W. S 1.5′ above top of Dam	T 110, B 58	6	23	237	237	

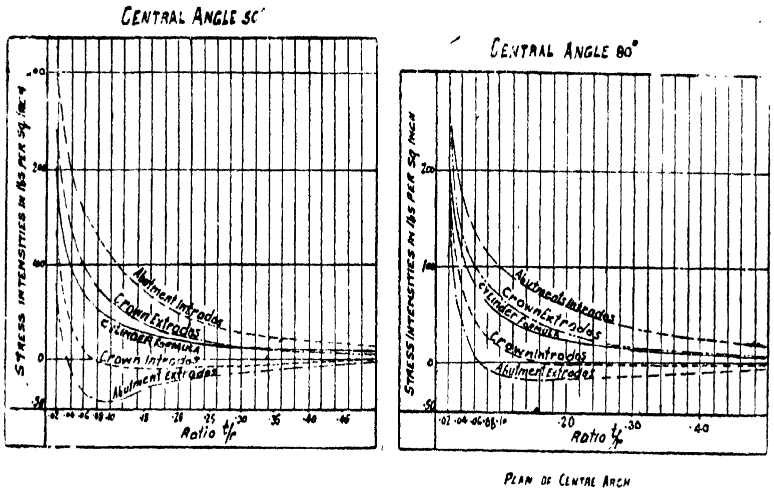


Fig. 12 (a) & (b)

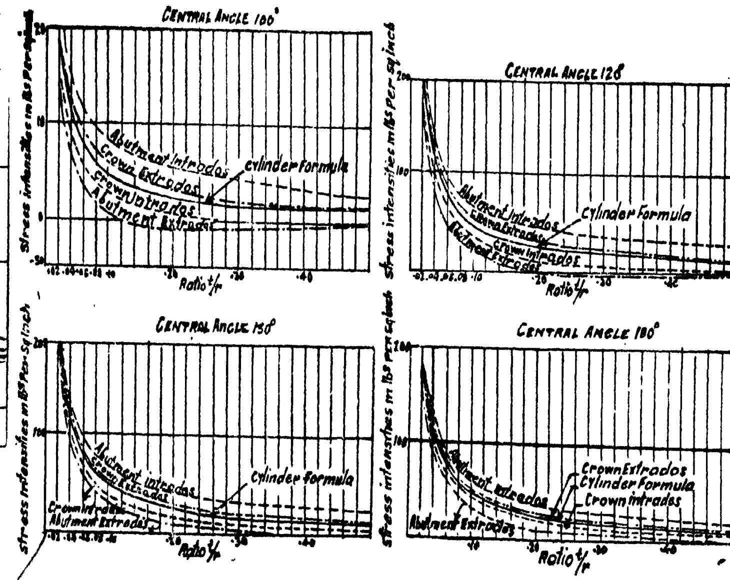


Fig. 12 (c) to (f)

11. Loads on Arch Dams.

(*a*) **Dead load and water pressure.** Loads on arch dams are essentially the same as loads on gravity dams, except that temperature changes, which usually are not important considerations in straight dams, cause important deflections and stresses in curved dams. The principal dead load is the concrete weight. The principal live load is the reservoir water pressure Additional loads may be imposed by tail-water pressure, uplift pressure, upward water pressure. under over hanging sections, deposition of silt on sloping faces, presence of silt in flood flows and formation of ice surfaces. Earthquake accelerations cause momentary changes in water pressure and an additional live load due to the inertia of the concrete.

(*b*) **Uplift pressure.** Uplift pressure seldom has an important bearing on the safety of an arch dam. If no cracking occurs, it can be neglected. If cracking occurs, uplift pressure in the cracks causes increases in downstream deflections, changes in load distribution and increases in maximum compressive stresses in both arch and cantilever elements. Uplift in horizontal cantilever cracks usually has a greater effect on stress conditions than uplift pressure in vertical arch cracks.

(*c*) **Ice pressure.** Ice pressure causes a continuous concentrated load along the arch element at the elevation of the ice. This load is carried partly by arch action and partly by cantilever action. The actual distribution can be determined by a trial load analysis. The transference of ice loads to the foundation and abutments can be facilitated by placing vertical reinforcing at the faces of the dam. Concentration of reinforcing at the downstream face, along the elevation of the ice, increases the proportion of ice load carried by arch action.

(*d*) **Temperature loads.** Temperature changes cause internal forces that move the dam upstream during the summer and downstream during the winter, the former condition working against the reservoir load and the latter with it. Conseqently the winter condition is usualy the more important in the stress analysis.

Since zero temperature stresses occur at the time of closing the arches, the closures should be made after the setting heat has been developed and dissipated and after a winter season has reduced concrete temperatures to their minimum values. However, pracitical considerations usually prevent closure at the most desirable time. Unless the concrete is artificially cooled as at Boulder Dam, it may be necessary to include some of the setting heat effects in analysing temperature stresses.

If clossure can be deferred until the setting heat has been fully developed and completely dissipated, the designer may assume that the temperature changes to be considered in the arch analysis will be the reduction from mean annual to minimum concrete temperatures expected during full reservoir load. Fig. 8 shows the maximum drop in average concrete temperature, below mean annual, which may occur in arches of different thickness.

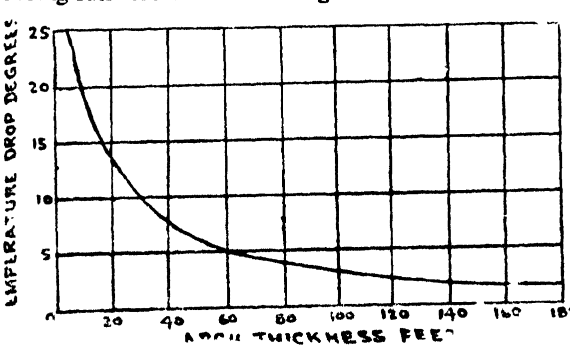


Fig. 8. Maximum drop in average concrete temperature below Mean Annual.

12. Stress in Arch Dams.

(*a*) **Stress distribution.** The distribution of stresses in an arch dam varies with the horizontal curvature, shape of vertical cross sections, general dimensions of the structure and uniformity of canyon profile. Pronounced humps in the rock surface cause stress concentrations in adjoining concrete, some times resulting in the formation of diagonal cracks. Maximum cantilever stresses often occur at such humps, even though the elevations are appreciably higher than the base of the maximum cross section.

(*b*) **Cantilever stresses.** Maximum cantilever stresses in arch dams, built at sites free from pronounced irregularities, usually occur at the base of the highest cantilever. During full reservoir load maximum compressive stresses usually occur at the downstream full reservoir loads, maximum compressive stresses usually occur at the downstream edge of the base but may occur at the upstream edge in comparatively high and thick dams provided with an upstream batter. Tension often occures at the upstream edge of the base in relatively thin arch dams. During the empty condition of the reservoir maximum compressive stresses in the crown cantilever usually occur at the upstream edge of the base in constant-radius dams and at the downstream edge of the base in variable radius dams.

(*c*) **Arch stresses.** Arch stresses in the central and upper portions of arch dams are commonly higher than in the lower portions. Maximum arch stresses usually occur at the crown and abutment sections. At the crown section, relatively high compressive stresses usually occur at the downstream face. At the abutment sections, stress conditions are usually reversed in accordance with the change in moment sign which generally takes place near the quarter points. Stress conditions at the abutment may be somewhat different in the top arches of long thin dams, owing to the upstream deflections that sometimes occur near such locations. Shearing stresses at the crown section are zero in symmetrical arches symmetrically loaded.

(*d*) **Principal and secondary stresses.** Major principal stresses along the contact between concrete and rock usually act in planes approximately horizontal at the top of the dam, practically vertical at the base of the maximum cross-section and at gradually varying inclinations along the intervening parts of the profile.

13. Investigations of Arch Dams.

(A) Investigations should include measurements of rock deformations, radial concrete deflections, circumferential concrete moments, concrete strains, crack formations, opening and closing of construction joints, seasonal concrete temperature variations, temperature changes caused by generation and dissipation of setting heat, water temperatures at the upstream face of the dam, uplift pressures at the base of the dam, uplift pressures at horizontal construction joints, efficiency of drainage instalations and other miscellaneous factors that affect the structual action and safety of arch dams. Only the more important investigations and resulting conclusions can be discussed herein.

Stevenson Creek Test Dam. The Stevenson Creek Test Dam, a constant-radius structure 60 ft high, was built solely for research measurements. It was perposely designed with a relatively thin section, so that strains and deflections would be of measureable magnitudes. It was constructed under rigid technical control and was arranged so that the full water load could be applied or removed in a few hours. Testing operations were conducted by a staff of research specialists, supervised by the Engineering Foundation Arch Dam Committee. The measurements, obtained were so accurate and comprehensive that the research enigneers were able to satisfactorily analyse the action of the structure. For instance, they were able to determine the proportions of water load carried by arch and cantilever action in different parts of the dam ; the moments, thrusts and shears in the arch and cantilever elements ; the vertical, horizontal and principal strains at the downstream face ; the spreading of the canyon walls ; the radial movements of the dam ; and the relations between the actual deformations of of the structure and those calculated by deflection formulae.

Results of Investigations. The results of the investigations at Stevenson Creek Test Dam, together with careful studies of experimental measurements at other arch dams, lead to the following conclusions :—

1. Concrete temperature changes are especially important, not only as effecting stress magnitude and distribution, but also as effecting structural movements and Crack formations.
2. Chemical heat generated during the curing period should be throughly dissipated before closing the arches by grouting radial contraction joints or filling radial slots.
3. Appreciable proportions of horizontal water loads are carried by cantilever action, even in dams of the thin-arch type.
4. Horizontal cracks may develop along the upstream face of thin-arch dams, near the foundation, owing to tension stresses in the cantilever elements.
5. Movements of arch dams may be satisfactorily estimated from calculations by elastic formulae in proper allowances are made for rock deformations, twist action and tangential shear effects.
6. Proportions of horizontal water loads carried by arch elements may be uniform crown to abutment sectione at certain elevations in symmetrical dams, but are usually too variable to warrant the preparation of important final design on such a basis.
7. The trial-load method of analysing arch and cantilever action in curved concrete dams furnishes a satisfactory basis for the design of arch dams of any type and size.

14. Model Tests of Arch Dams.

The results of the model investigation agreed satisfactorily with data secured at existing dams particularly in the Stevenson Creek Tests. Trial-load analysis of the Stevenson Creek, Gibson and plaster-celite Boulder models showed close agreements between calculated and measured deflections. Although tests of the litharge-rubber model of Boulder Dam showed a fairly close agreement of deflections, they were not wholly satisfactory, owing to the high Poisson's ratio of the model material about 0·50, and the large variations in the modulus of elasticity in different directions.

Conclusions. The general conclusions which may be drawn from the model investigations are as follows : –

1. Satisfactory predictions of deflections and strains in thin arch dams may be made from tests of celluloid models loaded with mercury, particularly if tension stresses are relieved at locations where cracks would be expected in the prototypes.
2. Satisfactory predications of deflections and strains in thin-arch dams may be made from tests of concrete models, loaded with mercury.
3. Plaster celite is the best material thus far developed for use in building models of large thick arch dams or models of horizontal or vertical elements of such structures.
4 Trail-load analysis of model strains and deflections check the experimental measurements satisfactorily.
5. Model tests of horizontal and vertical elements of arch dams furnish valuable data regarding the stress distribution in such structures.
6. In the case of usually large, thick arch dams, where dead-load stresses are important and where prototype construction is necessarily complicated by contraction joint, contraction joint grouting, concrete temperature control, variable concrete curing, partial water loads during construction, irregular rock profile, differences in geological formations and other field factors that cannot be definitely determined until the work is completed carefully controlled model tests are more valuable in checking fundamental methods of design than in making direct predications of deformations and stresses in the prototypes.

15. Failure of Arch Dams.

No important Dam has failed so far. A small arch dam near Manitou, Colo., about 50 ft. high and 300 ft. long, suffered a partial failure in 1924 owing to disintegration of poor concrete used in its construction. A thin-arch dam on Moyie River near Bonner's Ferry, Idaho, 53 ft. high and 154 ft. long, failed in 1926 owing to undermining of the timber lined spillway channel which had been built in soft stratified rock near the left abutment. A low arch dam on Vaughn Greek at Lake Lanier, N. C., 62 ft. high and 236 ft. long, also failed in 1926 owing to washing out of a cyclopean masonry abutment built on soft decomposed material at one end of the arch section. The Moyie River and Vaughn Greek dams both stood intact under nearly full reservoir pressures, while the abutments were being washed away ; so that they were still in place after the water escaped.

A small arch dam on Purisima Greek, San Mateo County. California, failed completely during the first filling of the reservoir. It was 40 ft. high, 100 ft. long, 2 ft. thick at the top, 4 ft. thick at the base and was curved on a radius of 120 ft. It was buill in a loosely bedded, shale gully and apparently failed by shearing of the foundation and abutment rock. No other records of arch-dam failure have been found in engineering literature.

16. Multiple Arch Dams.

The earliest example of a multiple arch dam in world is the Mir Alum Dam, built across a valley about a mile wide near Hyderabad (Deccen) India by a French Engineer at the beginning of the nineteenth century. The dam was constructed of rubble masonry, and consists of a series of semi-circular horizontal arches : there spans varying from 70 to 147 feet. The maximum height of the dam is 39 feet. A plan of the centre arch with section through the arch and abutment are shown in fig. 9. The line of the resultant pressure, when the reservoir is full, cuts the base of the buttress some distance outside its middle third and consequently there is tension on the upstream face. The friction of the buttress on its base would be insufficient to prevent its sliding so the stability of the dam depends on the shearing strength of the material of which it is constructed. 100 yards later a dam of some what similar character, about 50 feet high, was built in Alwar by Captain (afterwards Lt. col.) A. F. Garrett, R. E.

Until the beginning of this century, multiple arch dams found no favour with engineers. A well-known English writer on waterworks, in a book of comparatively recent date summarily dismissed this type of dam remarking, "it is extremely doubtful whether any economy in the total cost can be secured except under very peculiar circumstances. From a theoretical point of veiw arch and buttress dam combines the disadvantages of both types of dam" (curved and gravity) "this type, therefore, seems unlikely to be adopted and will not be further discussed." It is interesting to observe that, at about the time that the book, in which these remarks appeared was published, an American dam-expert stated, in a paper on the subject, that in a multiple arch dam with sloping arch barrels, "the advantages of arch and gravity dams are combined without involving certain undesirable features inherent in structures of these two types." The multiple arch dam seems to have been originated in the United States by Mr. J. S. Eastwood. A considerable number of these dams have built in that country during the last 30 years. In Italy this design of dam became popular after the War and has been adopted for dam over 200 feet high. No multiple arch dam has, so for as I am aware, been built in England, but there are examples in other parts of the British Empire. English Engineers are, as a rule, cautious about adopting new forms of design and it has been thought that the water might percolate through the comparatively thin arches and that in winter the seepage might freeze and gradually destroy the concrete on the intrados of the arches. To some extent these fears have been justified in places where the winter cold is very severe; but on the other hand dams of this type have been built and have remained in good condition, in situations where very low temperatures are experienced. In a temperate or hot climate like that of India there is no reason why multiple arch dams, if paoperly designed and constructed, should not be completely satisfactory and in fact, nearly all the dams of this type have been so. The prejudice against them, which existed until quite recently, has thus been proved to be generally unfounded and has now disappeared in those countries where this type of dam has been adopted.

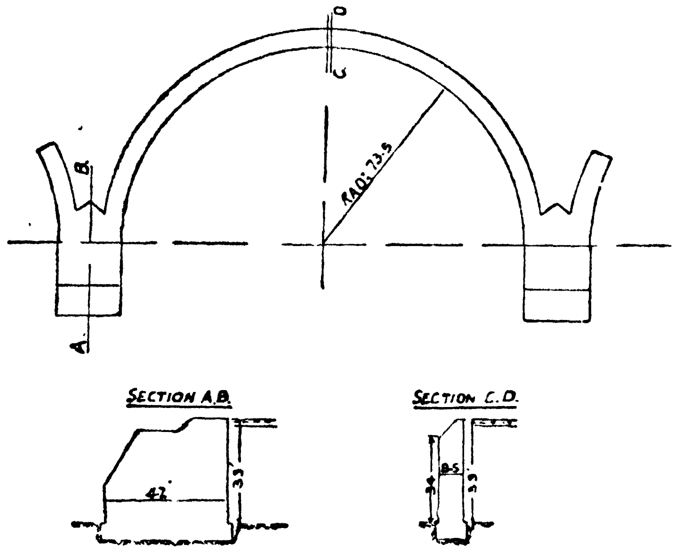


Fig. 9

17. Design of Multiple Arch Dams.

Design of the arch. The modern multiple arch dams are constructed with sloping arch barrels. The intensity of water pressure on a slice of the arch normal to the axis of the barrel is consequently not uniform from the crown to the abutment The former being at a higher level has less water above it than the latter. The difference is proportionately greater near the surface than at the base of the dam. In order to provide for the irregularity, it has been suggested that the arch should be made circular in the horizontal direction, which would entail its being elliptical in the direction normal to the face of the dam. This suggestion has, however, not been usually adopted. Near the top of the dam the eccentricity of the line of resistance in the arch is a matter of comparatively small importance, for other considerations require that the upper portion of the arch should be made thicker than is theoretically necessary to take the stresses produced by the water pressure. At the bottom of the dam the eccentricity is much smaller than that at the top. In low dams the arch is usually made circular in the plane normal to the face of the dam. In higher dams a three-centred arch can be adopted which follows the curve of the line of resistance in the arch sufficiently closely.

In the past the thickness of the arched dams have been usually calculated from the "cylinder" formula t=PR/f or f=PR/t where t is the thickness of the arch ring; P the intensity of pressure of the water ; R the radius of the upstream face of the arch ; and f the working compressive stress intensity. The assumption that an arched masonry dam acts as part of a cylinder under uniform radial pressure, which is free to deflect radially, is obviously inaccurate. In the large, single arch and curved gravity dams that have been errected in the United States and elsewhere. the cylinder formula has been abandoned in favour of the trial load method of analysis, based on the deflections calculated according to the theory of elasticity. In this system a division of load is arrived at between the horizontal and vertical elements of the dam. The load in the former is assumed to be carried by arch action and then on the latter by cantilever action. In the multiple arch dam cantilever action is not of sufficient importance to be taken into account. But even in this case the cylinder formula is for from being mathemati cally correct. Its simplicity, however, makes its use convenient, if possible, to the extent of its inaccuracy and the possibility of modifying it so as to overcome these inaccuracies will now be examined.

18. Rib-Shortening.

The effect of rib-shortening under external water pressure, in an arch fixed at both ends is illustrated in Figs. 10 (a) and (b). Let ABC be the centre line of the circular arch of uniform unit thickness and uniform unit width under a uniform radial pressure of p lbs. If ABC radial were free to move in a radical direction it would shorten uniformly in length and take up the new position A′ B′ C′ in Fig. 10 (a). But as the ends of the arch are fixed, it can only shorten to the line AB′C in Fig. 10 (b). If we imagine that the arch is first allowed to shorten to the position A′B′C′ and then again to extened to the position A B′ C without removeing the external pressure, horizontal tensile force H must be applied to bring the ends out from A′ and C′ to A and C. It is found that this force H acts through the centre of gravity of the arch which is approximately at the distance h/3 from the crown. It produces a moment tending to deform the arch at any point on it in proportion to the distance of that point from H. Thus at F the moment is dH, whilst at G and K the moments are nil and at these points there are no deformation stresses. At the crown, the stresses are tensile at the intrados and compressive at the extrados, whilst at the abutments they are compressive at the intrados and tensile at the extrados. The intensities of the deformation stresses at the abutment are approximately twice those at the crown. Rib-shortening produced by a fall of temperature causes deformation stresses of a similar character to those produced by the water pressure which must be added to them.

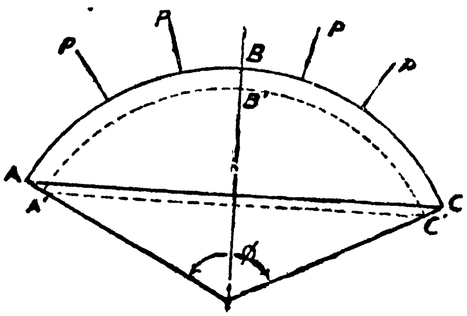


Fig. 10 (a)

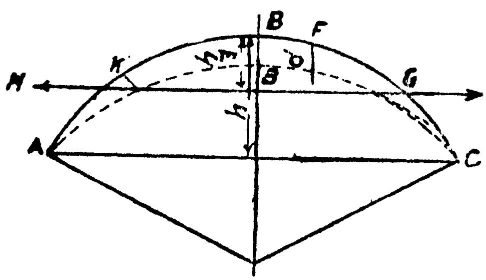


Fig. 10 (b)

19. Professor Cains Formulae.

Elaborate theoretical researches into the subject of stresses in arched dams have been made by various methematicians, particularly in the United States. It is impossible to deal with all these within the limits of this chapter, but some of the results can be summarised and compared with those from the the cylinder formula in order to indicate the extent of the inaccuracy of the latter in different conditions. The object aimed at is to find some simple method of designing dams of this character which will arrive at safe and economical results.

If it is possible to do so without actually employing any of the complicated theoretical calculations referred to, it is a waste of time attempting to follow these here. They cannot be accepted as completely accurate, for they start with assumptions adopted for convenience which are in many cases only partially correct The late Professor Cain of the University of North Carolina was one of the best known mathematicians who have investigated the stresses in arch dams. Amongst his many activities in this direction he developed formulae for computing moments and thrusts in arches of uniform thickness with fixed ends, under uniform normal radial pressure, both including and excluding the influence of transverse shear. These he published in more than one paper. He considered the thrust and moments at the crowns and abutments of the arches as depending on the normal radial pressure. The radius of the arch and the angle subtended by it. The stresses at the intrados and extrados .of the crown and abutments were calculated from the axial thrust end bending moment by means of the known formula

$$f=\frac{P}{t} \pm \frac{6M}{t^2} \qquad (1)$$

P being the total thrust ; t the radial thickness of the arch, M the moment (taken in this case clock-wise) and f the stress at the intrados and extrados. The question for the complete mathematical treatment of this problem became very complicated and the solution extremely laborious.

20. F. H. Fowler's Diagrams.

In 1928 F. H. Fowler published in a paper read before the American Society of Civil Engineers a series of Diagram showing graphically the stresses at the crowns and abutments of horizontal arches, under a head of 10 ft. derived from Professor Cain's formulae. These stresses were worked out for angles ϕ from 40° to 180° and for ratios t/r (see Fig. 11) from 0·02 to 1·00. Fowler pointed out that in an imaginary arch dam of fixed central angle and uniform thickness throughout its entire height, the stresses in the arch rings at any two levels would be in proportion to the heads. So the computed stresses for 10 feet head would only require to be multiplied by the ratio of any other head to 10 feet to give the stresses at that head. For example if the stresses in an arch under a pressure of 100 feet of water are required, it was only necessary to multiply the stresses for an arch of the same central angle and ratio t/r by 10.

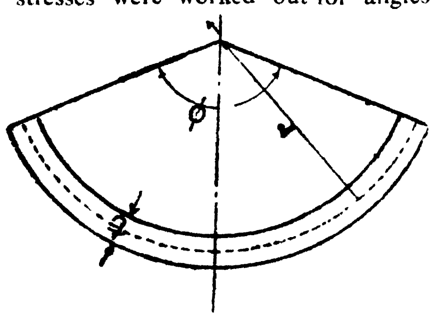


Fig. 11

Graphic comparsion of Professor Cain's formula with Cylindar formula.

The Fig. 12 (a) to (f) (folder) shows the same information as Fowler's diagrams in a different form, for central angles from 60° to 180° and for ratios t/r from 0·02 to 0·50. The diagrams apply to arches with fixed ends, of constant radial thickness, under constant radial pressure and include shear. From them it is possible to ascertain the comparative accuracy of the cylinder formula when applied to similar conditions Figs. 12 (a to f) measured by Professor Cain's formula. In the cylinder formula the central angle is not taken into account. The head of water (in this case 10 feet) and the ratio t/r are the only factors to be considered.

If f_r is the stress intensity according to the cylinder formula in lbs. per sq. inch.

$$f_r=\frac{10\times 62\cdot 5}{144}\left(\frac{r+t/2}{t}\right)=4\cdot 34\left(\frac{1+t/2r}{t/r}\right) \qquad (2)$$

The curve plotted from this equation is shown on each diagram. It will be observed that the variation in the respective curves becomes proportionately less as the central angle increases. That is, as the arch becomes flatter it acts more and more as a girder and less as an arch until it finally ceases to act as an arch at all and acts solely as a girder. With a central,angle of 60° Professor Cain's formula shows tension at the abutment extrados in ratios of t/r > 0·04 ; for a central angle of 80° in ratio >0·07 ; for 100°>0·12 and for 122°>0·17. When the angle is 150° or over the possibility of tension does not usually need to be considered.

21. Practicle Examples.

In order to make practicle use of the diagram, a maximum permissible working stress intensity is to be decided on and the ratio t/r found from the diagram, which shows this intensity as a maximum under the head of water which the dam will have to sustain. This can then be compared with the stress intensity that would be obtained for the cylinder formula for the same ratio t/r. In order to make this clear a few examples are given. In these the maximum permissible pressure intensity on the concrete is taken as 600 lbs. per square inch.

Example 1. An arch with a central angle 120° the ratio t/r is required for a head of water of 30 feet. As $30=3\times10$, the maximum stress intensity on the diagram must be $600/3=200$ lbs. per square inch. This is found to occur at the abutment intrados with a ratio $t/r=0{\cdot}025$.

The corresponding stress intensity according to the cylinder formula for the same head and ratio is 527 lbs. per square inch.

Example 2. In the same arch with a head of 100 feet the maximum stress intensity to be found on the diagram is $600/10=60$ feet. This occurs with a ratio $t/r=0{\cdot}12$.

The corresponding stress intensity according to the cylinder formula is 384 lbs. per square inch.

Example 3. An arch with a central angle 180°. Head of water 30 feet. The ratio t/r is 0·022. The corresponding stress intensity according to the cylinder formula is 596 lbs /sq. inch.

Example 4. The same arch with a head of water of 200 feet $t/r=0{\cdot}28$, stress intensity according to cylinder theory 353 lbs. per square inch.

These examples are sufficient to show that if the cylinder formula is to be adopted to the results arrived at by professor Cain the working stress intensity f_c will have to vary under different conditions, lower figures being taken for greater depths of water and smaller central angles. If tension is to be eliminated, the maximum height of dam, under the accepted hypothesis is 120 feet for a central angle of 120°. If this condition is to be observed in a high dam the central angle must be 150° or more.

22. F. A. Noetzli's Formula.

In 1924 the late F. A. Noetzli contributed a paper to the American Society of Civil Engineers on an "Improved Type of Multiple Arch Dam," in which he gave the following stress formula for "rib-shortening" due to water pressure :—

At the crown.

$$f_1=-kf_c\,t/h\,(t/h\pm2) \qquad (3)$$

At the abutment.

$$f_2=-kf_c\,t/h\,(\phi/h\pm4) \qquad (4)$$

Where h is the rise of the arch and k a co-efficient depending on t/h and the central angle ; f_c is the axial stress obtained from the cylinder formula.

Noetzli's results may now be compared with Professor Cain's. Take example 1 above :

As $h=r\,(1-\cos\phi/2)$; and $\phi/2=60°$, $h=0{\cdot}5r$

$\therefore\ t/h=2t/r=2\times0{\cdot}050=0{\cdot}050$

From Noetzli's diagram Fig. 13 the co-efficient k for an angle of 120 and ratio $t/h={\cdot}05$, is ·75. So from equations (3) and (4), the stresses at the crown due to rib-shortening are $f_1=-{\cdot}75f_c\times0{\cdot}05({\cdot}05\pm2)$ and at the abutments $f_2=-{\cdot}75f_c\times{\cdot}05({\cdot}05\pm4)$. The — sign in the last equation gives the pressure for the stress at the intrados of the abutment, which is thus $148f_c$. The axial stress has to be added, so the maximum stress intensity is $f_{max}=f_c\,(1+{\cdot}148)$, f_c at the thickness of arch found from professor Cain's formula has already been shown to be 527 lbs. per square inch. According to Noetzli's formula the maximum stress is $527\times1{\cdot}148=604$ lbs. per square inch, which is thus slightly more than the figure derived from Professor Cain's formula.

Other examples will be found to give similar results and generally speaking Professor Cain's and Noetzll's results agree fairly well.

Diagram showing stress intensities to be used in cylinder formula for arched dams to allow for deformation stresses.

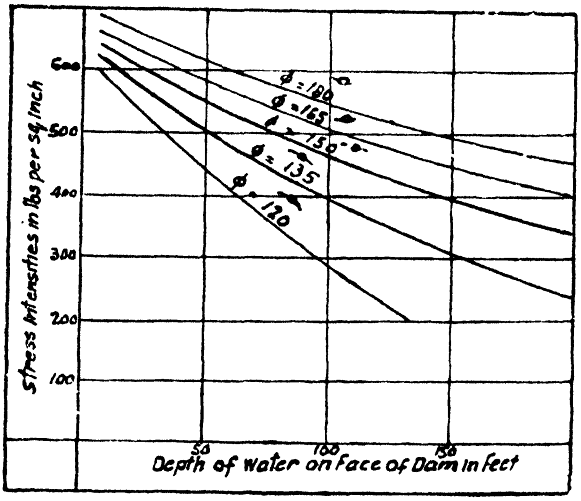


Fig. 13

Noetzli's equations for the deformation stresses due to change of temperature are :—

$f'_1 = kEcT\ t/h\ (t/h \pm 2)$ (5)

At the abutment

$f'_2 = kEcT\ t/h\ (t/h \pm 4)$ (6)

Where T is the change of arch temperature in degrees Fahrenheit, E the modulus of elasticity, which may be taken as 2,000,000 lbs. per sq. inch. c the co-efficient of expansion, taken at ·000006 ; and k the same co-effcient as adopted for the arch shortening due to water pressure for corresponding values of t/h and ϕ

A rise of temperature is considered positive and a fall negative. In the latter a *minus* sign must be prefixed to the right hand side of the equation and the figures obtained added to those arrived at from equations (3) and (4).

As k in eqautions (5) and (6) is the same for the values of t/h as in equations (3) and (4) it appears that the deformation stresses produced by any change of temperature T°F are equivalent to those produced by an axial stress of 12 T lbs. per square inch. For example, if there is a fall of temperature of 15° in the arch ring, the stresses produced will be equivalent to those produced by an axial stress of $15 \times 12 = 180$ lbs. per sq. inch.

If the dam is situated in a place where the cold in the winter is not very severe and is completed at a time when the temperature is below the annual mean the following falls in temperature, below that at which the arches were closed, are as great as are likely to be experienced in the arches at the respective depths, when the resevoir is full.

10	feet below	the surface	30°F
50	do	do	25°F
100	do	do	20°F
150	do	do	15°F
200	do	do	10°F

23. Diagram of Working Stress Intensities.

As a result of a number of experimental calculations the curves in Fig. 13 have been plotted to show values of f_r to be used in the cylinder formula which will allow for the deformation stresses due to full water pressure and maximum estimated fall of temperature in multiple arched dams with central angles 120° to 180° and for depths of water from 10 to 200 feet. On the basis of the forgoing theories these values will ensure that the maximum stress intensity in any part of the arch under the most unfavourable conditions is less than 700 lbs. per square inch. As a practical illustration, take an example of an arch ring with a central angle of 120° under a head of 100 feet.

In this example $T \times 20$ and the deformation stresses for this fall of temperature are equivalent to those produced by an axial stress $20 \times 12 = 240$ lbs. per square inch.

$r = 2h$ and f_r (from the diagram) $= 290$ lbs. per square inch ; $t = \frac{100 \times 62\cdot5}{290 \times 147} \times 2h = \cdot32h$; $\frac{t}{h} = \cdot32$;

For this ratio Noetzli's coefficient $k = \cdot62$; The maximum stress is, therefore, $f_{max} = 290\cdot62(290 + 240) \times \cdot32 \times 3\cdot68 = 677$ lbs. per square inch.

As another example, take an arch with a central angle of 180° and a head of water of 150 feet.

T=15 ; equivalent axial stress is 180 lbs. per square inch from the diagram ; f_c=490 lbs. per square inch ; $t=\frac{150\times 62\cdot5}{490\times 144}\times h; \frac{t}{h}=0\cdot13$; In this Noetzli's coefficient k=0·54 ; The maximum stress is quality of $f_{max.}=490+\cdot54(490+180)\times\cdot13\times3\cdot87=673$ lbs. per sqaure inch.

A working stress intensity of 700 lbs. per square inch can be considered safe in a reinforced concrete structure, if the concrete is of first class quality. The calulated stresses in some of the large concrete arch dams in America reach a higher maximum in one over 900 lbs. per square inch. In the dams discussed in this chapter, the maximum stress intensities would only occure when the highest possible flood level in the reservoir coincided with the maximum fall of temperature in the dam—a somewhat improbable combination of circumstances. In any dam in which there is any doubt as to the quality of concrete, the maximum stress intensity should not exceed 600 lbs. per square inch. This limitation would entail plotting a fresh set of curves for f_c in the diagram

There is, however, a further consideration that the formulae referred to above assume a constant radial pressure on the upstream face of each arch slice which is only found, when the arch is horizontal and the upstream face of the dam vertical. As already pointed out, in a dam with a sloping face, the pressure is not constant. Noetzli provided for this by assuming an average load on the arch equal to that produced by a head of water, which is the mean of the heads at the abutments and the crown. This he considered allowed sufficiently for the water pressure and also for the component of the weight of the arch itself at right angles to the slope of the inner face of the dam. The component parallel to this slope is assumed to be carried by the arch barrel. Subsequent investigators have developed more accurate methods of dealing with this question, but these involve very long and combersome calculations and the results obtained do not seem to differ sufficiently from those arrived at by Noetzli's assumption to justify the labour required to make use of them, at all events for perliminary designs. If considered necessary more complete methods determining the variation in the stress due to the difference in head could be used for checking the final designs.

24. Design of the Buttress.

For the purpose of calculating the stability of the buttress, this may be treated as a slice of a gravity dam, the water pressure on the face of the dam, as well as the component of the weight of the arch itself normal to the face being considered as being transferred to the buttresses In the dam with sloping arch barrels as usually designed, the point where the resultant of all forces cuts the base of the buttress when the reservoir is full, does not differ much from the point cut by the resultant when the reservoir is empty and is not sufficiently eccentric to produce tension at the inner toe. The dam is also safe from sliding. In a high dam there is some tendency for the buttresses to buckle. This has been guarded against, in some dams, by bracing the buttresses together. In a large dam the cost of bracing becomes a serious item. Noetzli, in the paper referred to, proposed a design of hollow buttress, with cross walls and struts which would avoid the necessity of bracing between the buttresses. In many dams the buttresses have been strengthened by battered counterforts and in some the buttresses are made wedge-shaped being splayed outwards in the downstream direction.

25. Failures of Multiple Arch Dams.

(*a*) There have, however, been a few failures and these failures are instructive. Conspicuous among them is the Gem Lake in California. The causes of the deterioration of this dam were fully explained in a paper read before the American Society of Civil Engineers in 1925, and discussion thereon. The dam was built in 1915-16, and consists of 16 arches of 40 ft. span. The maximum height from the crest to the deepest part of the foundation is 112 feet. The upstream face is inclind at an angle of 42° to the vertical. The dam was built in Sierra Nevada, in one of the most remote situations in the Westeren U. S. A., at the exceptionally high elevation of 9,050 feet above sea level. The temperature in the coldest part of the year is as low as 25° F below zero. After some years, disintegration of the concrete was found to have taken place, due to thin layers of the concrete freezing consecutively. This increased rapidly. Attempts were made to remidy matters by waterproofing the face, "Ironite" being used for the purpose. After this treatment the dam was water-tight for some time, but then deterioration set in again. It is noteworthy that the top 30′ of the dam remained perfectly sound and that bottoms of the arches were little affected. The damage was confined to the middle belt, where the concrete had become a dead, inert material, with little strength. The buttresses remained in perfect condition.

(*b*) Gleno Dam was built in the Alps near Bergamo, at an elevation of 6,000 feet. In this case also the site was inaccessible. This work failed due to bad design, poor workmanship and lack of supervision at the time of construction as explained by Mr. Luiggi in a paper on the subject.

Multiple arch dam for 150 feet depth of water

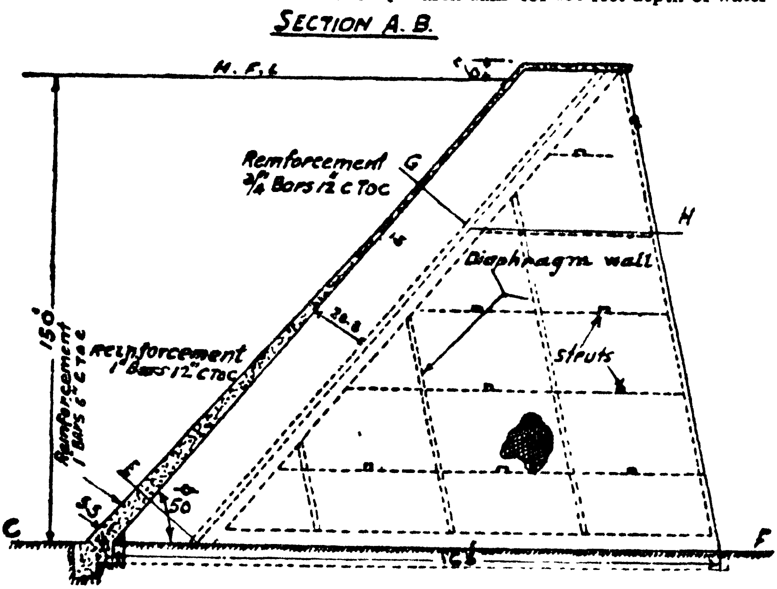


Fig. 14 (a)

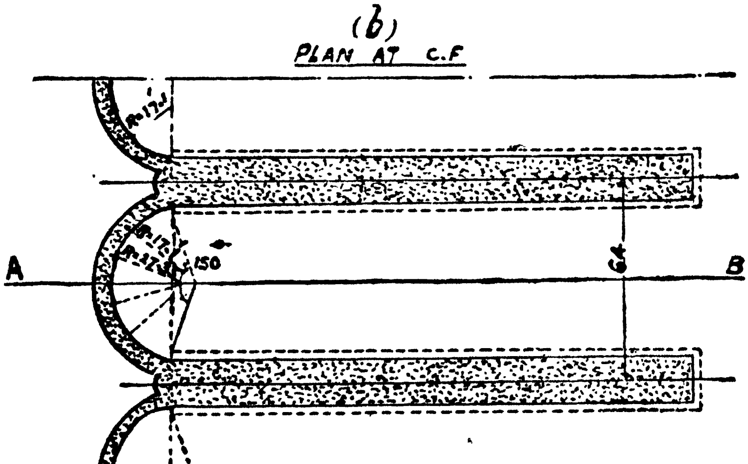


Fig. 14 (b)

(*c*) It is not possible to discover any examples of failure of multiple arch dams that are not attributable to intense cold or defective design or materials. In fact, the failures described only serve to emphasis the success of those which have been properly designed and built in suitable places. The failure of the Gem Lake Dam is, however, a warning of the risk of situating a dam in a place where there are very low winter temperatures, although even in that dam it is to be noted that no deterioration on the part of the dam above water level took place. In temperate and hot climates multiple arch dams have much to recommend them. They are particularly suitable for a country like India where damage by frost need not be feared except at extremely high altitudes and where economy is such a very important consideration. As will be shown later, within the limits of height of the majority of Indian dams, this type of dam is much cheaper than a masonry gravity dam.

26. Comparative Cost of a Multiple Arch Dams, Other Types of Dams.

Example of a multiple arch dam. The details of a multiple arch dam have been

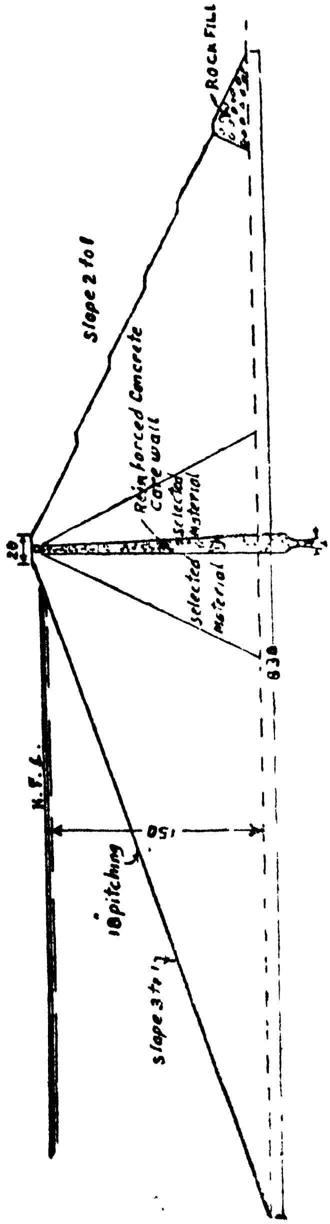


Fig. 16

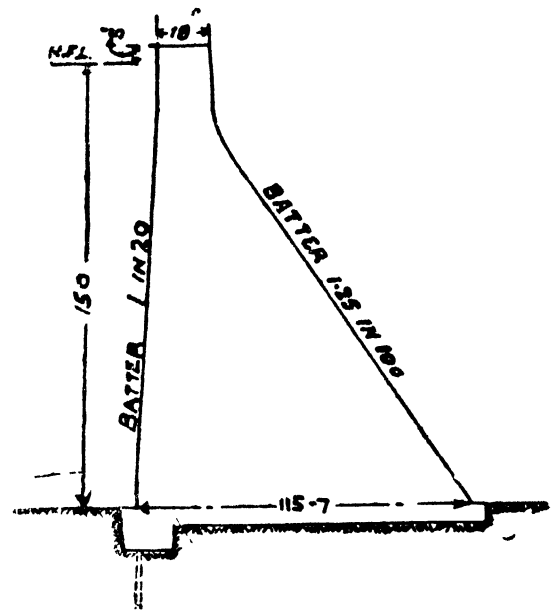


Fig. 15

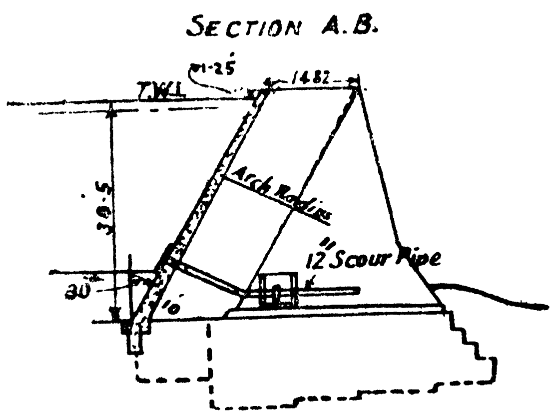


Fig. 18 (a)

worked out by C. B. William M.I C.E (C. P. India) for the purpose of comparing its estimated cost with (a) a masonry gravity dam and (b) an earthen dam. For the purpose of this comparison the rates of materials and labour recently current in Central India have been taken. These,

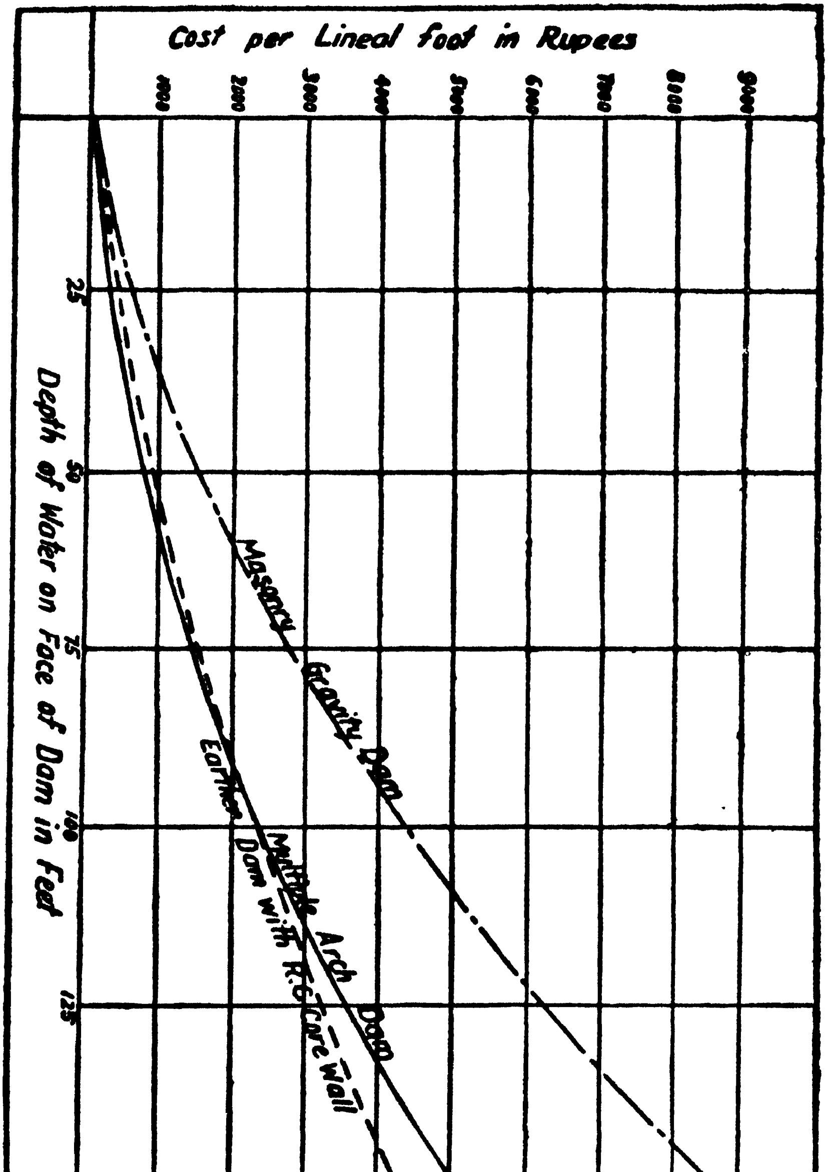


Fig. 17.

of course, vary in different places and from time to time so the comparison cannot be taken as more than roughly correct. It is, however, sufficient to show the marked advantage in this respect possessed by the multiple arch dam.

The design of the dam illustrated in Fig. 14 (a and b) is based on the principle already described. The maximum head of water on the dam is 150 feet. The intrados of the arch barrel and upstream face of the buttresses are inclined at an angle 40° to the vertical. The buttresses are spaced 64 feet centre to centre. The intrados of arch is three centred. The span is constant from top to bottom of the dam. The extrados is a segment of a circle, the radius of which increases with the thickness of the arch. This is 1 foot 6 inches from the crest to the point at which this thickness becomes theoretically correct. Below that point it increases to a maximum of 5 feet 6 inches. The butteress are hollow down to 25 feet above the base. The walls increase in thickness from 1 foot 6 inches at the top to 5 feet 3 inches at the bottom and are interconnected by reinforced diaphragm wall and struts. Both buttresses and arches are lightly reinforced. The main function of the reinforcement is to prevent shrinkage cracks and the cracks that have appeared in many dams in unreinforced buttresses at the inner toe. The rest of the details will be sufficiently clear from the figures. The design is not intended as a final one, but merely for the purpose of arriving at a comparatively accuiate estmimate of the cost.

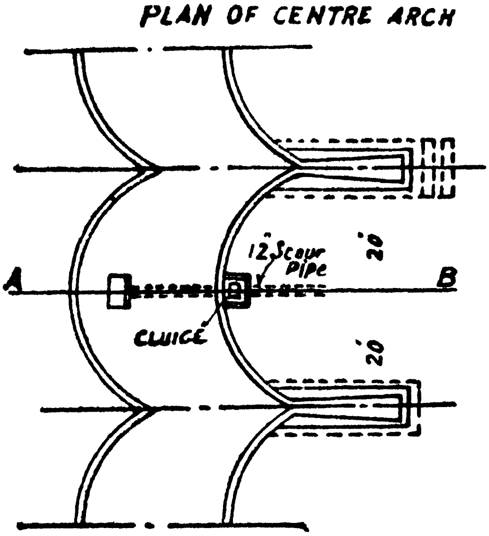


Fig. 18 (b)

The cross section of the masonry dam compared with the multiple arch dam is shown in Fig. 15. This is of the same height as the latter and its cross section in the standard profile for material with a spcific gravity of 2·40 which has been described elsewhere. This cross section of the earthen dam is shown in Fig. 16. The reinforced concrete core makes this somewhat expensive, but it is doubtful whether a cheaper form of dam would be safe for a dam over 100 feet high.

Comparison of costs.

The comparative estimated cost are shown graphically in the diagram Fig. 17 for depths of water up to 150 feet. It will be seen that the cost of the multiple arch dam is less than 2/3rd that of the masonry dam. The cost of the former is nearly the same as that of the earthen dam upto 100 feet depth of water, for higher dam the cost of the multiple arch dam increases relatively more rapidly and it becomes somewhat more expensive. In this connection, it must be considered that an earthen dam for a depth of water over 100 feet is a comparatively high one and although there are many dams of this type in existence that are higher, their construction requires great care and the choice of suitable materials, which often do not exist in the vicinity of the dam site.

27. Examples of Existing Multiple Arch Dams.

Ingleburn dam. As an example of a recently construced small multiple arch dam, the Ingleburn Dam near Sydney, New South Wales, may be mentioned. It consists of 6 arches. The angle subtended by the arches is 132°. The butteresses are spaced 40 feet centre to centre. They are solid and of the shapes shown in the drawing. The cement concrete in the arches is in the the proportion 1 : 1½ : 3 with hydrated lime added in proportion of 5 percent by weight of the cement. For the buttresses and foundations the proportion is 1 : 2 : 4. The arches and buttresses are reinforced. The maximum depth of water in reservoir is 40 feet. Its capacity is 10,800,000 gallons. This is a good example of a well-designed and economical dam of its size. Fig. 18. (a) Page 549 and 18 (b).

28. Bibliography.

1. "Stresses in thick arches of Dams," by B. F. Jakobson, Trans. A.S C E., U.S.A. 1927.
2. "Elastic theory of arches," by William Cain in discussion of (Bib. 1) 1927.
3. "Graphic method of Determining the stresses in circular arches under normal loads by the Cain formula," by F. H. Fowler. Trans. A. S. C. E., U. S. A. 1928.
4. "Analysis of thick arch dams including abutment yield," by P. Craviter, Trans. A. S. C. E., 1936.
5. "Arch deflections and temperature stresses in curved dams," by I. E. Houk, No. II. Engineer, 1937.
6. "Das Talsperrenge Wolbe", (the arch Dams), Lee man and Company, Zurich 1936.

7. "Design of Dams," by Hauna, W. Frank and C. K. American MacGraw Hill Book Company, New Nork 1931.

8. "Engineering for Dams," by W. P. Creager, I. D. Justin and J. Hinds, Chapman & Hall Ltd., London and John Weley & Sons. I. N. C , Now York, 1944.

9. "Hand book of applied Hydraulics," by C. V. Davis Mac Graw Hill Book Company New York and London 12,94

10. "Reportion arch Dam Investigations," by Arch Dam Committee U. S. A. Vol. III, the Engineering Fondation, 1934.

11. "Tests of models of Arch Dams", Report on the Dams by Arch Dam Committee, Vol. II, the Foundation, 1934.

12. "Study of Arch Dams," by Stucky Alfred, Bull. Tech. Suisse Romande, Lausame, 1922.

13. "Trial Load Method of Analysing Arch Dams," Bureau of Reclamation U. S. Department of Interior, Boulder canyon Project, Final Report Part V, Technical Investgatiions, Bulleten I, 1938.

14. "Analysis of Arch Dams by Trial Load Method," by C. H. Howell and A. C. Jaquith, Trans. A S. C. E., 1929.

15. "Trial load Analysis of curved concrete Dams," by I. E. Houk, Engineer July 1935 U. S. A.

16. "Arch Dam Analysis by Trial load simplified," by H. M. Westerguard, Engineering News Record 1931, U. S. A.

17. "Cleber die Berech nungder Fundament deformation," by Fredrik Bogt., Det Norske Vidonsks—Akademi, 1925.

18. "Dams and Control work," by Bureau of Reclamation U. S. Department of Interior, 1938.

19. "Improved type of Multiple arch Dams," by F. A. Noetzli, American, Society of Civil Engineers, 1924.

20. "Experience wit-Maintenance of concrete Dams in High Altitudes," by W. L. Chandwick U. S. A.

21. "Treatise on Dams," United States Bureau of Reclamation.

PART III

TANK IRRIGATION

(Storages and Dams)

CHAPTER VI

Buttress And Reinforced Concrete Dams

1. Introduction.

The principal structuaral elements of a buttress dam are the water-supporting upstream face or deck and the buttresses. These water-bearing upstream members are supported upon the buttresses and span between them ; the buttresses are equally spaced, triangular walls proportioned to transmit to the foundation the water load and the weight of the structure. Solid gravity dams resist the forces acting against them primarily by weight alone. Strength of masonry is critical only when the height is great and then only over limited areas. In massive arch dams, described in Chapter V of this Part, the strength of masonry is more fully developed. However, not all dam sites are suitable for massive arches.

Frequently, a reduction in cost with no sacrifice in safety can be effected by a dam of structural form. On accunt of efficient development of latent strength, masonry quantities are reduced. More intricate form work and the need for reinforcement increase unit costs, but under favourable conditions and appreciable net saving in total cost may be achieved. This is particularly true in locations where the cost of procuring or transporting the cement requied for a more massive structure is prohibitive or where other construction materials are scarce. The more efficient use of masonry strength does not necessarily mean higher maximum presssures than are permited in gravity structures. However, with carefully placed reinforced concrete, increased stresses may be allowed. If these occur at the base, better and more carefully prepared foundations may be needed. This can be offset by the use of spread footings.

Buttressed dams are more subject to damage or destruction by sabotage or military attack than massive dams. Because of the thinness of their members they are sensitive to even moderate deterioration of the concrete ; hence they must be carefully built and careful consideration must be given to any unusual exposure conditions.

2. Forces at Buttressed Dams.

Buttressed dams on rock foundations are subject to the same forces as other dams except that the downward component of the water pressure is greater and uplift is less. In fact, because of the easy lateral escape of pressure under the buttresses, uplift from headwater in the case of rock foundations is usually neglected. However, where the rock is liable to uplift pressures on horizontal seams, the foundation should be drilled for drainage Full uplift from tail water should always be included. Buttressed dams on pervious foundations may require footing slabs designed to resist uplift loading. Wind pressure, which is neglected in other dams, may merit consideration if a diagonal wind of high velocity can reach the dowstream side. On high thin buttresses, such pressures may increase the danger of buckling. Because the wind cannot strike the buttress face normally, a pressure of 10 lbs. per sq. ft. over a width not exceeding the clear distance between buttresses should be safe. Authoritative data are lacking. For well-braced or double-walled buttresses, such pressures are of little importance.

3. Earthquake Loading for Buttressed Dams.

Earthquake forces are computed by rules established in Chaper IV of this part. Slab and round-head buttress dams are particularly efficient in resisting such forces, because of small mass relative to rigidity for individual units, coupled with ability of the dam as a whole to yield slight permanent displacements. For general stability, the most unfavourable direction of motion for such dams is upstream horizontally. For the deck slabs, the maximum masonry inertia load is for motion normal to the face. Definite rules for the computation of the increased water pressure on the inclind face are lacking. The approximate rules established in Chapter IV of this part may be followed. Cross-stream acceleration is unimportant for straight-faced dams. Unreinforced buttresses may need checking for slab strength under lateral loading, but the buttresses cannot overturn sidewise.

For multiple-arch dams, the most important earthquake effect may come from transverse motion. The force resulting from the inertia of the masonry in the arch barrels is readily computed. Unless the buttresses are stable in themselves or are securely braced against lateral displacement. they must be held against over-turning sidewise by the arches. This introduces an accumulating transverse load, applied at the spring lines and transmitted to the abutments by the arches. The resulting stress may be of appreciable magnitude. The load is difficult to compute but may be approximated by dividing the buttress into blocks and computing the force required to prevent overturning, proceeding step by step from the top downward. The small volume of water in the troughs over the piers adds to the uncertainty of increased water pressure from cross-stream acceleration. This effect has generally been ignored, although not on the side of safety.

4. Spacing of Buttresses.

The spacing of the buttresses is governed by economy If the spans are short, face slabs or arches may be thin with a small volume of masonry. On a simple unit stress basis, the buttress thickness would be proportional to the span and total thickness for the dam would be constant. Practical considerations preclude the use of very thin walls; hence beyond certain limits the volume of individual buttresses remains constant regardless of spacing. The result is a more or less definite economic limit to the spacing. Many factors enter into the determination of this limit. Two buttresses each 2 ft. thick cost more to construct than one buttress 4 ft. thick. The cost of excavation and foundation treatment is also greater for two thin buttresses than for one thick one. For very long spans the cost of falsework for the facing may be high and secondary stresses in the haunches may be troublesome. Economic buttress spacing increases with the height of the dam. Usually the height is variable, giving a variable economic spacing. Variable spacing is usually avoided by adoption of a standard for the entire dam. Separate standards for the abutments and the central portion of the dam may be used if desired, but this not the usual practice.

5. Classification of Buttress Dams.

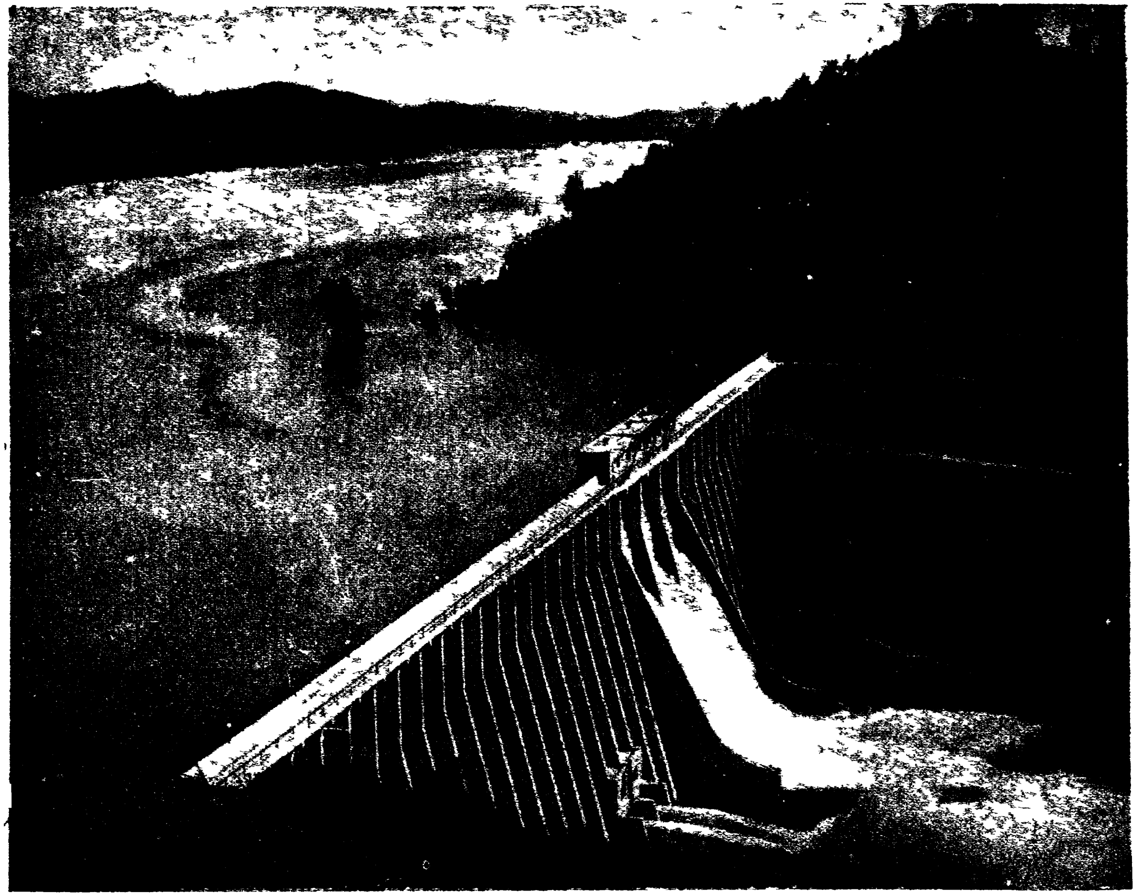
Buttress dams, as generally classified with respect to the principal types of decks most commonly used, are of three kinds;

(1) The flat-slab type. (2) The multiple-arch type; and (3) The massive-head type,

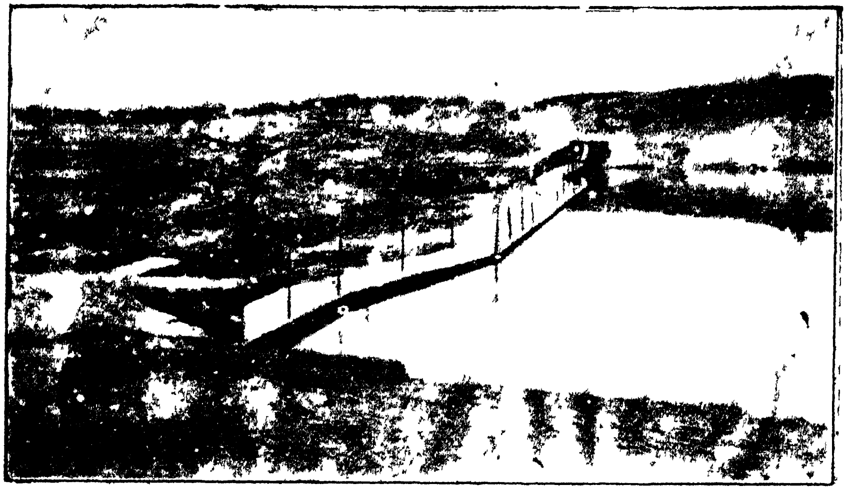
(*a*) **Flat-slab type.** In the flat-slab type the deck consists of reinforced concrete deck slabs spanning the distance between adjacent buttresses, the slabs being separated by the buttress tongues and supported by reinforced haunches or corbels which are usually constructed monolithically with the buttresses. To provide watertight expansion joints and to obtain a measure of flexibility through articulation, so that the deck may conform to slight unequal settlement of the buttresses, the surfaces of the corbels and buttress tongues that contact the deck slabs are coated with an asphalt or mastic filler. The usual shape and arrangement of deck slabs, buttress corbels, and buttress struts in a typical flat-salb type of buttress.

Stony Gorge Dam in California, and Thief Valley Dam in Oregon, are examples of the flat-slab type of buttress dam as constructed by the Bureau of Reclamation. Both dams have overflow spillway sections.

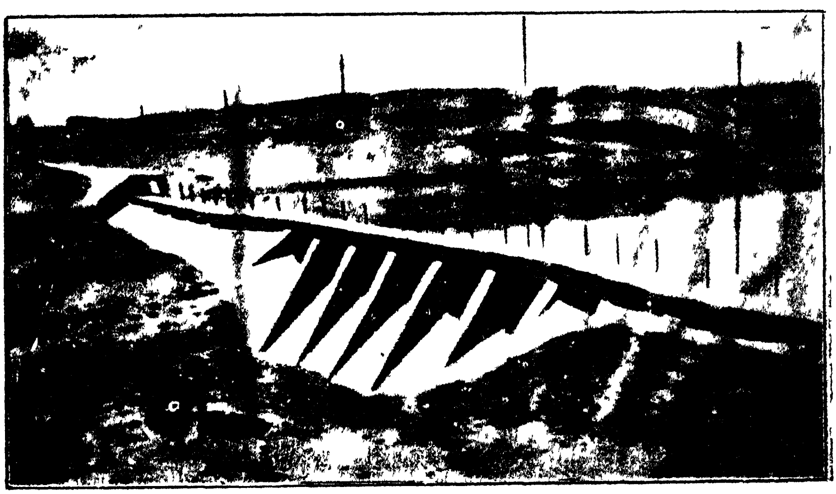
(*b*) **Multiple-arch type.** In the multiple-arch type buttress dam each unit of the water-supporting member consists of an inclined arch barrel supported by adjacent buttresses. The buttresses may be of the single-wall or the double-wall type. In some dams the walls of the double-wall buttresses are continuous with the arches. Corbels are commonly used to



Stony Gorge Dam (Concrete Buttress Dam) Completed in 1928 height 125



Bhowra Concrete Dam upstream view



Bhowra Concrete Dam downstream view.

transfer the water load from the multiple-arch deck to a single-wall buttresses. Bartlett Dam on the Verde River, Arizona, is a notable example of the multiple-arch type of buttress dam as constructed by the Bureau of Reclamation. Big Dalton Dam, constructed by the Los Angles Flood Control District is an outstanding example of the multiple-arch type with single-wall buttresses.

(*c*) **Massive-head type.** In the massive-head type of buttress dam, shown in the water supporting member consists of the upstream portions of the buttresses themselves which are enlarged to full span, with the upstream faces curved or shaped in such manner that the water load is transmitted to the buttresses in compression. A flashing and key are provided at the joints between the buttress heads to prevent leakage. This type of construction offers the advantages of mass-concrete methods and compressive throughout the deck. This type of dam is comparatively recent design, having been proposed about 1925 by the late F.A. Noetzli, and only a few designs have reached the construction stage.

(*d*) **Other type.** The truss buttress and the columnar buttress types of dams are modifications of the flat-slab type, and are characterised by reinforced-concrete trusses or a series of inclined columns of reinforced concrete substituted for the conventional solid buttresses. Only a few of these types have been built, principally because they require foundations of exceptional strength and stability and moreover they are practically as expensive to build as the more conservative and conventional types.

In the multiple-dome type of buttress dam, a modification of the multiple-arch type, the water supporting member consists of dome-shaped deck sections spanning the spaces between adjacent buttresses The Coolidge Dam on the San Carlos River, Arizona, completed in 1939 by the United States Indian Irrigation Service, is the only outstanding example of this type in the United States. This dam, consisting of three domes supported by massive buttresses spaced at 180 feet between centres, is 250 feet high. This type was found to be the most economical for the foundation conditions existing at this site. A few small dams of this were built in Italy buring the early days of Irrigation.

6. Site Requirements for a Buttress Dam.

Buttress dams are suitable for a wide variety of topographic and foundation conditions. The most desirable canyon is one having gently sloping walls. Such a condition makes the connection between dam and abutment less difficult, and more stable buttresses are secured than when precipitous walls are encountered.

For a site requiring a long dam, the buttress dam, if not precluded by other conditions, will require less material and will usually cost less than a solid cancrete dam. At a site where conditions are such as to require an overflow spillway or outlets extending through the dam, the buttress dam may be more economical than an earth-fill or rock-fill dam.

Some sites having foundations which are not of sufficient strength to support solid concrete dam may be entirely feasible for the construction of buttress dams because of their lighter weight and because the foundation load is supported over a wide area.

7. Considerations Affecting Choice of Type.

Until a few years ago the opinion was quite prevalent among many Engineers that the buttress, type dam was suitable only for structures of low or medium height. It is the consensus of engineers having wide experience in the design of such dams that there is no reason why they can not be built to more than twice this height, and that height limitation does not preclude the use of the buttress type for even the highest dams.

Choice of the type of buttress dam best suited to any site found to be favourable for a dam of this general type is usually determined on the basis of tentative designs and cost estimates of each of the various types that are selected as most suitable to the general requirements and the conditions of site. In making the choice, consideration is given to the following general advantages and disadvantages of each type.

1. **Flat-slab type.** Since the units of this type are structurally independent, ordinary foundation deformations or settlements have little or no effect on the distribution foundation stresses. It is therefore often preferred, if otherwise satisfactory, for construction on jointed or faulted foundations, particularly in regions subject to seismic disturbances where unequal subsidences or deformations may be expected. Although it is extensively used for low dams because of its simplicity and economy of construction, this type is by no means restricted to low dams. The principal disadvantages of the flat-slab type are the complete dependence upon the tension steel of the slab for support of the imposed loads, and the difficulty of transmitting the slab load to the buttresses without causing objectionable and excessive stress concentrations in the buttress corbels.

2. **Multiple-arch type.** The multiple arch type is, in general, most suitable for the higher buttress dams with buttress spacings of 50 to 60 feet. Such a dam is a continuous structure in which the stability of each unit may be dependent upon the stability of adjacent units. Despite some objection to the multiple arch type for this reason, no failures of this type of dam are known to have occurred due to structural faults in design. With 180 degree arches which are usually used, the lateral thrust is so small that stability of one unit is not dependent upon stability of adjacent units.

3. **Massive-head type.** For lower dams the spacing of buttresses may be such that a massive head design may be more economical than the multiple-arch, and may compare favourably with the flat-slab type. Since the head is designed so that dead weight and water load induce only compressive stress in the structure, very little or no reinforcement may be required with the massive-head type, which may be a factor favoring its selection. This type of dam is considerably heavier and has greater sectional area on horizontal plans, so that resistance to sliding is greater and the shear-friction factor is higher than with other types. Since the units are structurally independent, small unequal foundation settlements may be permissible.

8. Design of Buttresses.

Buttresses for all of the dam types shown in Fig. 1 are analysed for stability in a manner similar to that used for gravity dams, Chapter IV of this part. The design element instead of being a slice of unit thickness, is taken as a full panel. In addition to meeting the stability requirements for gravity dams, the buttress must conform to the design rules for structural concrete members.

The buttresses may be considered as vertical cantilever beams of variable cross-section Both the width and the thickness may vary. The width must be sufficient to avoid tension at the upstream face when fully loaded and also to avoid excessive compression at the down stream face. As in all beams, simple bending stresses are smallest for a given cross sectional area if the buttress is made wide and thin. However, if too thin, failure may occur by buckling.

In order to fix the required thickness of buttresses to prevent buckling, they are considered to be bearing walls instead of beams, the minimum allowed thickness being the same as for columns. A reduction in stress must be made where the unsupported length exceeds ten times the thickness.

It is usual to reduce the unsupported length by means of struts or to increase the width of the compression face by adding a flanged section. Both of these devices are illustrated in Fig. 2 and both may be used. In high dams, additional flanges or pilasters may be added along the width of the buttress, either in place of or in addition to the struts. The Florence Lake Dam. California, shown in Fig. 2, illustrates the use of pilasters. There are no stablished rules for the dimensions or spacing of pilasters and struts. However, the unsupported length in the highly stressed portions of the buttress should not exceed ten times the effective thickness. At other places the unsupported length may be increased to 15, provided the stresses are not in excess of 50 percent of allowed stresses.

The reinforcement in the struts is usually continuous through at least three days, but in some cases it has been carried continuously throughout the structure with no deleterious

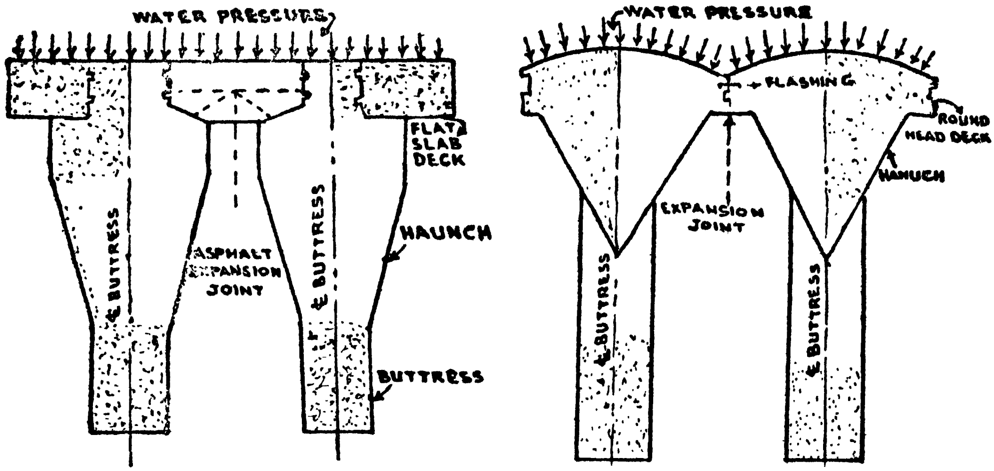


Fig. (a) Ambursen Massive Buttress Type — Section through Deck and Buttress

Fig. (b) Round-Head Buttress — Section through Head and Buttress.

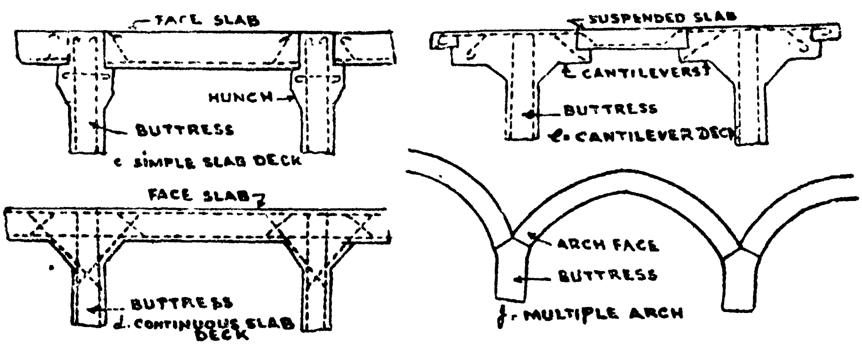


Fig 1 Types of Buttress Dams.

effects from contraction. The struts should abut solidly against the abutments. The horizontal building joints in the buttresses should be at the elevation of the struts if practicable. In multiple-arch dams, where the economic span is usually greater than for other buttressed types, the need for struts and pilasters may be eliminated by using double-wall or hollow buttresses, each stable within itself. This type proposed by Noetzli in 1924 has been used in a number of dams. A system of tie-walls and struts between the webs assures unity of action A buttress of this type can be made secure against buckling without resort to long struts between buttresses.

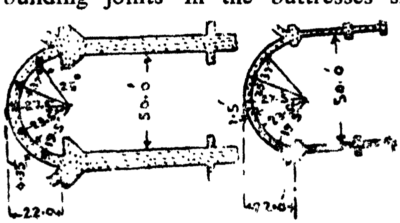


Fig. 2 Typical Horizontal Sections.

Double-walled buttresses have a distinct advantage as to appearance particularly in high dams.

Some useful data for preliminary design of multiple arch buttress Dams is given Fig. 3, (Folder).

9. Stresses in Buttresses.

Horizontal and vertical forces and moments on the buttresses are computed as for gravity dams. The joint between the facing and the buttress is sufficiently rough that the facing will follow the buttress in case of overturning ; hence the full weight of the facing and of all other parts of the dam above any section being analysed is included with the downstream forces.

Examples of alternative buttress-slab connections are shown in Fig. 1. For monolithic deck and buttress, as illustrated in Fig. 1 (d) and for multiple arches actually ti d to the buttresses, Fig 1 (f), the buttress and a half-span of deck on each side act as a T-beam. In the simple slab type, Fig. 1 (c), T-beam action is rendered somewhat uncertain by reduced shearing strength along the joint between the slab and the buttress. However, friction is assumed to hold the slab against movement on this point and it is not unreasonable to assume monolithic action. Also, because of increased eccentricity of the vertical loads, the maximum compressive stress in the buttress, which occurs at the downstream face, is greater for the buttress-slab T-beam combination than for the buttress alone. The reverse is true at the upstream face, but buttress compression there is not critical, hence it is on the side of safety to assume unit action. If there is any possibility of a critical upstream cantilever stress in the buttress, the condition of buttress alone should be investigated. As in structural T-beams, only the web (buttress) is assumed to resist shear. With the cantilever deck, Fig. 1 (e), the suspended deck is excluded in computing butress stress. For round head buttresses, the buttress and butrress head are treated as a unit. Vertical-unit pressures are assumed to be linearly distributed, as for gravity dams. If uplift is considered, it is treated in accordance with the rules of Chapter IV of this part. Inclined stresses at the faces are computed according to equation in Chapter IV. The normal pressure at the upstream face is the water pressure, as for a solid dam. Shearing stresses and principle stresses at interior points may be computed according to principles discussed in Chapter IV. Because of complexity of form and action, the applicability of the ordinary assumptions of stress distribution to buttresses is more uncertain than in the case of gravity sections. However, they give a general idea of buttress action. Any uncertainty is absorbed in the factor of safety used for stresses. In especially important cases resort may be had to model studies. There is an intensified normal force along the junction between the buttress and the buttress head or facing. As as approximation, the principal stress along this junction plane may be assumed to be equal to the "normal" pressure due to the panel water load and the normal component of the weight of the facing and buttress head. This approximate stress usually will be less than the true principal stress by a small percentage, which may be assumed to be absorbed in the factor of safety. If the stress computed in this manner approaches the danger point, an internal stress analysis may be made.

The resultant must be so located that tensile stresses are produced the foundation level. This is accomplished by adjustment of the buttress width. In fact, the resultant usully can be made to fall near the centre of gravity of the buttress, thus approximating a uniform distribution of vertical pressures.

10. Upstream Face Dams.

The upsream face of a buttressed dam is inclined to provide the vertical water load required to insure stability. The downstream face is inclined only as required to provide an adequate buttress width. In most existing dams, the upstream slope, tan ϕ ranges from 1·00 to about 0·70. The steepest slope that will satisfy stability requirements is economical. Near the top of the dam the face can usually be steeper than at lower elevations, because the width of the top is greater than needed for stability and the amount of masonry may be reduced by using a variable slope for the water face. This Principle has been utilized in a few dams, but the general practice is to use a straight upstream face except for a short vertical lift at the top.

11. Shrinkage Cracks in Buttresses.

Concrete in buttresses as in all other structures is subject to shrinkage. The base of the butress is prevented from shrinkage on account of contact with the foundation, particularly when or rock. As a result, vertical or inclined shrinkage cracks tend to form in the buttresses. Such cracks observed in many dams, usually run more or less in the direction of the planes of minimum principal stress,although apparently not specifically related to stresses caused by loading. Should such a crack assume a disadvantageous direction, it would weeken the buttress. Cracking can be avoided or controlled by reinforcement, by contraction joints or by a combination of these means.

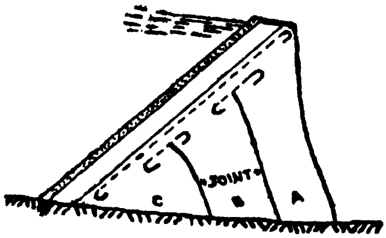


Fig. 4. Big Dalton Dam

The use of contraction joints is illustrated in Fig. 4 which shows a buttress of the Big Dalton Dam, California. This buttress is reinforced and part of the steel passes through the joints which no doubt influences their effectiveness. No appreciable cracking has been observed. Similar joints were used in the Possum Kingdon Dam.

12. Masonry Buttress Dam.

A summary of (Col W. M. Ellis) analysis of buttress dam is briefly given below : –

(A) buttress dam consists of a number of buttresses of piers. dividing the space to be dammed into a number of spans. To hold up and retain the water between these buttressses, panels are constructed consisting of horizontal arches or ferro-concrete beams and slabs or steel beams and planting.

(B) **Stability of buttresses.** In considering gravity dams, the stresses and stability are examined with reference to a slice or profile of the dam, one foot in width, but in the case of a buttress dam, it is necessary to consider the thrusts and weights of one buttress and two adjacent half spans. With the exception, the methods of examining the force on the buttresses are generally similar to those employed for the gravity dam.

(C) **Panelling between buttresses.** If the panelling between the buttresses consists of masonry arches, the stresses in these are those in the arches of an arched dam.

Where ferro-concrete or steel panelling is employed, the designs may be worked out from the loading due to water pressure in a similar way to the road bearers and platform of a bridge.

In the case of dams and many other irrigation works, weight in the structure is a feature tending to stability and thus ordinary masonry, even at slightly higher prices, is preferable in such works to ferro-concrete or steel structures.

(D) **Arched buttresses dams.** The arched buttress dam is only type of buttress dam likely under existing conditions to come into extensive use in India and the design of this type alone is dealt with in detail.

It is desirable in such dams to build the arch with an axis inclined to the vertical from 30° to 45°

While this increases the total pressure on the face of the arches, it does not increase the intensity of stress and it has the effect of depressing the intersection of the line of resultant of the water pressure and weight of masonry and of making the resultant to make a steeper angle to the horizontal and thus increases the frictional resistance to sliding which is a danger to which this type of dam is specially liable.

(E) **Spacing of buttress for arched dams.** The spacing of the buttresses will depend on the depth of water and the nature of the foundations and should be considered separately for each case. Generally the spacing would not be less than three fourths of the depth of water or more than the depth. This only refers to arch-panelling and when ferro-concrete facing slabs or other panelling is used, the buttresses would be closer together than three-fourths of the depth. The width of the buttress should be sufficient for the skew backs of the arches to rest on. The arch may be such as to subtent an angle of 100 to 120 degrees at the centre.

(F) **Stresses due to the weight of masonry of an inclined arch.** Part of the weight of each arch-ring of an inclined arch is carried by arch action and supported by the reaction of the abutments and the remainder is carried from each arch-ring to the successive rings below and is supported, as in a gravity dam, by the reaction of the foundation.

If A be the angle with the vertical of the axis of an arch, R_c the radius to the centre of the arch-ring then for each cubic foot of masonry of weight w_p, a part equal to w_p sin A is supported by arch action and a part w_p cos A is supported by the reaction of the foundations.

If S be the stress in the arch induced by the weight of the arch itself.

$S = w_p \sin A \times R_c$ (A)

and S must be added to the stress induced on each arch-ring by the water load to get the total thrust on the arch-ring. The stress on the base or any joint of the arch at right angles to the axis is found as in the case of a gravity dam allowing $w_p \cos A$ instead of w_p for the weight of each cubic foot masonry in the profile above the joint.

(C) **Example of a design of an arched buttress dam.** An example of design of an arched buttress dam subject to the following conditions will now be worked out.

Conditions. Depth of water H below the top of the dam 40 feet.

Arches to be in stone concrete and buttress rubble masonry of specific gravity $2\frac{1}{4}$.

Design of arch. The axis of the arches to be inclined at 30° to the vertical and the limiting stress in arch masonry to be 10 tons.

The clear spacing between buttresses will be made $1\frac{1}{2}$ times the depth = 60 feet.

The radius of the arches to the intrados will be made $37\frac{1}{2}$ feet which gives a central angle 106° and a versed sine of 1·65 feet.

The top width of the arch will be 3 feet, this being about $\frac{1}{2}\sqrt{H}$.

The slopes of the up and downstream faces will be the same. The bottom width (b) to give the limiting stress will be somewhat more than that given by equation (F) Paragraph 3 Chapter II of this Part, because of the extra stress due to the weight of arch. H, suitable width, has been found by trial to be 6 feet and the suitabilty is tested as follows.

The value of (*b*) being considerable in reference to the radius, the long formula equation (D) Article 3 Chapter V Part III must be applied to calculate the arch stress due to water pressure.

$$S = \frac{2Hw}{b/R\,(2-b/R)} = \frac{2 \times 40 \times 1/36}{6/43{\cdot}5\,(2-6/43{\cdot}5)} = 8{\cdot}7 \text{ tons.}$$

The arch stress due to weight of arch from equation (A) Paragraph 14 is $S_a = w_p \sin A \times R_c = 1/16 \times \frac{1}{2}\,(37{\cdot}5+3) = 1{\cdot}3$ tons.

Therefore, the total arch stress $S + S_a = 8{\cdot}7 + 1{\cdot}3 = 10$ tons.

Design of buttress. The buttress may be made 10 feet thick at the base and batter at 1/20 on each side (Fig. 5).

The general method of designing the buttress is by trial, tested by computation or graphic statics, in a similar way to a gravity dam profile.

There is, however, an intermediate factor which must be discussed before protecting to design the buttresses, *viz.* : to what extent the weights of the arches may be expected to contribute to the stability of the buttress.

It is evident that they must do so to a considerable extent as the buttresses can neither slide nor overturn without the same happening to the arch or at any rate, to a part of it. The greater the angle at the centre of the arch, the greater will this assistance be and *vice versa.*

In dealing with this equation, Bligh considers the whole of the two half arches on either side of a pier as constituing an upstream prolongation of the pier split into two diverging branches.

He plots the equivalent weight of the two half arches distributed as a subsidiary part of the pier extending upstream to a plane tangential to the upstream faces of two adjacent arches. This appears to be an overbold assumption, but it is considered safe to include 1/3 of each half arch and consider these as constituting a split upstream prolongation of the buttress teminating at a′ a′, a a, (in Fig. 5) the intersection with the prolonged pier of the plane intersecting the arches an 1/3 their length along extrados.

The longitudinal section of this prolongation of the pier is the portion a′ a′, a a. It is assumed that the prolongation is of the same side batters and top and bottom width as the rest of the buttress. This is a fair assumption as the content of the $\frac{1}{3}$ lengths of the two half arches nearly correspond to the content of prolonged buttress.

Under this assumption the upstream slope of the buttress is a a, in section Fig. 5 and a similar slope is given to the downstream end of the buttress. This latter is merely a matter of convenience and any other suitable slope may be given.

If *l* is the base of each of the sloping ends of the buttress and the thickness at bottom

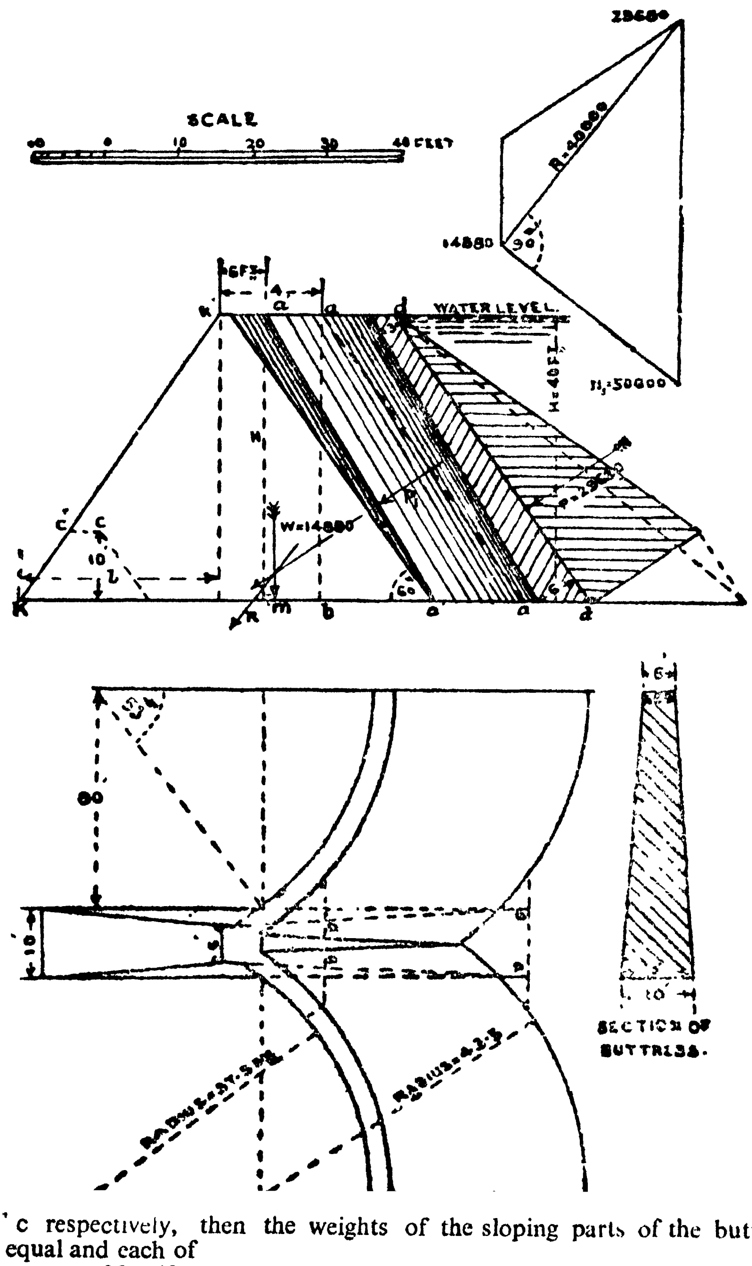


Fig. 5

and top f and' c respectively, then the weights of the sloping parts of the buttress up and downstream are equal and each of

volume; $\frac{lH}{6}$ $(2f+c)=\frac{30\times40}{6}\times26=5200$ cft.

In this case as the the sloping ends of the buttress are exactly the same upstream and downstream, the center of gravity of the whole buttress must pass through the center of the portion of the buttress of which the top is horizontal.

The volume of this central portion per foot length is H/2 $(f+c)=320$ cft.

The buttress should be made of such length as to bring the centre of pressure near the centre of the base so as to make the maximum pressure on the foundations nearly equal to the mean pressure.

The water pressure (P) is the area of the triangle of pressure Fig. 5, multiplied by the distance (70 feet) between buttresses, centre to centre and as taken from the diagram, equals 29,680.

This is applied along (P) in direction at right angles to the extrados slope d d at a point along a a' one-third of its length from the base.

It is found by trial, by means of the diagram of forces, that by giving a length of fourteen feet, measured from a, to the horizontal from top of buttress, the line of resultant comes near the centre (m) of the base a k and it is, therefore, so designed.

The weight of this portion of the buttress is $320 \times 14 = 4480$ units.

The whole weight of the buttress is thus $10,400 + 4480 = 14,880$ units. Plotting this is a diagram of forces (Fig. 5) the resultant equals 40,000 and cuts the base a k three feet downstream of the centre.

The base scales 69 feet long. The mean pressure on the base due to N_1 is $508800/(69 \times 10 \times 16) = 4 \cdot 6$ tons.

The centre of pressure is 3 feet from the centre of the base.

Therefore, the maximum pressure $= S_a\left(1+\frac{6e}{b}\right) = 4 \cdot 6\left(1+\frac{6 \times 3}{69}\right) = 4 \cdot 6 \times 1 \cdot 26 = 5 \cdot 9$ tons per square foot which is suitable.

The amount of masonry per span works out to approximately.

	14,560	cubic feet,	archwork	
	10,733	cubic feet,	buttress	that is; $\frac{25293}{70} = 361 \cdot 3$ cft. per ft. run.
Total	25,293	,, ,,	per span	

A gravity section with a 6 feet top width and a section of the elementary profile has a content of 559 cft. per foot run.

The arch section has, therefore, a great deal less masonry.

The cost, however, of all masonry and especially the archwork would be more per unit for the buttress dam than for a gravity one.

On the whole there would be frequently a saving is cost by using this type of dam which is the one likely to be used more in future.

13. Slab Buttress Dams.

(A) The basic flat slab and butterss type of dam has borne the name of its inventor since 1903. These articulated buttress dams are provided with expansion joints between the decks and the buttress, as shown by Fig 1 (*a*), a typical section through the deck of an Ambursen massive buttress type dam. The deck consists of reinforced concrete deck slabs, separated by the buttress tongues and supported by reinforced haunches which are constructed monolithically with the buttresses.

Of the Ambursen type of dam there are recorded 391 examples completed or under construction, varying in height from a minimum of about 6 ft. to a maximum of 250 ft. distributed geographically as follows :—

United States	335	Australasia	4
Canada	25	Japan	3
Europe	16	Africa	2
Latin America	6		

The foundations of these stuctures represent a wide range of materials, varying from fine sand through coarse sand, gravel, boulders, clay and hardpan to ledge rock and in varying combinations of these materials.

(B) **Ranson buttress dam.** This flat slab type of dams was introduced by W.M Ranson about 1908. In this design, the buttresses were constructed at an angle of 30 degrees with the axis of the dam instead of at 90 degrees as in the case of all other buttress dams. Alternate buttresses where turned 30 degrees to the right, with the adjoining buttresses turning 30 deg. to the left, this construction resulting in a honeycomb or cellular interior. Although the deck was of conventional Ambursen type with a reasonable degree of articulation, the intersecting buttress construction was monolithic throughout the length of the dam, a very undesirable feature.

economy was found in this type of structure and the only two recorded examples are found at Columbia, N. J. and near Cleveland, Ohio.

14. Columnar Buttress Dam.

The columnar buttress dam, a modification of the typical Ambursen dam, substituted for each butress a series of inclined columns terminating in spread footings on the foundatian and carrying on their tops or upstream ends a heavy inclind girder which supported the deck slabs. This type of design appears to have been originated about 1910 by W. S. Mortion and only one example is recorded as having been constructed, 300 ft. long and 45 ft. high, built about 1927 in Missouri. It was suitable only for the best of ledge rock foundations and little or no economy was obtained through the substitution of heavily reinforced concrete columns for the plain concrete buttress of the conventional Ambursen dam, particularly as costly struts and diagonal braces were required.

15. Truss Buttress Dam.

This type of dam, of which the only recorded example was constructed in French Indo-China about 1912 was similar to the columnar-buttress dam described above except that heavy vertical trusses of reinforced concrete instead of columns, took the place of the customary solid buttresses.

16. Cantilever Deck Dam.

Various engineers have advocated designs for buttress dams where in the sloping upstream deck slab is constructed monolithically with the buttress and rigidly tied to it, with the deck slab cantilevered out on each side so that the contraction or construction joint comes in the centre of the span. An objection to this design is that it is less readily adjustable to foundation settlement and only half as many joints for expansion and contraction are pravided as compared with the conventional Ambursen design. This cantilever design has also been frequently proposed with the decks continuous over two or more buttresses, on the theory that an advantageous reduction in deck slab thickness of deck slabs is not considered desirable, the present tendency being to increase the thickness for practical reasons, rather than to reduce it to minimum theoretical limits. Moreover the excessive rigidity of the proposed form of construction is undesirable and the widely separated joints require special flashing. Only one example of cantilever deck construction is recorded, constructed about 1924 in Maine.

17. Round-head Buttress Dam.

Fig. 1 (b) The upstream water supporting member of the round-hand buttress dam, as originated by F. A. Noetzli in 1926, is provided with a radial face, which transmits the water pressure in direct compression through the flared water bearing member to the buttress blow. This type of deck has several distinct advantages : (1) the entire deck is in compression under full water load ; (2) little steel reinforcement is required as bending and diagonal tension stresses are theoretically eliminated ; and (3) savings in constrution cost may in some intances be effected through the use of mass concrete construction methods. Of the round head buttress type of dam the first example built was the Don Martin Dam for the National Irrigation Commission of Mexico. In 1936 a second example 85 ft. high and 279 ft. long was built in Switzerland. It requires firm ledge rock foundations.

18. Diamond-head Buttress Dam.

This structure is a modification of the round head buttress design. As its name implies, it substitutes for the curved upstream face of the buttress head a series of three planes. The only structure of this type recorded as having been built is at Haweswater in England.

19. Steel Buttress Dam.

To complete the history and description of dams utilizing the buttress principal wholly or in part, it is necessary to discuss dams built of steel, of which three examples have been built. F. H. Bainbridge appears to have been the first to advocate, in 1895, the construction of an all-steel dam consisting of an inclined upstream face supported on steel columns. In 1898, a dam of this type was constructed near Ash Fork, Aris, 180 ft. in length and 46 ft. high. The Redridge Dam of somewhat similar design was constructed in Michigan in 1901, 464 ft. long and 74 ft. high. Both of these dams were built on good rock foundation and are still in service. The third and last dam of this type to be built, the Hauser Lake Dam in Montana, with a length of 630 ft. and maximum height of 81 ft. was constructed upon a rock foundation at each end but about 300 ft. in the middle of the river was placed upon gravel into which a steel pile cut-off wall 35 ft. deep was driven an upstream blanket of fine material 20 ft. deep and extending 300 ft. above the dam being installed to prevent underflow. The dam was completed in 1907 and failed the following year. The failure, however, being due to inadequate foundation provisions and not to the design of the dam. If the question of high cost is disregarded the principal objection advanced to steel dams has been the possibility of excessive corrosion of the water-supporting steel face. The Ash Fork Dam has been unwatered and repainted at cosiderable intervals of time, approximately every seven years, whereas the Redridge Dam is said to have been unwatered and repainted even less frequently.

20. Connection of Facing and Foundation.

Particular care must be taken with the connection of the facing of a buttressed dam with the foundation rock. A connection tight against leakage under high pressure must be secured in a relatively short distance. Usually a cutoff trench is excavated into sound rock. Fissures are closed by grout or other means. Important fissures may need so be cleaned out and refilled with concrete. At the Rodriguez Dam in Lower California, Mexico, a shaft was excavated to a depth of 300 ft. in a (presumably) dead fault and refilled with concrete. Similar but much less extensive treatment is frequently required.

The facing may be made monolithic with the cutoff or may be joind to it in any satisfactory manner. The joint must be tight under all conditions of deformation. For multiple arch dams, the effect of foundation restraint on normal arch action must be donsidered.

21. Buttressed Dams on Soft Foundations.

Any of the buttressed dams described in this chapter may be adapted to soft foundations by the use of spread of footings, or if necessary complete foundation slabs inverted arches may be provided to reduce the foundation pressure to allowable limits. If the foundation is porous the upstream cutoff must be carried to an impervious stratum or if this is not possible, the foundation slab must be designed for uplift and made of sufficient length to provide the required percolation distance. (See Chapter VI for treatment of porous foundations). If uplift is not allowed for, the foundation slab should be provided with large weep holes which should be protected from freezing and the foundation must be safe against piping through the weep holes.

22. Reinforced Concrete Dams.

Reinforced concrete dams are as yet mainly confined to America. Possessed of many theoretical advantages, it is probable that the few failures on record are principally due to over-confidence placed in the principles of there design and insufficient care given to workmanship and good foundations.

When properly constructed, there is little doubt that, as a type, reinforced concrete dam is by far the most satisfactory form of dam. But is evidently quite useless to construct a dam which is satisfactory in itself, if the foundations are insecure and many of the earlier dams possess foundations evidently designed in accordance with sound rules for houses, or bridge piers, but which are quite useless when applied to dams.

A normal R. C. dam consists of a series of buttress of triangular section, carrying a flat slab of reinforced concrete on their upper face.

The dam may be designed as an overflow dam or may be provided with a separate waste weir, as circumstances require.

Let the horizontal distance between the centres of the buttresses be l feet.

Let at any depth h below the high flood level of the water passing over the spillway (which, as a first approximation, we can assume as 5 feet above the crest of the dam, the pressure per square foot of the slab face be 62·5 h lbs. and neglecting the effect of the upstream face batter the bending moment per foot width of the slab at the centre of the slab assumed as non-continuous, is ; $62{\cdot}5hl/8$ foot lbs. or say equal to $K/8$.

If the slab is contionuous over several buttresses, theoretically, the bending moments in each span vary according to the total number of spans. The variation is of importance only when there are less than 7 spans. Owing to the fact that long lengths of concrete are liable to crack by expansion, it is doubtful whether continuous slabs are advisable. If, however, the slabs are built continuous, it is safe to provide for a bending moment of $K/12$ at the centre of each span, producing tensile stresses on the downstream side of the slab and a bending moment of $K/20$ at each support, producing compressive stresses on the downstream side. For the two end slabs, close to the point where the dam is joined with the hill side, the theoretical bending moments largely depend on the exact manner (freely supported or builtin in which the end buttresses and slabs are connected to the hillside. In good construction it is probable that the connection is so complete as to justify the assumption that the slabs are built in, but it is safer to provide for $K/9$ over the two end buttresses and $K/8$, at the centre of each of the two slabs. So also, theory shows that the pressures on each buttress are not exactly those given by the rules for non-continuous beams, being roughly $1{\cdot}01 \times 62{\cdot}5hl$ and $0{\cdot}99 \times 62{\cdot}5hl$, alternately. Such differences are negligible, except in the case of first buttress at each end of the dam, where $9/8 \times 62{\cdot}5hl$ (exactly $1{\cdot}134$ for 9 spans) should be provided for per foot width of the slab.

Calculations can easily be done for the design of the slab following the usual theory of R. C.

The percolation must be carefully guarded against. This is easily effected in a non-continuous slab, as all the reinforcement lies on the downstream face of the slab. In a continuous slab, however the water face is in tension over the supports and some steel must be placed near the water face. Thus, it is probable that the extra thickness of concrete necessary to protect this steel from action by water will counter-balance any decrease in thickness or percentage of steel, that might theoretically be obtained by continuity.

It also seems probable that future experience will show that a layer of waterproof material at or near the water face of the slabs is advisable, although so far as I am aware, no such construction has yet been adopted.

We can now proceed to proportion the buttresses. Theoretically the work is carried out just as for a solid dam, the buttresses having to support water pressure indicated by 62·5 lbs. at each foot of height and their own weight.

In actual practice the dam is not usually founded on hard rock and its base is therefore, about one and a half times to twice its height. In such cases tension in the buttresses does not occur, as can be seen by merely inspecting the annexed Fig. 6.

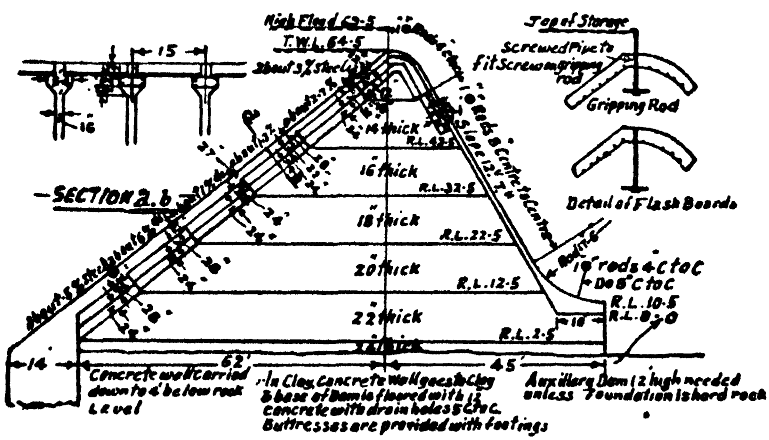


Fig. 6

It will be found by actual practice that the easiest method of design is to proportion the buttresses so as toproduce a safe intensity of pressure on the foundation and to make them of triangular section from this level upwards. It will then be found that such buttresses are of ample strength when tested on any other section; and as a matter of fact, in actual dams, passage ways and arched openings are frequently made in the buttresses, either to save material or to provide a means of communication along the dam.

The downstream face of the dam is (in a spillway type, at any rate) covered with slabs. The thickness of these cannot be determined by any ordinary rule. If we regard the depth of water flowing over the dam as the determining factor, we find a pressure of at the most 312 lbs. per square foot, assuming a depth over the dam of 5 feet and taking no account of the velocity of the flowing water, which would tend to diminish the pressure.

As a matter of practice, we find in spillway dams, that such damage as occurs is apparently due to a partial vacuum induced by the flowing water and the form of crest that theoretically, at any rate, prevents the formation of this vacuum is with an adherent jet throughout.

In a hollow dam, such as we are now considering it is plain that a few holes in the slabs would prevent any vacuum. The author understands that the best practice in America usually makes the thickness of the slabs about two-thirds t_1, say 12 inches.

Foundations.

The design of foundations depends on the character of the material on which the dam rests.

In a really soild rock, a shallow seepage trench is perhaps all that is necessary but in gravel or fissured rock it appears to the author that the only safe rule is to follow the practice evolved for earth dams. We have one great advantage - our impermeable wall being of concrete, cannot be injured by burrowing animals and we can therefore put it right in front of the dam.

A concrete core wall carried down either to an impermeable stratum or to such a depth as investigation of the material, conducted on the lines discussed under earth dams, shows to be necessary. Behind the core wall is a small stone drain, as discussed under earth dams, which should be connected with one or more vent pipes.

The whole floor of the dam is covered with a layer of concerete, the thickness of which need only be 4 inches for good foundations and in case of bad soil, may be reinforced so as to spread the pressure of the buttress, if any doubt exists as to their foundations being of sufficient width.

At the tail of the dam is another core wall, the depth of which is fixed by the scour produced below the dam by the overflowing water. The section on Falls and Weirs may be consulted when determining the site and thickness of this tail wall.

It may also be pointed out that lines of steel, or cast iron sheet piling may be substituted for the core wall or walls, but such work unless carefully executed, is liable to prove faulty; and I doubt whether it will be satisfactory even from the point of view of cost ; since each core wall should probably be replaced by a double line of piles.

Some dams exist which depend on several shallow core walls in place of two deep ones. Personally, the author doubts whether such foundations are trustworthy, but they appear to be satisfactory for heads up to 30 or 40 feet.

In such cases a very wide foundation similar to that of an Indian weir is necessary. It therefore, seems doubtfull whether expense is saved, more especially as in any soil fit for a dam foundation it is usually possible to sink a deep trench and fill in with concrete.

23. Reinforced Concerete Dome Dams.

This dam is situated in Arizona. It is shown in Fig. 7. It is multiple dome dam of a novel and unique design. It consists of three domes 180 feet centre to centre. The height of the dam is 250 feet. The extrados of the domes is a surface of revolution generated by rotation round an inclind axis and through an arc varying from 180° at the base about 80° to the crest. The intrados is a series of curves on three centred arches. The maximum compressive stress in the domes is calcu'ated to be 586 lbs. per square inch, and in the buttresses 400 lbs. The domes are heavily reinforced to prevent temperature and shrinkage cracks. The buttresses are reinforced in horizontal layers 28 feet apart. Two contraction joints have been placed along the line of principal stress in each buttress. The

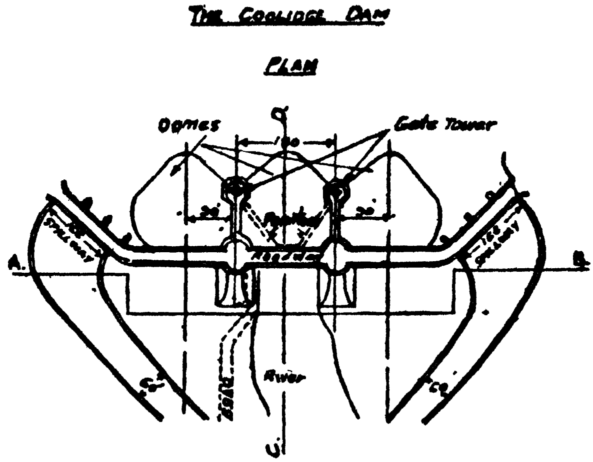


Fig 7 (a)

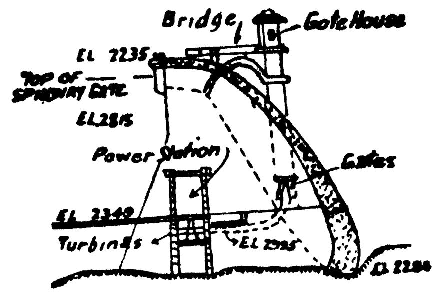


Fig. 7 (c)

SECTIONAL ELEVATION AT A.B

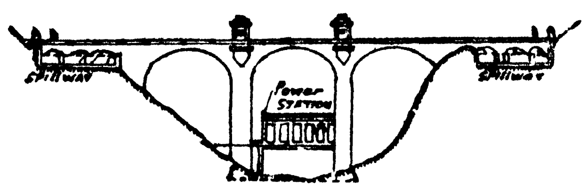


Fig. 7 (b)

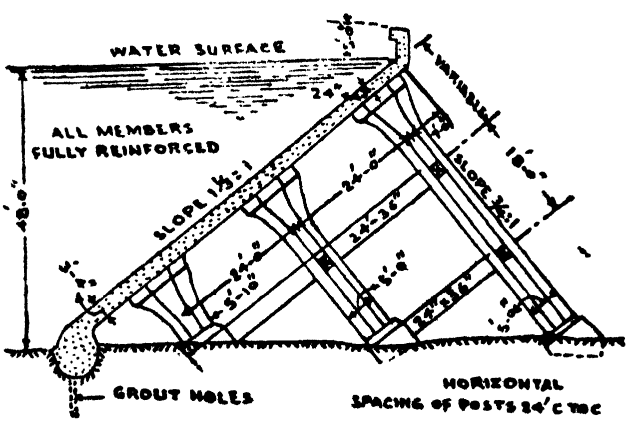


Fig 8 Slab and Column Dam.

upstream face of the dam is faced with a coating of "Gunite" 1½ inches thick.

24. Reinforced Slab and Column Dams.

The dam shown in Fig. 8 is taken from an unpublished preliminary study by the U. S. Bureau of Reclamation for a dam in Idaho (1924). In consists of a continuous flat slab designed and reinforced in accordance with concrete floor practice and supported on inclined columns. The columns are stiffened by a two directional system of struts. It is claimed that this type eliminates the uncertainties of buttress action. The authors have no information concerning actual itstallations of this type.

25. Biblography.

1. "Stresses in Buttressed and gravity Dams" Trans. American Society of Civil Engineers by F. A. Noetzli, 1933.
2. "Design and construction of Dams." by Wegmanon, Canadon Cave Creek Dam, 1927.
3. "The Buttressed Dam of uniform strength" by S, Harman, A.S.C.E. 1932.
4. "Dams and control work" by S E. Rockwill, U. S. Bur. Reclam, 2nd Edition 1938.
5. "Buttress Dams" by Edger. H. Burroughs in hand Book of applied hydraulics, Mac Graw Hill Book Company, New York, London 1942.
6. "Multiple arch Dam on Soft Foundations" A. S. C. E. 1923.
7. "Round head Buttress Dam" Don Martin Dams A. S. C. E. 1932.
8. "Treatise on Dams" United States Bureau of Reclamation.

PART IV
DRAINAGE ENGINEERING
CHAPTER I
Rainfall And Runoff

1. Hydrology.

Hydrology is the science which deals with the phenomena of water in all its states ; of the distribution and occurence of water in the atmosphere, on the earth's surface, and in the soil and rock strata ; and of the relation of these phenomena to the life and activities of man. Our present knowledge of the subject is indeed fragmantary, in complete and scattered throughout the literature of engineering and other sciences. Most of the phenomena of hydrology are exceedingly complex, and to the causal observer the irregularities and apparen inconsistencies are often so great as to make the existence of fundamental laws and cause-and-effect relationship seem hopelessly obscure and even completely improbable.

The total quantity of water with which mankind is concerned must always remain substantially the same, but its occurnce and its distribution over the surface of the earth is continually changing. As an artical of use and consumption, it is one of those few natural resources, the supply of which remains substantially undiminished because, through the action of natural laws, it is continually passing through an ever-recurring cycle of evaporation, condensation and precipitation. *ad infinitum.*

The field of hydrology, like that of more other sciences is not sharply demarcated. The subject matter is largely drawn from the sciences upon which hydrology is based. In consequence, it is difficult to determine in a book of this kind, what material shall be used and what shall be excluded. The outher has simply attempted to deal with the essential parts of this vast subject relating to Drainge Engineering in this part of this book and those relating to the groundwater engineering in Part V of this book. The attempt is intended to be elementry and to introduce the student to this vast subject which has been greatly developed in the United States of America.

2. Rainfall.

Rain is condensed moisture in the atmosphere falling on the surface of the earth after becoming too heavy for the atmosphere to support. Various causes operate in the production of rainfall. The major potion of the precipitation is, however, caused by the expansive cooling of air as it ascends.

Blanford states in Volume XXXIX of "Nature", "As a result of a long study of the rainfall of India (and perhaps no country affords greater advantages for the purpose), I have been convinced that dynamic cooling, if not the sole cause of rain, it at all events the only cause of importance, and that all the other causes so frequently appealed to in popular literature on the subject, such as intermingling of warm and cold air, contact with cold mountain slopes, *etc.*, are either in operative or relatively insignificant."

The ascensional movement of moist air which results in dynamic cooling and consequently in precipitation is brought about in one of the following three ways.

(*i*) By convective current (convective rainfall) ;
(*ii*) By hills and mountains (orographic) ;
(*iii*) By cyclonic circulation (cyclonic).

3. Definitions.

(*a*) **The rainfall, or mean annual rainfall.** Is the mean of the annual rain-fall observed over a period which is sufficiently long to produce a fairly constant mean value. In the British Isles is can be stated that this period is about thirty to forty years, and that the probable variation of the mean value (*i. e.*, the residual irregularity) is $\pm$ 2·5 percent when compared with the mean of another record of equal duration for the same locality.

(*b*) **The evaporation, or mean annual evaporation.** Is the mean value, in inches, of the depth of water annually evaporated from a free water surface the period of observation being of adequate duration to secure approximate constancy, as in the case of rainfall.

(*c*) **The runoff, or mean annual runoff.** Of a catchment area, is the value of annual volume of water discharged by a stream draining the area, expressed in inches depth of water over the catchment area, the period of observation being sufficiently long to secure a fairly constant mean. As the rainfalls, a part of it is held by leaves and foliage af trees and crops. The rest falls on the surface of earth and may be called the ground rainfall. If the ground is dry and parched, a light shower would be entirely spent up in saturating the thin crust of the earth's surface. Any water surplus to it would begin to collect and flow over the ground. From this flow there is a continuous obsorption into the soil. The equation of runoff in volumetric units may thus be written as:

Runoff=ground rainfall−Rainfall losses.

(*d*) This runoff first goes to fill big and small cavities and irregularities on the surface of the earth ; and, when all such depressions fill up and start to overflow, active runoff begins. It is this active runoff which we desire to determine.

(*e*) We also define the rainfall loss for a catchment area as the difference between the rainfall and runoff for any period, both being measured in inches over the catchment area.

(*f*) **The percolation, or mean annual percolation.** Is the depth of rain water measured in inches that annually soaks into the earth ; it being presumed that the period of observation is of sufficient length to secure an approximate constancy.

As yet we are unaware as to how long a period of observation is required in order to produce fairly constant mean values of the last four quantities : although it is highly probable that the pericds are less than those for rainfall in the case of percolation, but greater for evaporation and possibly also for rainfall loss and runoff.

4. Effect of Climate on Rainfall.

It will be shown later that annual rainfall in any locality varies from year to year within certain limits. These variations are largely determined by the general character of the local climate, and, consequently, it becomes necessary to define broadly the types of climate that influence the probable variations.

Parker classifies climate as Insular and Continental. The distinction is primarily a geograpical one. Localities close to the oceans have an Insular climate while the continental type of climate occurs either in the interior of continents, or in places separated from the oceans by high mountain ranges.

The characterisitics of the two types are well known. Continental climates have a very hot summer, followed by a relatively cold winter, while the difference between the mean winter and summer temperatures in an Insular climate is by no means so marked, and in some cases is almost imperceptible.

In the Temperate Zone, the climate of the British Isles is typically Insular, while the Middle United States, or Southern Russia, possess a climate of continental character. The dividing line may be practically illustrated by the fact that an Englishman's wardrobe does not usually include either furs or white suits ; while an American of the same class invariably possesses both.

So also in Tropical Regions, such as the Punjab, fur coats are common in the cold weather, while in the hot weather *pankhas*, or electric fans are necessities for Europeans, and are appreciated by all races The contrast with, say Ceylon, where *pankhas* or fans are less essential, but are used all the year round by those who can employ them, is very marked.

The distinction between a tropical and a temperate climate is somewhat difficult to define. Geographically, for instance, the Punjab is not in the Tropical Zone, yet the temperatures obtaining, there are generally higher than those occuring in the adjoining extra tropical regions, that is Persian Gulf and Salton desert.

From the point of view of an Engineer, Tropical climates may be defined as those in which the native workman is unable (during some seasons of the year, at any rate) to perform hard manual labour continuously during the hottest part of the day.

In all tropical climates (except a few extremely Insular examples), and in most temperate Continental Climates, there are well defined rainy seasons, usually one each year, but in some cases two. In such instances, the major portion of the rainfall, and all that has any practical influence on the runoff, occurs during well-defined periods of the year usually not exceeding four months in length, and during the remainder of the twelve months the rain that does fall is insignificant in quantity and accidental in occurence.

Generally, it may be stated that an Insular climate is (comparatively speaking) a wet one throughout the year.

5. Variability of Annual Rainfall.

(A) The fall of rain at any locality, measured in inches per annum, varies from year to years. A study of rainfall-records extending over periods of many years, such as exist in England, Furope and the United States, has led to the conclusion that the average of the yearly rainfall tends towards a constant quantity, as the number of the years over which the average is taken increases, and it appears that the average of 30 to 40 years' rainfall varies but little, whatever period of 30 to 40 years in a long rainfall record is selected.

(B) Binnie, in a paper on "Variation of Rainfall" (P. I. C E., Volume 109) analysed the data of annual rainfall for a large number of stations all over the world. He considered the mean annual rainfall as 100 and expressed the actual as percentage as shown blow.

TABLE 1.

Locality	Number of stations	Wettest year	Average of two consecutive wettest years	Average for three consecutive years	Average for three consecutive driest years	Average of two consecutive driest years	Dries year	Maximum number of consecutive years with a fall above the mean	Average fall of these years	Maximum number of consecutive years with a fall less than mean	Average fall these years
British Isles	44	145	130	123	73	71	66	5 52	117	5 57	84
N. W. Europe	5	148	133	126	75	66	61	3·80	123	5·40	83
France	23	161	142	131	74	68	59	5·22	122	5·43	81
Italy	15	159	139	129	76	70	55	4 26	121	5·60	83
N. Germany	17	139	127	121	77	70	61	5·53	114	5·59	82
S. Germany and Austria	9	144	133	127	76	68	56	6·11	120	5·55	81
Russia	12	166	146	135	63	63	53	5·42	122	7·66	78
India	9	162	142	130	72	66	52	4·77	123	5·33	78
Canada and Eastern United States.	10	141	131	125	79	75	68	5·70	119	6·80	85

(C) It is interesting to study the individual station on similar lines. As an example, the following figures hold for the rainfalls for the years 1879-1908, at Amritsar (Punjab), which possesses a typically Continental climate, with a mean rainfall of 25·26 inches.

Wettest year	=309%	of the mean
Average of two consecutive wettest years ...	=237%	,,
Average of three consecutive wettest years ...	=214%	,,
Average of three consecutive driest years ...	= 52%	,,
Average of two consecutive driest years ...	= 48%	,,
Driest year	= 34%	,,
Maximum number of consecutive years with a fall above the mean	= 5%	,,
Average fall of these year	=170%	,,
Maximum number of consecutive years with a fall less than mean	= 5%	,,
Average fall of these year	= 54%	,,

Similar case exist in India, where the average fall "is even" greater than that at Amritsar and several stations in Siberia and China show even larger variations.

(D) **Space variability of rainfall.**

In arrange of hills the rainfall generally increase as we procced towards the crest, but small areas of maximum rainfall almost invariably exist, not at the exact crest, but a little below it and to the leeward of the crest in relation to the prevailing rain-bringing winds.

In plains, the variation in rainfall from place to place is generally accidental, due to summer thunder-storms of small extent in respect to area, which not only produce short but heavy and very local falls of rain over comparatively small areas but tend to follow the track of the first storm of the hot season throughout each summer. This seasonal tendency should be confused with the general habit of thunder storms (whether covering a small area and following a sharply defined trak, or covering a large and not well-defined area) in an undulating conuntry, of following year after year, some natural feature, since this will be more or less clearly disclosed in the mean summer rainfall records.

In a hilly country, it will usualy be found that the narrow valleys have approximately the same rainfall as the adjacent hills, the difference, if appreciable, inclining towards a decrease in fall.

Apart from this exception, and the one mentioned in the first rule there is a general and undoubted tendency for the rainfall to increase with the altitude but such rules as have been proposed seem only to be applicable to limited areas. Isohyets, showing the rainfall variation in the Punjab are shown in Plate Vol. III (1).

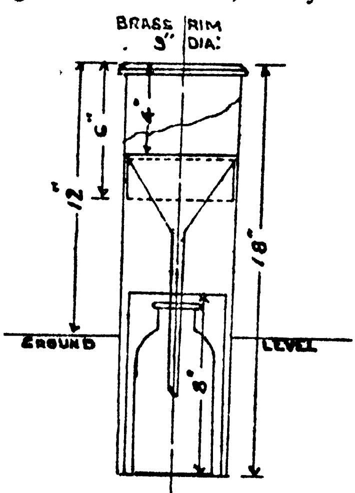
6. Measurement of Rainfall.

Two types of Rain Gauges are used in the Punjab :—

(A) Symon's Rain Gauge. (B) Integrating Rain Gauge.

(A) The five inch Symon's rain gauge is used in the Irrigation Branch, Punjab, for the registration of rainfall at the officially recognised stations. It has been found that a rain gauge placed in a perfectly open space registers less than the true amount of rain, because the wind forms an eddy over the mouth of the gauge and carries away small drops which would otherwise have fallen in the gauge. It follows that a certain amount of protection from the wind is advantageous. Gauges should, therefore, be erected in such places that the distance between the gauge and the nearest object should not be less that twice height of the object; and no tree should be planted within adout thirty yards of a gauge.

A gauge should never be situated on the side or the top of a hill if a suitable site on level ground can be found, & only under exceptional circumstances should a gauge be exposed on a roof.



STANDARD ENGLISH RAIN GAUGE
Fig. 1

The Symon's gauge consists of three parts as shown in Fig. 1.

(*a*) the base, which is built into a masonry or concrete foundation.

(*b*) the body, in which the glass bottle collecting the rain water is placed.

(*c*) the funnel, which collects the rainfall.

The foundation for the gauge will consist of a masonry or concrete cube of two feet side, sunk into the ground with its top two inches above the genral level of natural surface. Into this foundation the base (a) of the gauge, will be firmly built so that top of the complete gauge is exactly one foot above natural surface level. Great care must be taken when setting up the gauge to ensure that the mouth is perfectly level.

In order to protect the gauge from damage, it should be surrounded by an open fence of such size that the top of the fence is not higher above the mouth of the gauge than half its distance from the gauge.

Rainfall is measured by pouring the rainwater found in the receiver into the glass measuring cylinder provided. The cylinder to contain one inch of rainfall is generally used and is so graduated that each of the divisions represent one-hundredth of an inch of rainfall. If there is more water than the measure glass will hold, the glass should be carefully filled to the top graduation mark; this water poured away and then the glass re-filled. The total rainfall is the sum of all the measurements. The receiving bottle, does not as a rule, hold more than three or four inches of rain. During heavy falls this quantity is frequently exceeded, which must be measured three or four times in a day, and the some total of all the measurements during the preceding 24 hours entered as the total rainfall of the day.

The rainwater in the gauge shall be measured every day at 8 A. M. and the rain-gauge examined every day at that hour, even when, in the observer's opinion, no rain has fallen. If it is necessary during heavy rain to measure the rainfall more than once, the last measurement should be taken at 8 A. M.

(B) Integrating rain gauge.

The Symon's Rain Gauge can measure only the total rainfall that occurs in a known period but this type is an integrating one which records automatically the total rainfall at any time from beginning of a storm to its end. A graph is produced The abscissa shows the time in hours and the ordinate at any instant shows the total rainfall in inches from the beginning of the storm. A typical graph is given in Fig. 2.

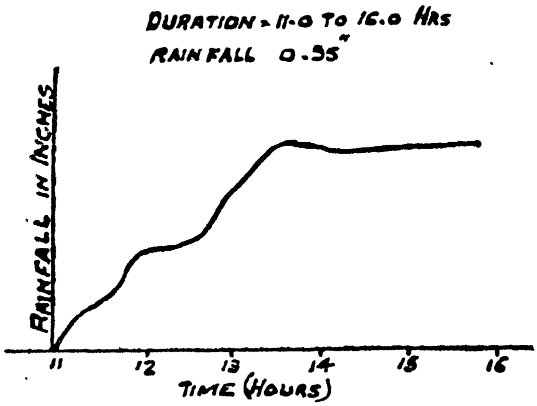
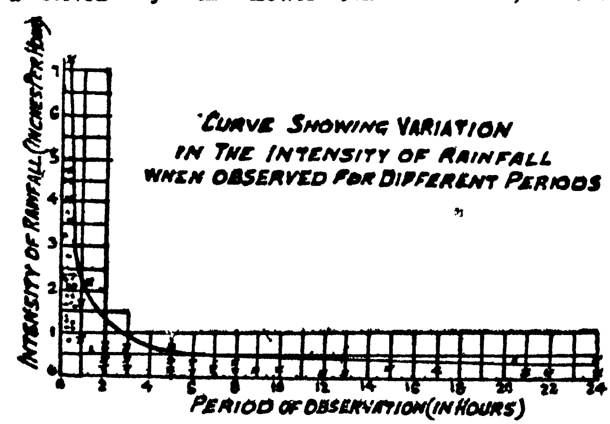


Fig. 2

From the above, it will be seen that total daily rainfall as recorded at most of the Indian stations does not give any idea of how the rain has fallen whether the total rain fell in one hour or in five. Again, it may happen that the rainfall starts half an hour before the scheduled time for reading the gauge and continues until half an hour after this scheduled time. The rain which has actually fallen during a period of one hour will be shown in the records as having fallen during a period of 48 hours, while the Integrating Rain Gauge records the intensity of rain automatically

7. Rainfall Curve for Maximum Intensity.

For the observed intensities of rainfall in the area served by the Lower Jehlum Canal, S. D. Khungar and N.D. Gulhati have produced a summation curve, or a graph of rainfall of maximum intensity as has been shown in Fig. 3. The ordinates of this graph for any time represent the total rainfall upto that time. Such a graph would be recorded by an Integrating Rain Gauge, if rain fell with



Fig· 3

the maximum possible intensity for a period of 24 hours. A little consideration will show that all rainfall curves for any total rainfall will generally fall below the graph of the most intense rainfall shown in the figure. In other words the graph in Fig. 3. is an enveloping curve for rainfall graphs of all storms that are likely to occur in this area.

The graph of the heaviest storm obtained by the Integrating Rain Gauge at Lahore, has been superimposed in Fig. 4.

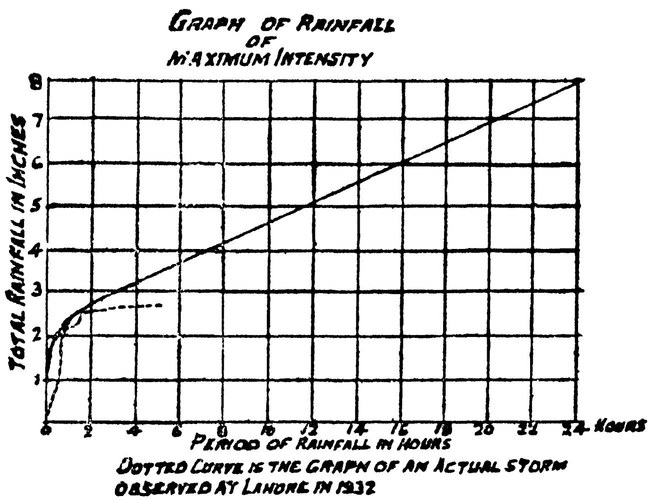


Fig. 4

Variation in Intensity and Distribution of Rainfall.

It will be seen that it very well conforms with the graph obtained analytically for the most intense rainfall. Any one acquainted with the monsoon rainfall in India can very readily appreciate the variation in the intensity of rainfall from place to place. While one portion of a village may receive a good shower, the other end might have remained quite dry.

Even when Rain Gauge Stations have been spaced a short distance apart they have recorded vary different rainfalls for the storm. At Cherapunji (India) there are two Rain Gauges, one at the Police Station and the other at the Welsh Mission Hospital. The following statement show some of the rainfall recorded at the two gauges.

TABLE 2.

Date.	Welsh Mission Hospital Gauge	Police Station Gauge.
June 30, 1913	22·10	17·61
June 13, 1923	30·67	10·85
June 14, 1923	11·76	28·15
July 17, 1926	13·50	10·30
October 8, 1926	19·25	14·60

Consider now a large area (Fig. 5) with Rain Gauges installed at G_1, G_2, G_3.................G_n.

If hyetographs for a particular storm were drawn as shown in Fig. 5 the following facts seem to emerge;

(1) That the rainfall recorded at any Rain Gauge is a true index of the intensity of the storm for only a limited area round it.

(2) That there may be areas between neighbouring rain gauge stations that may either receive no rainfall at all, or very much in excess of, or less than that recorded by any of those stations.

Apart from the above, the time at which rain falls at different Rain Gauge stations may not be the same.

8. Spacing of Rain Gauge Stations.

In view of what has been said above, it is obvious that the closer the rain gauge stations and the more uniform their spacing in a catchment area the more accurate will be the estimate of total rainfall over it.

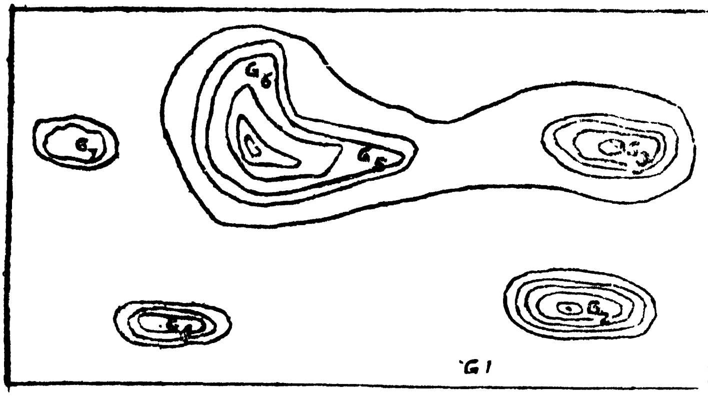


Fig. 5

"In the British Isles, waterworks engineers prefer to have one rain gauge to about every 1,000 acres of gathering ground; but it must be remembered that the rainfall of the British Isles (and more especially that of England), varies from place to place for more rapidly, (and falls in patches) than is the case in countries possessing topographical features on the larger scale."

"In plains, variation in rainfall from place to place is generally accidental due to summar thunderstorms of small extent in respect to area", which produce short but heavy and very local falls of rain over comparatively small areas.

In a Note dated 1906 Benton, Inspector General of Irrigation, wrote; "It appears to me that the least number of rainfall stations inside the boundaries of a catchment area which will afford a reasonably safe estimate of the rainfall may be assumed to be as follows :—

Area in Square Miles.	Number of Stations
0 to 50	1
50 to 100	2
100 to 200	3
200 to 350	4
350 to 500	5
500 to 700	6

The gross area on the Lower Chenab Canal is 5,721 square miles and there are 47 rain gauge stations in it - an average of 122 square miles per rain gauge station. The existing number is very inadequate, particularly when their are a large number of drains with catchment area of less than 50 square miles.

Rain gauge stations in the irrigated areas of the Punjab should be spaced, generally, not more than seven miles apart in either direction and that in fixing their site due consideration should be paid to their positions with regard to the catchments of the existing or proposed drains. The number of rain gauge stations in the area of the Lower Chenab Canal on this basis would be 115 and at least 10 percent of these should be of the integrating type.

9. Factor Affecting Runoff.

Factors affecting Runoff are Rainfall characteristics and Watershed characteristics.

(A) **Rainfall characteristics.**

(i) **Vegetation.** Rain Gauges are always installed in the open. However, when actually dealing with runoffs, it it not the rain that falls on the ground in the open that needs to be considered but a part of it which is initially held by trees, plants and other vegetable cover on the ground which has to be taken into account. The rain so held may fall to the ground by wind action during or after the currency of the storm or it may be directly re-evaporated from the surface of leaves, etc. The rain that actually falls on the ground and can produce runoff is thus always less than that recorded by a rian gauge.

No accurate estimates exist of the quantity which should be deducted on the above account. From observations made by D. W. Mead of rainfall under shelter of trees in America it was concluded that rainfall reaching the ground was on an average about 70 percent of that caught in the open. It has also been estimated that the wedge like capillary spaces between grass blades may hold at least ¼ inch depth of water. These figures cannot, of course be applied in general way The amount of water held by foliage, crops and undergrowth will vary with.

(*i*) The intensity of rain fall;
(*ii*) Wind action during and after the storm;
(*iii*) Thickness and nature of foliage, kind of crop and nature of undergrowth; and
(*iv*) Dry or wet state of plants prior to commencement of the rain.

It will be appreciated that the deduction from the total rainfall the effect of vegetable cover will vary even for the same area from time to time on account of the wind. If the wind is blowing, there will be hardly any rain held by the foliage of trees and thus the figure of 30 percent obtained in certain experiments mentioned above may be too high for such and other cases. The folloing figures are proposed to be adopted tentatively by S. D. Khungar and N. D. Gulhati for the canal irrigated areas in the Punjab :—

TABLE 3.

Serial No.	Kind of Catchment.	Deduction from total recorded rainfall for amount initially held by vegetable cover.
1	For area under trees	1/4 inch
2	For area under crops	3/8 inch
3	For area with thick undergrowth	3/8 inch
4	For area with light undergrowth	1/4 inch
5	*Banjar* land with no undergrowth	Nil

(*ii*) **Surface evaporation loss.**

Buckley's experiments showed that surface evaporation losses in India were of the order of 1/100 inch per hour. The total monthly surface evaporation (average of 16 stations in U. S. A.) observed by D.W Mead during 1887—88 was 5·5 inches during June and 6·7 inch in July. This works out to about 1/120 inch per hour. This subject is dealt with in detail, in Chapter II, Part II.

(*iii*) **Losses due to transpiration by plants.** Evaporation through plants is spoken of as transpiration. Out of the total quantity of water used in transpiration about 10 percent is utilized to build the tissues of the plants and the rest is evaporated from the surface of the leaves. W. Harrington made an estimate of transpiration losses. At the locality where the investigation was made, the transpiration was found to be 6·5 inches During the same period the evaporation from free water surface was 8·39 inches The transpiration, was therefore 77 percent of the open water evaporation in this case or say, 1/130 inches per hour

(*iv*) **Infiltration into soil.** The loss due to infiltration into the soil is the principal loss. Most of it is brought up as soil evaporation and only a small part of it serves as an addition to the subsoil water table (Chapter III, Part V). The following table gives results of observations by S. D. Khungar and N. D. Gulhati as published in Paper No. 245, Punjab Engineering Congress. It is based on actual observations. on the second day of putting water in a field. In the rainy season, the soil is supposed to be partially wetted by previous showers. In the actual flooding tests, the first day's loss was about twice that on the second day.

TABLE 4.

Kinds of soil.	Absorption loss in inches per hour.		
	Fallow land.	Land with crop.	*Banjar*
Good soil	0·49	0·50	0·57
Kalrathi soil	0·32	0·18	0·22
Sandy soil	1·69	1·33	1·88
High spring level	0·37	0·08	0·39

In general the losses due to infiltration into the soil depend on :—

(*i*) Temperature changes.

(*ii*) Packing of the soil surface and in-washing of fine material to pores and openings in the soil surface by rain.

(*iii*) Soil moisture content.

(*iv*) Cultivation.

(*v*) Earthworm and insect perforation of the surface soil and sub soil and perforations left as a result of the decay of plant roots.

(*vi*) Shrinkage and swelling of surface soils which contain colloidal material, particularly sub-checking of the soil surface during dry periods.

(B) **Watershed characteristics or Topography of the catchment area.**

Topography of the catchment area affects run-off as below :—

(*a*) **Shape of the catchment and the distance from the watershed to the drain.** When the width of catchment measured from the drain to the water-shed is narrow, the time taken for removal of rain-water is comparatively short resulting in quicker drainage and higher intensity of runoff.

(*b*) **The slope of the country from the watershed to the drain.**

A steep slope has the same effect as a narrow width, *viz.*, water takes a comparatively short time to be drained off.

(*c*) **Natural or artificial pondage in the area.** The effect of natural storage provided by a catchment is considerable. Apart from big ponds or natural reservoirs the effect of each of which must be considered separately, all surfaces, even the paved ones in urban areas, contain depressions which must fill up and overflow before the active runoff can begin. These depressions are of all kinds of and their size may vary from cavities to large flats covering many acres. In addition to the above, on small sodded areas water is held in wedge-like capillary spaces between grass blades.

(*d*) **Effect of cutivaltion.** In a cultivated area the field dowels provide immense storage capacity depending on the height and strength of the dowels. It is not perhaps widely known that the runoff from cultivated areas is little execpt in an extraordinarily heavy downpour. This view is confirmed by observations made during experiments on the determination of runoff performed in the Irrigation Research Institute. In an area of 70 acres on the Upper Chenab Canal near Khambranwala no runoff was received from cultivated areas although the rainfall on one day was as high as 3·6 inches and intensity of rainfall as much as 0·72 inch per hour.

10. Runoff Estimate in a Small Catchment.

When the width of the catchment area for a drain is less than ¼ mile on each side of it, it is considered to be a small catchment.

The analysis given below is due to S. D. Khungar and N. D. Gulhati in their paper No. 245, Punjab Engineering Congress, Lahore.

(A) **Simplified case with certain assumptions.**

(*a*) **Assumptions made.** In the solution of the simplified case the following assumptions are made.

(*i*) Area considered is small, such that it can be wholly covered by a storm and the intensity of the storm over it does not vary appreciably ;

(*ii*) Losses are considered uniform over the area and constant in the duration of runoff ;

(*iii*) Velocity of storm water flowing over the catchment is considered uniform over the whole width throughout the period of runoff. In other words, there is on relative movement in the different parts of the sheet of water over the catchment ;

(*iv*) There is no vegetable to cover the catchment ;

(*v*) There is no natural or artificial pondage over the area ; and

(*vi*) The quantity of water flowing over the catchment during the period of the storm is ignored.

It is also taken for granted that a rainfall graph of the storm is available and that the inlet time of the catchment is known. Rainfall graphs can be obtained by means of an Integrating Rain Gauge installed in the area. The detemination of inlet time is described latter.

(*b*) **Runoff estimate.** Let AB (Fig. 6) be the watershed, and DC the drain. Let ABCD be a sub-area of the catchment which has a small width (AD or BC). Let T hours be the time taken by water to travel from watershed AB to the drain CD (inlet time). Now if this sub-area of A square miles be divided. as shown, into 'n' number of equal narrow strips of area 'a' square miles, and storm water flows with a uniform velocity v, water will take $T/n = t$ hours in travelling from one strip to another.

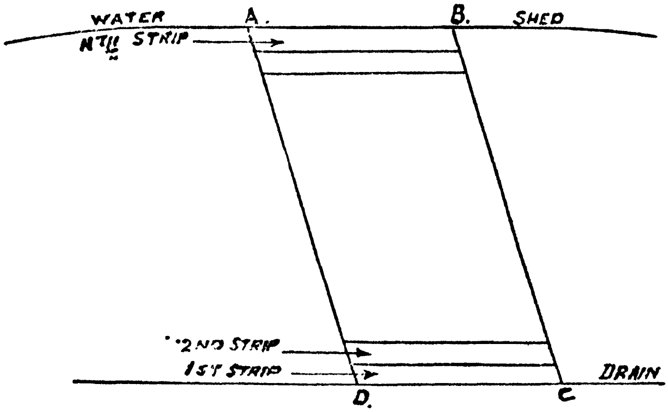


Fig. 6

Let the graph of the storm be as shown in Fig. 7. The abscissae of the graph represent the time from the beginning of the storm, and the ordinates show the total precipitation of rainfall up to that time.

Draw ordinates R_1, R_2, R_3........................R_s giving the total rainfall up to end of the time intervals t, 2t, 3t...............st.

If r_1, r_2............be the rainfall in inches during successive time intervals t, then : $R_1 = r_1$ $R_2 = r_1 + r_2$; $R_3 = r_1 + r_2 + r_3$; and so on.

If the losses are considered uniform over the area and constant during the period of runoff, and are equal to D inches per hour, then the loss during the time intervel t is equal to tD inches of depth.

Assume that ; $r_1 > tD$; $r_1 + r_2 > 2tD$; $r_1 + r_2 + r_3 > 3tD$; and so on.

This implies that every strip will contribute its runoff from the very beginning of the storm.

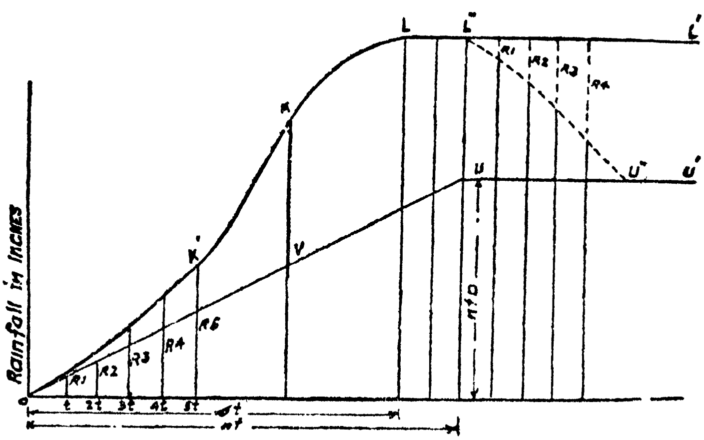


Fig. 7

Note that (in Fig. 7 above) at the intersection of lines O U and K' 5t, V' is omitted by mistake

Now let the duration of the storm be equal to 's' time intervals or, 'st' hours, when 'st' may be greater than, equal to or less than the inlet time nt.

(*a*) **Duration of storm less than inlet time. (s<n).**

The discharge q_1 reaching the drain during the first time interval t is only from strip

No. 1, water from the second strip does not yet arrive, and ; $q_1 = \frac{a(r_1 - tD)}{t} \times 645$ cusecs (one inch of rain in one hour over one square mile is equivalent to 645 cusecs).

In this expression the quantity of water held on the surface to give the depth required tor flow has been ignored.

In the second time interval both the first and the second strips will be contributing. The runoff from the fiast and second strips will be due to rainfalls of r_2 and r_1 respectively. Discharge from the second strip reaching the first strip is equal to $645 \times a/t\ (r_2 - tD)$.

To this will be added the rainfall at that time on the first strip, and from the total must be deducted the absorptian in the first strip. Thus discharge reaching the drain in the second time interval.

$$q_2 = \left(\frac{a(r_1 - tD)}{t} + \frac{a(r_2 - tD)}{t} \right) 645 = \frac{a}{t}(r_1 + r_2 - 2tD)\ 645 = \frac{a}{t}(R_2 - 2tD)\ 645 \text{ cusecs.}$$

Similarly during the third time interval discharge reaching the drain will be ;

$q_3 = \frac{a}{t}(R_3 - 3tD)$ 645 cusecs and during the 's'th time interval ; $q_s = \frac{a}{t}(R_s - stD)645$

$$s = \frac{na}{nt}(R_s - stD)\ 645 = 645\frac{A}{T}(R_s - stD) \text{ cusecs} \qquad (1)$$

Just after the 's'th interval time, that is, when rainfall has just ceased, the last strip adjacent to the drain will cease to contribute. In its place the runoff from the (s+1)th strip will commence to reach the drain. The number of strips contributing will thus still remain the same, *viz.*, s. The absorption would however, occur in s+1 strip, and

$$q_{s+1} = 645\frac{A}{T}\ R_s - (s+1)tD \qquad (2)$$

It follows that for finding the maximum discharge, it is not necessary to consider any time interval after the sth.

Draw on it a line OU through the origin such that this line cuts the first ordinate R_1 at a height tD above the x-axis, cuts the second ordinate R_2 at a height 2tD above the x-axis, and so on.

It will be noticed that the lengths of the ordinates betweeh the graph of the storm and the line OU are equal to $(R_1 - tD)$ $(R_2 - 2tD)$............$(R_s - stD)$............and thus represent graphically the values of q_1, q_2............q_s, etc.

$$\text{Thus } q_s = 645\ (A/T)\ K'V' \text{ cusecs} \qquad (3)$$

The longest of these intercepts KV will give the maximum discharge that reaches the drain at any time due to the storm, and this can be easily scaled off the graph.

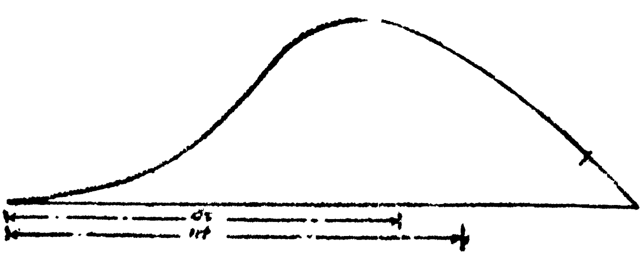


Fig. 8

To draw the hydrograph of runoff, a little modification and extension of the above graph is necessary. It has been shown above that the intercepts between the line OU and the graph of the storm represent to a definite scale, the rates of discharge at each moment. Flow into the drain will however, not cease with the storm, but will continue for some time atter the storm is over. Fig. 7 explains how the rate of flow after the storm has eased, could be determined.

The intercepts between the lines OKL″U′ and OUU′ when plotted on a horizontal base as shown in Fig. 8 give a hydrograph of runoff.

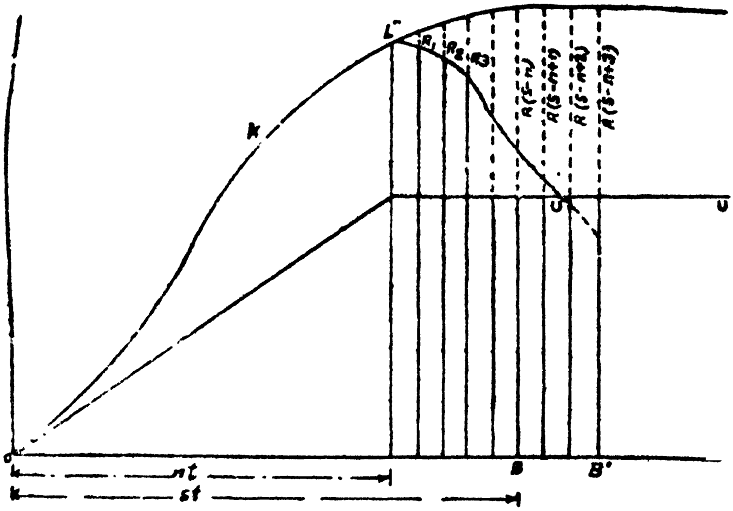


Fig. 9

It is clear from equation (1) that to get the discharge in cusecs the ordinates of the hydrograph require to be multiplied by 645 A/T.

(*b*) **Duration of storm equal to inlet time, (s = n).**

If s is equal to n, the hydrograph can be drawn in the same way as when s is less than n.

(*c*) **Duration of storm greater than inlet time (s > n).**

If s is greater than n, the diagram will take the form shown in Fig. 9.

The differences are :

(1) That the absorption after time T becomes constant.

(2) In the (n+1)th interval, the discharge reaching the drain will be :—

$$q_{n+1}=645\ (A/T)\ (r_2+r_3+\dots\dots\dots r_{n+1})-t_nD=645(A/T)\ (R_{n+1}-R_1)—ntD \text{ cusecs.}$$

After time 'st', the diagram has been extended in the same way as Fig. 8 and the intercepts should be taken from the modified curve OKL″U′.

(*d*) **Correction to be made when rainfall over part of the area is totally absorbed.**

If the condition assumed does not hold good, that is, if r_1 is not$>$tD; r_1+r_2 is not$>$2tD ; But $r_1+r_2+r_3$ is$>$3tD the diagram will take the form shown in Fig. 10.

In the first and second time intervals no discharge will reach the drain. During the third time interval the discharge reaching the drain will be due to rainfall r_3 in the first strip. The second and third strips will not be contributing and $q_3=(r_3—tD)\ (a/t)\ 645$ cusecs.

$$\text{Now } r_3-tD=(r_1+r_2+r_3—3tD)—(r_1+r_2—2tD)=(R_3—3tD)—(R_2—2tD)$$

$$\therefore\ q_3=645\frac{a}{t}(R_3—3tD)—(R_2—2tD) \text{ cusecs.}$$

$$\text{Similarly } q_4=645\ \frac{a}{t}(R_4—4tD)—(R_2—2tD) \text{ cusecs.}$$

In Fig. 10, R_2-2tD is equal to the length b c and $R_3—3tD=\sqrt{K'V'}$. If, therefore, a line c′U″U‴ is drawn parallel to the line OUU′ and the graph will represent the rate of discharge to the same scale as before.

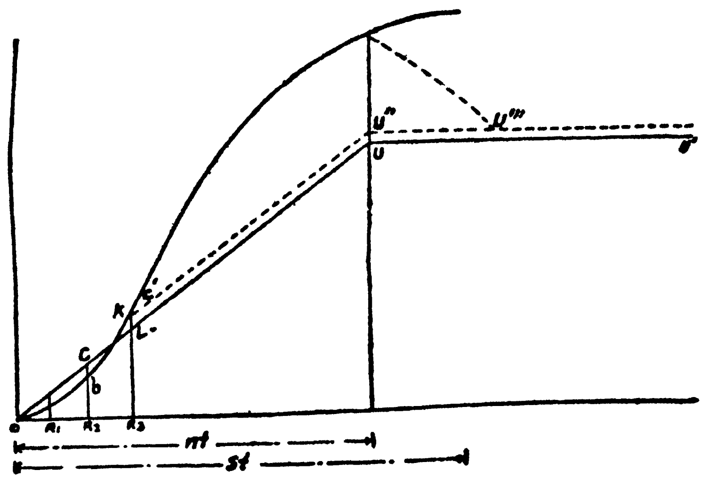


Fig. 10.

The same method would be applicable if water started to flow into the drain at any interval after or before the third.

(e) **Combination of hydrograph of various sub-areas.**

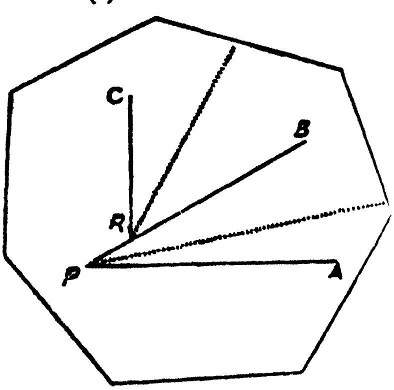


Fig. 11

Consider now an area shown in Fig. 11 and assume that the length of drain is so small that there is no appreciable decrease in intensity of flood discharge as it travels down.

Divide the entire area into sub-areas. Assuming that rainfall graphs for each of the sub-areas are available, it is possible to draw hydrographs for each of the sub areas by the method explained on the preceding pages. The resultant hydrograph at the point P will be a combination of the three hydrographs and can be prepared as shown in Fig. 12. The resultant hydrograph is shown as AA_1GDE.

AA_1A_2 is the hydrograph for sub-area A, B B_1B_2 that of sub-area B and BC_1C_2 of sub-area C. The distance AB represents the time taken by the flood to travel from point R to P.

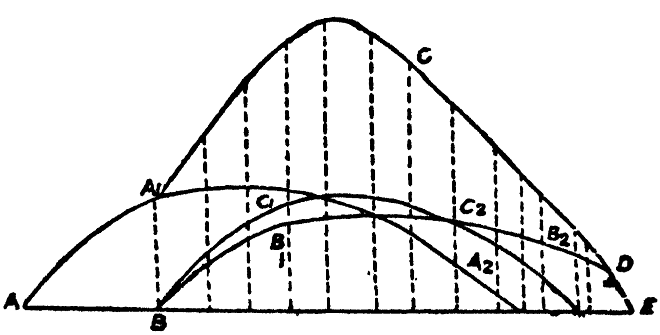


Fig. 12.

(f) **Determination of inlet time.**

The inlet time for any drain depends on :—

(a) The distance of the watershed from the drain;
(b) The slope of the catchment towards the drain;
(c) The intensity of rainfall and permeability of the soil;
(d) The vegetable cover on the soil and the nature of cultivation ; and
(e) The natural and artificial pondage on the area

It will be relized that it is almost impossible to evaluate all these varied factors accurately.

If the entire catchment area is covered by a storm and if v is the average velocity of runoff, then; $T \times 60 \times 60 = L/v$ (4)

Assume now that water is flowing in a thin sheet all along from the watershed to the drain. This flow may be laminar, turbulent or mixed. R.E. Horton has given a very interesting discussion of the mathematics of overland flow. He refers, however, to some experimental determination of the law of sheet flow over soil surfaces made by Lewis and E H. Neal at the Agricultural Experiment Station at Moscow These experiments were performed in a trough 20 ft. long, 1·9 ft. wide, and 1·5 ft. deep, so arranged that the slope S could be varied. Depths were measured by means of hook gauges and discharges by means of orifice. The experiments comprise of 62 satisfactory observations with slopes varying from 1/2000 to 1/300 and depths from 0·07 inch to 0·54 inch. The results obtained can be expressed by the formula :—

$$v = 1100\delta^{0.9} S^{0.7} \qquad (5)$$

where δ is the average depth of sheet flow in feet and v the velocity in feet per second.

A little considration will show that the average value of KV of equation No. 3 is equal to δ, which can thus be found from the hydrograph.

We have two equations :—

$T = L/3600v$; and $v = 1100\, \delta^{0.9} \times S^{0.7}$

From these two equations we can determine the value of T.

The above is a fairly simple method of determining the value of T, but it assumes sheet flow in the entire width of catchment from watershed to the drain. Those who have ever watched surface flow can appreciate that rain water over a catchment never flows to the main drain like a sheet in its entire length. What happens is that a number of small streams are formed and the sheet flow is restricted to the area between the various small streams. It would, therefore, be necessary on this account, to reduce the inlet time as calculated above by an amount depending on the configuration of the country.

(*f*) **Intensity of Normal flood.**

To obtain the intensity of runoff which would have a frequency of once in three years the maximum discharge obtained for the most intense storm should be reduced suitably. In rivers, a discharge which has a frequency of once in three years is about ¼th to ⅓rd of the maximum flood discharge ever recorded.

It will be seen that the ratio of the intensity of a flood that occurs on an average of once in three years to that of the highest recorded flood is as follows.

River Jhelum at Mangla	22 percent
River Chenab at Marala	28 percent
River Chenab at Khanki	37 percent

Khanki is comparatively away from the main catchment—the hills—and the results, obtained from Marala and Mangla should be adopted for drains. On this analogy, obtain the intensity of what may be called the normal flood, the maximum discharge determined for the severest storm should be reduced to, say, ¼th.

For design of drains in an open country, it would not cause any appreciable damage if the very maximum discharge obtained for a short period of an hour or so was ignored. It would be sufficient to design a drain for the average discharge for the period of flood. Thus if the average height of the hydrograph during time T is denoted by Z, the discharge for which a drain may by designed is obtained from the equation $Q = \frac{1}{4} \times 645A/T \times Z = 161A/T \times Z$; It would be clear from assumption (i) made paragraph A (*a*) that the area A in the above equation is that part of the catchment which contributes to flow in the drain and is so small that every point in it simultaneously receives rainfall of the intensity shown by the graph. For the sake of distinction, let such an area be called A_0, then;

$$Q_0 = 161A_0/T \times Z \qquad (6)$$

(B) Corrections for the assumption made in the solution of the simplified case.

(*i*) **Variation in absorption losses.** As already explained, losses are not uniform during the enire period of runoff. Further, they vary from place to place in a catchment area. The correction of the hydrograph for the variations in the absorption losses during the period of runoff is a simple affair. In the beginning of a storm, the soil is dry and parched and losses due to evaporation and absorption into the soil are comparatively high. As the soil gets satutated, absorption decreases. This decrease in losses due to absorption can be accounted for by a slight modification of the absorption line as shown by the dotted line in Fig. 13.

To account for the difference in the infiltration capacity of different parts of the catchment is not so easy. A good approximation can, however, be obtained by taking the weighted average of the absorption capacity of different parts of the catchment. As an illustration, let 20 percent of an area be good soil and 80 percent *kalrathi*. Also let the area of each class under crop be 30 percent. These figures can accurately be obtained from a survey plan of the catchment.

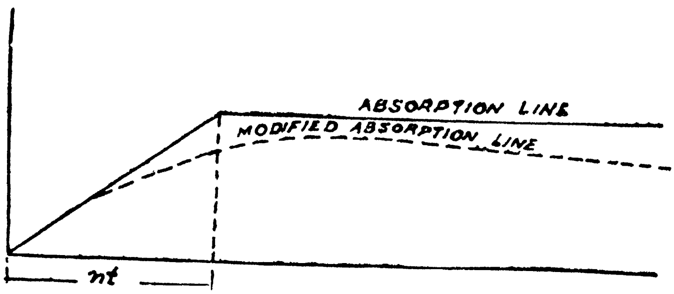


Fig. 13

From the results given in the table 3 of this chapter the absorption loss is :—

For good soil under crop 6% × 0·5	0·03
For good soil under *banjar* 10% × 0·57	0·057
For *kalrathi* soil under crop 24% × 0·18	0·043
For *kalrathi* soil under *banjar* 56% × 0·22	0·123
Total	0·253

Say ·25 inch per hour.

(*ii*) **Variation in velocity.** The velocity of runoff across a catchment varies for two reasons—space (distance from watershed to drain), and time (duration of runoff). Throughout the period of flow a continuous increase in runoff from the watershed to the drain results in a

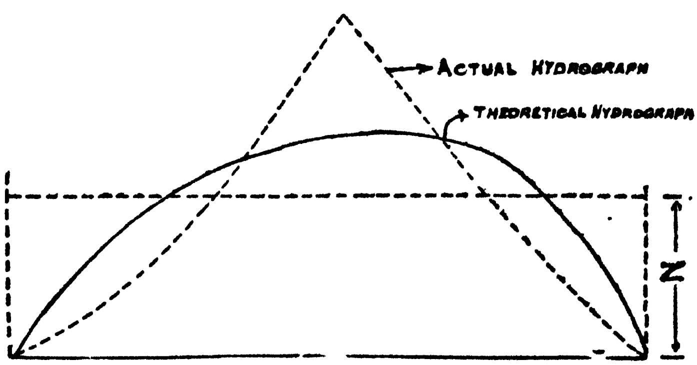


Fig. 14

corresponding increase in velocity towards the drain. Also there is a variation in velocity from nothing to a maximum due to an increase in runoff from zero to peak and back to nothing when the runoff ceases. On the chart below, an indication is given of the way in which the velocities would vary both across the catchment and during the time of flow, For simplicity, assumed velocities of flow have been shown.

			0	0	0	0	0						Watershed
Width of catchment		0	·4	·5	·6	·5	·4						
			·8	1 0	1 0	1·0	·8	0					
			·2	·9	1·2	1·4	1 2	·9	·4				
			·5	1·2	1·5	1·8	1·5	1·2	·6	0			
		0	·8	1·5	1·7	2 2	1·7	1 5	·9	·5	0		
		·2	1 0	2·0	2·5	3·0	·5	2·0	1·5	1 0	2		
		·5	1·2	2·4	3·0	3 7	3 0	2·5	1·8	1·2	4		
		·8	1·5	3·0	3 6	4 5	3·6	3·0	2·2	1·5	·8		
	0	1	2	4	5	6	5	4	3	2	1	0	
	Period of flow												Drain

In the simplified case it has been assumed that water takes a uniform time t in flowing from one strip to another throughout the entire width of the catchment and during the entire period of flow. The average velocity v for the data shown on the chart is 1·7 feet per second It is clear from a perusal of the above chart that the actual hydrograph of runoff would be different from the theoretical and would roughly take the shape shown by the dotted line in Fig. 14. But in the beginning and at the end of the period of flow, velocities being below average the actual runoff will be less than the theoretical. On the orther hand, the actual peak will be higher than the theoretical The value of Z in equation No. 6 has been taken as the average value of the hydrograph during the period of flow, and this value will remain unaltered as the error is small and counterbalanced by factors mentioned below.

(*iii*) **Correction due to rain held by vegetable cover.**

The amount of rain held by trees, crops and undergrowth can be treated as an intial reduction in the total rainfall. If a curve is drawn parallel to and below the maximum rainfall curve at a distance equivalent to the amount of rain held by vegetable cover, the ordinates for the hydrograph should be measured between the absorption line and the new curve.

Generally it would be found that the conditions of vegatable cover differ very widely over a catchment. A small portion may be forest land, a major portion may be under crops of different kinds, and the rest may be barren land. A good approximation of the amount of intial deduction can be made by obtaining a weighted mean ualue for the entire area. Let this mean weighted value be denoted by F inches of depth. Equation No. 6 will thus be modified to :—

$$Q_o = 161\ A_o\,(Z - F/T) \qquad (7)$$

(*iv*) **Correction for natural pondage over the catchment.**

This correction can be made in exactly the same way as for the effect of rain held by vegetable cover. Natural pondage represents an initial deduction on the total rain because active runoff will begin only after sufficient rain has fallen of fill the natural depressions.

For maximum conditions of runoff which will generally occur when a heavy storm is preceded by one or more light storms, the initial pondage may be nil and it is not proposed to make any allowance for it in the design of drains.

(*v*) **Correction for storage provided by the quantity of water flowing over the catchment.**

A certain amount of water must be initially held on the catchment to provide sufficient depth for the flow to take place. This quantity of water will reach the drain after the rain has ceased. The effect of this water initially held on the surface of the catchment will be slightly to lower the hydrograph in the begining of the storm and to increase the runoff towards are latter part or slighty to extend the period of flow. This has, however, hardly any effect on the maximum runoff.

(*vi*) **Correction for the effect of cultivation.**

On account of the presence of field dowels little runoff takes place from cultivated area. For this reason such areas which do not contribute to flow should be excluded from the effective catchment area. If *banjar* areas are regarded as wholly contributing to runoff, cultivated areas may, for simplicity, be reduced to equivalent *banjar* areas and the total *banjar* area so arrived at may be considered as the effective catchment area. The following recomendation is made as a result of general observations so far made in this connection.

(*a*) *Banjar* areas to be considered as wholly effective.
(*b*) 10 percent of the total canal irrigated area to be considered as equivalent to *banjar* area ; and
(*c*) 80 percent of *barani* and *chahi* area may be regarded as *banjar* area.

(*vii*) **Non-contributing areas.**

When the distance of watershed from a drain is long, the water from more distant parts of the area would, unless branch drains have been provided or some natural *nallahs* exist, never reach the drain. This will be clear if it is considered that as inlet time increases, absorption loss also increases, and a point may be reached when the absorption loss is in excess of the total rainfall. Experience shows that effective area of a drain in flat irrigated tracts does not exceed a strip of 1·15 miles width on either side of the drain. All areas lying outside this strip should be excluded from the effective catchment area.

(*viii*) **Value of Z for use design.**

The determination of Z in a particular case depends on the value of T. The value of T in turn depends, among other factors, on value of $Z(Z-F)=\delta$ of equation (5). To solve a particular problem it is necessary first to assume a value of T, draw a hydrograph and determine Z, and than recalculate T, and see if the calculated value agrees with the assumed one. Otherwise another assumption for T should be made the process repeated, till the assumed and the calculated values of T agree.

To simplify this work, hydrographs were drawn for different values of inlet time and absorption, and the values of the maximum intercept and Z determined. The values are given in Table 5.

The solution of equation (7) is now easy. Assume a value of T, obtain Z from table given below and then calculate the value of T by equations (4) and (5). Repeat the process till the assumed and the calculated values of T agree.

TABLE 5

T in hours	Z in inches Absorption loss in inches of depth per hour				Maximum intercept in inchs. Absorption loss in inches of depth per hour			
	0·2	0·3	0·4	0·5	0·2	0·3	0·4	0·5
1	1·52	1·45	1·40	1·25	2 09	2 00	1·90	1·82
2	1·86	1·71	1·63	1·55	2·09	2·05	1·92	1·82
3	1·97	1·81	1·65	1·53	2·32	2 08	1·90	1·83
4	1·96	1·82	1·58	1·33	2 33	2 09	1·92	1·80
5	2 13	1·86	1·66	1·43	2·35	2·06	1·90	1·82
6	2·09	1·81	1·55	1·21	2·41	2·07	1·92	1·82
8	2 18	1·80	1·41	1·06	2·50	2·05	1·94	1·83
10	2 34	1·85	1·33	1·11	2·60	2 08	1.89	1·81

10. Runoff From Large Catchment Area.

A catchment area over 50 spuare miles is considered as a large catchment for a drain.

(A) The discharge for which a drain may be designed is given by the equation :

$$Q_o=161\ A\ (Z-F/T) \qquad (7)$$

In this equation A_0 has been defined as that area :—

(*a*) which is wholly effective, and
(*b*) which is so small that it is wholly covered by a storm and in which the intensity of rain does not vary appreciably.

If A represnts a large catchment area which is wholly, effective then from the variations of runoff with respect to small catchments which are wholly effective, shall take the form:

$$\frac{Q}{Q_o}=\left(\frac{A}{A_o}\right)^m \text{ when m is less than unity } \therefore Q=Q_o\left(\frac{A}{A_o}\right)^m \text{or } Q=161\ A_o\ \frac{Z-F}{T}\left(\frac{A}{A_o}\right)^m \quad (8)$$

To be able to use this result in the determination of discharges for drains, it is necessary first to evaluate A_o and m. This could only be done if a large number of discharges are observed from different catchment areas, of which the inlet time T, mean absorption capacity D, and cropping etc., are fully determined. It is also necessary to obtain graphs of rainfull at a number of places in the catchment to enable a correct idea being formed of the actual rain over the area that produced the runoffs. The collection of this data is very important, and unless this is accomplished it will not be possible to design drains and drainage works accurately.

To obtain some idea of the values that A_o and m will take, an attempt was made to analyse, as best as possible, some of the available data in the Lower Chenab Canal area. The results obtained are not conclusive, but there are indications that A_o and m will have values in the neighbourhood of $m=0{\cdot}3$; $A_o=0{\cdot}25$ sq. miles.

For these values equation (8) can be written a follows :

$$Q=161\times\frac{Z-F}{T}(0{\cdot}25)\left\{\frac{A}{0{\cdot}25}\right\}^{0{\cdot}3};\text{ or } Q=161\ (0{\cdot}25)^{0{\cdot}7}\times\frac{Z-F}{T}\times A^{0{\cdot}3}$$

$$\text{or } Q=61\ \frac{Z-F}{T}\ A^{0{\cdot}3}$$

It would bear be repetition to state that the discharge obtained from equation (9) would occur in a drain on an average of once in three years and that a higher discharge upto four times this intensity may have to be faced at longer intervals.

It should also be mentioned that equation (9) is applicable to the drain in the Lower Chenab Canal tract or similar areas. At places with different conditions of rainfall values of A_o and m would also differ and the general equation (8) should be applied.

For areas having value of A_o less than 0·25 sq. miles, the dispersion factor is unity and equation (7) would apply.

Both A and A_o represent effective catchment areas duly corrected for effect cultivation in terms of reduced *banjar* areas.

(B) **Example.**

Gross catchment area of the drain = 55 square miles. Of this five square miles are cut off by the embankment of a railway line and thus the effective catchment area = 50 square miles.

Assume that the area is similar to the Lower Chenab Canal area for which ; $m=0{\cdot}3$ and $A_o=0{\cdot}25$ square miles.

Assume that this area is *kalrathi* and that 30 percent is cultivated.

Then cultivated area = 15 square miles ; *Banjar* = 35 square miles.

Equivalent *banjar* area = 10 percent of 15 square miles + 100 percent of 35 squre miles.
= 1·5 + 35·0 = 36·5 square miles.

$$\text{Absorption}=\frac{1{\cdot}5\times0{\cdot}18+35+0{\cdot}22}{36{\cdot}5}\text{ (from table 4 of this chapter)}$$

= 0·22 inches per hour approximately ; Slope = 1/2500.

Assume average direct distance of the watershed from the drain is equal to 4,000 feet, and the average distance which the water travels is equal to 5,000 feet.

Assume inlet time = 2 hours ; From Table 5 value of Z is 1·83

Let ¼ in. rain be held by trees, crop and undergrowth = F

$$\text{Then } \delta Z-F=\frac{1{\cdot}83-0{\cdot}25}{12}\text{ ft.; } v=1100\left(\frac{1{\cdot}58}{12}\right)^{0{\cdot}8}\left(\frac{1}{2500}\right)^{0{\cdot}7}=0{\cdot}73\text{ ft. per second.}$$

$$\text{Inlet time } T=\frac{5000}{3600\times0{\cdot}73}\text{ say } 1{\cdot}9\text{ hours.}$$

Therefore inlet time assumed is correct, and $Q=\frac{1\cdot58}{2\cdot0}\times61\ (36\cdot5)^{0\cdot3}=142$ cusecs.

The drain can thus be designed for 142 cusecs which it will have to run on an average of once in three years. The maximum discharge that can ever be expected from the catchment will be 568 cusecs.

11. Runoff from Very Large Catchment Area.

(A) The catchment area of large rivers of Northen India or natural drains which have catchment area larger than 500 square miles fall in this category. In ordinary practice there are two methods of estimating maximum flood discharges of a river at sites when it is proposed to construct a weir or a bridge accross it. Both of them should be used, one to serve as a check on the other.

(*i*) Measurement of the mean cross section of the highest flood and estimation of the mean of velocity either (a) by application of Kutter's or Bazin's formula or (b) by direct observation of velocity by means of floats or a current meter.

(*ii*) Measurement of the drainage area, observations of the maximum rates of rainfall on this area, and estimation of the amount which reaches the site of the bridge.

(B) **Calculation of discharge from mean flood section and mean velocity.**

By the first method the discharge is obtained by multiplying the area of the mean flood section by the meam velocity, either calculated or observed.

In order to determine the mean flood section, the following procedure should be adopted. First determine by enquiry the height of the highest flood ever known and correct the information if possible, by flood marks Then take an accurate section of the river bed at right angles to the course of the channel at the site of the proposed bridge and calculate the area contained between the highest flood line and the bed. Do the same at sections one mile above and one mile below the proposed site and find the mean area by adding together the three areas thus obtained and dividing by three

To find the mean velocity by calculation it is necessary to know the average hydraulic mean depth and the slope of the bed or water surface. To ascertain these, proceed as follows. Measure the length of the undulating line of the river bed in each cross section, *viz* ; the wetted perimeter, divide each area by this length, the quotients will be hydraulic mean depth of each section. Add together the three mean depths so found and divide by three. The quotients will be the average of the three hydraulic mean depths to be used in the calculation. Then ascertain by means of a levelling instrument the difference of level of the water surface at the margin at the upper and lower sections, *i. e.*, the fall of the water surface in two miles length. This will give the slope in the from of a fraction of which the numerator is the fall and the denominator the length between the upper and lower sections *viz.*; 2 miles, both expressed in feet. Having thus ascertained the mean flood and section the slope of the water surface, the velocity is obtained from the formula $v=C\sqrt{rs}$; where r represents the hydraulic mean depth in feet, s the slope as explained above, v the mean velocity in feet per second and C a coefficient which, far from being constant, varies within a wide range and depends on the hydraulic mean depth and surface slope as well as on the rugosity of the channel. Bazin's formula for the calculation of this coefficient is the similar of the two in common use, but Kutter's is supposed to be more accurate and specially suitable for large rivers and irregular sections.

The value of the coefficient C will be found to depend largely on the value assigned to f ; the value of f can only be assigned according the judgment of the engineer.

In calculating discharges velocities obtained by the foregoing method of calculation it should be borne in mind that, if the section is very irregular, it is necessary to divide the whole section in two or more fairly regular figures and calculate the hydraulic mean depth, velocity and discharge of each figure separately to give an approximately correct result. The total discharge will be the sum of the discharges of the different figures.

(C) **Calculation of flood discharge from catchment basins.**

The other method of estimating the discharge is by calculating the runoff from the catchment basin, above the site of work, during the heaviest known rainfall. The main data required for this calculation are the area of the catchment and the maximum rate of rainfall. Thearea can be found readily in most cases from a contoured survey map, as its boundary is defined by a watershed, the drainage of which on the inner side flows to the basin in question while that on the outer side discharges into other adjoining basins. Part of the rainfall fails to reach the point of final discharge at the site of the bridge owing to absorption and evaporation. The amount lost depends chiefly on the nature of the soil the fall of the country and the shape of the basin. The maximum rate of rainfall for a given basin must be obtained from the register kept at the nearest recording station, the period taken for estimating the rate ranging from two hours for very small catchments to 24 hours for large ones. The rate of discharge from the basin will not, however, be directly proportional to the fall because :—

(*i*) Very heavy rainfall is often only local, and sometimes occurs over a limited area not greater perhaps than 5 square miles round the station. Equally heavy rainfall may occur at other points in the basin. but possibly not at the same time.

(*ii*) The larger the basin the greater the probability that the flow from the ground near the point of discharge will have ceased before the flow from remoter portions had time to come in.

To provide for the proportionate reduction thus required for large areas, various empirical formulae have been proposed those chiefly employed being.

Ryves' formula $Q = C\ M^{2/3}$; Dicken's formula $Q = C_1\ M^{3/4}$

Where Q is the discharge in cubic feet per second, M the area of the catchment in square miles, and C, C_1 local coefficients depending on the raifall, soil and slope of the district. Values of these co-efficients suited to particular districts are best deduced from measured maximum flood discharges from known catchment basins. For catchments in the plains of Upper India. C is generally assumed to be 675 and C_1 ; 825.

J. Craig, M. I. C. E., has evolved a formula for discharge from a catchment area which appears to give reliable results :—

$S = 184{\cdot}2\ B \times \log\ 8L^2/B$; where S the sectional area in square feet of maximum flood at the discharging point.

B Mean width of area in miles ; L Mean length of area in miles.

The maximum flood dircharge can be obtained in the usual way by applying the total sectional area found by the formula to the cross section of the stream at the discharging point and ascertaining therefrom the hydraulic mean depth, and by measuring the declivity of the bed.

This formula is founded mainly on a consideration of the shape of the catchment area, and the accuracy of the results has been demonstrated by numerous results worked out by the author.

To obtain B & L, the perimeter, or ridge line, of the catchment area should be rectified by means of straight line as CD which, with the distance CA and DA, will divide the area into triangles with their apices at the point of discharge Fig. 15.

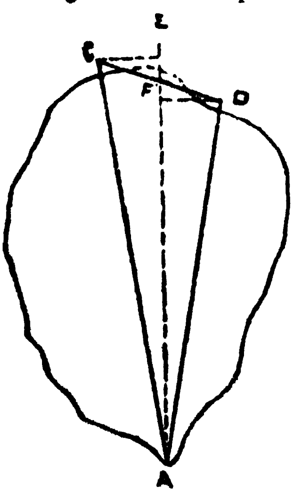


Fig. 15

For each triangle L is the mean length, and B is half the sum of the perpendicular distances from C and D to AF or L *i. e.*, its mean width, so that $B \times L$ gives the area of the triangle ADC.

The sum of the discharges from all the triangles into which the figure is thus rectified will give the total discharge of the basin

As a general rule it is a good practice, when calculating discharges by catchment basin formulae, to test the results by comparing them with the actual discharges of other basins under similar conditions and with those obtained from flood section and bed slope formulae. To do this it is necessary, first, to have the usual data of sections, transverse and longitudinal, of the river bed. Then to determine by a few trials the flood level which would give the discharge required by the catchment formula. This should be compared with the reputed flood level. If they differ materially, the latter should be again investigated. After this it will be safe to adopt whichever result gives the highest flood discharge.

No formula, however ingeniously constructed can cover the entire range of conditions presented by very large catchment basins. A snow-fed river, with a long course in the hills and then passing through an arid tract, can hardly be compared with one rising from springs and flowing through a highly saturated country ; the rainfall is rarely uniform over a large area, and the length of the basin has a controlling and regulating influence on the floods passing through it. Perhaps the most difficult cases to determine are those in which the floods that, previous to the throwing up of embankments, passed with an almost imperceptible flow over a wide expanse of nearly level country, are checked and have to be concentrated at the lowest depression : here sufficient waterway must be provided, as records are not available to check the results of the formulae. In such cases the demarcation of the various basins must be complete and accurate and careful observations should be made to determine the highest rate of flow off the area per square mile when saturated by long continued rainfall.

12. Examination Questions.

(*i*) Describe briefly the factors affecting variability of annual Rainfail.

(*ii*) (*a*) Describe the various methods of recording daily rainfall.
(*b*) Give your view about the spacing of rain gauges.

(*iii*) Descirbe briefly the various factors affecting the runoff from a catchment area.

(*iv*) Describe with sketches the procedure to make a hydrograph from the rain intensity curve at a station.

(*v*) Gross catchment area of a drain is 55 square miles ; of these 5 square miles are cut off by the embankment of a railway. Assume inlet time 2 hours. $Z=1 \cdot 83$, $M=0 \cdot 3$, $A_0=0 \cdot 25$. Calculate the Discharge of a drain with direct distance, 4000 ft. using Gulhati's farmula.

(*vi*) Describe the method to assess the maximum flood discharge at the site of a River from the previous years highest flood marks.

(*vii*) (*a*) Explain what is meant by run-off end mention the different factors affecting it.
(*b*) How would you determine and verify the maximum run-off catchment basis.
(*Mysore U. 1940*)

(*viii*) What are the different methods of determining (*i*) Minimum (*ii*) Ordinary (*iii*) Maximum discharge of a river ? Compare and criticise the various methods.

PART VI

DRAINAGE ENGINEERING

CHAPTER II

Surface Drainage. (Open Drains or Ditches)

1. Introduction.

Drains are constructed with the object of relieving waterlogged or flooded areas, to drain off ponds and marshes and to dispose of the surplus rain water not required for normal operations of the agricultural crops. The quick disposal of surplus rainwater is very essential to reduce the additions made to the ground water by its percolation down to the water table. In the irrigated tracts the seepage from the canal results in large additions to the groundwater reservoir and water-table tends to rise. In order to arrest the rise it is necessary both to reduce seepage from canals as well as to remove quickly the surplus rain water.

The drains may be natural or artificial. The irrigation channels usually run on the ridges and the lowest valley lines between the two ridges usually from natural drains. Rivers are natural drains. Sometimes drain-sections are artificilly constructed and maintained for quick disposel of surplus rainwater before it gets absorbed into the soil.

2. Classification of Drains.

The drains are classified in three categories according to the purpose for which they are constructed.

(a) Storm water or surface drains.
(b) Seepage drains.
(c) Surface-cum-seepage drains.

Storm water or surface drains are meant to drain off surplus rain water. They mostly run in the monsoon season. They are supposed to carry the flood discharge from the catchment area served by them.

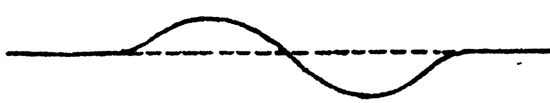


Fig. 1

With the advent of the canal irrigation, there has been a general rise in the sub soil water-table. In some cases water table comes upto the natural surface and even appears above it making the land unfit for cultivation. In order to relieve such tracts seepage drains are constructed to lead away sub-soil water to some outfall. These drains are generally small in size as they have to deal with small seepage discharges.

Surface-cum-seepage drains perform the function of seepage as well as storm water drains. During the major portion of the year they work as seepage drains in the high water-table area and even in the monsoons and in the case of winter rains they also carry the surplus rain water. Normally the seepage water flows in a cunnete dug in the center.

3. Alignment of Drains.

Normally the alignment of drains should follow the drainage line, the lowest contours in a valley.

When an existing drain is to be canalised, it will be expedient to reduce the length of construction and maintenance of the drain by cutting loops shown in Fig. 1.

If the alignment passes across any ponds or marshes, the drain should not pass through the middle of such depressions because the excavation work in such places is virtually impossible and the subsequent maintenance difficult. The alignment should pass near the pond and a feeder drain should be constructed connecting the pond with the main drain as shown in Fig. 2.

Normally the alignment of a drain should not be across the irrigation channels but where the drains are constructed after the development of the irrigation system there may be cases when drains have to cross the water-courses or distributing irrigation channels such as distributaries or minors. In such cases, the siphon or aqueduct crossing should be constructed or the irrigation should be arranged from other channels or by pumping installations for the areas served. If no such arrangements are made, compensation shall have to be paid for converting *Nehri* into *barani* cultivation and shall in additions provide temptation for the cultivators to put in *bunds* in the drains and to resort to irrigation from there which will greatly impair the efficiency of the drains.

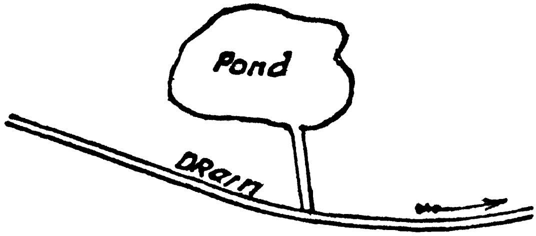


Fig. 2

4. Capacity of Drains.

In the last chapter on "Rainfall and runoff", the method of arriving at the correct flood discharge of drains has already been shown. The usual practice in the Punjab is to design the drains for a flood capacity of 4 cusecs per square mile of the catchment area in the canal irrigated tracts. This is considered to be the maximum capacity while in some cases, the drains are designed for a capacity of 1 to 2 cusecs per square mile of the catchment and the design of the sections and the masonry works provision for future widening.

5. Drain Sections design.

(A) **Valocity formula.** Usually the drain sections are designed by using Chezy, Kutter or Manning's formulae in India. In America, Elliott's open ditch (drian) formula is commonly used :—

$v = \sqrt{\frac{a}{P} \times \frac{3}{2}h}$; where v mean velocity in feet per second ; a is area of cross section of ditch in square feet; p wetted perimeter in feet ; h fall in feet per mile

A typical design of a drain cross-section as used in America is shown in Fig. 3.

(B) **Discharge of drain.** In order to obtain the dischasge of a drain, it is necessary to know the mean velocity of flow as obtained above, which when multiplied by the area of the cross section of the drain in feet, will give the discharge in cubic feet per second ; or expressed as a formula : $Q=av$; where Q is discharge in cubic feet per second.

a is area of cross-section of drain in square feet.

v is mean velocity of flow in feet per second.

(C) **Number of acres a ditch will drain.** With the discharge of the ditch known, the number of acres drained can be obtained by dividing the ditch discharge by the amount of water that is to be removed from each acre or expressed in a formula, as follows :—

$A=Q/Z$; where A is number of acres

Q discharge in feet per second.

Z amount of water to be removed from each acre in cubic feet per second

It will be noted that Z is the same as the drainage co-efficient, except that its unit in this case is cubic feet of water to be removed per second per acre, while, in the other case, it was inches in depth per 24 hours. The conversion of the drainage co-efficient to Z is a simple problem.

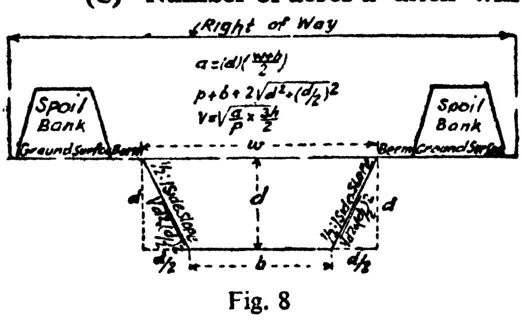


Fig. 8

For example, what would be the value of Z, if the drainage co-efficient was ½ inch for an area of 1 acre ? ½ inch equals 0·04166 foot ; 1 acre equals 43·560 square feet.

Water ½ inch deep over 1 acre equals 0·04166 times 48·560 or 1,814·709 cubic feet of water to be removed each 24 hours. There are 60 minutes to an hour, 60 second to a minute;

so in 24 hours there would be 3600 times 24 or 86400 seconds. The amount of water that must be removed each second will be equal to 1,814·709 divided by 86,400, which is 0·0210, the value of Z for a drainage co-efficient of ½ inch.

The table below has been computed in order to give the values of Z for the different drainage co-efficients.

TABLE.

Values of Z for various Drainage Co-efficients.

Drainage Co-efficient, inches per 24 hours.	Z cubic foot per second per acre.	Drainage Co-efficient, inches per 24 hours.	Z cubic foot per second per acre.
1	0·0420	1/2	0·0210
15/16	0·0394	7/16	0·0184
7/8	0·0367	3/8	0·0156
13/16	0·0341	5/16	0·0131
3/4	0·0315	1/4	0·0105
11/16	0·0289	3/16	0·0079
5/8	0·0262	1/8	0·0052
9/16	0·0236	1/16	0·0026

(D) **Side slopes.** In selecting the slope for a ditch, it will be necessary to consider the kind of material through which the ditch is to be dug. A rock formation could be used vertical sides ; a loose rock or hard pan soil, ½ : 1 ; a clay soil 1 : 1 ; a sandy loam 2 : 1 ; a very sandy soil 3 : 1.

(F) **Grade in drains.** The grade that can be given to a ditch is more or less limited by the fall of the land through which it is located. This grade, however, can be modified by shortening or lengthening the ditch, or by digging it deeper, but to lower the grade by digging a longer ditch is an expensive method and should be used only in extreme cases. The velocity can better be lowered by the design of the ditch itself. An ideal ditch is one so designed as to obtain a velocity sufficient to make the ditch self-cleaning, yet not enough to cause it to cut more deeply or erode the sides. In order to get this desirable velocity, it is necessary to design the ditch for the soil through which it is to run.

Etcheverry gives the following as the maximum values of mean velocities safe against erosion :—

Material	Mean velocity in feet per second.
Very light, pure sand of quicksand character	0·75 to 1·00
Very light loose sand	1·00 to 1·50
Coarse sand or light sand soil	1·50 to 2·00
Average sandy soil	2·00 to 2·50
Sandy loam	2·50 to 2·75
Average loam, alluvial soil, volcanic-ash soil	2·75 to 3·00
Firm loam, clay loam	3·00 to 3·75
Stiff clay soil, ordinary grave soil	4·00 to 5·00
Coarse gravel, cobbles, shingles	5·00 to 6·00
Conglomerats, cemented gravel, soft slate tough hard pan, soft sedimentary rock	6·00 to 8·00
Hard rock	10·00 to 15·00
Concrete	15·00 to 20·00

The experience of United States Engineers indicates that an average velocity from 2 to 3 feet per second will prevent the deposit of silt.

In reality the velocity needed to prevent silting or erosion varies with the depth of flow according to the following relationship (for fine sandy silt).

Depth in feet.	Feet per second.	Depth in feet.	Feet per second.
1	0·84	6	2·64
2	1·30	7	2·92
3	1·70	8	3·18
4	2·04	9	3·43
5	2·35	10	3·66

In designing an open ditch, cheapness and efficiency should be main object. In American practice a drain which is twice as wide as it is deep will be the cheapest, as for as the cost of excavation goes. A desirable depth for the drain is from 6 to 12 feet. A bottom width less than 4 feet is difficult to construct and clean out, except by manual labour.

6. Longitudinal Sections of the Drain.

(*a*) The following information should be collected while carrying out surveys along the proposed alignment of drains.

(*i*) Cross sections at every 500 feet to the full land width.
(*ii*) Natural drain bed and water levels at every 500 feet.
(*iii*) The spring levels every mile.
(*iv*) Inlet sites of subsidiary drain with other hydraulic data.
(*v*) Bridge sites.
(*vi*) Ful data for the irrigation channels crossing the drain.
(*vii*) Soil sample every half mile to determine the transmission constant and the probable side slope which the soil will stand below spring level.

(*b*) To prapare the longitudinal section plot the F.S.L. line of the drain and the natural surface levels in the longitudinal section paper. Fix outfall water surface level considering the highest levels in the river or outfall drain and the "backing up" which may result in the drain. The permissible slope will depend on the ground slope and the permissible maximum slope for the discharge of the drain. No falls are given in a drain except far metering purposes. It is perferable to construct headless meter flumes in drains as all drop available must be used in the slope for efficient working of a drain.

Divide the drain into convenient reaches preferably from inlets or bridge sites to the similar masonry works and work out capacity of the drain in various reaches for the catchment area served by the drain according to para (4).

Design the sections for the discharge and the slope so arrived at. Relatively deeper sections are preferable. Keep C. V. R. about 1·2 or silt factor about 1·1.

Then plot the resulting bed and full supply levels. Compare the bed line with spring level line. If the drain is to be dug by manual labour the bed should not be deeper than 2 feet below the spring level. If the excavation is to be carried out by the excavator, it may be deeper.

7. Cross Sections of a Drain.

A typical cross section of a drain in digging is shown in Fig. 4 and one for a drain in filling is shown in Fig. 5.

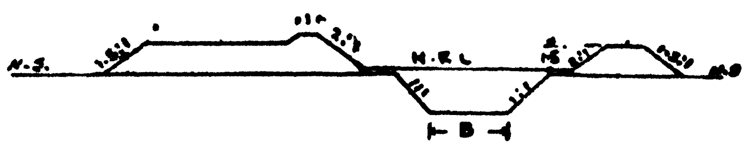


Fig. 4.

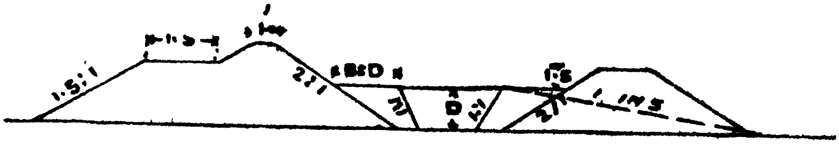


Fig. 5.

The road width should be 15 feet at least. In the case of high ground, it may be on the natural surface and in the case of low ground subject to flooding it should be at least 1·0 foot above natural surface. The spoil bank should be away from the road leaving a drain at least 2 feet wide and one foot deep below the road level. The road and the spoil should be on the side of the drain which has relatively smaller catchment area. The slopes of all earth-work should be 1½ to 1.

The inner slopes of the drain sections are usually dug 1 to 1 but in the case of seepage, they may be flatter according to the soil conditions.

On the sides other than the road, there should only be small dowel 3 feet wide on the top and with 1½ feet free-board above highest flood level. Dowel should be constructed sufficiently away so that the bed of the drain could be subsequently widened by at least 50% and should leave enough room for the excavator to work on the berm. The dowel is useful to prevent shoaling, silting and ravining. The water from the catchment should be admitted through properly constructed inlets.

Cunnete sections are desirable as shown in Fig. 6 for the seepage-cum-storm water drain where the bed is below spring level. The berms should be at the spring level. The cunnete should be designed for the excepted seepage discharge with the available slope and total sectional area should cater for the flood discharge. No cunnete is required when the bed is above spring level.

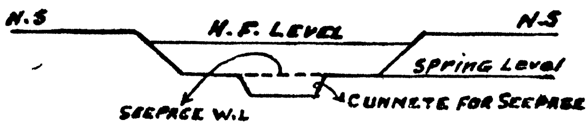


Fig. 6.

8. Distance Marks.

The reduced distances are measured from the outfall upwards unlike the canals so that the lengths may be increased as the drains are extended upwards.

The cast iron distance marks are expensive and are likely to be stolen in the out-of-way places on the drains. The distance marks should be of stones as shown in Fig. 7. The (reduced distance) R. D. number should be inscribed on the sloping face of stone on both sides. These also serve as land boundary pillars.

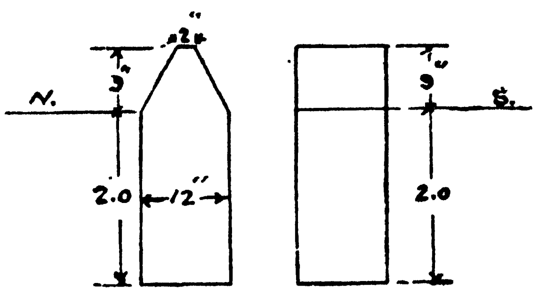


Fig. 7.

9. Maintenance of Drains.

The maintenance of drains usually entails the following types of works :—

(*a*) Maintenance of actual section of the drain including the inner side slopes.
(*b*) Maintenance of driving road.
(*c*) Filling of the raincuts.
(*d*) Repairs to bridge and other masonry works.
(*e*) Painting distance marks.
(*f*) Silt clearance of drains.

The items of work (b) to (f) are usually of the same type as done on the canals and therefore need no special mention. The sub-head (a) is very important for efficient working of the drains generally includes the following items of work.

(A) Removal of obstructions.
(B) Removal of weeds or *Jala*.
(C) Prevention of sloughing of sides.
(D) Removal of grass and jungle from inside slopes and berms.
(E) General silt clearance.
(F) Prevention from caving banks.

(A) **Removal of obstructions.** It has been observed that at *ghat* sites and other crossings, the bed tends to become higher. The maintenance gangs should be employed to remove such obstructions.

(B) **Removal of weeds or *Jala*.** In the case of seepage drains the most important work of maintenance is the clearance of weed growth. Weed growth is unavoidable in seepage drains on account of water being clear. (See paragraph No. 22 Chapter VI, Part II).

(*i*) **Weed clearance by manual labour.**

The weed growth is maximum in winter when water temperatures are very low and men cannot work in cold water. Hand clearance is only possible up to 2·0 feet depth but the weed growth is generally very excessive. The author during his service as an Engineer-in-charge of seepage drains has occasions to see a normal depth of flow of 1·0 foot headed up to 4·0 feet. Fortnightly removal is necessary to keep the channel in good condition. This is usually done in the Punjab by maintaining gangs who remove the weeds by hand.

In some large drains racks or drags are, used which are attached to a rope thrown in water and the gang pulls the drag which a huge load of weeds coming with it Complete clearance is not possible in this way but this serves to make the water muddy and the weed growth is retarded because in opeque water the light cannot reach the roots of weeds.

(*ii*) **Weed clearance by machinery.**

Mowing machines, scythes, sickles, brush hooks and axes should be used at frequent intervals to keep the banks clear but cutting vegetation from the bottom and sides is a more difficult undertaking Where there is perceptible current, long, pliable bandsaws, weighted to hug the bottom and drawn back and forth while moving upstream diagonally, have been used with advantage. Men should be posted downstream with pitchforks to catch the floating stems and throw them out.

Another device which has been used extensively in England and also in India, Egypt and Australia is described below.

An English device for clearing rushes, water lilies, weeds and other growth from drainage and irrigntion ditches, rivers, lakes and ornamental waters consists of power driven cutters operated from a small motor boat of 12 inch draft Fig. 8.

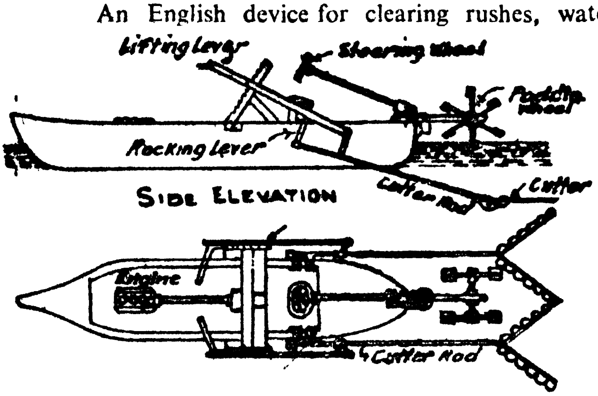


Fig. 8

Each cutter consists of a steel blade, v-shaped in plan with a wavy or scalloped cutting edge and having the apex attached to a long rod. There is a rod on each side of the boat, the upper end being attached to a rocker arm and also to a link lifting level by which the cutter is raised and lowered as desired. The rocker mechanism is drived from the launch engine and as the boat moves forward, the cutters are given a rapid oscillating motion, so that they engage the weeds at high velocity and strike them with a diagonal cut.

In shallow water the cutters trail on the bottom, being provided with spring shoes to keep them out of the mud and to guide them over logs or other obstructions. In deeper water they are set usually to cut at a depth of 4 to 6 feet. It is stated that in ordinary service, the boat will clear as much as 4 acres is an hour. A small double paddle wheel is used for propulsion as it will not be clogged with the weeds. This wheel serves also as a rudder, being carried by a pivoted frame which has a radial movement in a horizontal plane. For small ditches a row boat or a scow pulled by two lines may be fitted with a single, hand-power cutter, the oscillating mechanism of which is operated by a crank handle. This wheel cutting divice is built by the Saunderson Tractor and Implement Company, Bedford, England.

(*iii*) **Weed destruction by the use of poison.**

The use of poison is not desirable because in addition to the plants, all fish and the cattle drinking water are killed. White arsenic and potassium permaganat where tried at the Machhianna drain by the author with some success.

(C) **Sloughing of side.**

In seepage drains sloughing of sides is very troublesome. It is due to two causes; firstly due to the low angle of repose of the wet soil especially when it consist of fine soil partical and secondly due to the velocity of inflow being greater than the critical velocity which would lift up the soil particles. In the latter case the sloughing of sides is also accompanied by the heaving of the bed. Open seepage drains are impossible if velocity of inflow is greater than the critical velocity or floatation velocity necessary for the soil conditions in a drain.

The following measures may be adopted to reduce the sloughing of sides.

(*i*) **Flattenings side slopes.** In some cases the trouble has been cured by flattening that side slopes to 1 in 4 or 1 in 5. This can only be done if the land is available.

(*ii*) **Staking and bushing.** It is very expensive and of limited utility because it is removed if silt clearance is done by mechanical excavator. It may be in the form of rolls behind the stakes but the most successful type is known as Trestle Bushing, as sketched in Fig. 9.

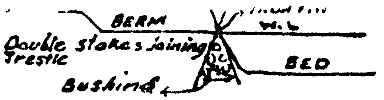


Fig. 9

(*iii*) *Gachi* **pitching.** The side slopes are pitched with large divots of berm earth from nearby irrigation channel. This is usually quite successful but can only be done if berm clods are available nearby.

(*iv*) **Tile pitching.** Porous tiles or perforated concrete pitching is very expensive and has not been tried on a large scale in India.

(*v*) **Perforated hume pipes.** The seepage drain is dug down to a convenient depth so that the bed does not heave and the sides do not slough. The perforated Hume Pipes are sunk vertically in the bed plugged at the bottom Fig. 10. Water collects into the open drain. The efficiency of the drain is very much increased without increasing the depth which cannot be maintained.

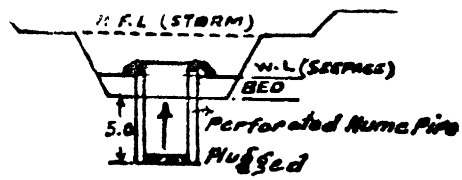


Fig. 10

(D) **Removal of grass and** *jungle* **from inside slopes.**

(*i*) During the winter months the seepage cunnete section is maintained to ensure unobstructed seepage flow and before the monsoons, say in June, the *jungle* and grass growth should be cut from the berms and inner side slopes. They should not be uprooted but cut flush with the ground leaving the roots which strengthen the berm and sides against erosion in floods.

(*ii*) In some localities,woody growth such as willows and other young trees, predominate. Soon after cutting, young shoots sprout from the old stumps and grow rapidly, thus necessitating recutting within a short time. In an attempt to prevent such sprouting O. C. Kulicka, a drainage Engineer of Greenville, U. S. A., has experimented with various chemicals which where intended to destory all the live tissues of the stumps and roots. He found that a solution of granulated caustic soda and white arsenic, 10 pounds of each dissolved in 8 gallons of water, proved most effective. After the mixture cools, two more gallons of water added, and the compound is applied to fresh cuts and splits in the stumps by means of a homemade mop. The cost of this work averaged about Rs. 35/- per mile and the rate of progress for a crew of eight men was from ½ to 3 miles per day, working on a drain of 125 feet base width with a very dense of growth. On smaller drains, only three or four men were used.

Great care must be taken not to inhale or swallow any of the dust or poisonous fumes arising while the preparation is being made and all live stock should be kept away for ar least 10 days from the drain undergoing this treatment. Vinegar or some other acid, acts as an antidote in case of accident.

(E) **General silt-clearance.**

The silt-clearance as defined for irrigation channels is not found in drains. However rain water entering a drain brings with it a lot of debris and earth from the adjoining lands All such deposits have to be cleared out for efficient working of the drain. The frequency of silt-clearance depends on the class of the drain. Efficiency of a storm water drain is not appreciably impaired even though it silts up a couple of feet so long as the banks are not overtopped in floods in areas where damage could be caused to valuable crops or property.

In the case of seepage drains every inch of silt accumulated on its bed tends to reduce its efficiency. This will not only reduce the effective area but also the infiltration head causing flow into the drain. Such drains should be silt-cleared as soon as a depth of 6 inches has accumulated on the bed.

If the seepage water runs more than 2·0 feet deep, then the efficient silt-clearance cannot be done by manual labour. On the other hand, the machine (mechanical excavator) cannot tackle anything below 18″ of site clearance and if used for lesser depths is extremely uneconomical and spends all its time in crawling. Moreover silt-clearance by machine is not possible in a drain narrower than 8 feet because correct sections and slopes cannot be properly shaped even with the smallest machine. A programme of mechanical excavation should be made sufficiently in advance, more than a year; so that the machines could be arranged according to a planned programme. Dragline Excavators with buckets ½ cubic yard to 2 cubic yards are used in the Punjab for such silt-clearances.

(F) **Caving banks.**

There are two common causes for caving banks—either the grade of the drain is excessive resulting in erosive velocities or the side slops have been improperly constructed and are too steep to be permanent.

If the natural fall of the ground is excessive, the drains may be designed with slopes necessary to produce safe velocities, and the surplus fall can be absorbed in a series of "drops" (falls). If there is only a moderate excess in grade, caving can be prevented by protecting the banks at critical points with masonry riprap, willow matteresses or brush retards or by deflecting the current with some form of spur fence or dike. Sometimes engineers can take advantage of moderataly high grades to construct the drains much smaller than necessary and depend upon erosion to enlarge them.

On the other hand, it is not easy to secure side slopes which will be absolutely permanent in any kind of soil. It is very inconvenient and difficult for floating dredge operators to dig flatly sloping banks, so much so that they prefer to dig the drains with the prescribed top width put with steeper side-slopes and greater depth than specified, without pay for the extra excavation. Many good engineers in American practice in the past have allowed this to be done on the theory that the banks would soon cave and side just enough finally to cause the drain to assume the form of cross section which it was originally designed to have. In some cases, this idea has worked out fairly well, while in others, it has been disastrous. Experience has shown that, in general it is wise to undergo the extra trouble and expense to construct the drains in the first place with smooth side slopes which are flat enough to ensure permanence.

10. Prevention of Silting in Open Drains.

The methods that prevent the silt from entering the drains or that tend to carry it through without being dropped by water, fall under this category.

(*a*) **Soil washing.**

If by proper cultivation, terracing or other methods soil-washing from drains and watersheds can be kept under control much of the silt evil will be eliminated, because silt will not have a chance to get into the drain. This subject is fully treated in Chapter IV of this part and is mentioned here only to show its relation to the problem of drain maintenance.

(*b*) **Diversion drains.**

The greatest trouble from silting generally occurs where water from upland drains with steep lopes is emptied into a flat drain in bottom land. The velocity of flow is decreased to such an extent in the flat drains that the silt settles out rapidly and necessitates almost constant maintenance to keep them open. Sometimes the trouble is minimized by the use of diversion drains running around the lower edge of the high ground, which collect the hill water, highly charged with sediment and carry if for some distance to a fovourable point where is is emptied into a drainage channel. This protects bottom drains to some extent, but the diversion drain itself needs to be cleaned out occasionally and its burden of silt generally causes trouble farther d/s.

(*c*) **Settling basins.**

Another method that has proved effective in U.S.A., where the topography permits its use, is to collect the hill water in settling basins and allow the silt to settle out before the water enters open drains. Frequently the natural depressions and hollows can be utilized for this purpose, but unless there is another into which the water can be turned when one is filled with sediment, they too must be cleaned out at regular intervals.

(*d*) **Control of silt by flooding.**

The most sensible way of all, is to treat the silt as a friend rather than as an enemy and to utilize it to build up and enrich low bottom lands. This method was orginated by Roy N. Towl a consulting engineer of Omaha, Neb., and it has been used by him with much success on one of his farms near Tekamah, Neb., since 1917. Rich silt has been deposited at the rate of about 1 foot per yard, and the land, formerly sandy and level, has now assumed a considerable surface slope away from and right angles to the over-flow drain. Water is caused to spread out in a slowly moving, thin sheet which does not injure growing crops and does not prevent cultivation a short time after it recedes. Good crops are thus produced at the same time that the land is being continually enriched and built up.

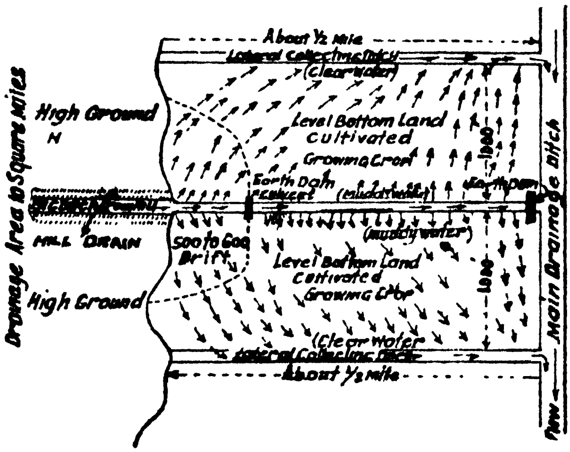


Fig. 11

Fig. 11 illustrates clearly the action that takes place. The central or overflow drain is made just large enongh to take care of a normal flow. Any excess deliverd to it from the larger, non-overflow, hill drain passes through the culvert in the upper dam and is prevented from entering the main drainage ditch by the lower dam. The water continues to rise until it slowly overflows the banks and travels laterally in both directions, depositing a thin film of silt as it goes. By the time it reaches the lateral collecting drains, 1000 feet away practically all the silt has been dropped, and the water is delivered to the main drainage ditch in a clarified condition. Any drift or other debris that comes down in times of flood is caught by the upper dam and deposited within the small area shown by the dotted line, thus leaving the cultivated land free from trouble in this respect. If the culvert should become clogged, no paticular damage would result before the water receded and it could be cleaned out.

The layout shown in the sketch is suited to take care of the run-off from a watershed of 10 square miles. A larger watershed would require proportionately more land for the water to spread over. Great care is necessary to see that the water thins out uniformly and travels slowly. The bed and banks of the overflow drain build up at an equal rate so that, in a few years, the drain flows along the crest of a ridge which may have a lateral slope of as much as 1 foot per hundred.

11. Inlets.

In order to prevent ravining of sides and washing in of earth caused by inflow of rain water into the drain, inlets are necessary. They should be provided on liberal scale as their usefulness cannot be over-emphacised. The number of inlets depends upon the catchment erea, its slope towards the drain, the nature of the soil and whether the drain passes through the cultivated or uncultivated land. Carefull observations by the engineer incharge during the rains will indicate where they are necessary. It should be considered, bad mantenance to see standing water even in small areas for any length of time and, therefore additional inlets should be provided. It is a good practice to have a large number of small, cheap inlets rather than a few large and expensive ones. In irrigated tracts in the Punjab plains an inlet about a mile apart will do and in the unirrigated areas they may safely be 2 miles apart.

There are no standard design for inlets. The designs depends upon the prevailing bed levels of the subsidiary and the main parent drains and the discharge of the subsidiary drain.

The design will also vary according as the inlet is on the road side or the other. When there is no appreciable difference between the bed levels, a flume pipe inlet or just an open culvert will do ; but if there is a consididrable drop then a proper fall will have to be designed.

12. Discharge Observations on Drains.

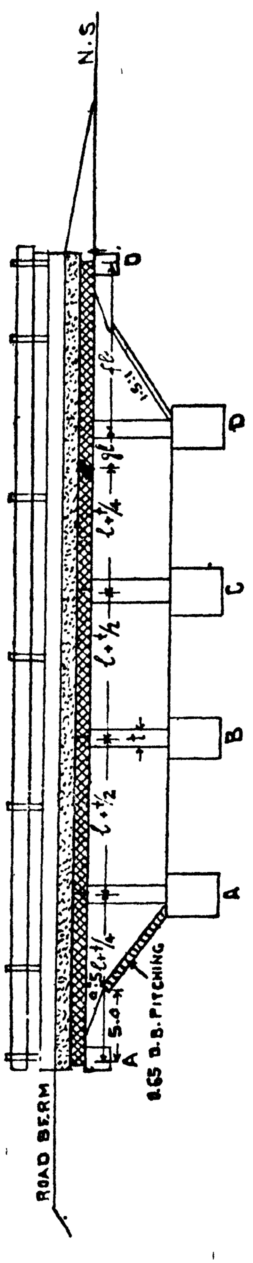


Fig. 12

It is very important to have an accurate idea of the daily discharges and the flood discharges of the drain to watch their future development and the effect in attaining the object for which a particular drain has been constructed such as for lowering water table, for draining swamps, for reclaiming *thur* land, for reclaiming waterlogged land or for public health reasons.

The discharge observations are very difficult and inaccurate in the case of drains on account of ever-changing bed and cunnete sections. The pitched permanent discharge sites are waste of money on account of an ever-changing bed level. The country side is generally flooded and the discharge sites are unapproachable. The selection of suitable discharge sites on drains is a problem.

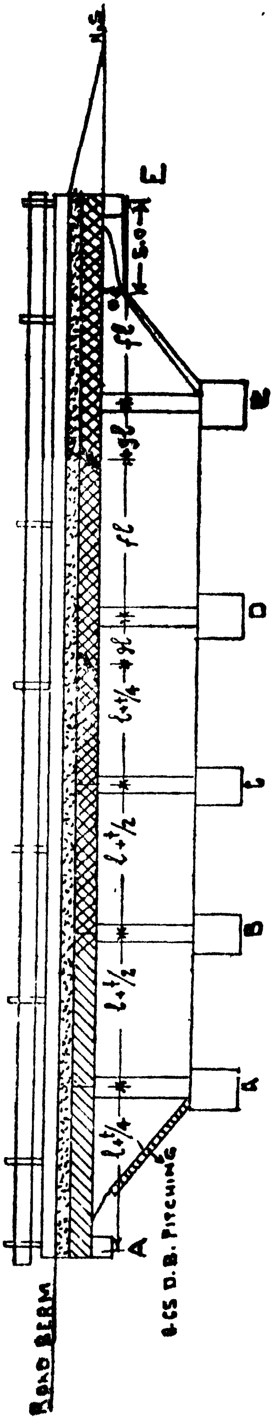
The discharge sites should be in reaches with high banks where loops have been cut out. The observations will, no doubt, in such a case be absolutely correct because there is a lot of ponding up and storage in the reaches upstream of the straight cuts. In the case of *Budhi Nallah*, Upper Jhelum Canal, the author had occasion to observe discharges in its tributaries such as Machhinna, 250 cusecs and P.R.K. Drain 1200 cusecs and other 200 cusecs but the net effect at the headless meter flume at R.D. 160,000 never exceeded 400 cusecs. As there are no falls, accurate metering is not possible. Headless meter flumes, as given in Chapter XXII part II of this book have proved very useful. The current meter observations are possible at bridge sites with *pacca* floors which remain clear of silt. The accurate gauging of drains has defied solution.

The discharge observation of seepage drains are still very difficult on account of weed growth, small depths and small discharges. Current meter discharges are possible in specially maintained sections where velocity exceeds one foot and depth is more than one foot. Velocity rod discharges are also possible when velocity exceeds one foot and depth is more than one foot. When velocity is less than 1·0 ft. per second and depth less than 1·0 foot, observations may be taken measuring mid-depth, surface-widths and surface velocity. The discharge should be calculated applying an overall coefficient of 0·7.

13. Bridges on Drains.

(*a*) The bridge design on drains has many novel features which do not exist in the case of canal bridges such as wet foundations below spring level, saturated soil behind the wing walls and abutments, submersibility and the necessity for future development and widening of drains. Shallow foundations with *pacca* floors between the piers are not desirable on account of the fact that the alluvial beds are liable to scour in floods. The foundations have, therefore, to be in the from of wells rectangular or circular with enough depth below the probable maximum scour to take up the loads coming on the piers and abutments To begin with, ordinary R.C. slab and T beam bridges supported on strong piers and abutments were designed ; but there occured numerous failure due to out-flanking and submersibility Such bridges were very expensive on account of heavy wing walls and abutments with expensive well foundations.

(*b*) The modern practice is to design on large drains either in the form of Irish bridges or in the form of submersible and extensible bridges described below. The first type has a limited



utility in the case of drains which are dry for the most part of the year or where the seepage depths do not exceed more than a foot. They usually prove to be cheap and successful but they are not liked by the public because they canont be used even in small floods.

(*c*) **Submersible and extensible bridges.**

The idea of extensibility without additional cost is a great asset and an achievement for drains which have yet to develop and require widening. It is explined in Fig. 12

An extended bridge without practically any loss of material or extra labour would be as shown in Fig 13.

For extension of the bridge beyond dummy abutment D′ a new pier E is to be built at a distance of fl which generally gives enough space For extra convenience, the span D D′ could be spported on temporary dry brick pillars founded on sleepers and even the dummy abutment D could be removed ; and new pier E could then be conveniently sunk. The bridge will be extended as shown in Fig 13. It is capable of extension similarly. It is very important that the end of the slab meant for extension should be made slanting and reinforced.

The bridge is also submersible with low parapets having G I. pipe railing. The bottom of the slab should essentially be at N.S and right hand berm and bank should also be at N.S. The bridge is safe against outflanking as the upstream and downstream water levels will be the same Light projection of the earth berm (not false) under the bridge is provided in the form of 0·65 ft. brick pitching under the bridge only, using wells or casing of piers as toe walls. *Pacca* protection of these berms is not needed as no heading up is likely in any condition of working.

This is the cheapest design for a drain bridge as the main cost of bridge on drain is that of abutment and wing walls for saturated soil pressures and their well foundations. This saves both of them. The cost of extra length of slab is trifling in comparison with the cost of abutments and wing walls on wells.

It is not expedint to stick up the bridge higher than the natural surface on drains for *katcha* roads. In case of earthen village and district roads, when storm water floods them by six inches, no traffic is possible The bridge is not then in use. There is no harm if the bridge is submerged when the road is not in use.

The standard loading of different type of bridges (foot village road, district road) requires specific proportion of the cantilever and the simply supported span as given in the calculations as per author's publication No. 8 available in the Punjab Irrigation Library.

Although the cantilevered spans are 1·31 to 1·41, the moment shears and the reaction at the free end are in no case more than those in a corresponding ordinary simply supported span of length l. This device, therefore, makes the bridge relatively cheaper.

Brick pitching is a very unsuitable material as filling over the R.C slab of bridges. It connot splay loads except to the extent of the size of the brick. Bricks generally break and crush under the cart and roller loads. Ballast would probably be a bit loose against water action and traffiic at bridge is liable to be over-topped in floods. It is always best to use cement concrete, say 1 : 4 : 8 in brick ballast as filling over R.C. slabs. This stands the normal use in out-of-the way places and is not washed away even when it is submerged under water.

14. Other Masonry Works.

The other masonry works on drains are the crossings for water

(Fig. 13)

coures and irrigation channel. It has already been said that they should be avoided as far as possible in good alignment of the drains. There are no standard designs. The water-course crossings are usually in the from of C. I. Pipes or R.C. troughs (Khosla type). They can usually be designed where they have enough clearance above the maximum flood and have in some cases been successfully used when above the winter-flood or seepage full supply levels and submersible in the monsoons.

The crossings of minors and distributories are generally not many and are designed as R.C. trough aqueducts well above the highest flood levels. Siphon designs should be avoided altogether on account of the future development of the drain and the uncertainty of maximum flood discharge.

15. Seepage Discharges of Drains.

The seepage flow can now be estimated in the drain with a fair degree of accuracy because it is now possible to determine the seepage or percolation intensity co-efficient of the soil as existing at site according to the method described in Chapter III, Part VI of this book. The previous practice to get soil samples and the determine the transmission co-efficient in a laboratory was very erratic, because natural soil can never be reproduced in laboratory.

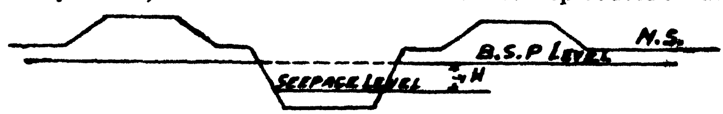


Fig. 14

Let P be the Perimeter of the drain section Fig. 14 ; H the Percolation Head (Difference between B. S. P. Level and the water level in the drain): σ the Percolation intensiry co-efficient per million sft. at 20°C. per unit head in cusecs ; q the Discharge in cusecs per thousand feet length of drain.

$\therefore q=\sigma HP\times 10^{-3}$ per thousand.

16, History of Drain.

It is advisable to maintain history of the drain to watch the object for which the drain has been constructed. The history may preferably be divided into four parts.

(*a*) **Narative description.**

The original conditions should be compared with the existing ones every year. This part should explain how far the object of the drain such as lowering water-table, draining swamps, reclaiming *thur* land, reclaiming waterlogged lands and public health requirements have been attained or progressed.

(*b*) **The technical part.**

This part should deal with the technical notes on the working of the drain covering the following points :—

(*i*) Alignments and its improvements.
(*ii*) Change in capacity.
(*iii*) Improvements in L. section.
(*iv*) Improvements in cross section.
(*v*) Improvements in maintenance methods.
(*vi*) Desired changes in masonry works.

(*c*) **Plans to accompany the history.**

(*i*) Index plan.
(*ii*) Long section original and revised.
(*iii*) Existing cross section compared with the original.

(*d*) **Appendices.**

Statements should give the following information :—

(*i*) Annual cost of maintenance.
(*ii*) Annual statement of flooded areas.
(*iii*) Statement of benefits to land derived from drains.

The history should be posted every year giving the information briefly in the order mentioned above.

17. Examination Questions.

1. (a) Give the usual classification of surface drains.
(b) What considerations govern the alignment of surface drains ?

2. Sketch the typical cross-sections of a surface rain fully in digging and with its bed above natural surface.

3. Sketch a suitable section for a seepage-cum-surface drain. What consideration governs the design of cunnete in this case ?

4. Describe the various methods used in the Punjab for weed clearance from seepage drains.

5. Describe the various measures usually used to stop sloughing of sides in seepage drains. *(P. U. 1943)*

6. What measures can be adopted to prevent or reduce silting of open drains ?

7. Explain the use and function of submersible and extensible bridges in the case of surface drains with sketches.

8. Describe the function of surface and seepage drains to reduce waterlogging.

9. (a) How would you keep a balance between surface irrigation and waterlogging.
(b) What main points govern the alignment a d design of drains. *(P. U. 1952)*

10. Sketch cross section of a typical surface andepage drains. H cw would you fix the designed scharges for such drains. What are the important maintenance features of these drains. *(P. U. 1955)*

11. What steps will you take to stop sloughi sides in seepage drains. *(P. U. 1957)*

12. How would you carry out the following maint nance in an open drain.
(a) Removal of weeds (b) prevention of sloughing of sides (c) prevention of silting (d) prevention of caving in of the banks. *(P. U. 1958)*

PART IV

DRAINAGE ENGINEERING

CHAPTER III

Subsurface Drainage

1. Introduction.

It has been shown in the last chapter that open seepage drains cannot be efficiently maintained when the depth of digging is more than 2·0 feet There is trouble due to the sloughing of the sides and the bed. When it is required to lower the water-table by more than 2·0 feet, the subsurface drains in the from of covered underground tile drains must be resorted to. The primary object of the subsurface drainage is to lower the line of full saturation sufficiently below ground level so that the plant roots get the requisite free circulation of air.

The subsurface drainage has not yet been used in the Punjab. In the United Provinces in India, it has been used with success for the colony at the Sarda canal headworks The subject has been developed scientifically in America, A list of books referred to is given at the end of this chapter.

Formerly three poles were used to leave a space in between for the seepage water to collect and flow out. The poles usually decayed in a couple of years and became clogged, The drains became useless and the replacement of poles used to be very costly. Then drainage by the use of stones was accomplished in the same manner as by poles. If large, flat stones were available, they were placed at the sides of the drain with one over the top, thus forming a small channel which was more durable than the poles. If flat stones were not to be had, small stones were thrown into the drain to the depth of 6 or 8 inches. The water percolated around them to the outlet. Instead of the poles decaying in this case, the spaces between the stones become clogged with silt, thus rendering the drains useless. This is the most effective method. Tiles are in the same way as the poles and stones were used. They are more durable when properly laid and should last for many years doing good work. The tiles of burnt earth or concrete are called in India as earthen-were or *colaba* drain.

2. Benefits of Subsurface Drainge.

Subsurface drainage has three major beneficial effects.

(*i*) It gives the root zone greater depth, which increases the available supply of plant foot and moisture. It has been found that plant roots will extend as for as 3 or 4 feet below the surface of the ground where conditions are favourable. (*ii*) It results in a warmer soil. Water clogged soils are slow to warm up, because more heat is required to raise the temperature of a a given volume of water by 1° than to raise the temperature of a like volume of air upto the same extent. When air is in the soil in place of water, the soil warms up more quickly and maintains a higher average temperature. This allows earlier planting of crops and lengthens the growing season. (*iii*) The physicial condition of the soil is improved. Drainage permits the development of the granular structure so essential to good tilth and lengthens the period during which the optimum soil moisture for crop content prevails. Drained land is, therefore, more safely and easily worked, reducing the effort of the farm operations.

Interlinked with the three major benefits given above are several outstanding benefits that become very noticeable to the farmer. It will perhaps be well to examine these a little more in detail. The aeration and higher temperature due to drainge increase the activity of soil organisms, and help to render the plant food of the soil available. The improved physicial condition of the soil results partly in an increased ability of the soil to retain a larger amount of film water which, in dry periods increases the supply available to plants. There is also a better internal circulation of water through the soil. The accumulation of toxic substances in the soil is greater where poor drainage conditions prevail. Sanitary soil conditions often form one of the problems of good management. The formation of these toxic substances is less in well-drained soils.

The removal of the surplus water from the soil eliminates dampness, which reduces the vitality of the people and the stock on the farms, rendering them more susceptible to disease. The removal of the stagnant water also reduces the danger from malarial troubles. Drainage reduce injury by frost, since air instead of water occupies the interstices of the soil. The heaving action of frost is not so harmful on well-drained soils ; this is, of course, more noticeable in the north than in the south. The increase in the depth of the root system zone enables crops better to withstand conditions of draught. Since excess moisture is readily removed by drainage, crops withstand wet-weather conditions better. This is highly important at planting time since it tends to prevent the drowning of the sprouting seed. Soil erosion is reduced by drainage, since much of the surplus water passes out through the tile instead of over the surface. Although the need for commercial fertilizer is reduced on well-drained soils, their efficiency when used is increased by drainage, since they are permitted to sink more deeply into the soil and are less likely to be washed away in the surface or runoff water.

The improved drainage of the surrounding fields maintains the foundations as well as the surface of the farm roads and walks in better conditions for travel, thus reducing the expense of upkeep.

If all the benefits are derived from proper drainage, then there will be better crops produced on the farm at a reduced cost of operation. This will mean that the farm will return greater profits. Often, the increase in production for the first year pays for the cost of the tile installing. The increase in production for the years following is a clear gain.

The bettering of farm conditions as to sanitation, appearance, and crop yields means that the farm is more valuable ; hence there follows an incresse in property value.

3. Systems of Tile Drainags.

A main tile drain is one into which several lines of tile empty their water. The mains are usually of 6 inches tile or larger upto 12 inches.

The mains usually have outfalls into open drains. A submain is a short main which collects the water from a number of lines of tiles and empties into the main. A lateral is a line of tile which collects the water from the soil and empties it into the main or submain. In the best American practice, there are eight different systems in general use the natural, cutoff, Elkington, Herrigbone, Gridiran ; double main, grouping, and across-the-slope.

(1) **Natural System.** This system consists of placing the tile drains in various ways in order to drain certain wet spots in the field. It should be used only where are a few depressions which hold water, these remaining wet while the rest of the land drains well.

(2) **Cutoff system.** This system is used on seepy hillsides where the water come to the surface and makes the hillsides as well as the bottoms, wet and unproductive. Tiles placed along the hillsides intercept this seep water and cut it off thus preventing it from going into the bottom Fig. 1.

(3) **Elkington system.** This system was first used by Joseph Elkington of Warwickshire, England, in 1764, Fig. 1. It is practically the same as the cutoff system and is used for the same purpose, to intercept seepage water—but has in addition, wells dug at regular intervals to let the water come up from a lower stratum and enter the tile ; or in other words, wells are dug which allow the water to rise to the tile which drains it away. If the earth is not firm enough these wells can be curbed with lumber or masonry, or filled with gravel. The system is precticaIly applicable to bogs and spring. It is also used to a considerable extent in the draining of irrigated lands.

(4) **Herringbone system.** The herringbone system is used on a bottom having two sloping sides. The main objection to it is that it involes considerable drainage. The mains and the ends of the laterals entering the main drain in the same land. This is double drainage and not, of course, economical. Fig. 1 shows this system, and the distance between the two dotted lines indicates the extant of the double drainage.

(5) **Gridiron system.** This system is shown in Fig. 1. It will be seen that there is not so much duoble drainage here as there is in the Herringbone system. It is, therefore, to be preferred to that system wherever a preference is possible.

(6) **The double main system.** This system Fig. 1, is the same as placing two gridiron together. It should be used where it is desirable to have two lines of small tiles in place of one big one in the centre. Such a condition might be where there is a slough or boggy place in the centre. This system is not used so much as the others, as conditions which require it are not often met with.

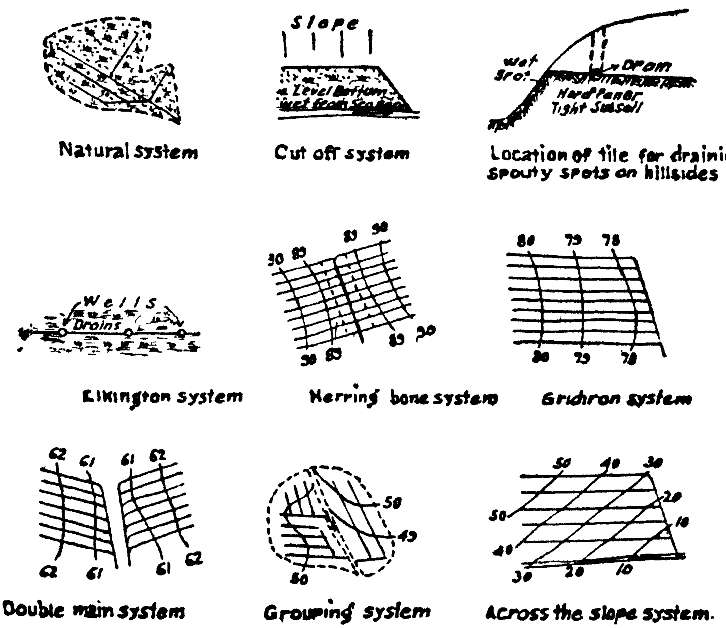


Fig. 1

(7) **Grouping system.** This system Fig. 1, consists of collecting the water from a number of small systems and discharging it into the ditch through one outlet.

(8) **Across-the-slope system.** This system Fig. 1, is also known as the Keythrope system It consists of running the lines of the tile across rather than down the slope, the idea being that often the sub-soil lies in such a way as not to be free from small valleys which allow the water to flow along the sub-soil directly down the slope. Then, if the tile were run across the slope, it would intercept this water and thus more readily drain it. This system has been used with good results where the sub-soil is in the condition named above. A soil-auger can be used to determine the lay of the sub-soil.

The following considerations govern the selection of a system or a combination of these :—

(*a*) The mains should be put along natural drainage lines. The reason for this is that water naturally runs that way.

(*b*) Mains and laterals should be, as for as possible, placed in straight lines. Where the direction of these is to be changed it should be done by long, easy curves. One reason for this is, that in changing the direction of flow of water, its velocity is lowered. The sediment carrying power of water depends on its velocity, so if muddy water is slowed up, it drops its sediment and thus fills up the tile, rendering it useless. It is often desirable to cut through small banks in order to keep the lines straight.

(*c*) Laterals should be given the greatest fall possible by running them down the slope, except in cases of intercepting seep water.

(*d*) Laterals should be in long lengths rather than short ones, in order to eliminate double drainage due to the mains and laterals draining the same land.

(*e*) A system should be selected and the drains so placed that all the land is equally well drained.

(*f*) The number of outlets used should be Kept to a minimum.

4. Effect of Tile Drains on Spring Level.

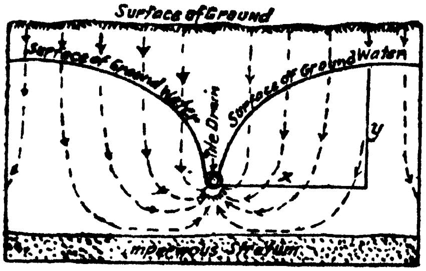


Fig. 2

If a drain is placed below the phreatic surface of the ground water, the surface is depressed towards the drain as shown in Fig. 2. The resulting surface curve is illustrated there in which also shows in dotted lines the general trend of path that water is behaved to flow just after the rain, the entire depth may be saturated and the stale air may be driven out of the pores of the soil. Fresh air will be drawn in, when the surface is dropped. Gross at the Engineering Experiment Station, Iowa State College, has proved in his researches that the surface curves for fine sand are hyperbolic and of the form given below where origin of co-ordinates is as the bottom of the drain.

$y = \frac{x}{a+bx}$; where x is abscissa ; y is ordinate ; a is constant for a specific curve ; b is constant for a spicific curve.

5. Spacing of Tile Drains and Their Depth.

Consider two lines of tile, both placed at a given depth D Fig. 3 and spaced a certain distance apart. The function of the tile is to collect and carry away surplus ground water. Rainfalling directly over the tile or at the point A will owing to gravity, work through the soil until it reaches the tile, through which it will flow away. The water which falls on the ground at some point B to one side of the tile, will not flow dıectly to the tile but owing to action of grarvity, will work down in a vertical direction shuc as along the line BB′. To reach the tile this water would have to move horizontally, but since it cannot flow in that direction by gravity alone, it must remain relatively stationary until more water "stacks up" above it. When this occurs to a sufficient extent the weight of the higher water will exert pressure and force the water in the vicinity or the line *xx* to move sidewise towards the drains along the paths of least resistance. Upon arrival it is forced upwards and enters the tile through the lower portion of the joints, or opening between individual tile, whence it flows away inside the tile by garvity. After the rain stops, flow will continue towards the drain at a constantly decreasing rate until finally all movements will cease when the height of the "stack water" is just sufficient to balance the resistance at any given point.

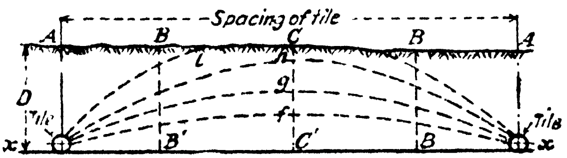


Fig. 3

Take the point C : water falling on the surface here works downword towards the line *xx* and then must move laterally towards the tile. Obviously more pressure is required to force water to the tile from the point C′ than from the nearer point B′. It follows that the height of the "stack water" must be correspondingly greater at C than at B. By considering each point in succession between the two drains it will be seen that there would be varying amounts of pressure needed to force the lower water in the tile, This pressure would vary form nearly zero at the tile to a maximum at a point about midway between the tile. The heights of "stack water" needed to produce this pressure would vary form a minimum at the tile to a maximum height at the midway point. A line joining the tops of successive "stacks" would then, form a curve which is known as the ground water curve or phreatic surface curve.

If the tile is placed in a sandy soil, the water can work through it easily ; consequently not much pressure is needed to force the water into the the tile, and the curve takes a flat shape, such as curve f. Should the soil be sandy loam, the pressure required would be greater, and it would take a shape such as curve g. In clay loam, the curve would be sharper and would take the the shape of h, while in a heavy clay it might come to the surface in the centre and take the shape of curve i.

From the foregoing discussion the following conclusions can be drawn regarding the proper spacing and depth of tile drains.

In an open soil, such as sandy the tile can be placed far apart, because the plane of saturation is a flat curve and will not come near the surface except in extreme distances.

In a tight soil, such as stiff clay, the tile should be placed close together, because the plane of saturation is a sharp curve.

C. G. Elliott gives the following guidance for spacing tile drains.

"In close, dense soils, largely clay, 30 to 40 feet ; coastal plain lands composed of mixed clays with fine sand and uniform structure, 60 feet ; alluvial gumbo or heavy soils but with granular structures, 70 to 80 feet, alluvial glacial drift and sandy loam soils, with clay sub-soils, 100 feet; sandy lands and soils containing considerable quantities of vegetable matter and those without sub-soils having a liberal supply of sandy or gravely material 150 to 200 feet." A thumb rule is known that a tile line will draw 1 foot on either side for each inch of depth.

The depth of tile laterals varies from 18" to 4 feet in sandy soil and in dense soil 3 to 4 feet. The mains and submains are usually 6′ to 8′ below natural surface. The depth is taken to be at the bottom of the tile.

6. Grade of Tile Drains.

The proper grade or fall for tile is the greatest obtainable. The idea that a tile can have too much fall is wrong. The minimum grade is absolutely flat. Tiles, where it was necessary, have been laid flat, and the land has been drained, but this is to be advocated only in extreme cases. To install a line of tiles with a flat grade requires much time and labour, as it must be absolutely level, for if a little downgrade is made in the tiles a corresponding upgrade must be made, and water will not run uphill without pressure. A fall of 1/10 foot per 1000 feet is as flat as tiles should, as a general rule, be placed, and it will be found difficult to maintain even this grade. A fall of 3/10 per 1000 feet is a nice grade to work to, and of course, for falls greater than this, the tile is correspondingly easier to install. These larger falls are not so desirable as it is necessary to go through considerable trouble (such as digging for grade) and expense to get them. By digging for grade is meant placing the tile shallow at the upper end and deeper at the lower end, thus getting the fall by deepening the ditch.

7. Flow inTile Drains.

The following factors govern the discharge of tile drains :—

(*a*) The slope on which the tiles are laid, or effective head ; this is sometimes spoken of as the grade or gradient.

(*b*) The resistance or "skin friction" developed by the water in sliding against the tile walls : this is expressed by an empirical co-efficient : and.

(*c*) The shape of a cross-section through the drain, which is a measure of its hydraulic efficiency and for drain tile, is always assumed to be circular. The effect of shape is expressed by the term hydraulic radius, which is the ratio of cross-sectional area to wetted perimeter. By wetted and perimeter is meant the length of the line of contact between the water and the channel through which it is flowing ; for tile flowing full, it is equal to a circumference.

The usual formula used for tile flow; $v = 138R^{2/3}\, S^{1/2}$.

where v is the mean velocity in feet per second : S the hydraulic grade or slope ; R the mean hydraulic radius ; $S = h/l$ or fall in ft. per foot expressed as a decimal fraction ; h the fall in feet between the points considered ; l the length in feet between the points considered ; $R = a/p$; a the area of tile opening or cross section in square feet ; p the wetted perimeter or circumference of the tile in feet.

When v is found, then substitute it in the following formula:

$Q = av$; whore Q is discharge of tile in cubic feet per second ; a the area of tile opening in square feet.

When Q is determined, then substitute in the following formula $A = Q/Z$ where

A is the area in acres drained by tile ; Z is the rate of run-off expressed as a decimal fraction.

8. Capacity and Size.

The amount of water to take off each 24 hour in drain tile depends more upon the rainfall than upon any other factor. The character of the soil, whether open or dense, as well as other factor of run-off, has considerably to do with it but not so much as rainfall. If only this one factor were taken into account the following statement, consistent with current practice, may be accepted as a rough guide ; $\frac{1}{4}$ to 5/16 inch for regions with annual rainfall less than 30 inches ; 5/16 to $\frac{3}{8}$ inch for regions with annual rainfall between 30 and 40 inches ; $\frac{3}{8}$ to $\frac{1}{2}$ inch for regions with annual rainfall between 40 and 50 inches; $\frac{1}{2}$ to 1 inch for areas with rainfall between 50 and 60 inches ; and one inch or more for areas where the average rainfall is greater than 60 inches. The size of laterals is never less than 4 inches and sometimes 5 inches when the lateral is more than 500 feet long. The submains and mains range upto 12 inches according to the laterals joining them.

The surface drainage is separately arranged in open drains. If inlets are provided into the tile drain to take up surface water, the co-efficients mentioned should be doubled to calculate the size of tiles.

9. Tile Selection.

Good drain tiles must be regular, dense, strong and sound. By a drain tile being regular is meant that it is round with ends clean cut. A tile which it oval or out of round and thus not true is hard to lay to grade in a ditch. A tile with ragged ends is difficult to lay up against the end of the tile next to it. Tiles which are not regular will cause unnecessary labour in installing and are apt to be unsound.

As a rule, tiles which absorb large quantities of water are weaker than those that absorb small quantities. The denser a tile is the lower absorption it will have and, hence, the stronger it will be.

The absorption test can be made by first drying the tile and weighing it then placing it in water for 24 hours and again weighing, and then computing the percent gain in weight. The usual allowable figures are as below :—

For farms tile made of concrete	
Allowable absorption percent — — —	8 to 11
For farm tile made of clay :	
Allowable absorption percent — — --	8 to 16
For large tile drains made of concrete :	
Allowable absorption percent — —	6·5 to 9
For large tile drains made of clay :	
Allowable absorption percent — — –	6 to 7

The idea has long been prevalent that a good tile should be capable of absorbing large quantities of water in order to ensure that the water gets into the tile. The fact is that very little water gets into the tile by soaking through the wells, even when the former is very porous. The vast quantity of the water enters the tile through the joints.

Farm drain tiles, upto 12 inches in diameter, should have sufficient thickness of wall to enable them to withstand a load of from 600 to 800 pounds per foot length. Generally, if small tiles will support the some what concentrated load of a heavy man without cracking, they are satisfactory.

The tiles should be sound, that is, free from cracks, chips soft spots, blisters, and all other imperfections Common tiles which are overburnt are likely to develop these defects and become brittle like glass. One weak tile, once in the ground, will endanger the long life of a whole system.

10. Concrete Versus Clay Tiles.

A well-burnt and properly made clay tile is the equal of a well-made and properly cured concrete one. They are both very durable and will give about the same degree of satisfaction. If a first class product of either variety is compared with an inferior product of the other the former should be selected. If there is reason to believe, however, that the soil water is strongly alkaline, it is well to have careful tests made of the water to determine the nature of the impurity and the quantity in which it is present. In such localities, the safest plan is to use only good clay tiles, as the alakli attacks the cement and gradually disintegrates concrete tiles.

11. Laying of Tile Drains.

(*a*) **Digging of the ditch.** There are two methods of digging trenches for tile—by the use of machines and digging with hand tools.

(*i*) **Machine Ditching.** There are two general types of machines in the market for digging ditches for tile – those having a revolving wheel or endless chain with attached buckets, and those which are various forms of plow ditchers. A diching machine must be capable of cutting true to grade, it must operate efficiently in any kind of soil and it must be able to work for long periods of time without breaking or otherwise getting out of order.

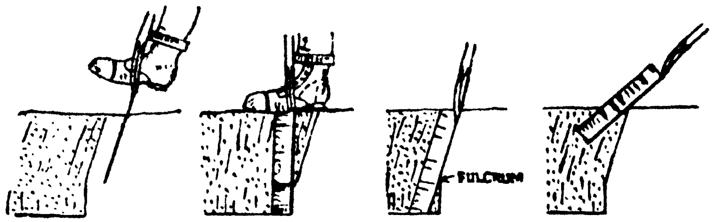


Fig. 4

(*ii*) **Hand ditching.** Hand ditching is a very common method, There is however, in this form of ditching, a certain knack that can be acquired only by experience as shown in Fig. 4. Special form of spades should be obtained and used for the work. These spades are called drain or tile spades and can be obtained with the spade part solid or open and in length of from 14 to 20 inches. The open or skelton spade seems to work best in wet, sticky ground as it allows the water to run out and reduces the friction. Spades may also be obtained with square or round points square for the top spading and round for the bottom layer. A trench for tiles of 4 to 6 inches diameter, which are to be laid at ordinary depths, should be started only 12 inches wide at the ground surface the object being to remove the smallest possible amount of dirt and save needless work. To a novice, this will seem rather narrow but with a little experience it will be found to be wide enough.

The loose earth in the bottom of the ditch should be taken out with a long-handled, round-pointed shovel.

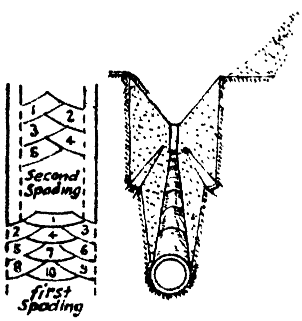


Fig. 5.

The last spading should not take the earth out lower than 2 or 3 inches above the grade line or bottom of the ditch Fig. 5. The man tacking out this last spading should be provided with a gauge 2 or 3 inches shorter that 5 feet (full length).

The last part of the ditch should be taken out with the ditch cleaner, or scoope, which shapes the bottom of the ditch so that the tile will fit Fig. 6. The man using the cleaner or scoop should stand on the unfinished part of the ditch and draw the cleaner towards him.

(*b*) **Checking grade.** It will be enough to check the correct levels of the bottom of the tile lines with a level at every 100 feet and then to correct levels in between by means of boning rods. Grade should also be checked for laying the tile line by spot levels.

(*c*) **Laying tile.** For laying small tile, say upto 8 inches, a tile hook should be used. This tile hook has a handle like that of a hoe and on one end it has a $\frac{3}{4}$ or $\frac{1}{2}$ inch rod inserted, which projects a short distance and then turns a right angle. Tile hooks can usually be purchased at the local hardware stores. By using an ell, they can be made from $\frac{3}{4}$ to $\frac{1}{2}$ inch water pipe. The use of this hook is very simple. The tile layer stands astride the ditch. He inserts the hook into a tile and lower it into the ditch. Then, by means of the hook, he turns the tile over and pushes it up close until it fits well against the next tile. The joints can be

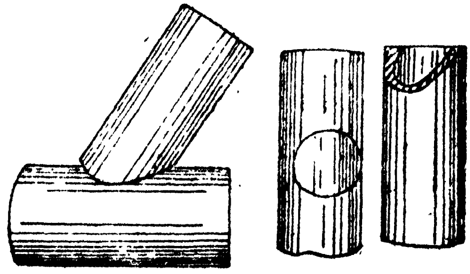


Fig. 6.

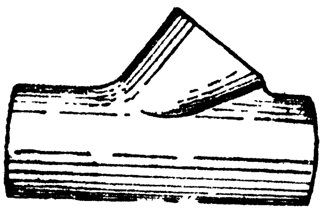


Fig 7

made by cutting tile as in Fig 6 or they may be factory made as shown in Fig. 7.

(*b*) **Refilling ditches (back filling).** The first earth should be carefully shaved off the sides of the trench and placed at the sides of the tiles. Then 3 or 4 inches should be place on top of the tiles with shovels. All the rest of the dirt can then be placed in the ditch in any way that seems best and cheapest. Often quicksand or tight soil which would heve a tendency to fill tile or pack so close around it as to hamper drainage, other materials are placed in the ditch on the top of the tile Straw, hay, cornobs, broken stone, cinders, coarse sand, and gravel have all been used with good success to serve as shrouding.

12. Outfall of Tile Drains.

There are five things against which the outlets or outfalls must be protected —caving bank, small animals, back water, tramping of stock, and in cold climates, damage from alternate freezing and thawing.

(*i*) **Caving banks.** Caving banks are caused by the water running out of the end of the tiles and washing away the bank underneath the tiles thus allowing the first joint of the tile to fall into the open ditch. The water repeats the same thing on the next piece of tile after the first tile has fallen, and this process goes on until the outlet has receded far into the field This can best be remedied by providing a suitable face wall as shown in Fig. 8.

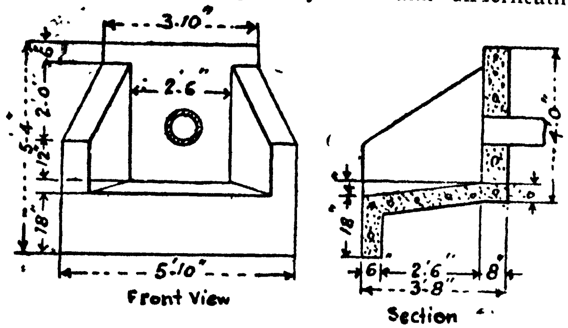


Fig 8.

(*ii*) **Animals trouble.** Small animals such as muskrats, rabbits etc., often seek habitation in the tile drains, especially during a dry time. They build their nests or get themselves held fast in the drains, causing the drains, to stop up. To protect against this trouble, iron gratings should be placed across the end of the out let of the tile to prevent the animals from gaining entrance. The gratings may either be fixed permanently in place of may be removable.

(*iii*) **Back water.** This protection is necessary only where a submerged outfall is used. Automatic outlets flap value is the best practice as shown in Fig. 9.

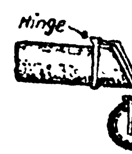


Fig. 9.

(*iv*) **Tremping.** Well located outfalls are seldom damaged by cattle tramping. The remedy is to keep it deep and to make a masonry or concrete face wall

(*v*) **Freezing and thawing.** In regions where the winters are severe, some thought needs to be given to the disintegration action of alternate freezing and thawing. The more porous a tile is the greater the damage is likely to be, because more water is held in the pores to be frozen. Ordinarily tile laterals and mains are buried below the frost line which seldom penetrates more deeply than 2 or 3 feet, but wherever they are exposed to drafts of air, as below surface in takes and for 20 to 25 feet back from outlets, hard burnt, vitrified tile should be used or some other non porous material.

13. Maintenance of Tile Drains.

The tile line is to be maintained so that its effective sectional area is not reduced firstly by silt or sediment deposits and secondly by growth of roots.

(*i*) **Cleaning-tile drains.**

(*a*) The usual procedure is to dig holes down to the tile 25 feet apart and the amount of sediment is removed at each hole as for as possible. Next a light stout string is threaded through the tile between two adjacent holes and a rope is attached to one end of the string and pulled through. A strong bag filled with straw and large enough to occupy about three-fourths of the area of the tile, is then securely fastened to one end of the rope. By pulling at the other end, sediment is gradually forced at out the opening towards which the bag is being drawn. Here it is bailed out. It is a good plan to tie another rope to the opposite end of the bag so that it can be drawn back and fore several times and thus clean the tile thoroughly. The only difficulty is to get the first string through. The most satisfactory way is to use specially constructed flexible sewer rods. These are usually in 4 ft. lengths and can be attached and detached accoring to requirement after inserting first piece in the drain with string tied to the front end.

(*b*) If large quantities of water are available under good pressure, the tile may be cleaned by flushing, but this is seldom possible.

(*c*) Common diaphram pump is used to remove the sediment if it can be disturbed and muddied with enough water is allow it to flow.

(*ii*) **Root growth in drain.** Root growth through the joints of the tiles takes place only when trees are within 50 to 75 feet of the tile line. Sometimes the growth of fibrous mass becomes so dense that it completely chokes the tile. This is usually in long snake like form and can sasily be pulled out by opening a tile 50 feet away.

It is said that soaking tile in a carbolineum solution before laying them or mixing rock silt in the blinding soil will prevent or discourage the entrance of roots. The only safe rule is to keep the tile outside the danger zone or else sacrifice the trees.

14. Surface Inlet into Tile Drain.

(*a*) There are three principal methods of building surface inlets with broken stones,

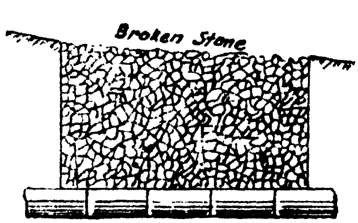


Fig. 10 (a)

Drain-Tile Accessories

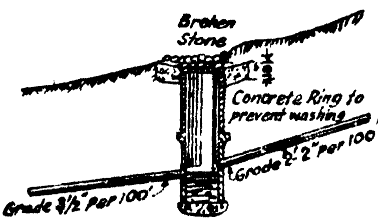


Fig. 10 (b)

with vitrified tile and with concrete. They are used to allow surface water directly into the tile drain without going through the soil. Broken stone inlet is the simplest, cheapest but least durable. Fig. 10 (a) Vitrified tile inlet is reasonable in cost and very durable Fig. 10 (b).

(*b*) **Silt basins.** Silt basins are where a considrable amount of silt is washed into the surface inlets, or where there is a change of grade sufficient to cause silt to be put down as shown in Fig. 11.

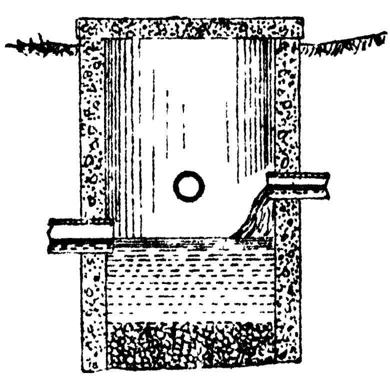


Fig. 11

15. Vertical Drainage.

It is the custom in many localities scattered over the United States to drain sink holes and other land having no gravity outlet into vertical wells which penetrate into a water-absorbing stratum some distance below the ground surface. This kind of drainage is more or less speculative in character, because it is impossible to secure exact and complete data concerning the underground formation into which surface water will be discharged. Sometimes the wells funcion perfectly for a few weeks or months and then cease to take any more water because there is no escape from the stratum into which the water is being drained. Where conditions are favourable, however, this method of drainages possesses considerable merit. In northen Iowa a number of wells exist which have given complete satisfaction for more than 30 years, and they make possible the drainage of acres of many excellent land which would otherwise remain unreclaimed. Most of these wells discharge into a stratum of pervious rocks, connection with which is maintained through an iron casing extending to within 4 or 5 feet of the surface.

Other such wells penetrate a stratum of lime stone, found at depths of 50 to 60 feet. and extend 20 feet farther, where a good outlet can usually be secured Fig. 12. One of these lime-stone wells receives the discharge from 2 to 12 inch tile drains and 320 acres of wet land.

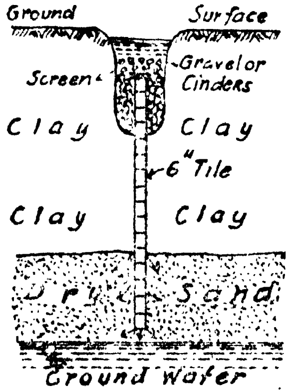
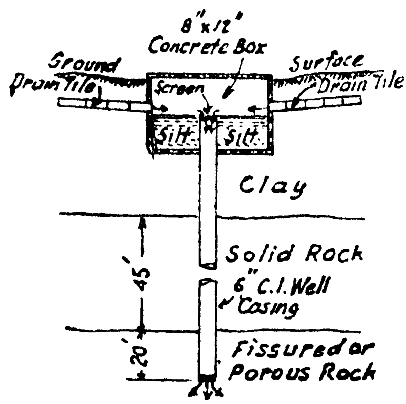


Fig. 12



Filg3.

Another well located within 12 miles of two successful ones, was drilled to a depth of 350 feet, and no limestone was encountered. This well was not a success Fig. 13.

16. Biblography.

1. Ohio State University, 6 Vol. XVII 1922.
2. King James. A Tile drainage, Mason City Buik and Tile Company, Mason City, Town, 1918.
3. Hayne R.A. "Drain the wet land" International Harvest Company Chicago, 1921.
4. Ayers and Scoats, "Land drainage and Reclamation" Megraw Book Company, New York 1939 2nd Edition.

PART IV

DRAINAGE ENGINEERING

CHAPTER IV

Soil Erosion

1. Introduction.

In Chapter III of this part on Surface Drainage an essential principle was to remove all obstructions resulting in quick disposal of the surplus rain water and to save the lands from water-logging. Near the foot of the hills, the surface slopes are generally very steep and in the mean annual rainfall is relatively very high. The Drainage across the countryside and in the natural drains (*gullies*) results in soil erosion. The valuable soil with fertilising properties is washed away. The process is very much accelerated if the deforestation be not checked and the grazing is not controlled. The lands in course of time become absolutely barren. This chapter deals with the measures necessary to control the drainage so that it does not result in Soil Erosion. This is a very live problem in the Punjab. While the hillsides in Hoshiarpur, Gujrat, Cambellpur and Gurgaon districts are losing the fertile soil the lands near the foot of the hills are being surcharged with deposits of sand. The lands are deteriorating in both cases and in some cases they have become barren.

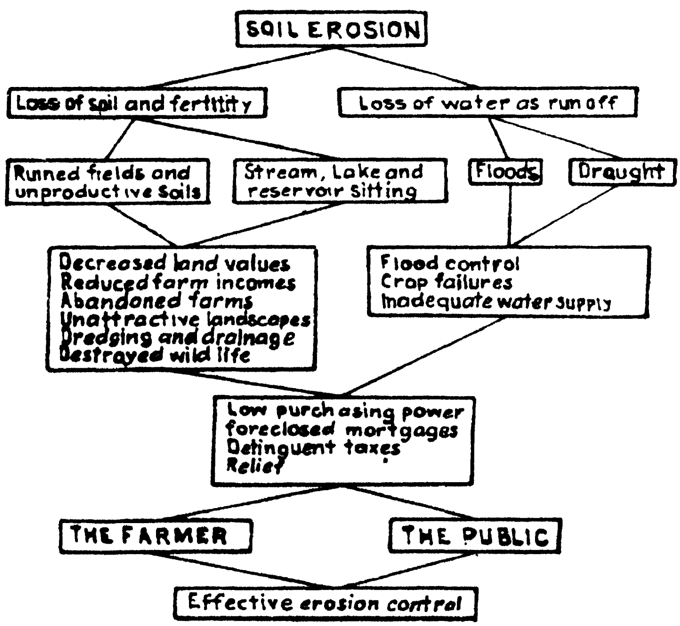


Fig. 1

2. Effective of Soil Erosion.

A diagramatic representation of the effect of soil erosion as devised by L. D. Baver is shown in Fig. 1.

3. Soil Washing.

Water erosion is generally considered to manifest itself in two forms :—(1) sheet washing (2) *gullying*. Usually these two forms represent different stages in the same process ; and *gullies* as a rule, do not appear until sheet erosion has been under way for a considerable time. *Gullies* however, sometimes occur without being preceded by sheet erosion ; and conversely, sheet erosion has been known to continue indefinitely without the formation of *gullies*.

Sheet erosion is a rather uniform skimming off the cream of the top soil with every heavy rain. The process is insidious because it may or may not leave any visible trace of damage and may continue for years under the eyes of an owner who does not realize his loss and cannot understand why the productivity of his land is rapidly decreasing.

Gully **Erosion.**

Once the water begins flowing in definite channels or *gullies*, the eroding power of a given volume on a given slope is greately increased, in accordance with certain well-known laws of hydraulics On fields where erosion has been allowed to proceed unchecked long enough for this to happen, not only are losses accelerated but the expense and difficulty of reclamation and control are greatly increased.

There are two general types of *gullying*. Ditch erosion where the head, and sides of the *gully* are usually sloping and erosion occurs at the head, sides and bottom of the *gully* in varying degrees by the action of water, freezing and thawing. Waterfall erosion is caused by water falling over the edge of a *gully* or ditch bank and is often responsible for many of the deepest *gullies*. The falling water undermines the edge of the bank, which caves in, and the waterfalls moves upstream The undermining action of the waterfall goes on rapidly if the sub-soil is of an easily erodable type. Both types of *gully* erosion are often found in the same *gully*. Lateral *gullies* tend to from the main *gullies*, and ultimately a network of *gullies* develops.

4. Methods of Controlling Soil Erosion.

The usual methods may be summarised as below :—

(*a*) Reducing the slope by terracing.

(*b*) Reducing the erodability of the soil by plant growth.

(*c*) Reducing the runoff by damming or *bunding* and using storage water for other proposes such as irrigation or water supply.

(*d*) Reducing the volume of sediments in water by constructing silting basins and using the deposits to reclaim eroded land.

(*e*) Reducing run off by increasing absorption.

(*f*) Reducing run off by *Wat bunds* (*bunds* around fields).

5. Terracing.

(*a*) Modern terraces are nothing more than a series of broad flat fields built with a slight longitudinal grade to carry the excess water of the field at low velocity. In fact each terrace acts as a shallow diversion ditch that intercepts the run off from its individual catchment area and leads it away slowly before the water has a chance to attain harmful velocity or volume.

On cultivated lands subject to erosion, terracing is the most successful control measure for widespread adoption and this has been throughly tested and found acceptable under actual farm conditions.

As developed to suit modern field cultivation practices, terraces can and do save rich topsoil in the magnitude order of 30 to 40 tons per acre per year when fields are in clean-tilled row crops and half as much when in small grain.

The disadvantages of terraces are; of course, that they are somewhat costly to build and troublesome to maintain ; they may require adrupt changes in traditional farming practice, damage may be caused by diversion and concentration water at unnatural exits.

Terraces emphatically are not a cure-all and are dependent upon the mutual support of good farming and croping practices to produce best results.

(*b*) **Terrace types.**

(*a*) Bench type, (*b*) Broad base ridge type just like one shown in Fig 2.

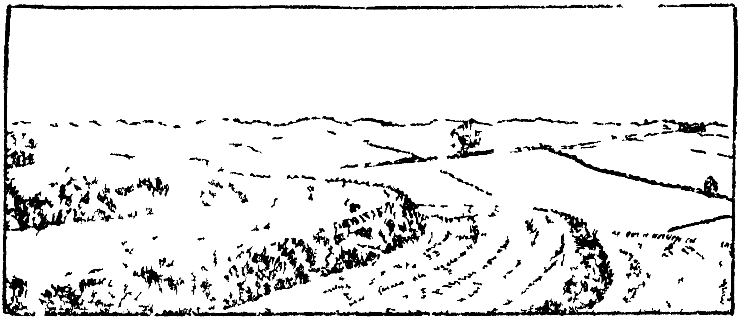


Fig 2

The broad base terraces are divided into two categories according to the purpose for which they are used

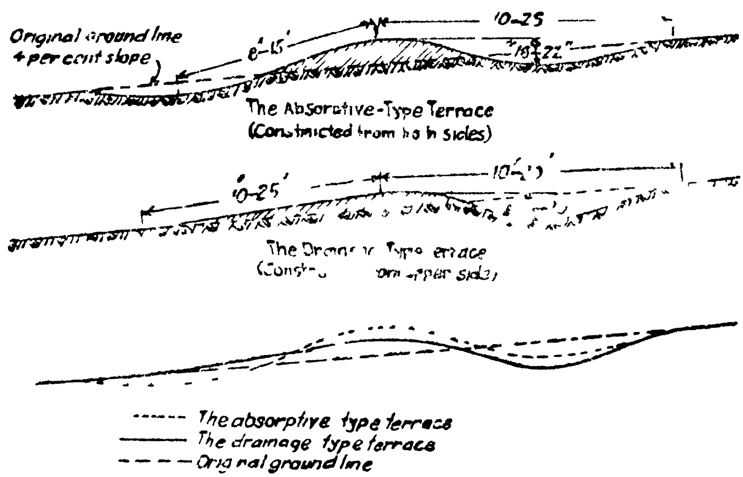


Fig 3

(*i*) Drainage or Channel Type (*ii*) Absorption or ridge type.

The typical cross sections of these are shown in Fig 3

(*c*) **Terrace spacing.**

Since the allowable spacing is governed mainly by the need for preventing appreciable erosion between the terraces it will vary with the land slope for given soil and cultural conditions For the sake of economy in construction and minimum interference with cultivation, it is desirable to space the terraces as widely as the soil and slope will permit without requiring too much maintenance at a time when the land is in the least resistant crop Wide spacing calls for more capacity in the terrace channel to serve the larger watershed. Good cultural practices permit maximum spacing and by increasing the time of concentration, reduce to a minimum the critical rates of rainfall and the run-off co-efficient

In the best American practice trial spacings are usually expressed in feet of vertical drop from one terrace to the next as a function of the land slope For instance, the vertical interval (or drop) in feet may roughly be expressed as 2, plus the land slope in percent divided by 4, and for Northern States 2 plus the land slope in percent divided by 3. Thus, if the terrace system is located in Mississippi on a slope of 8 percent the vertical interval would be 4 feet.

(*d*) **Terrace grades.**

Successful terraces have been built with a variety of gradients (fall along the terrace water channel) as follows :—Level end to end ; graded uniformly from one end to the other or from a point near the middle toward each end ; with a grade varying from a minimum in the upper portion to a maximum at the lower end.

It is desirable, within practical limitations, to keep terrace grades as slight as possible and still impart sufficient velocity to the water to prevent overtopping of ponding.

(*e*) **Cross section of terrace chrnnel.**

Typical cross sections for the water collecting channel in terracing are shown in Fig. 4.

6. Vegetation or Plant Growth Affecting Soil Erosion.

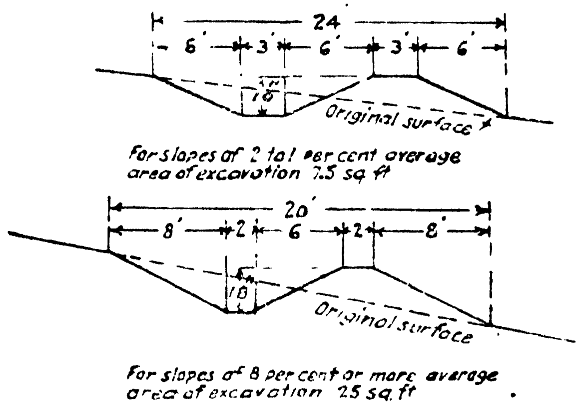


Fig. 4.

(*a*) The soil erosion primarily depends on the nature of the soil. The clay or *pubby* soils can very much resist soil erosion, while the loam and the sandy soil are easily erodable. The protection provided by vegetation serves to reduce erosion by holding water thus reducing flow and also by ensuring skin protection against sheetflow

(*b*) Suitable rotation of crops.

In normal operations of the crops, the land has to remain ploughed for some time before a crop is sown and it is also required to be vacant for rest before the next crop is sown. The cropping programme should be arranged in such a way that land is without the plants protection for the minimum period . In Northern India, the problem is very simple becaus the rainfall is concentrated in the monsoon season, where all land could be under the crop and could be allowed to be vacant in the dry months.

(*c*) **Contour farming.**

The plant protection also varies according to the methods of Farming. Contour farming is supposed to yield the best results. When row crops are planted crosswise of a slope rather than running up and down, the rows act as miniature terraces and tend to hold rain water where if falls, thus increasing absorption and reducing run-off. If the rows are run up and down the slope, as is constomary in many localities, each plow or cultivated furrow serves as a channel for rapid flow and results in inevitable *gully* cutting.

(*d*) **Strip cropping.**

The methd of Strip Cropping also tends to increase the protection against soil erosion provided by agricultural plants growth. By strip cropping is meant the practice of growing intertilled crops in alternation bands or strips with close-growing hay or small grain running crosswise of the slope and laid out to follow the contour of the land as nearly as possible. While not entirely new in some localities, practically no experimental, data have yet been collected.

The practice is best suited to gently rolling land with uniform slopes but may occasionally be used on slopes quite steep. Usually strips of densely growing or fibrous-rooted crops, such as oats, wheat, barley, sorghums, alfalfa, clover or grasses, are planted between strips or cleantilled crops, such as corn, cotton, tobacoo, soybeens, etc The wider and closer together the strips of non-erosive crops are placed the more effiective they are in checking erosion from the entire field, the object being, of course, to slow down the run-off velocity, filter out the soil and increase absorption.

(*e*) **Different crops.**

The soil erosion also varies according to the different kinds of crops sown. The reduction in both the soil erosion and run-off brought about by the various crops and on different soils is apparent from the comparison of some of data table below (U. S. Year Book of Agriculture 1935, page 302).

TABLE :--Comparison of soil and Water Losses by Surface Run-off at Several of the Soil-Erosien Experiment Stations.

Area-soil type inches rainfall.	Prot treatment,	Soi! loss, tons percent.	Loss of rainfall (run-off) percent.
Upper Mississippi Valley, Le-Crosse, Wis. Clintontsilt loam. 16% slope. (1933 only) rain, 29 11.	Bare soil, uncultivated	51.5	15·9
	Continuous corn	59·9	19·2
	Continuous barley	12 0	17 8
	Continuous bluegrass	0·003	2 9
Mo.: Bethany, Mo. Shelby silt loam. 8% slope (Av. 3yrs. 1931–1933.) Av. annual rain 33 53.	Bare	112 48	22 98
	Continuous corn	61·16	28 38
	Continuous bluegrass, timothy	... 0·36	... 7·72
	Continuous alfalfa	0·22	3·40
Red Plains; Guthrie, Okla. Vernon fine sandy loam, 7·7% slope (Av. 4 years 1930–1933.) Av. annual rain 32 92	Bare	14 59	26·04
	Continuous cotton	28·06	14·18
	Bermuda grass	0·04	1·51
Tex ; Ark.; La. sandy land region; Tyler, Tex. Kirvin fine sandy loam. 8 72 % slope. (Av. 3yrs. 1931–1933) Av. annual rain 42·31.	Bare	12·20	18·20
	Continuous cotton	19·06	18·00
	Bermuda grass	0 20	1·50
Central Piedmont Statesvilla N C. Cecil sandy clay loam. 10% slope. (Av. 3 yrs 1931–1933) Av. annnal rain 42·9.	Bare	65 3	32 0
	Continuous cotton	14·0	9 7
	Continuous grass	0·8	5·2

Note:—All plots 72 6 feet long, 6 feet wide—1/100 acre.

7. Dams to Control Soil Erosion.

(A) *Sailab Bunds.*

The practice of constructing *sailab bunds* against torrents to produce sufficient moisture in the soil for growing *Rabi* Crops is pretty old in the Punjab. Advantages of *Sailab bunds* are summarised below :—

(*i*) The sand coming down from the torrents is prevented from spoiling cultivated land away from the hills.

(*ii*) The broken ground near the foot of the hills is levelled and reclaimed for cultivation.

(*iii*) The spring level of wells near the *bund* is raised thus facilitating the working of the wells.

(*iv*) The water of brackish wells in the vicinity of *bunds* is made sweet by percolation of water stored by the *bunds*.

(*v*) The land downstream of *bund* receives moisture by percolation enabling *zamindars* to grow *jhil* crops with out any waterings.

A *sailab bunds* is built at a site where spring level is low and *sailab* irrigation would be welcomed. The alignment of the *bund* is on a contour where water would spread on as large an area as possible. The *bunds* have an upstream slopes on 2 : 1 and outside slope sufficient to give an hydraulic gradient of 5:1. The *bunds* having 5:1 or flatter side slopes are most economical in maintenance. The weakest place in a *bund* is the site of crossing the torrent. Where the torrent bed is deep and narrow, sand or shingle must be removed to a depth of at least 5′ before the *bund* is built and the upstream slope should be about 10:1.

The regulator is a small opening in the *bund* about 5′ × 5′ in size near the torrent bed with a cill about a foot above natural surface to allow for silting. This remains close during floods and is opened on about 10th September to drain off water still standing against the *bund* so that land may dry up for *Rabi* ploughings. It has been found by experience that a coefficient of creep of 1:15 in bed and 1:10 round wing walls it the minimum required.

The type of water weir generally adopted is given in Fig. 5. This is able to pass only 1′ depth of water over the cill and is the cheapest form of construction. The discharge for which the waste weir is designed, is calculated on figures of run-off given in Appendix 8 of

Strange's book. These discharges are meant for storage reservoirs. In the case of *sailab bunds*, a co-efficient is applied which may vary from 9 percent to 18 percent according to the nature of catchment, and is the same as given in Appendix 2 of Strange's book used to give the run-off and yield.

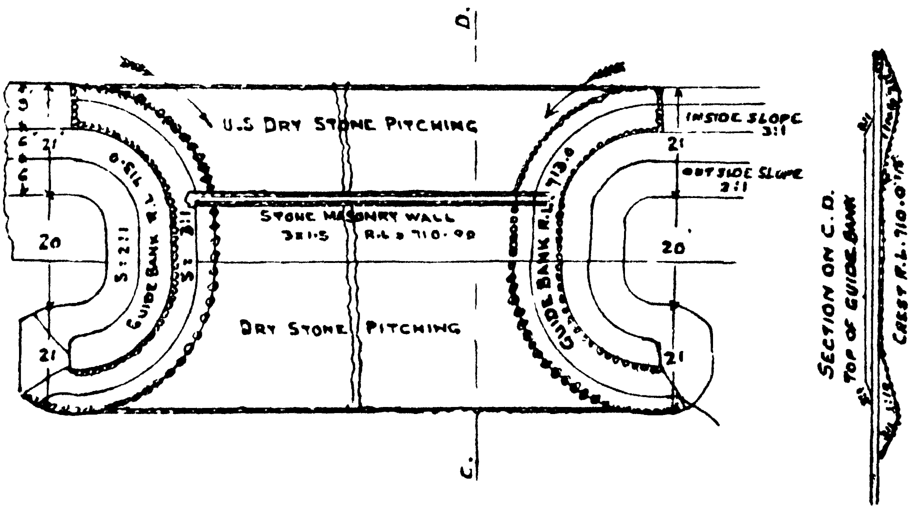


Fig. 5 Plan of waste weir on Bhond Scale 1/200

An example below gives the method of cultivation :—

Catchment area 2·56 sq. miles.

Rainfall during monsoon 23″

Average run-off assumed 12·3% vide Appendix 2 of Strange's book "Indian Storage Reservoirs with Earthen Dams."

Yield of the torrent is thus 17 million cft.

(Refers to capacity and area curve)

17 m. cft. will fill the reservoir up to R L. 710·0 and this is taken as the Full supply level.

Top of *bund* is 713·0.

(*a*) **Waste weir.**

Assume intensity of run-off at 2 65″ per hour (See Appendix 8 of Strange's book).

As this is a *sailab* bund and not a Storage reservoir, apply a further co-efficient of 13·8%. Appendix 2 of Strange's book (1).

Flood discharge to be expected :—

$$\frac{2{\cdot}56\times5280\times5280}{60\times60}\times\frac{2{\cdot}65}{12}\times\frac{13{\cdot}8}{100}=604 \text{ cusecs.}$$

Discharge of waste weir is calculated by the formula 2·8 L. H. 3/2. Keeping height of water above cill=1′, length of waste weir=604/28=21·6′.

(*b*) **Check dams.**

Erosive velocities are reduced by constructing a series of checks which, in time, transform the longitudinal gradient of the *gully* from a uniform, steep slope to a succession of "steps" with low rivers and long flat treads. These checks are usually of a temporary character and serve to hold the fill and prevent washing while vegetation for permanent control is becoming established. If the intension is to fill the *gully* completely, additional checks

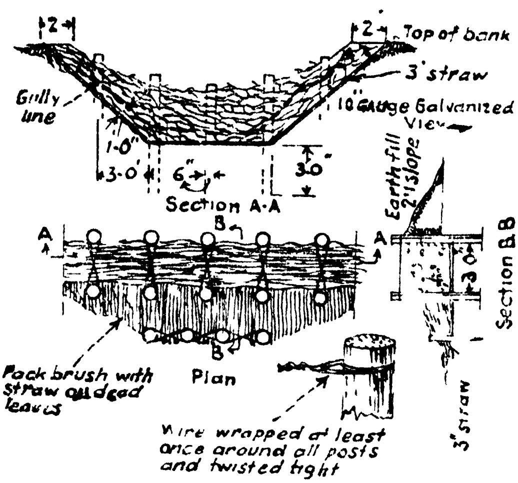


Fig 6

are later built midway between the original structures, and this process is, repeated as many times as may be necessary

When the fill is complete, future drainage must be diverted by terraces, or else the fill must be left under a cover of undisturbed sod to prevent a recurrence of scour. Substantial checks creosoted planks, masonry, or concrete are built where it is necessary to rely upon these alone for permanent or semi-permanent control Earth dams with drop inlet culverts are generally resorted to when both the *gully* to be reclaimed and its watershed are large.

(c) **Temporary dams.**

(i) **Brush dams.**

Although brush dams are the least permanent of all types, are tedious to build, and are very difficult to render and keep even partially water-tight, they are nevertheless recommended because of the nominal cash outlay required and because they can be made to function by giving almost constant attention to maintenance. Fig 6 illustrates a usual type of Brushwood dam

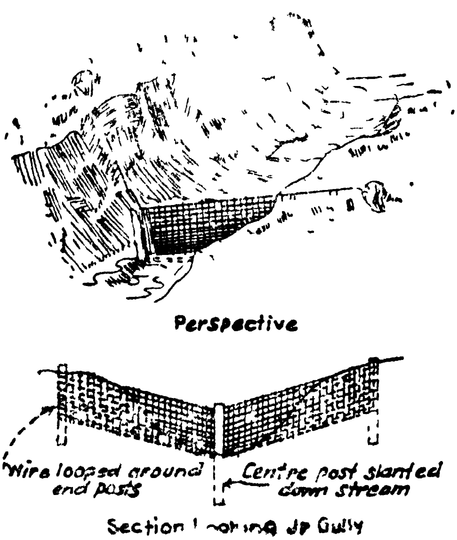


Fig. 7

(ii) **Woven wire dams.**

The V-type check dam shown in Fig 7 is adapted only to *gullies* of V-shape of those with very narrow bottoms and small watersheds The main construction features are the deepest centre post leaning downstrsam and bank trenches deep enough to bury all the wire

(iii) **Log dams.**

Where timber is plentiful or where a large number of logs 4" or more in diameter have accumulated as remnants in the construction of brush dams, or where, as in parks, the aesthetic value of rustic structure is important, log dams fit in very nicely On the other hand, they entail a wasteful use of both materials and labour and are suited only to the special conditions. Fig 8 shows a log dam

(d) **Permanent dams.**

They may be earthen, of masonry or of concrete They have been fully described in Part III of this book

8. Detention Basins.

As sand comes down by the denudation of hills the remedy must be found by constructing works in the hills by selecting favourable sites for storage of water and help percolation into the soil. This suggests that reservoirs or detention basins should be built in the various branches of the *choes* in the hills which should hold water sufficiently long to allow as much of it as possible to percolate into the soil, but at the same time the reservoir should empty quickly enough to recieve the water of the next rainfall. The torrent above the reservoir will assume a flatter slope and so diminish the denudation of the hills. The reduced discharge below the dam will mean less velocity in the main torrent and so less erosion of the bed and sides of the torrent. It is also clear that in order to gain appreciable results, the dams should be built in torrents of small catchment areas. Keeping all these points in view, branch torrents having catchment areas of about ½ sq. mile should be tackled by constructing rockfill dams (Loose Rock Dams). No waste weirs are necessary in this case as water can pass through and below the dams.

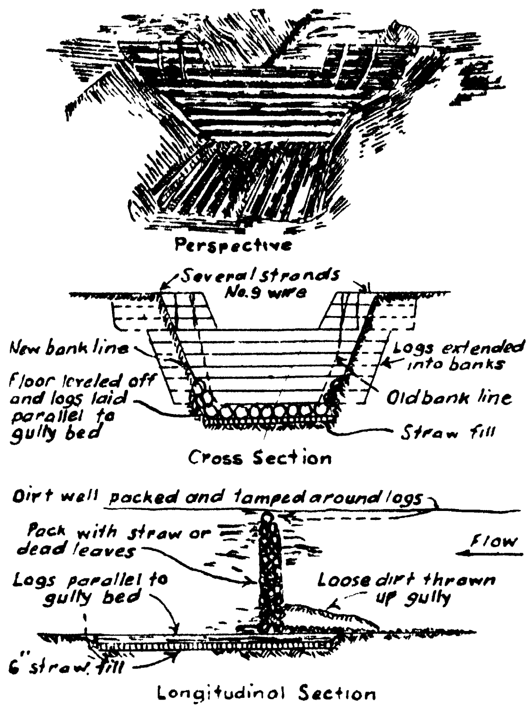


Fig. 8

9. Reducing Run-off by Increasing Absorption.

The water stored or checked will lose a large portion of it into the sub-soil. The greater the depth of storage the greater will be the loss of water into the sub-soil. The vertical drainage can also be arranged to increase the loss into the subsoil as described in paragraph 15 of the last chapter.

10. *Wat bundi.*

The construction of *wats* or small *bunds* at boundry of every field in a natural depression line which receives no water from hills has been tried to a large extent in Cambellpore district and found to be efficacious in holding the rain falling in a field itself. The *wats* are only a foot high. In places where slops are very steep, fields are stepped to hold the rain water and also to avoid ravining. By this means moisture can be retained in the field and probably this helps in maintaining the spring level. This method of holding water in fields needs to be encouraged at every place where spring level is low or falling down. It can only be useful if every *zamindar* has *wats* to his field. The efficacy of this method is now being realised in Jullundur district where spring level is falling at the rate of 1′ per year and it would be very interesting to watch its effects on the spring level of wells and on the discharge of torrents. The immediate effect of the system is that as the fields retain moisture, the draw off from the well becomes less.

PART V
GROUND WATER ENGINEERING
CHAPTER I
Soil Physics

1. Introduction.

Scientists and Agricultural Engineers have studied soil and its properties at great length. The subject has become so voluminous that its acount in the small space of this Chapter is essentially elementary and it is meant only to introduce the student to this vast subject. The student should refer to standard books on the subject such as "Physical Properties of the soil" by Dr. B.A. Keen (1931) and the publications of other eminent soil physiolists *viz.*, Fisher, Haine, Gardener, Buckingham etc.

Agricultural Engineers have always been is close touch with the subject and its development in connection with soil fertility and water requirements of crops, but it is only since recent years that the Civil Engineer has been faced with the necessity of knowing this subject rather in detail in order to deal with the problems connected with soil stabilization and the movement of moisture through soil resulting in water-logging and the rise of salts to the ground surface.

2. Soil and its Texture.

(*a*) Soil is a mixture of irregularly shaped particles of colloids, clay, silt and sand. The particles in the soil curst do neither maintain the shape not the size of the theoretical spherical particles of clay. They are often compound particles. The soil crust is essentially porous. The pore space is usually 35 to 44 percent. The average figure of pore space for the Punjab soils can safely be taken as 40 percent by volume.

The earlier investigators were misled by the apparent simplicity of the problem : the soil being porous, it seemed reasonable to regard it as a bundle of capillary tubes ; the simple concepts of surface tension and capillarity could then be applied to explain soil moisture distribution with, naturally, some small modifications on acount of the non-uniform nature of the material. What may, therefore, be called the "capillary tube" hypothesis, found an early place in the literature. It will still be found in a majority of present day text books of agriculture. Its survival is partly due to the simplicity of the hypothesis, and partly because most research warkers have endeavoured to explain anomalies by elaborating the original hypothesis instead of questioning the hypothesis itself.

In modern practice, the hypothesis of irregular thread-like capillary tubes has been abandoned ; the pore space is now pictured as an assemlage of small cellular units, communicating with each other through narrow necks. The equilibrium distribution of moisture with in such an assemblage is worked out on the principle that the water tends to reduce its free surface and hence its surface energy, to a minimum A pioneering investigation was made by Versluys which, in spite of some obscurity of expression is essentially sound. Later, the subject *wats*, clarified by important contributions by Haine and Keen at Rothamated, who considered in detail the effects of suction pressure, *i.e.*, the pressure deficiency necessary to draw an air-water interface into the cellular pores of the soil.

(*b*) Keen's description of the cellular pores in the soil is given below :—

"The irregular shapes and sizes of actual soil particles necessitate recourse to some regular and uniform material for purposes of theoretical development. We, therefore, consider the so-called "ideal" soil, consisting of spheres all of the same radius, packed together in a systemetic manner.

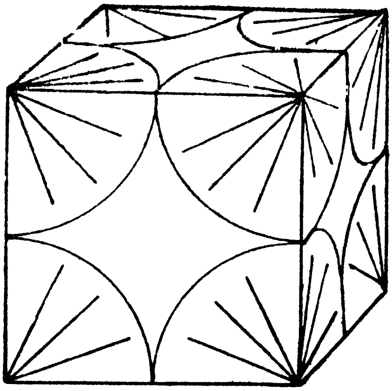


Fig. 1.

The most open system of packing is the cubical ; the spheres are arranged in columnar form, the lines joining their centres form cubes and each sphere touches six others. The unis cell which, repeated indefinitely, will reproduce this assemblage is shown in Fig. 1. It is a cube of side 2r, where r is the radius of the spheres. The eight portions of the spheres centred on each corner of the cube are equal in total volume to one complete sphere. The pore space in the cell is $(8r^3 - 4/5r^3)/8r^2$ or 47·64 percent, and is evidently independent of radius of the packed spheres.

The closest system of packing in given by the familiar pyramidal piles of cannon balls ; the unit cubical cell of Fig. 1 becomes a rhombohedron of sides 2r and face angles of 60° and 120° (Fig. 2). The eight segments of the spheres again equal one complete sphere in volume, the pore space is 25·95 percent, and as before, is independent of the diameter of the spheres used. This system is used in preference to the open packing, as actual soils approximate to it both in pore space and particle arrangement."

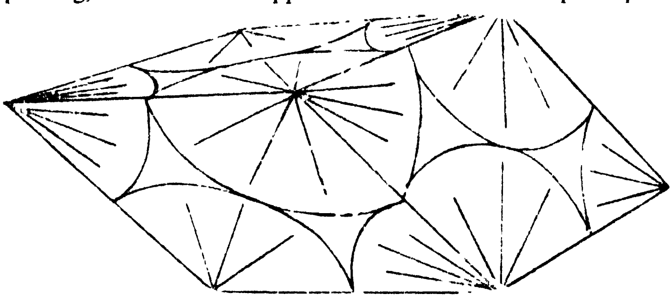


Fig. 2. Unit element of spheres in closest packing (Slichter) from the " 19th Annual Report of the U.S. Geological Survey".

The cellular nature of the pore space. Slichter investigated the pore space geometry in a system of spheres in connection with a theoretical survey of water flow in porous rock. This was supplemented by King's comprehensive experimental work. There joint work is one of the classical contributions to soil physics. As they were concerned with underground water flow, the pore spaces were assumed to be completely full of water. The dimension and shape of the "capillary tube" was accurately defined, so that the flow in various conditions could be calcuated with precision. Slichter regarded the tubes as having a curved apex following the general contour of spheres. They are triangular in section and pass alternately and regularly through maxima and minima of cross-sectional area. This general concept is illustrated by Figs. 2 and 3 ; the latter representing a plaster cast of the actual pore space. Slichter's analysis shows that the radii of the inscribed circles at the narrowest and widest part of the triangular pore are respectively 0·155r and 0·288r, where r is the radius of the spheres.

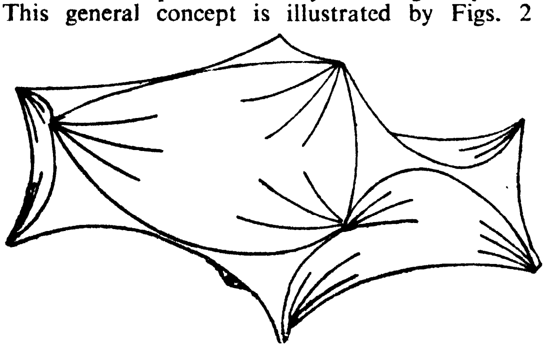


Fig. 3. Shape of pore space in unit element of spheres in closest packing (Slichter). (From the "19th Annual Report of the U.S. Geological Survey")

The idea of triangular capillary tubes which is satisfactory for flow problems in a completely saturated pore space, is not suitable in considering water movement in surface soil or subsoil, for here air is always (and necessarily) present. The manner in which air-water interfaces adjust themselves within the pore space can only be ascertained if its essentially cellular nature is clearly visulalised. Haines has shown that there are two types of cells. one of tetrahedral form and the other rhomboidal. The former lies at the centre of a group of four spheres packed in the form of an equilateral pyramid, and can be visualised without difficulty. The rhomboidal cell is enclosed by a group of six spheres.

(*c*) Soil is definitely a hetrogeneous mixtrue of particles of clay soil, silt and sand. A soil crust can never be reproduced in a laboratory, The soil constants determined in laboratory using, sand or "Ideal sand" cannot be applied to a natural soil crust. Even if the same soil be used for the laboratory tests, the natural stratification and texture can never be reproduced.

3. Soil Classification.

The methods of soil sampling and analysis are described in the last paragraph of this chapter. The American practice of soil classification is as given below.

Colloids below 0·002mm ; Clay 0·002 to 0·005 mm ; Silt 0·005 to 0·15 mm. Vary fine sand 0 05 to 0·1 ; Find sand 0·1 to 0·25 mm ; Sand 0·25 to 0·5 mm ; Coarse sand 0 5 to 1·0 mm ; Fine gravel 1 0 to 2·0 mm.

According to the practice of the Irrigation Research Institute, Lahore, clay is defined to be the stuff below 0·002 mm. diameter.

4. Physical Properties of Soil.

The physical properties are usually investigated under the following heads :—

(1) Apparent and real specific gravity. (2) Cohesion and plasticity. (3) Shrinkage on drying. (4) Moisture-holding capacity. (5) Rate of evaporation of water from soil. (6) Rate of uptake of water-vapour. (7) Heat of wetting. (8) Specific heat and thermal conductivity. (9) Rate of absorption of heat radiation. (10) Electrial conductivity. (11) Absorption of oxygen.

5. Soil Water.

Briggs suggested that the whole range of moisture content from dryness to saturation could be divided into three regimes which, in ascending order, were :—**Hygroscopic water,** the thin film of condensed vapour deposited on soil when in the presence of water vapour, held with considerable force by the soil, incapable of movement, and unavailable for plants ; **Capiliary water,** available for plants and capable of movement over the soil particles under the influence of surface tension, whenever the equilibrium was disturbed by evaporation at the soil surface, or by absorption by plant roots ; **Gravitational water,** which drains away under the action of gravity and is thus not directly used by plants. The term Gravitational water used here is a measure of the additions made to the water table as grouud water. Soil physics deals with the water available in the soil above the spring level or the Phreatic surface of the ground water reservoir.

There are three hypothesis which are usually set forth to account for the presence of the soil water above the spring level. These are detailed below :—

(A) Capillary tube hypothesis.

It is used to specify the following main aspects of soil moisture behaviour ;

(*a*) The capillary rise of moisture from a water table ;

(*b*) The rate of movement of water through soil.

(*a*) **Capillary rise.** The well-known simple formula for the height to which water ascends in a vertical capillary tube which is watted by water and whose lower end is in water. is ; $h=2Tg\rho r$ (A)

where h is the height of meniscus above water level ; T the surface tension between water and air ; g the acceleration of gravity ; ρ the density of the liquid ; r the radius of the tube.

Substituting this numerical values (C.G.S. system) in equation A, we have, approximately, for the height of rise in centimeters.

$$h=0\ 15/r \qquad \text{(B)}$$

Applying this formula to rise of water in soil, r is regarded as the equivalent radius of capillary tubes made up of elements of the pore volume. The minute size of the average soil pores implies very large values of h, and certain investigators have held the view that soil moisture will rise a considerable distance by capillarity. McGee from his work in the Great Plains of America thought that water could rise at least 10 ft. in a year, and 30 to 35 ft. in a favourable term of years. The behaviour of herbage on the Chalk Downs during a drought led Hall to suggest that soil moisture had risen some 200 ft.

(*b*) **Movement of water on the capillary tube hppothesis.**

The classical Poiseuille equation for flow through a cylindr.cal tube is $v = \frac{\pi gh \rho rt}{8\eta l}$ (C)

where v is the volume of fluid passing in time t, g the gravitational constant, h the pressure head, ρ the density of fluid, l the length of tube ; η the viscosity of fluid, r the radius of tube.

This hypothesis failed because the properties of a non-uniform tube in the soil could not be clearly defined There was vast disagrement between the actual values of capillary height and those worked out according to this formula, as shown by the research works of Green, Hamp and Hardy.

(B) Analogy with flow of Heat or Electricity.

The fundamental principle on which the theory of heat conduction is based is expressed by the equation ; $H_r = -K\delta T/\delta x$ (D)

where H_r is the quantity of heat passing in unit time through unit cross-sectional area,

K the conductivity of the material,

$\delta T/\delta x$ is the temperature gradient at x.

This equation enables us to measure the amount of heat passing a given cross-sectional area, if we know the conductivity of the material and the temperature gradient at that place. Similary, the strength of an electric current in a weir is obtained from a knowledge of the electrical conductivity of the material and a measurement of the potential difference between the ends of a wire.

By analogy, the quantity of water flowing through a soil should depend on capillary couductivity (the facility with which water flows through it) and on the gradient of the capillary potential (the gradient of the attraction of the soil water at any given point). The equation is therefore :— $Q_r = -\lambda \frac{\delta \phi}{\delta x}$ (E)

where Q_r is the mass of water passing in unit time through unit cross-section prependicular to direction of flow, and at distance x from the source,

λ is the capillary conductivity, or transmission constant, and

$\frac{\delta \phi}{\delta x}$ is the capillary potential.

The analogy, however, is only formal ; in heat and electrical problems, the conductivity and potential are practically constant and independent of current strength where as in waterflow they are not. The capillary conductivity in a soil may vary both with distance and time, since the case of water-flow will probably depend on the thickness and extent of the actual water film itself. Similarly, the capillary potential at any point -- the attraction between soil and water – will be some function of the actual moisture content at that point, and may, therefore, also vary with distance and time. Equation (E) is thus only of qualitative value until both quantities λ and ϕ can be expressed as definite functions of some measurable factor, *e. g.*, the moisture content.

This theory failed because the constants cou'd neither mathematically nor experimentally be found with reliability. Gardener, Buckingham and Wilsdon worked on this problem with no complete success. Gardener gave the following values.

The capillary conductivity of Greenville soil in various conditions of packing gave the following mean values :

Loosely packed	$1{\cdot}8 \times 10^{-3}$
Well packed in dry condition	$7{\cdot}4 \times 10^{-3}$
Further packed by moistening and drying	$5{\cdot}4 \times 10^{-3}$
Undisturbed field condition	$8{\cdot}7 \times 10^{-3}$
General mean	$5{\cdot}8 \times 10^{-3}$

(C) Theory of moisture distribution in the cellular pore spaces.

It is necessary first to determine how the water distributed within the cellular structure at different moisture contents. Keen's analy is is given on the next page.

The Single Water Wedge. The pressure under a curved water surface is less than outside, the pressure deficiency, p, being given by a well-known relation as the product of the surface tension. T, and the total curvature of the surface. The true surface, as Fisher has pointed out, has the same total curvature at all points, and is generated by the revolution of Plateau's Nodoid about the line joining the centre of the two spheres. However, we may take with sufficient approximation the simpler anchor ring surface with zero angle of contact between particle and water and consider the total curvature at the neck (Fig. 4), which gives -

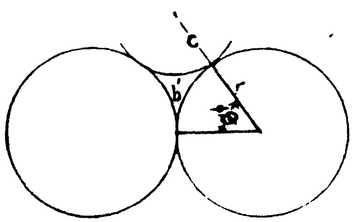


Fig. 4

$$p = T\left(\frac{1}{c} - \frac{1}{b}\right) \qquad \text{(F)}$$

From the geometry of Fig. 4
$c = r(\sec\theta - 1)$; and $b = r(1 + \tan\theta - \sec\theta)$
substituting in the above equation –

$$p = \frac{T}{r} \cdot \frac{(2 + \tan\theta + \sec\theta)}{r(1 + \tan\theta - \sec\theta)(\sec\theta - 1)} \qquad \text{(G)}$$

The pressure deficiency is, therefore, defined as the product of T/r, a constant for a given assemblage of spheres and a trigonometrical foctor. For progressive decrease in the volume of the wedge, θ decrease, tan θ opproaches zero and sec θ approaches unity. Hence, the numerator in the trigonometrical factor approaches the limiting value 3, and the two expressions in the denominator each approaches zero. The pressure deficiency, therefore, progressively decreases as the water volume of the single wedge increases. For any given value of θ, the numerical value of the trigonometrical expression is easily obtained. In the discussion of pressure deficiency which follows, we shall employ a numerical factor, multiplied by T/r ; the latter may be regarded as the natural unit for its measurement in any given assemblage of spheres.

Haine classifies the soil moisture in the three stages : Pendular, Funicular and Capillary.

(*a*) **Penduler water.** At a very low moisture content the water is distributed in discrete rings, one around each point of contact between the spheres, Fig. 5. As the moisture content isincreased the rings become larger, and eventually their edges will come into contact at certain places. Contact is first established in the planes joining the centres of the four spheres enclosing a tetrahedral pore, and from Fig. 5 this is seen to occur when θ = 30°, and hence, from the relation $c = r(\sec\theta - 1)$, $c = 0{\cdot}15r$ approximately. 24 percent pore space is filled with water. When the upper limits of Pendular stage are reached in case of increasing moisture, the water wedges are fast coming in contact, the pressure deficiency which has been continuously falling from very high values is reduced to 4·1 T/r. In the case of reducing moisture, removal of water causes the waists to open until the penduler stage replaces the funicular stage ; but this takes place at a pressure deficiency of 12·9 T/r.

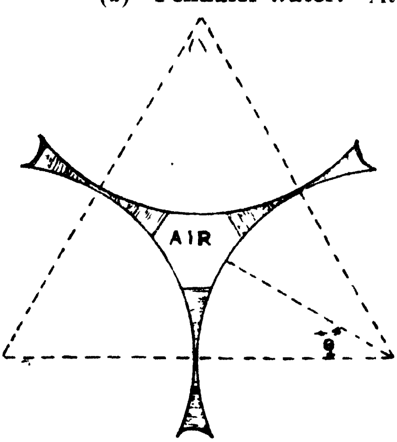


Fig 5.

(*b*) **Funicular water.** If the volume of moisture is increased beyond the pendular stage, changes in the form occur in the water surface. The water wedges are now in contact and it is possible to pass from any one point in water film to any other within the liquid phase Fig. 6. The water forms a kind of continuous net work and so, of course, does the air. The film shape can be pictured as rounded cells, drawn out at the communicating corners into hour glass shaped waists which supply an outward tension and counteract the tendency of the films in the cells to collapse.

Connected Funiculer Water

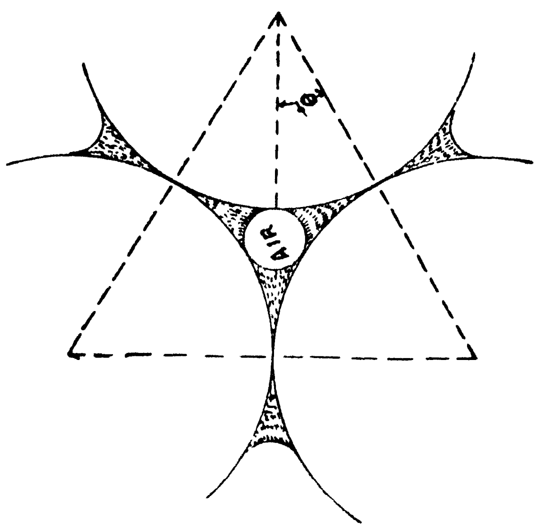


Fig. 6

The pressure deficiency when the collapse of waists occurs is 6·9 T r in the case of increasing moisture and 12·9 T/r in the case of reducing moisture.

(c) **Capillary water.** In the case of increasing moisture, on further additions of water, the saturated zone will progressively extend by sudden saturation of the cells under practically constant value of pressure deficiency. Eventually the "capillary" stage (*i.e.*, pores completely full of water) which has been extending at the expense of the funicular stage is reached throughout the whole mass. On reaching this point the pressure deficiency rapidly falls to zero as the menisci in the external pores disappear, and by rapid transition the soil passes from saturation to flooding.

In the case of decreasing moisture initially the soil is flooded to the top and the air-water boundry plane has zero pressure deficiency. As water is removed the menisci are formed in the surface pores which may be called the meniscus surface above capillary water. The capillary meniscus surface drops into the soil at a pressured eficiency of 12 9 T/r, if water is removed furthei.

Note :- Haines experiments were carried on "Ideal Sand", a sand with rounded grains made of spherical glass of mean radius 0·19 cm. called "glistening dew." and the limits mentioned above were found to be very nearly correct except for the hystirises effect due to change of cycle from increasing to reducing moisture.

These limits are not at all applicable to an actual soil crust, although the three phases will be reproduced. Actual moisture content distinguishing the three phases shall vary according to the pore space in the soil which may be from 35 percent to 44 percent. For 40 percent pore space, the author considers that the soil pore in the capillary stage may be considered to be effectively saturated with moisture content 25 percent to weight. In funicular stage: the moisture content shall vary from 25 to 10 percent and below.10 to 12 percent it shall be Pendular stage. The pendular water may aptly be considered to imply the disconnected hygroscopic water as called by some scientists. The term moisture content signifies the percentage of the moisture in the soil expressed in terms of the dry weight of the soil particles.

6. Soil Characteristics.

The Low Moisture Content or Field Range covers the field conditions from dryness to full saturation, that is 23 to 25 percent by weight. The term **field capacity** is sometimes used to denote the water in the soil above the requirment of the crops. It may be anything from zero to 23 percent moisture content. In the field range the soil may have the apparent specific gravity characteristic of field conditions, or it may be worked up with water to a plastic condition like the modelling clay. In the case of soils rich in humus, field capacity may reach as much as 100 percent, moisture content on a dry weight basis.

The medium, or paste, range is self-descriptive. It may be regarded as the ultimate stage of the plastic range, when sufficient further water is added to permit the mass to flow easily while still retaining some solid properties. The moisture contents in the paste range may sometimes be several hundreds percent on a dry weight basis.

In the high, or suspension, range the water is present in great excess. The concentration of soil is only of the order of 1 to 5 percent. This range is useful in mechanical analysis and in the numerious studies of flocculation, cataphoresis, etc., connected with the colloidal behaviour of soil. The clay, as the finest fraction of the soil, is of most importance in this range.

(*a*) **Cohesion.**

Cohesion in course grained soil due to the moisture films is relatively low, with very fine soil particles high stress values are found experimently ; but these are not due entirely to the moisture films. The organic material acts as a binding or cementing agent and physical forces associated with colloidal material also operate. It is a common observation that clay soils in the field may dry into extremely hard colds. The relation between the forces needed to rupture a block of soil with a steel wedge, and the moisture content was used by Atterberg as part of an extensive scheme of soil classification in Sweden based on the behaviour of soil at different moisture contents. Atterberg (Fig. 7) dealt with the whole range of moisture contents from field condition of suspensions. Here we are concerned only with the measurements of cleavage of blocks of soil made with water to the plastic condition initially, and carefully dried to various moisture contents before use. In general the force needed for cleavage increases with decreasing moisture content, as would be expected At the wetter end of the scale the cohesion is mainly due to the moisture films and the wedge makes a smooth cut, indicating that plastic deformations occurs. As the dry end is approached the block splits suddenly under the wedge with an irregular fracture ; over this range the particles may be regarded as cemented together and the values of the force of rupture as measuring soil cohesion. Other experimenters like Hains do not find the systematic behaviour of soil as depicted by Atterberg.

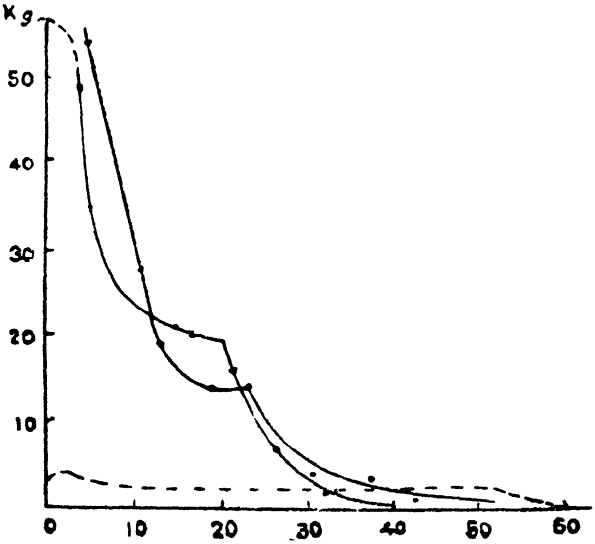


Fig.—7—Typical cohesion-moisture content curves (ATTERBERG) from the "Journal of Agricultural Science")

(*b*) **Liquid limit.**

Soil mixed with water so that it can flow in a liquid state and the minimum moisture required to bring it to that stage is the Liquid Limit.

Laboratory tests have determined the following comparative figures for liquid limits for soils : –

Sands 20, Silts 27, Clays 100, Diatoms 163, Colloids 399, Mica Flakes 123 and Peats 445.

Thus a liquid limit between 20 and 40 might indicate a mixture with sand or silt predominating, while limits higher than 40 point to the presence of mica, diatoms, organic matter, clay or colloids.

Mica is the name given to a group of Silicates having a perfect bassal cleavage into thin tough and shining plates, formerly used instead of glass. Mica-sheet and Mica-Slate are both laminated rocks.

Diatoms. A diatom is any individual of the genus Diatoma or of the order Diatomceae, a group of microscopic algae with siliceous coverings which exist in immense numbers at the bottom of the sea, multiplying by division or conjugation and occurring as fossils in such abundance as to form strata of vast area and considerable thickness.

Colloid. Chemically an uncrystallizable semi-solid substance, capable of only very slow diffusion or penetration. Colloidal matter is the gelatinous or gluey substance found in clays of a sticky nature in soil classification anything in clay measuring 0·001 millimetre in diameter is termed Colloid.

Peats embrace all kinds of decayed and partly carbonized vegetable matter found in boggy places and used as fuel.

(*c*) **Plastic limit.**

The minimum amount of moisture needed to knead a soil so that it can be rolled out into strands of 1/8th inch diameter is termed the Plastic Limit. Silts, clays and colloidal clays are plastic, while sand, peats, mica and diatoms crumble away when rolled and consequently have no plastic limit. It has been determined that silt has an average plastic limit of 20, clay an average of 45 and colloids an average of 46.

(*d*) **Plasticity index.**

This is taken as the difference between the liquid limit and the plastic limit. This index shows the cohesiveness of a soil and indicates its power to change is shape without appreciably altering its volume. The plasticity index of sand is fixed at zero, because it has no plastic limit. The same applies to peat, mica and diatoms. The classification of soils by Atterberg with regard to plasticity indexes is as under :—

Friable less than 1; Feeble Plastic 1 to 7; Medium plastic 7 to 15 ; Highly plastic greater than 15.

(*e*) **Shrinkage limit.**

Evaporation of water causes shrinkage in a soil up to a certain degree, beyond which decrease in volumes does not occur. At this stage the soil has reached its shrinkage limit, in other words the moisture content at the stage, when the soil changes from the semi-fluid to the solid state is the shrinkage limit Between the plastic and the shrinkage limits no direct connection has been established, but it is generally known that the smaller the shrinkage limit, the greater the volume change corresponding to a given variation in the moisture content.

Refering back to the plasticity index classification the shrinkage limit may be given as follows :-

Friable soils, between liquid limit and 50 percent of that limit ; Feebly plastic, 25 to 30 percent ; Medium plastic, 20 to 25 percent ; Highly plastic, 15 to 20 percent.

(*f*) **Moisture equivalent.**

It is necessary to distinguish between Impermeable, Permeable, and Porous Soils. This is effected by subjecting the soils under centrifugal force to pressure of 28·5 pounds per square inch. The moisture content of a soil, which has reached saturation point and then, for an hour, subjected to centrifugal force equal to one thousand times the force of gravity, is called its centrifugal Moisture Equivalent. In this manner we can readily distinguish the impermeable soils-clays and colloidal clays, from permeable soils, soils with sand, silt, flacked-clay mica, peat, diatoms etc., predominating and from sand, which is porous This test shows also the Capillarity of the Soils. Thus sand, which is porous, and allows the free passage of water through the soil, has but little capillary attraction and merely bulks slightly when wet. On the other hand, capillary action in permeable soils results in frost lift and expansion fractures.

Moisture equivalents of Six Soils from Vernon Series (U. S. A.)

	Percent
Sand	3 6
Fine sand	4·3
Sandy loam	10·3
Fine sandy loam	12·3
Silt loam	19·8
Clay loam	23·1

(*g*) **Field moisture equivalent.**

A. G. Bruce defines field moisture equivalent as the maximum percentage of water a soil will absorb when its moisture content is gradually increased by adding water. For sands the field moisture equivalent indicates porosity whereas in the case of moist compressed soil, it indicates the degree to which they can absorb water and expand and the degree of cohesion they possess.

A soil is apt to contain expansive materials, such as mica, in detrimental quantities, when the field moisture equivalent is either equal to or greater than its centrifugal moisture equivalent. Thus harmful micacious silts can easily can be detected.

(h) Lineal and volumetric shrinkage.

Roll out, as uniformly as possible to a length of about 18 inches and diameter of about 2 inches, a quantity of soil that has been wetted with water equal to field moisture equivalent and measure it while wet ; next let it dry and then measure again and work out the percentage shrinkage. In order to determine the volumetric shrinkage the following curve diagram shall have to be used. Below it, is another diagram for determining the shrinkage limit with the field moisture equivalent as the basis for determining the shrinkage. For reliable comparison of volume changes the amount of water to be applied to soils should be based on the total surface area of soils particles, hence the field moisture equivalent has been useful as a basis in the diagram Fig. 8.

It is generally accepted that a lineal shrinkage of 5 percent for which the corresponding shrinkage in volume is 17 should be regarded as the maximum permissible value for a good soil for stabilization.

When the liquid limit of a colloidal soil exceeds 35 (Fig. 9) the shrinkage in volume is likely to exceed 7 percent and hence the lineal shrinkage is likely to exceed 5 percent. In arid regions in the hot season the subgrade will dry out and shrink leaving the hard road surface without support. Clays change more in volume than silts while sands remain or less constant depending on their degree of porosity. From this it is clear that volume change depends on soil particles the finer the particles the greater will be the change in volume, while coarse sands will show no appreciable change.

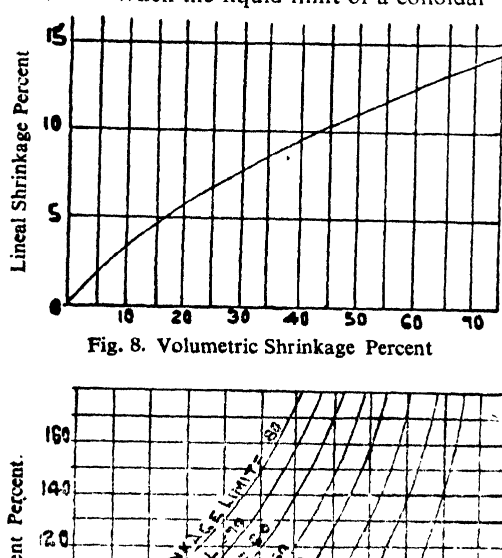


Fig. 8. Volumetric Shrinkage Percent

It might here be pointed out that the field moisture equivalent, the lineal shrinkage and the shrinkage limit are interrelated and with the help of the curves in the diagram the shrinkage limit can be determined from the other two volumes. First of all locate the intersection of the lineal shrinkage (vertical line) and the field moisture equivalent (Horizontal line). Then estimate the value of the shrinkage limit by finding the position of the point with respect to the curves.

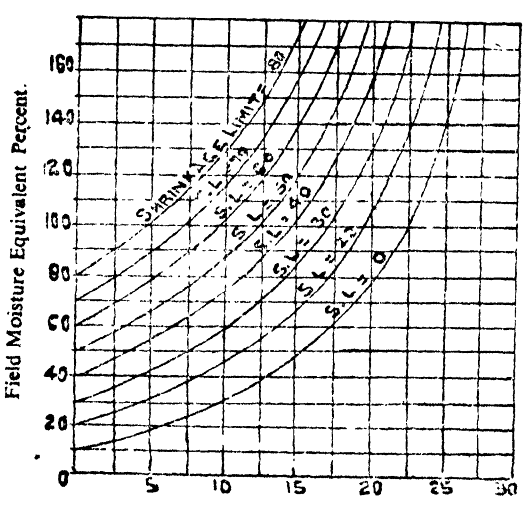


Fig. 9. Lineal Shrinkage Percent.

(i) Porosity.

The interstices, such as pores or voids in a soil determine its porosity and soil density is measured by its porosity, which in itself depends on the shape and disposition and proportion of the soil particles. The loosest soil consists of round particles of uniform size. A more compact soil contains particles, both large and small. Angular particular like chips, do not compact so well as round particles of varying sizes. Cubes pack better than flats. Whether soil particles are large or small, porosity does not alter as long as particles are of uniform size.

Porosity can be graded as under :

Small	When less than 5 percent
Medium	Between 5 to 20 ,,
Large	Greater than 20 ,,
High	Greater than 50 ,,

Most soils fall under the last named class.
Porosity may be found by the formula :—

$$P = \frac{V_v}{V_v + V_s} \times 10 \qquad \text{(H)}$$

in which P is the porosity ; V_v the total volume of voids in mass of soil, V_s the absolute volume of soil particles. There is also another formula

$$P = \frac{e}{1+e} \times 100 \qquad \text{(I)}$$

in which e is the void ratio; given in the following table for the sake of simplification :—

Soil	Void ratio at Liquid Limit
Sand	0·54
Silt	0·71
Organic matter	2·00
Clay	2·65
Diatoma	3·19
Mica	3·44
Colloids	8·18

When a soil changes volume on account of moisture content, the rate is proportional to the difference in the volume of moisture in it, and the constant that expresses the change ratio in soil volume to moisture content is called its shrinkage ratio.

7. Other Soil Constants and Equilibrium Limits.

(*a*) **Wilting coefficient (Briggs).**

It is the moisture content at which permanent wilting of plants occurs. The plant roots are assumed to exert a definite maximum pull on the soil moisture, which is given up by the soil less readily as the moisture content decreases, until eventually no further water is available and the plant wilts. The moisture content at this stage is the wilting coefficient. For determination of wilting coefficient plants are grown in pots, the soil surface being covered with an impervious material to prevent direct evaporation from the soil. Briggs found that the value was largely independent of the kind of plant used, and could, therefore, be regarded as a measure of the water content the soil could hold against a given extractive force. The value increased with increasing fineness of the soil particles and was about three-fourths the figure for the moisture equivalent .—

Wilting co-efficient for *Kubanka* wheat in :—

Fine sand	Fine sandy loam	Clay loam
2·59	9·66	16·3

(*b*) **Maximum capillary water capacity.**

It is the maximum water content held against gravity and therefore, marks the upper limit capillary division. The optimum capillary water is that at which the soil is in the best physical condition for plant growth.

Lintocappillary point is the moisture content at which the capillary movements become very sluggish.

(*c*) **Water holding capacity (King).**

This is an approximate measure of the pore space in small shallow container of the gently packed soil and is determined by saturating the soil from below permitting air to escape and measuring the percentage increase in weight due to water. This is the upper limit of gravitational water (ground water).

(*d*) **Unfree water (Bonyoucos).**

Some of the moisture in the soil is unfree and in close physical union with the colloidal matter in the soil. Unfree water is not available for plants.

(*e*) **Hygroscopic coefficient (U.S.A.)**

It measured the amount of moisture the dry soil would take up in order to come to equilibrium with an atmosphere of saturated water vapour. It was regarded as measuring the water which was held to the soil particle surfaces by intence forces. The experimental method was simple ; the dry soil was placed in a shallow layer close to the surface of water in a closed vessel and left for 24 hours, after which the increase in weight was measured. There is no doubt that many workers regarded this determination as a definite soil constant which, but for inevitable irregularities in the soil samples, would give results as reproducible and as significant as, for example, the determination of water of crystallisation of a salt. The measurements of vapour pressures were ignored in the beginning.

The hygroscopic moisture content was found to vary with humidity as determined by observing vapour pressure as shown in Fig. 10.

Vapour pressure curves of typical soils

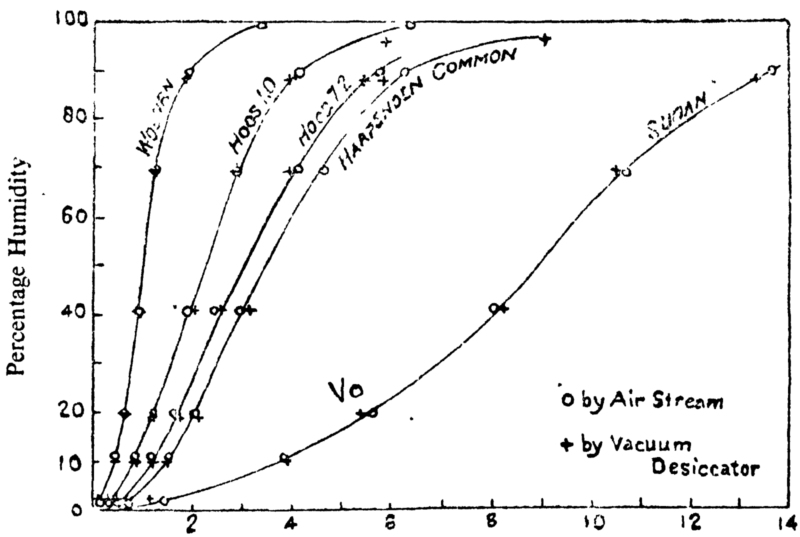


Fig 10

The curves clearly show that hygroscopic co-efficient has no real existence and is very much variable with humidity.

(*f*) **Imbibitional moisture capacity (Fisher).**

The measurements were made with xylene as well as with water, thus giving a "Xylene Equivalent " Fisher regarded the moisture equivalent as composed of (*a*) water held in the smaller soil interstices, and (*b*) that imbibed by the colloidal material. He assumed that xylene was not imbibed, hence the "xylene equivalent" gave the interstitial liquid only Therefore the imbibitional moisture capacity would be the water retained by unit volume of soil less the volume of xylene retained by the same soil after standared treatment in the moisture equivalent apparatus The high imbibitional capacity was associated with high silica alumina ratios.

(*g*) **Sticky point.**

In preparing soil for shrinkage determinations, the mass is kneaded with successive additions of water until it shows signs of becoming sticky or fluid. The point at which the mass is just not sticky or adhesive to the fingers is well defined and can be accurately determined after a little practice. It can be regarded as the moisture content at which the attractive power of the soil for water is just satisfied and is now generally referred to as the "sticky point " The determination was advocated by Hardy, who considered that it measured the maximum imbibitional capacity of the soil colloidal material for water.

(*h*) **The loss on ignition (Keen and Coutts).**

It measures the sum of organic matter and the so-called water of constitution of the hydrated alumino-silicates which characterise the clay fraction. The importance of the latter was not fully recognized by some earlier writers who associated the ignition loss mainly with the organic matter. Coutts has compared the organic matter content and ignition loss, untilizing the results of Robinson and Jones that hydrogen peroxide removes some 75 percent of the total organic matter and that this value is equal to $(I_o - I_p)$ (Table below).

TABLE 1

Ignition loss and organic matter content (Coutts)

	I_o (Original)	I_p (Peroxide treated)	Organic matter
English soils	11 7	6 5	6·9
	5·1	3 0	2 8
	3·6	2 0	2 0
	11 5	6 2	7·1
Natal soils	19 4	13 9	7 3
	16·5	13 7	3·7
	21 0	16 1	6 5
	18·1	14 3	5 1

The table shows that although the amount of organic matter is about the same in the two set of soils, the ignition loss of the Natal soils is much higher.

(*i*) **Heat of wetting.**

This measures the heat evolved when moisture is added to dry soil, both water and soil being initially at the same temperature. The evolution of heat when hygroscopic substances are moistened is a familiar phenmenon Muntz and Gaudechon showed that the heat of wetting increased with fineness of the soil particles ; it seemed therefore suitable as a measure of, *e. g.*, the clay content or the colloidal portion.

A thermodynamical relation exists between the heat of wetting of the hygroscopic material and the vapour pressure at different moisture contents ; if heat is evolved when water is absorbed, then the vapour pressure for a given moisture content should also increase with temperature.

$$Q = \rho T^2 \frac{d}{dT} \log \frac{\rho}{P} \qquad (J)$$

where Q is the heat of dilution ; ρ/P the ratio of vapour pressures or relative humidity ; the T absolute temperature ; and ρ the gas constant for a unit mass of water vapour.

This equation, suitably adapted, states that the heat of wetting of a material at a given moisture content is proportional to a constant, depending on the temperature and on the logarithm of the ratio of the percentage humidities corresponding to that moisture content at different temperatures.

8. Swelling and Shrinkage of Soil in Field Conditions.

In the field soils are not normally in the plastic condition described in the preceding section, but have a looser structure Nevertheless, marked shrinkage effects are shown by heavy clay soils on drying. Deep cracks up to an inch in width at the surface are common even in clay soils during drought, and much mechanical damage to root systems may result. In clay soils under tropical and sub tropical conditions larger cracks may develop, several inches in width and several feet in depth. In the older 'basin' system of irrigation in Egypt, the soil, after the winter crop of wheat or *berseem*, remained fallow from May to August, when the Nile flood water again became available. The extent of the carcking during the *Sheragi* or summer follow, on a typical square metre of the surface can be seen from Fig. 11 only the larger cracks are shown ; there are numerous smallar ones ramifying in all directions. The work of Mosseri show that this shrinkage not only improves the tractability of the heavy clay deposits in the Nile valley, but as the soil dries out, the deleterious salts concenterate on the outside of the lumps and are thus washed down by the subsequent irrigation or flood water. Mosseri has stated that this effect has been largely responsible for the past freedom of the Nile valley from alkali troubles.

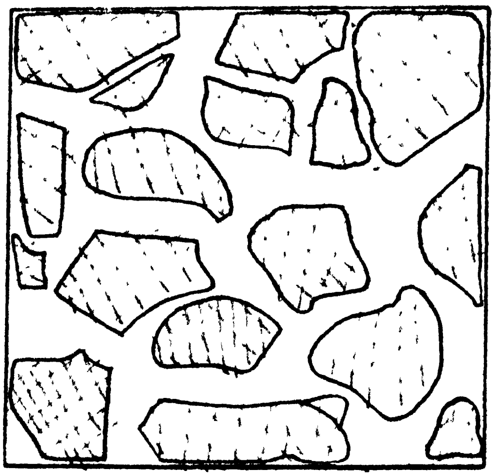


Fig. 11

When the dry cohesive lumps of soil are remoistened an apparent swelling occurs. This is not a swelling of the soil material, as such, but is due to an increase in the pore space coupled with a tendency to disrupt the block, so that after a limited sequence of wetting and drying it crumbles to pieces. It is in this way that alternations of weather produce a friable condition in field soils.

9. Stability of Soils.

The stability of a soil depends on the character of the soil and the resistance that it offers to loads passing over it is contingent on Internal Friction of its component particles, Cohesion between particles, Capillary Pressure. Elasticity, and Liability to change of state under external conditions.

Clays are highly cohesive ; sands are high in frictional resistance; micas, diatoms and organic matter are elastic and rebound when pressure on them is removed and since they cannot be permanently compacted, they make bad subgrades. sand has no cohesion, but great frictional resistance and its bearing capacity increases under heavy load. Clay has cohesion, but hardly any frictional resistance, especially when soft and its bearing capacity is very low. The finer the soil particles, the more variable are the cohesive and frictional properties of the soil under field conditions. The resistance due to cohesive does not depend on external pressure, whereas external pressure increases frictional resistance. Molecular attraction of water or capillary pressure, causes surface friction between soil particles and produces a certain amount of cohesion. Cohesion and internal friction depend not only on the composition of the soil but on the state of the soil-whether it be wet or dry.

Contraction and expansion in the volume of a soil are caused by the throwing off of water in the first case and the absorption of water in the next. The capillary pressure in a soil in liquid form is zero and no further change in volume can take place by the addition of water, although the particles will disintegrate and the soil flow. The semi-solid stage is reached when evaporation occurs and capillary pressure is developed. This pressure goes on increasing as evaporation progresses and shrinkage occurs until the shrinkage limit is reached, when capillary pressure and internal resistance to further reduction in volume are balanced. After this further evaporation merely cause capillary tubes to merge into the soil without causing shrinkage. Clays, colloids, and mica, which posses both cohesion and capillarity in large amounts, are highly expansive soils.

In the case of sand a certain amount of bulking does take place when a moderate quantity of water is added to it. This spreads in the form of a thin film over the sand particles and pushes them slightly apart, but this volume again reduces when the point of saturation is reached and the voids are filled. Thus wet sand reverts to the original volume of dry sand.

10. Properties of Clay and Colloids.

(a) The characteristics of soil in dry and paste stage have been briefly described in paragraph 6. The properties of clay and colloids are separately investigated in suspension stage. The clay fraction is easily estimated by treating the soil to effect complete dispersion suspending in water and siphoning off the supernatant liquid containing clay and colloids, The material does not settle even after a month, so that is the colloidal part. It is usually 1 to 2 percent in a soil. The amount of clay and colloids thus determined is, no boubt, affected by the organic matter in the soil.

(*b*) **Colloidal clay.**

Suspended clay consists of particles visible under the oil emmersion microscope but in the case of clay colloids seperate particles are not visible in the microscope. The colloidal clay sets to a hard mass when dried. Its binding power is greater than that of Portland Cement. The chemical composition is very much varied containing calcium, magnesium, potassium, sodium, phosphorous, chlorine and organic matter in varying degrees. The colour of colloidal material is not the same for all soils. Those of a red or yellow colour show evidence of greater bulking than grey and black colloids.

(*c*) **Properties of clay.**

(*i*) The physical and physico-chemical properties of clay suspensions have been the subject of a great number of experiments in recent years, and a connected account would be a difficult matter even if investigators had come to a general measure of agreement. But literature abounds in anomalies and contradictions, and there is every sign that this will continue for some time to come. The reason is twofold :- the experimental methods of colloid science have, with few exceptions, not yet reached the desired state of refinement ; and compared with the majority of systems studied in colloid science, the clay particle is exceedingly complex. Apparantly unimportant small differences in experimental conditions often unrecognised, and more often, not racorded-may profoundly affect the measurements, and in turn, the conclusions. Perhaps the greatest difficulty, apart from that of preparing clay for experiment in some definite and reproducible state, is that chemical decomposition of the clay complex and base exchange phenomena frequently occur in addition to the particular effect that the investigator is ostensibly studying. In consequence, phenomena ascribed to the original clay material may really belong to a different complex, and one, moreover, which has probably been changing continuously during the experiment.

The properties of clay particles of most direct interest to students of soil physics are those concerned in coagulation of a suspension, because the surface forces concerned in the process probably operate in field conditions in connection with the formation of compound particles.

(*ii*) **Dispersion, coagulation, flocculation of clays.**

The terms flocculation implies that the clay settles in flocs, leaving a clear supernatant liquid. This distinguishes it from coagulation which has been defined as the reduction in total number of particles in the system produced by the individuals sticking together or adhering. Coagulation may be regarded as the first stage of flocculation ; both perikitenic and orthkitenic coagulation may occur ; and their effects are not separately distinguished.

The following points are notworthy :—

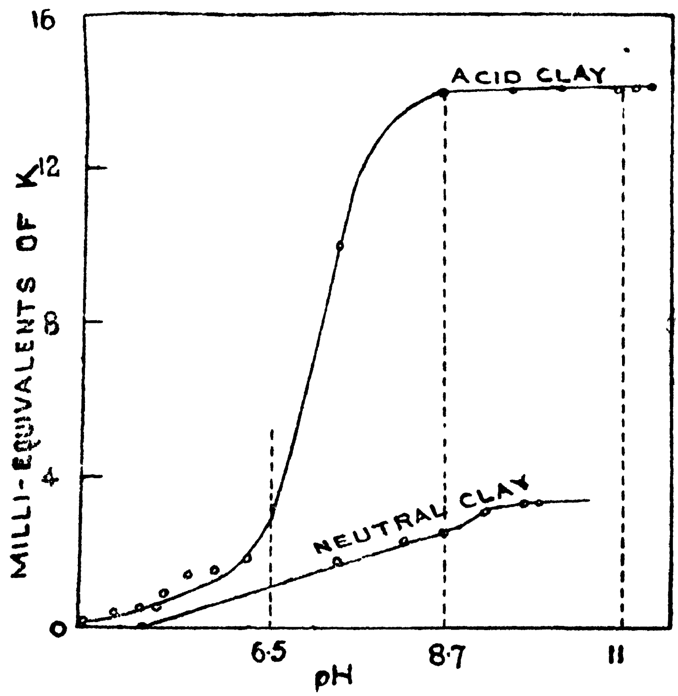
(*i*) Na-clay is more stable than the corresponding H-clay.

(*ii*) Na-ions are weak coagulators, hence flocculation by NaOH usually involves a very alkaline solution and conseqently some decomposition of the clay complex.

(*iii*) The H-ions of H-clay exchange weakly, since they may be regarded as forming very slightly dissociated complex acids with the aluminate, silicate and hydroxyl radicles on the clay particles surface.

(*iv*) Although in pure water, H-clay has a higher electrokinetic potential than Ca-clay yet $CaCl_2$ flocculates H-clay more easily than a Ca clay, in each case flocculation taking place at the same value of the electrokinetic potential. Bradfield studied flocculatton of a neutral clay and an acid clay over a rangs of pH between 3 and 13. The flocculating agent consisted of suitable mixture of KOH and KCl, or KCl and HCl, all having the same potassium concentration. His results are shown in Fig. 12 in which the pH of the supernatant solution is plotted against the milli-equivalents of K required to flocculate the clay. The two curves show striking differences. For the acid, clay the electrolyte requirement increases gradually with decrease in H-ion concentration to about pH 6·5 : between 6·5 and 8·7 the electrolyte requirement increases rapidly to a maximum which is maintained for higher values of pH.

The importance of base exchange reactions in flocculations phenomena was further illustrated by latter experiments of Bradfield He showed that the quautity of electrolyte needed for flocculation may vary widely with relatively small changes is pH concentration, concentration of the clay, and the nature and extent of the exchange reactions occuring when the electrolyte is added to the clay. His results for the amount of $Ca\ OH)_2$ and $CaCl_2$ needed to flocculate an H—clay as the concentration of the clay suspension was increased, are of considerable interest. After allowing for the amount of Ca needed to neutralise the clay, proportionality was found between clay concentration and milli-equivalents of Ca needed for floccnlation in the case of $Ca(OH)_2$, but with $CaCl_2$ the amount needed was much smaller and was, further, independent of the clay concentration. The difference is probable associated with the fact that when excess of $Ca(OH)_2$ is added to the clay after neutralisation (*i. e.*, to a Ca-clay), further absorption of Ca



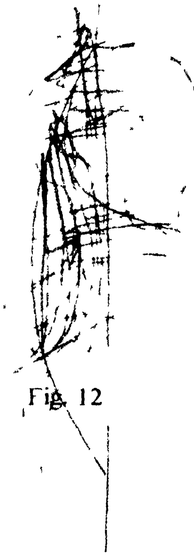


Fig. 12

occur owing to the formation of insoluble aluminates, and this absorption would naturally be proportional to the clay concentration. The linear relation found by Bradfield is almost certainly mainly due to this effect, and, but for the unavoidable experimental errores, it should therefore be possible to show that milli-equivalents of $Ca(OH)_2$ required to flocculate the clay are linearly connected with, but not quite propotional to, the clay concentration.

11. Distribution of Water in a Vertical Column in Soil Crust.

Theoretical methods described in paragraph 4 failed to determine correctly the moisture in a soil crust above the phreatic surface of the ground water reservoir on account of the difficulty of knowing the transmission constants in the partial saturation. Pressure deficiency of Haines and Keen is also not a correctly measurable commodity in soil crust. Maclean and Joffe employed porous pots filled with water buried in the soil and connected to a mercury monometer to record the pull of the soil for water, calling it pF value. They believed that pF value gave a correct idea of the moisture in the soil and was closely correlated with the so-called colloidal material in the soil. The measurements were mostly defective because even if the porcelain

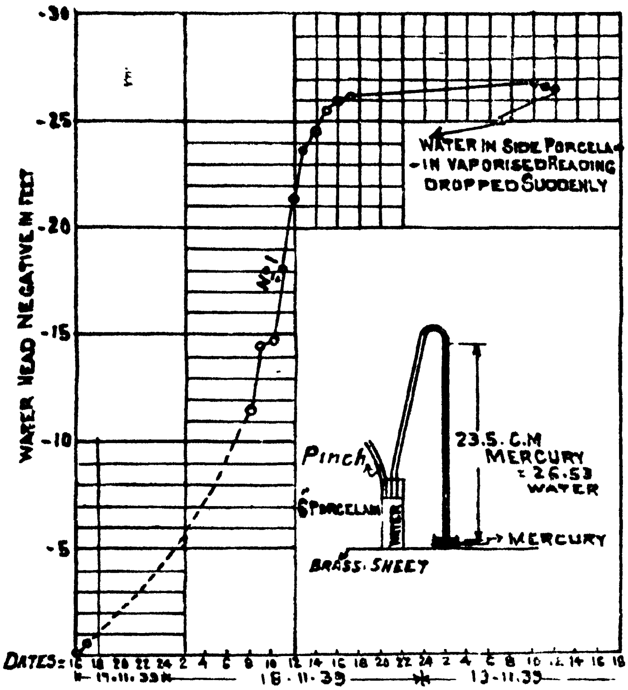


Fig. 13

pot be not buried in the soil the film surfaces are formed in the body of the porcelain pot itself recording very high negative pressure. The author carried out observations as shown in Fig. 13 which clearly shows that error due to the pot itself is as great as the abserved pressure. The observations of vapour pressures as used by Messrs Fisher and Puri to predict the moisture content using dilatometers in a lab. cannot be used in the field to predict the moisture content and the pressure gradient.

Wilsdon did very detailed work by taking the soil samples below an irrigation channel to determine the moisture contours. His results show that moisture content below the saturation line under the bed of a channel and above the capillary meniscus surface above the water table varies as log of the distance from the complete sturation line below the bed or the capillary meniscus surface. He further found an empirical relation between the hydrostatic pressure and the moisture content. His analysis as given in Punjab Engineering Congress Paper No. 78 of 1923, is given below :–

"Knowing the hydrostatic pressure or the water in a soil as a function of the water concentration and constants depending on the fineness of the soil. Its degree of packing and the surface tension of the soil solution, we can advance a step further and determine the conditions of static equilibrium in a column of soil which is saturated at the bottom.

The fundamental argument is that at equilibrium, the vapour pressure of water drop must be the same as the pressure of water vapour in the saturated column of air at the point considered.

Wilsdon mathematically deduced the expression given below assuming that the specific volume of the liquid did not change.

$$\phi = \frac{30{\cdot}5}{1033{\cdot}2} h - 0{\cdot}0295\ h \qquad \text{(K)}$$

Where ϕ is the hydrostatic pressure of the liquid in atmosphere ; h the height in feet above a given datum in feet.

He also found an empirical relation from his observation between hydrostatic pressure ϕ and the moisture content "η" as given below : –

$-\alpha\phi = A/\eta - B$; where α, A and B are constants. He gave tables of the values of the constants for a standard soil 0·0002 cm. radius.

His observations taken at Lyallpur showed that the moisture content below the bed of a channel reduced to 3 percent by weight, after it had attained equilibrium, although the channel was constantly losing water in absorption. Similarly above the ground water there is capillary fringe, the soil pores, in which, are effectively saturated. Above this moisture reduces as shown in Fig. 5, chapter IV of this part. The soil moisture in the region of reducing moisture contours may be divided into two parts, first funicular water which is continuous and the pendular water which is disconnected. When the moisture content is below 10 to 12 percent, it is considered to be disconnected, (pendular water).

12. Soil Temperature.

(*a*) The sources of supply of heat to the soil are radiation from the sun and conduction from the interior of the earth. Many measurements have been made of the temperature gradient in deep mines and wells because the results are of great value in determining the rate at which the world is cooling. Lord Kelvin and Forbes devoted much time to this problem, which was afterwards investigated in detail by a commitee of the British Association. The average increase of temprature with depth in Britain is about 1°C for 110 ft. Knowing the conductivity of the earth's crust, the amount of heat passing through and escaping from the earth's surface can be calculated. The final conclusion of the British Association Commitee was that on an average "41·4 gramme-degrees of heat escape annually through a sq. cm. of a horizontal section of the earth's substance". This quantity of heat has a negligible effect on the soil temperature, which is therefore primarily controlled by the supply of heat from the sun. It has been calculated that a ten percent increase in the radiation, other factors remaining constant, would raise the mean temperature of the earth's surface by 7°C. The amount of radiation reaching the earth from the sun is not constant. In the first place the total radiation emitted by the sun and the distance of the earth from the sun both vary. In the second place, the earth's atmosphere and matter suspended in it, modify the intensity of radiation to a vabriable degree.

The most reliable value of the solar constant is 1·925 calories per min. per square cm. This represents the energy received from the sun by the earth (at its mean distance), corrected for the loss by absorption in the earth's atmosphere.

The temperature of the soil determined by the balance between the gain and loss of heat. The radiation reaching the soil surface is decreased by dust and water vapour in the atmosphere, but dry air is transparent to the short wave-length within which the maximum of the energy distribution curve for the sun's radiation iies. The amount retained by the soil depends on its absorption spectrum, and hence, on its colour and amount of moisture present. The radiation emitted by the soil is chiefly of long wave-lengths. The ozone, CO_2 and water vapour in the atmosphere, having absorption bands in the wave-length, are effective in preventing unrestricted radiation into space.

Superimposed on the above factors are the "positional" ones, which may be grouped in six main devisions :—

(*i*) Latitude and slope of water.
(*ii*) Altitude (1°C for 550 ft. change).
(*iii*) Distribution of land and water.
(*iv*) Distribution of air and water currents.
(*v*) Extent of vegetation cover.
(*vi*) Nature of the soil.

(*b*) In general, any point on the earth's surface experiences a daily rise and fall, or a diurnal wave, of temperature. The amplitude of this wave, or the difference between the maximum and minimum temperatures for the 24 hours, progressively increases from winter to summer owing to the increasing elevation of the sun. When the soil surface ts heated, a diffence of temperature (temperature gradient) is set up between it and the subsoil, and a heat wave is propagated downwards. The wave travels comparatively slowly, and its amplitude decreases rapidly with depth, so that at a depth of about 3 ft. the daily fluctuations of temperature are inappreciable, and the changes observed are due to the daily average temperature that constitutes the annual seasonal wave of temperature. During the night period of the daily wave, and the autumn and winter for the annual wave, the soil surface is cooling and there is a flow of heat from the interior outwards. Both daily and annual waves therefore have the character of an ebb flow of heat. The seasonal temperature gradient is felt a couple of hundred feet below natural surface.

(*c*) The conductivety of moist soil increases with the moisture content over a wide range because of the better thermal contact between the grains produced by the water film. As the moisture content is increased indefinitely, the cunductivity must eventully fall tawards the value for water. The diffusivity increases under the combined influence of the improved conductivity and the relatively slow increase in effective specific heat until it reaches a maximum, at which point the rate of rise of temperature due to the application of heat has its maximum value ; subsequently the value decreases because at higher moisture contents the high specific heat of water becomes predominant.

13. Soil Atmosphere.

(*a*) The soil being a porous material will contain gas in its interstices, and the composition of the soil air will depend on biological activities evolving and absorbing different gases and on the rate of exchange these gases with those constituting the ordinary atmosphere. The predominant biological factors are the absorption of oxygen by plant roats and micro-organisms and the evolution of carbon dioxide, and it is evident that the composition of the soil air will depend on the rapidity with which carbon dioxide can escape into the atmosphere and be replaced by oxygen. It has long been known that the layer of air immediately, above the soil surface, when carbon assimilation is proceeding rapidly, has a higher content of CO_2 than the ordinary atmosphere ; hence, the normal production of CO_2 in the soil is at least equal to the uptake by aerial parts of the plant even at the season of most rapid assimilation.

(*b*) For a carbon dioxide production of 7 litres per square meter per day, the soil air would have to be completely renewed every hour to a depth of 20 cm. in order to remain at its usual average composition. This is taken by Romell as a convenient standard for assessing the relative efficiency of the different factors producing aeration, and is defined as normal-aeration. Systematic analysis of the soil air can be used to study the rate of escape of CO_2 from the soil. Thus high values would imply a slow rate persistent low values, a rapid rate due to some continuously acting factor, while wide fluctuations in composition would imply a varying rate controlled by some intermittent factor.

(*c*) The physical problem consists in examining the movement of gas through a porous moist material, whose upper surface is in contact with the air atmosphere and whose lower boundary is either the saturated zone of the groundwater level, or some impervious strata.

The factors concerned are the intrinsic physical property of diffusion and meteorological conditions causing bulk movement or streaming of air. The latter may be grouped as follows :—

1. Soil temperature changes causing expansion and contraction of air.
2. Change in the volume of pore space available for air, due to rain, irrigation and evaporation of soil moisture.
3. The effect of wind in forcing in or sucking out air, from the soil.
4. The influence of barometric changes in causing compression or expansion of soil air.

(*d*) **Effect of temperature gradient.** Owing to the small specific heat of air (about 0·0003 for 1.c.c. at atmospheric pressure) and to the minute size of the soil interstices, the air will rapidly assume the temperature of its immediate surroundings, at the same time the warmer and lighter air will tend to move upwards and be replaced by colder and denser air, and owing to its small specific heat a considerable convection interchange of air from one part of the soil to another could be affected without much appreciable effect on the temperature of the soil itself.

The effect would obviously be greater when the lower layers of the soil were at a higher temperature than the upper ones, *i.e.*, during the evening upward movement of the daily temperature wave in the day time, when the temperature gradient is in the opposite direction. much less movement of air would result in the condition being some what analogous to the slow equalisation of temperature in a column of liquid warmed only at the top.

(*e*) **The effect of diffusion.** The kinetic theory of gases shows that the molecules are in a state of rapid motion. If a vessel contains two or more gases originally separated from one another, molecules of each gas will penetrate into the space occupied by the other gas, until ultimately a uniform mixture is obtained. As applied to soil conditions, this action will tend to produce equality of composition between the soil atmosphere and air atmosphere. Carbon dioxide passes out of the soil and oxygen passes in. Th name diffusion is applied to this passes. Diffusion alone is able to effect the necessary removal of CO_2 (and water vapours) and the entry of oxygen, whereas the other factors considered—tem perarure, barometric pressure wind and rain are individually, insufficient for this purpose. Diffusion is a continuous process although temperature and barometric fluctuations have a certain regularity, their actual effect are small. Diffusion affects the soil atmosphere up to the water table even if it be several hundred feet below the natural surface.

14. Soil Sampling and Analysis.

(A) **Sampling.**

The usual methods of soil sampling.

(*i*) **1 inch dia: borer.**

It is a one inch dia: borer and works like an auger. It has a cutting edge at the bottom with a six inches high enclosed space above. There is a valve just above the cutter which closes when it is taken out and then the soil sample does not drop away. The samples are usually taken for every 6 inch depth of the soil crust and the bars connected to the borer can be lengthened according to the depth. The sample is bottled and stoppered as soon as it is taken out.

(*ii*) **Three inch borer.**

The working is exactly the same as described above and it is used when quantity of sample required is large and a high degree of accuracy is not required.

(*iii*) **Cutting a profile of the soil crust.**

The profile of the soil crust is dug down and soil samples bottled for various depths below the ground.

The author had an occasion to test the accuracy of these methods at the Chichoki Malian experimental site. The samples were taken 1·5 ft. below the ground by the above mentioned methods, bottled, sealed and sent to the Research Institute, Lahore. The percentage moisture content by weight was reported to 17·6, 14·3 and 22·3 respectively, showing great disparity in results of the same depth and the site.

The soil sample is tightly pulverised by a wooden pestle and then passed through a sieve to separate out silt below 0·07 mm diameter. The silt is analysed according to the methods described in paragraph 20, Chapter VI, Volume I and the stuff below 0·07 mm. diameter is analysed by the hydrometer or pippette method described below.

(B) **Soil analysis.**

(*a*) **Hydrometer method.**

The soil particles are originally found coalesced with one another and it is necessary to disperse them to estimate the percentage of clay, silt and sand present. A definite weight of sun-dried soil 33 grams, is taken and suspened in water in a bottle. About 2 to 2·5 c c. of NaOH is then added to made it alkaline above pH of 10·8 in addition to 1 c. c. of Na_2CO_3 to remove the exchangable calcium. The soil is then made to 1,100 c. c. in a cylinder and thoroughly shaken far four hours, after which it is kept overnight. in the morning it is again shaken for one hour and then allowed to stand.

A chain hydrometer is inserted in the mixture and the time taken by the silt particles to pass just beyond its centre of gravity is allowed. At this stage all the particles of silt have passed below and the density of suspension is simply due to the presence of clay. The density is then observed and the percentage of clay computed considering the average density of soil to be 2·65.

For determining the sand percentages, the soil is disperesd as above, diluted and after shaking allowed to settle for time calculated for the smallest sand grain (0·05 mm diameter) or settle to the bottom. The upper liquid is syphoned off and the process is repeated two or three times to drive off all the clay and silt which may be sticking to the sand particles. The sand is then dried and weighed.

The time of settlement of particles of different sizes was determined according to Stoke's Law :—

$$S^2 = \frac{30\,n}{g(G - G_1)V}$$

where S is the diameter of particles is millimeters ; n the co-efficient of viscosity of suspended medium ;

g the Gravitational constant (980) ;
G the Specific gravity of particle ;
G_1 the Specific gravity of suspended medium ;
V the Settlement velocity in centimeters per minute.

Substituting the following assumed numerical values : –

$n = 0{\cdot}0102$ (viscosity of water at 67° F) ; $G = 2{\cdot}65$ (specific gravity of average soil soilds) ; $G_1 = 0{\cdot}9984$ (density of water at 67° F ;

We have $S^2 = \frac{V}{5290}$ or $V = 5290\,S^2$ cm. per minute = 173 ft. per minute.

A convenient method of showing the distribution of particle sizes in a soil is to plot grain diameter as abacissas on a logarithmic scale with the corresponding total percentage as ordinates on an arithmetical scale. The curve connecting the points so plotted is termed "oartucke suze. *viz.*, accumulation curve".

(*b*) **Pippette method.**

This method is described by G.W. Robinson on page 309. Volume XII of 1922 of the Journal of Agricultural Science. Cambridge ; and an extract is given below : –

(*i*) **Theory of the method.**

The fundamental assumption underlying all methods of mechanical analysis by sedimentation is that particles in a column settle independently of each other. That their are limits to this assumption is obvious. According to Oden this condition is fulfilled in supensions of concentration not greater than one percent. Wiegner, on the other hand, brings evidence to show that concentraions of more than five percent can be used without serious inaccuracy, which may be due to the dangerous principle of compensating errors.

Let us assume a suspension of soil or other granular material to consist of a number of fractions, a, b, c, etc., each uniform in itself, having limited velocities, v_1, v_2, v_3 etc., respectively, and present in concentration A_1, A_2, A_3, etc., respectively, such that $\Sigma A = C$ the total concentration or (ΣA + organic matter = C, in the case of ordinary soil). Then if the fractions settle independently of each other, each fraction will behave as a separate fraction uniform in concentration from bottom to top and as may represent the state of affairs at the begining of sedimentation as in the upper diagram of Fig. 14 the relative amount of partial concentration of each fraction being represented graphically by the thickness of its columns on the diagram.

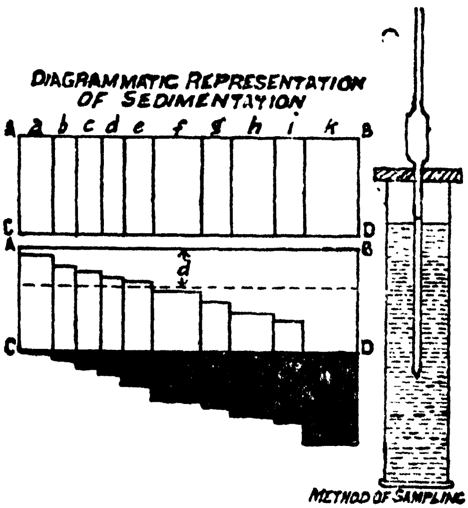


Fig. 14

As sedimentation proceeds each column will fall bodily at its own appropriate velocity and the disposition, after settling has proceeded for a certain time, may be represented by the lower Fig. of the diagram. The black portion below the line CD will represent the amount of each fraction accumlated on the bottom sedimenting vessel, while the concentration of the suspension at any depth will be given by the total width of columns at that depth. Thus at depth d, the concentration will be equal to the sum of the partial concentration of the fractions a to e, having velocities less than d/t. The ratio of the concentration at depth d after time t to the total concentration at the begining of the experiment will thus give proportion of material having velocities less than d/t.

By determining the concentration for different values of d/t the data are obtained for a summation curve showing the relation between percentage of material and log settling velocity.

(*ii*) **Experimental.**

The method used consists in allowing a soil suspension of known concentration to settle in a cylindrical vessel and with drawing samples for appropriate values of depth/time. By suitable choice of depth and time the concentration and hence the percentage of particles corresponding to any desired velocity can be obtained. Generally speaking a litre measuring cylinder about 40 cm. in height and 6 cm. in diameter is used. There is, of course, no necessity to use a graduated vessel, say cylinder of uniform cross section and suitable dimensions may be used. Sampling of the suspension is carried out by means of 20 c. c. pippette passed through a cork or shive and adjusted so that when the cork rests on the top of the cylinder the point of the pippette is at the desired depth below the surface of the liquid Fig. 14. The column having settled for the required time the pippette previously adjusted for depth, is closed at the top with the finger, in order to avoid sampling the upper layers, and allowed very carefully till the cork rests on the top of the cylinder. The finger is then removed and 20 c. c. of the suspension withdrawn. Every precaution is, of course, taken to avoid shaking or mixing the layer of the suspension at the point of sampling. With a column of the dimensions mentioned the withdrawal of 20 c. c. causes a fall in level of about 7 mm. This probably represents the extreme error in sampling. It is assumed that the 20 c. c. of suspension withdrawn represents the concentration at the point of the pippette. Probably the liquid comes mainly from above, but to some extent from below this point. A separate experiment with a column which had settled for several weeks and which had formed clearly defined strata, showed that it was possible to pippette to within 2 to 3 mm. of a stratum without disturbance. It can be shown that an error of a few millimeters in sampling involves a negligible error in the final result. Careful manipulation is, of course, necessary in the operation. The 20 c. c. of suspention is delivered into a flat porcelain dish which has been previously ignited and weighed. Dishes ordinarily used for the estimation of the total solids in milk are convenient for the purpose. The sample is taken to dryness on the water bath and if the estimation is to be made on unignited material, weighed after attaining constant weight. Ordinarily it is ignited in a muffle, an opperation which only takes a few minutes at red heat, and weighed after cooling in a desiccator. From the weight of ignited material the concentration of the sample of suspension is calculated. By sampling in such a way that successively smaller values of depth/time are used the same suspension may be shaken up and sampled over and over again. The partial concentration of any fraction at a given depth is unaltered until the top of the fraction column has sunk below that depth, as will be seen with reference to Fig. 14. The removal of a sample of suspension does not, therefore, affect the concentration of the suspension with respect to fractions of smaller velocities.

PART V

GROUND WATER ENGINEERING

CHAPTER II

Surface Evapora'ion, Soil Evaporation and Transpiration

1. Introduction.

Evaporation is the process by which water is changed fiom the liquid or solid into the gaseous state. As temperature is but a measure of the average rate of motion of the molecules of any substance, it follows that some molecules are always moving at a much higher velocity than the average. Some of these extra rapidly moving molecules are "bombarded" out through the surface film of water, into the atmosphere, sofar beyond the influence of the force of cohesion that they do not return to the liquid, but remain in the space above as vapour, When the vapour over the water surface is relatively dense, some of the vapour molecules are caught in the water and join the liquid again. When interchange of molecules is equal, evaporation is zero. This occurs when the dew-point temperature of the vapour above the water is just equal to the temperature of the liquid. When the dew-point temperrture of the vapour is lower than the temprature of the water, evaporation continues but when it is higher condensation occurs. For any given temperature, the fewer the number of molecules of vapour in a unit volume of space above the water surface, the more rapid the rate at which the upward moving molecules are lost from the liquid.

In as much as the process of evaporation consists of the abstraction of the more rapidly moving molecules from the liquid mass it follows that the average rate of motion of the remaining molecules must be reduced and, consequently, the temperature of the liquid lowered. In other words, evaporation "is a process of cooling". To distinguish it from other forms of evaporation such as soil evaporation and transpiration it is called surface evaporation in the pages of this book.

2. Effect of Temperature.

It was first pointed out by Dalton, over a century ago, that the rate of evaporation from a water surface, other condition remaining constant, varies nearly as the difference between the maximum vapour pressure corresponding to the temperature of the water and the actual pressure of vapour present in the atmosphere above the water. Vapour diffuses itself through the atmosphere somewhat slowly on account of the presence or the molecules of the dry gases. Principal means for the removal of the vapour which forms over all moist surfaces, is the bodily motion of the atmosphere. Since the air movement within a few feet of the land and water surface is very much slower than that at higher elevations their is always a considerable variation in the water vapour content of lower few feet of the atmosphere. This variation consists not only of a variation in the relative humidity bui in the actual amount of vapour present, as represented by the vapour pressure.

If we accept the principle enunciated by Dalton, that evaporation is governed by the difference between the vapour pressure correspending to the water temperature and the actual pressure of the vapour present in the air above ; and if the rate of reduction in the vapour contant of the air from the earth's surface upward is uniform. it follows that the vapour pressure measured at almost any elevation above the earth's surface, when subtracted from the vapour pressure corresponding to the water temperature will give a measure of evaporation.

In as much as the maximum vapour pressure is a function of the temprature, the actual pressure of the vapour present in the atmosphere must also be a function of the temperature, if the relative humidity remains constant. In other words, the rate of evaporation, according to Dalton's law, is approximately doubled for each 18° rise in the temperature, for constant humidity and wind velocity.

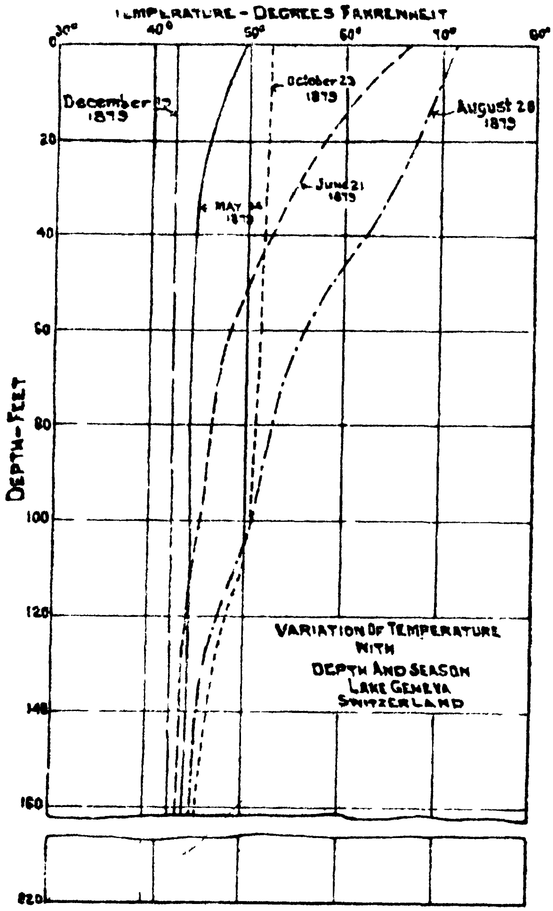


Fig. 1.

3. Effect of Barometric Pressure.

The evaporation is increased if Brometric pressure is reduced but no two investigators agree about the extent of this effect. Stefan, in 1873, represented the effect of barometric pressure by the following expression : $\log \left(\frac{P}{P-p} \right)$; where

P the barometric pressure and p the maximum vapour pressure at the given temprature. (The value of this expression becomes infinity at the boiling point).

4. Effect of Relative Humidity.

Relative humidity affects evaporation only when taken in connection with temperature. It is a measure of the amount of vapour present in the atmosphere. If the temperature of the water is higher than the temperature of the air, evaporation will continue even though the relative humidity a few feet above the water surface is 100 percent. For the conditions of uniform and constant air and water temperature, evaporation is proportional to the saturation deficit. It is equal to a constant time's the maximum vapour corresponding to the temprature multiplied by one, minus the relative humidity.

5. Effect of Wind.

The effect of wind velocity on the evaporation of moisture from a broad expense would appear to be primarily its effect in removing the vapour which forms more rapidly over the water surface than it can diffuse through the atmosphere above. The effect of wind is to lower the relative humidity at the point of observation, if this is within a few feet of the water surface. Where a small surface of water, is exposed to evaporation in quiescent air, a blanket of vapour soon forms above the water, which greatly reduces evaporation. If under these conditions, however, a measurement of relative humidity is made rather close to water surface, it will be found that the space is occupied by nearly saturated vapour. If the air is next yet in motion there will be a decided increase in the rate of evaporation which will also be indicated, however by a great drop in relative humidity. Observations of relative humidity made at some distance from the water surface would not reflect the wind effect ; consequently, it would appear correct to make an allownce for wind effect on the evaporation of moisture from small or non unifromly moist surfaces when relative humidity is observed in a standard Weather Bureau shelter nearby, but to make no allowance for wind effect when the observations for relative humidity are made above a large water surface.

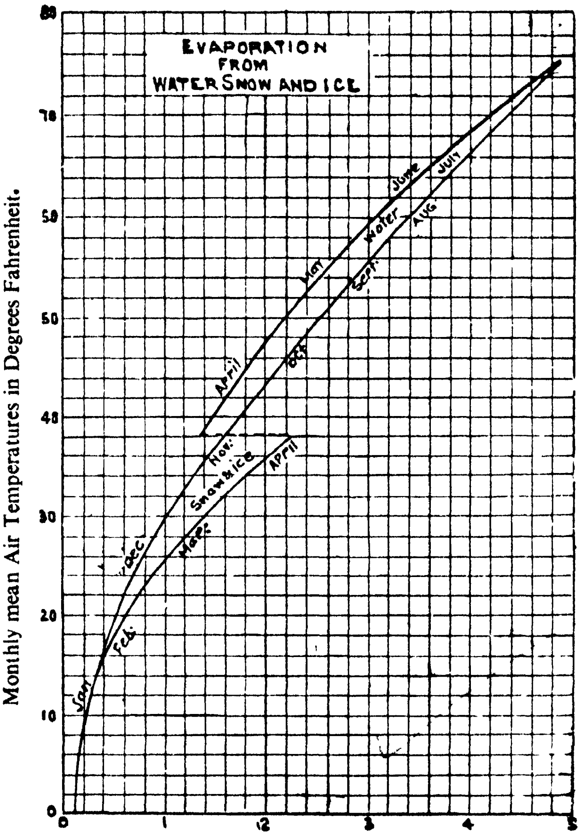


Fig 2.

The common method used in India to allow for the effect of wind velocity is given a part IV of this volume (Stevenson's formula). No formula can give accurate results, as the factors affecting the results are too many and very varid.

6. Effect of Depth of Water.

The depth of water has nothing to do with the surface evaporation but shallow water gets relatively more quickly heated than a deeper water and results in relatively higher surface evaporation. In very deep water, there is a temperature gradient and variation as shown in Fig. 1. taken from Meyer's Hydrology page 233. The rate of variatian of evaporation from water, snow and ice is compared in Fig 2.

Meyer's evaporation formula from small bodies of shallow water.

$E = 15(V-v)(1+w/10)$; E is the evaporation in inches per thirty day month. V the Maximum vapour pressure in inches mercury corresponding to monthly mean air temperature observed by Weather Bureau at nearby stations; v the actual pressure of vapour in air based upon weather Bureau determinations of monthly mean air temperature and relative humidity at nearby stations; w the Monthly mean wind velocity in miles per hour as observed by Weather Bureau at nearby stations, about 30 feet above general level of surrounding country or roofs of city buildings.

For computing evaporation from large bodies of deep water, when the necessary meteorological data available, are used.

7. Measurement of Surface Evaporation.

The author in his observations used Pitchi's evaporometer or a pan 24 inches long and 18 inches wide with depth of water 6 inches and surface levels measured with Hook gauge every 24 hours.

Pitchs's evaporometer is merely an inverted tube some 6 inches long graduated in cubic centimeters and covered on the bottom by a blotting paper disc the exposed surface of which is 10 square centimeters. Therefore, the instrument reads millimeters of evaporation direct for each c.c. mark. It is suspended in the open. The actual observations are available for the Lyallpur station as recorded in the Agricultural College and are shown in Plate XX. The annual evaporation is 130 inches in a year at a Lyallpur, Punjab, while the average value in the United States, America is about 80 inches and at Pretoria in South Africa only 60 inches.

The results published by M.A. Parker in his book "The control of Water" page 191 are reproduced in Fig. 3.

8. Soil Evaporation.

Soil evaporation is the vaporisation of water held in the soil pores and then its escapage into the atmosphere above the ground. This phenomenon is quite different from the surface evaporation described in the preceding paragraphs.

The water is avilable in the soil sacs in decreasing moisture contours from the ground water table to the natural surface of the earth It is held there round the points of contact of the soil particles forming menisci surfaces. The soil has great affinity for water and the force or pull for water is very high (may be many times greater than the atmospheric pressure). It was sometimes believed that the soil evaporation could not take place because water is attached to the soil particles with a force greater than the possible pull exercised by the forces causing evaporation at the ground surface which looks apparently dry. The soil evaporation was then considered to be a negligible quantity. All rain water when once below the ground surface was considered as addition to the ground water if not used in transpiration by the plant growth.

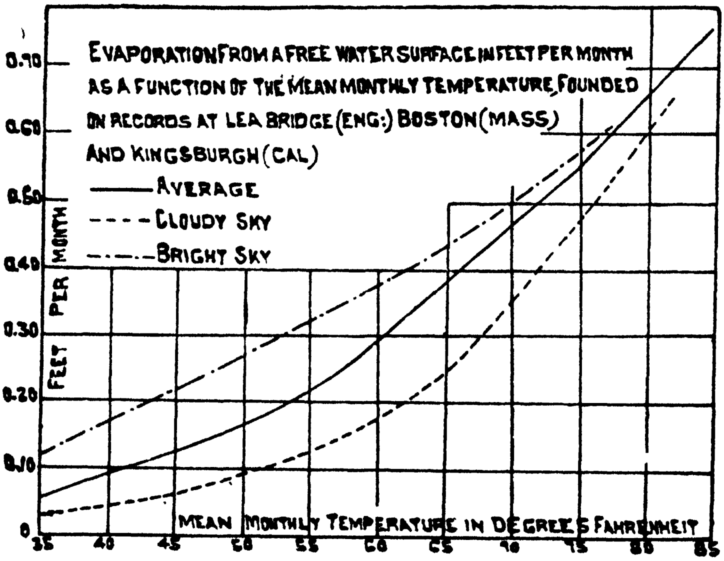


Fig. 3

This view was held because it was considered that the only factor which brought the water up, was capillarity. Keen states that for all practical purposes upward capillary movement is ineflective when the water-table is below 35 cms. in coarse sand, 70 cms. on fine sand and 80 cms. in heavy loam. He also states (Lecture to the Royal Meteorological Society 1932—Soil Physics in relation to Meteorology) that on the average soil water which has reached a depth of about 6 is not drawn back to the surface again by evaporation.

Keen's statement implies that there should be continuous water connection from the ground water to the ground surface by capillary action but Hain's experiment have shown that there is continuous water connection even in funicular water when moisture content of the soil is more than about 12 percent. In the author's observations described in the next chapter capillary fringe of 11 feet height was observed in the soil crust in the Punjab and funicular water height above it of about 5 feet. It may be that Keen's capillary heights were measured in the laboratory on artificially packed soils while in the actual soil crust much higher capillary heights are possible.

Water is under negative pressure in the capillary fringe and in the soil sacs above it. It is bound to be converted into gaseous state, *i. e.*, water vapour which becomes part and parcel of the soil atmosphere. It has been shown in the last chapter that the soil atmosphere is always changing due to the temperature gradient in it. Soil atmosphere with water vapour escapes from the soil pores and fresh air takes its place. In addition there is vapour pressure gradient which due to diffusion of gases is ever changing the soil atmosphere. It is, therefore, not essential that there should be a direct water connection from the ground water up to the ground surface for soil evaporation to take place. What actually happens is explained below.

With no soil evaporation taking place, as would be the case if the atmosphere above ground surface were fully saturated with water vapour—a state of equilibrium exists in the soil horizons between the capillary meniscus surface of the water table and ground surface. The vertical gradient due to gravity is, at all depths, exactly equal to the upward vertical gradient due to decreasing saturation and the corresponding gradient of negative pressure in the sacs of contact between soil particles. Thus, for a given soil, the effect of lowering the water table is to add more and more horizons of decreasing saturation and pressure.

When the atmosphere above ground is dry, this state of equilibrium is disturbed by the evaporation of water from the contact-sacs near the ground surface. The upward gradient due to decreased saturation and sac-pressure now exceeds gravity, the excess (or resultant gradient) being balanced by quasifrictional resistance to flow. With increased depth of water table the upward flow is, therefore, diminished not only by the increased length of the vertical column transversed, but also (a) by its increased specific resistance due to lower saturation, and (b) by the greater resistance to evaporation from the low pressure sacs near the surface, *vice versa* the rate of movement of moisture increases very rapidly as the depth of water table decreases.

In a soil crust containing 15 percent clay, Dr. Vaidianathan measured soil evaporation of about 5 percent of pan evaporation when the water table was 22 ft. below the ground near Lahore cantonment as described in Research publication Vol. 5, No. 3, Punjab Irrigation Research Institute. The author carried out observations involving a different technique to measure soil evaporation and found it to be 8 percent of pan evaporation in the month of December in the office compound of the watterlogging, Investigation Division, Lahore, where the water table was 21 ft. below the ground surface.

There is no limit to depth of the table below ground surface whence soil evaporation can extract water. The author considers that it may be even over one hundred feet because the medium causing removal of water (vapour) is soil atmosphere, which is ever changing due to seasonal temperature gradient in the soil crust, due to the rise and fall of the water table and due to the diffusion of the gases containing water vapour caused by vapour gradient.

Moreover there seems to be no limiting or minimum moisture content in a soil crust when the soil evaporation is likely to cease. Wilsdon's observation of the moisture content even measured as low as 3 percent in the soil crust near Lyallpur, Punjab Soil evaporation can extract the last trace of water in the soil, which in the American practice is called "Air Dry Soil".

9. Factors Affecting Soil Evaporation.

(*a*) Effect of temperature.

The effect of temperature is very predominent in the case of surface evaporation but in the case of soil evaporation the diurnal and seasonal temperature variation above and below the mean is important in producing temperature gradient in the soil crust as well as in the soil atmosphere. The theory is explained in para 12 (A), Chapter I of this part. The temperature gradient in the soil atmosphere results in its seasonal and diurnal renewals of soil atmosphere along with the water vapour. The range of temperatures in the Punjab plains is very great. Extreme temperatures being about 130° F and 40° F in shade. The atmospheric temperature of 140° F in the sun is rather common. Such a high range of temperatures does not exisit in the United States of America or in Great Britain where the hydrologists have measured the effect of soil evaporation not to go below 8·0 ft. of the ground.

(*b*) Effect of vegetation.

All forms of vegetation, particularly forests shade the ground to a certain extent and consequently reduce the rate of soil evaporation.

Transeau gives the following relative rates of evaporation observed at Cold Spring Harbour, Long Island.

	Percent
Bare sand and gravel slide	100
Open garden plot with low herbaceous vegetation	80 to 100
Upper beach areas	80 to 90
Light forest and gravel soil	50 to 70
Dense forest with abundant undergrowth	35 to 40
Dense ravine forest with abundant herbaceous vegetation ...	13

Dense swamp forest with abundant undergrowth and water near surface	10 percent
Fresh water marsh	45 „

The evaporation was measured with Livingstone porous cup atmometers placed about four inches above the surface of the ground. Meyer recommends that considering the rate of soil evaporation from the bare ground surface at a given mean temperature as 1·0, the rate of evaporation of free moisture from the ground in grain fields may tentatively be taken as 0·8 ; for grass land 0·7 ; for light forests, brush and second growth 0·6 ; and for dense forests with abundant herbaceous vegetation from 0·2 to 0·4 (for American conditions in U. S. A.)

(*c*) **Soil evaporation opportunity.**

The opportunity for given rate of soil evaporation to continue is determined by the available moisture supply.

(*i*) **Effect of interception.** If rain water is not allowed to flow away by vegetation or field *wats* then it saturates the surface, the soil evaporation will thus be provided with an opportunity to exercise its effect.

(*ii*) **Effect of percolation.** If the soil is such that all rain and irrigation water percolates down to the water table, then soil evaporation shall have little opportunity to exercise its effect. Fortunately the soil evaporation shall have little opportunity to exercise it effects. Fortunately the soil crust in the Punjab is not sandy but contains clay percentage from 15 to 35 percent. The effect of rain water in the Punjab which is very seldom more than 3 inches a day and irrigation which has normal depth of water about 3 inches per watering, does not travel more than 2 to 3 feet before the soil evaporation and transpiration absorb the moisture.

(*iii*) **Effect of precipitation.** In the Punjab plains the normal mean rainfall is generally less than 25 inches in a year. About two third is in the rainy season (July and August) and the remaining one third is distributed through the rest of the year. It is only once or twice in any rainy season that rain is about 3 inches a day, otherwise it is usually in showers of about one inch a day. A part of the rainfall flows away into the surface drain and the rest just moistens the soil crust to a small depth but it is lost in soil evaporation before it can reach the water table.

Such favourable conditions for soil evaporation do not exist in the temperate climates of the United States of America or in Great Britain.

(*iv*) **Effect of the height of the capillary fringe.**

The Punjab soil crust is rich in alkaline clay and capable of supporting relatively high capillary fringe above the water table. For a few cases, the capillary fringe heights are given in table No. 1 in paragraph 12 of the chapter. The normal height is about 4·0 ft. and in some cases above 11 ft. The effective full saturation is available up to these heights above the phreatic surface of the water table. Above the capillary fringe, there are similar height of funicular and penduler water. This peculiarity of the soil crust in the Punjab provides soil evaporation opportunity of unprecedental nature unobtainable in the soils of other countries.

10. Effect of Fluctuationg Water Table on Soil Evaporation.

The water table is fluctuating in the Punjab considerably. The effect of fluctuating water table on soil evaporation has so for escaped the notice of investigators. There is a swing of water table as measured by the B. S. P. levels (paragraph 3, chapter III of this part) more than once a year in the waterlogged areas, to the extent of 2·0 to 5·0 ft. Similarly in the unwaterlogged areas where water table is deep, there is a swing of over 1·0 ft. in all cases during the year. The water table is sometimes rising and sometimes dropping. The actual swing in the effective full saturation below the capillary meniscus surface is about 5 to 6 feet in the first case and 2 to 3 in the second case. Fig. 4 (a) and (b) explain the situation.

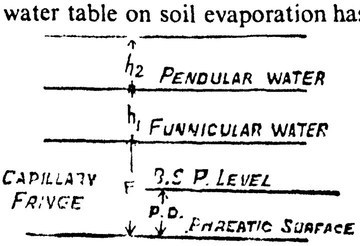


Fig. 4 (a)

The fluctuation of water table produces pulsation in the

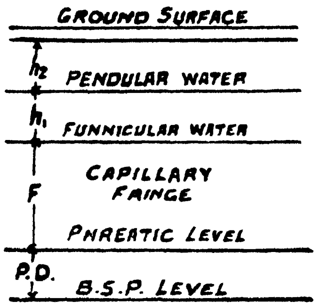


Fig. 4 (b)

soil atmosphere which escapes along with the water vapours and also alternately wets the dry soil above it. This is an important factor peculiar to the Punjab conditions causing increased soil evaporation.

11. Actual Measurements of Soil Evaporation.

The author carried out a large number of soil evaporation observations by developing the technique of hydrodynamic subsoil pressure observation as described in chapter III, part VI. The method was no doubt novel and used for the first time in the world as for as is known. The results are tabulted are table I below and plotted in Plate XX B Volume III.

The soil evaporation is more effective even than pan evaporation when the water table is at the ground surface. The fact has already been observed in America as shown in observations No 9 of the table. The soil evaporation can safely be taken to be 140 percent of the pan evaporation when the water table is at the ground surface. It decreases to about 50 percent when the water table is about 5 feet below ground surface and to about 8 percent when 20 feet below ground surface. The effect of soil evaporation is perceptible and measurable even when the water-table is about 50 feet bolow ground surface. These results shall, no doubt be surprise to the hydrologist in temperate climates but are true for the soil and the extreme climatic conditions of the Punjab.

TABLE No. 1

No.	Site.	Observer.	Spring level below N.S.	Soil evaporation expressed as % of pan evaporation	Depth of soil crest	Capillary Ht.	Clay %
1.	Lahore Cantt.	Dr. Vaidianathan	22 ft.	4·5	18 0	0·5	
2.	Lahore W. L. Divn : Office	Author	21 ,,	7·7	1·5	1·5	
3.	180,000 L C.C.	Author and T. Bench I S E.	5·0 ,,	50	13·0	3·0	35 to 40
4.	150,000 L C.C. left	Author	4 0 ,,	36	12·7	2·0	30
5.	do	do	3·0 ,,	42·6			
6.	do	do	2 0 ,,	46·3			
7	do	do	1·0 ,,	107·0			
8.	do	do	0 ,,	157 0			
9.	Coldwell Idaho U S A.	Meyer's Hydrology page 258	0 ,,	137 0			
10.	Chichoki Mallian	Author	4·5 ,,	57·8	16·0	3·5	30
11.	do	do	3·5 ,,	30·5			
12.	do	do	2·5 ,,	87 2			
13.	do	do	1·5 ,,	1·06			
14.	do	do	0·5 ,,	118 1			
15.	do	do	0·5 ,,	162·8			
16.	Hudiara Nalla	T. Blench I.S.E.	21·4 ,,	–	25·0	11·6	20
17.	Sheikhupura site	Author	5·5 ,,	24 0	14·0	4·0	30 to 35
18.	do	do	4·2 ,,	37·0			
19.	do	do	3·5 ,,	70 0			
20.	do	do	2·5 ,,	100·7			
21.	do	do	1·5 ,,	120·7			
22.	do	do	0·5 ,,	140·7			

12. Transpiration.

Definition. Transpiration is the process of vaporization of water from the breathing pores or stomata, of leaf and other vegetable surfaces. Most of the water used by the plants by the root action is used up in evaporation and only 10 percent is used to build up the tissues of the plants.

13. Factors Affecting Transpiration.

(a) Temperature.

Clements states that 95 percent of the light energy absorbed by the chloroplast of the leaf is converted into heat. Most of this heat is used in the vaporization of the water from the dilute solutions of mineral salts drawn into plants through the root system and used in building up plant tissue. The moisture retained in the plant tissues themselves is an inconsequential factor in the disposal of precipitation. Van't Hoff and Arrhenius have enunciated the principle that most chemical reactions and physiological processes double in activity for every similar increase in temperature. This law has been found by experiment to apply to a number of phases of plant activity. It has for example been found to be substantially correct for the rate of fixation of carbon dioxide by plants in sunlight; and in as much as transpiration occurs during the process of crabon dioxide assimilation, when the stomata open in the sun light, it is reasonable to conclude that the rate of transpiration, in so far as it is dependent on temperature, substantially follows Van't Hoff's law.

In applying this law, however, it is necessary to decide on a temperature at which plant activity begins. Koppen regards all monthly mean temperatures less than 48°C as included in the period of rest of plants. Other scientists hold that the protoplasmic contents of vegetable cells are inactive while the temperature is below 6°C.

(*b*) Effect of humidity.

All the experiments which have been made upon the water requirements of plant, for a given amount of growth, indicate that more water is used, per pound of dry material produced by plants growing in dry air than by those growing in moist air. From an engineering point of view the effect of relative humidity on the total amount of water used, rather than on the water used per pound of dry matter produced, is the result desired. This, however, does not appear to have been determined. It is probable, however, that the increased growth resulting from increased humidity, causes a total water loss in a humid atmosphere about equal to that in a moderately dry atmosphere, provided a reasonably sufficient amount of soil moisture is available for the plant to use.

(*c*) Effect of wind.

By hastening the removal of vapour from the leaf surfaces from which it is being transpired, air movement results in increased transpiration.

(*d*) Effect of light.

Transpiration is practically limited to the day light hours. In this respect it differs from evaporation, which continues throughout the 24 hours at a rate determined primarily, by the temperature. A graph is reproduced from A.F. Meyer's. "The Elements of Hydrology', to show hourly variation of transpiration rate in a day. (Fig. 5.)

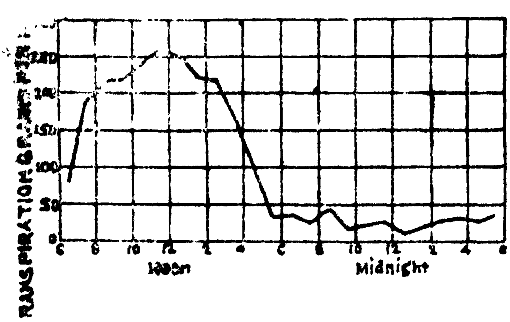


Fig. 5.

(*e*) Effect of soil moisture.

Most students of the subject of transpiration seem to be agreed that the quantity of water used by plants during the growing season depends mainly on the quantity available within reach of the root system It has been found that, in any given soil ; all forms of vegetation wilt when the moisture content is reduced to a certain percentage. This percentage however, known as the wilting coefficient varies greatly for different soils. Investigation of Bugg and Shauter Bureau of Plants Industry U.S.A., are shown in Fig. 6. (These are supposed to be laboratory observations on soil samples and not in a natural soil crust).

(*f*) **Effect of dry matter produced in the plant.**

Most experimenters have found that the quantity of water transpired by plants varies, approximately as the quantity of dry substance produced. The ratio of water used to dry substance produced has been found to vary with individual plants and with the plant environment. Conifers, in particular, have been found to use less than deciduous trees ; in fact ; some experimenters hold that they use less than one sixth as much. For grass and grain, the ratio of pounds of water used, to pounds of dry substance produced, seems to vary from about 300:1 to 600:1.

14. Amount of Transpiration.

(*a*) Meyer recommends that the following normal seasonal transpiration may be used as a base value estimating water losses for the north central portion of the United States ; 9 to 10 inches for grains, grasses and agricultural crops ; 8 to 12 inches for deciduous trees; 6 to 8 inches for small trees and brush; 4 to 6 inches for coniferous trees.

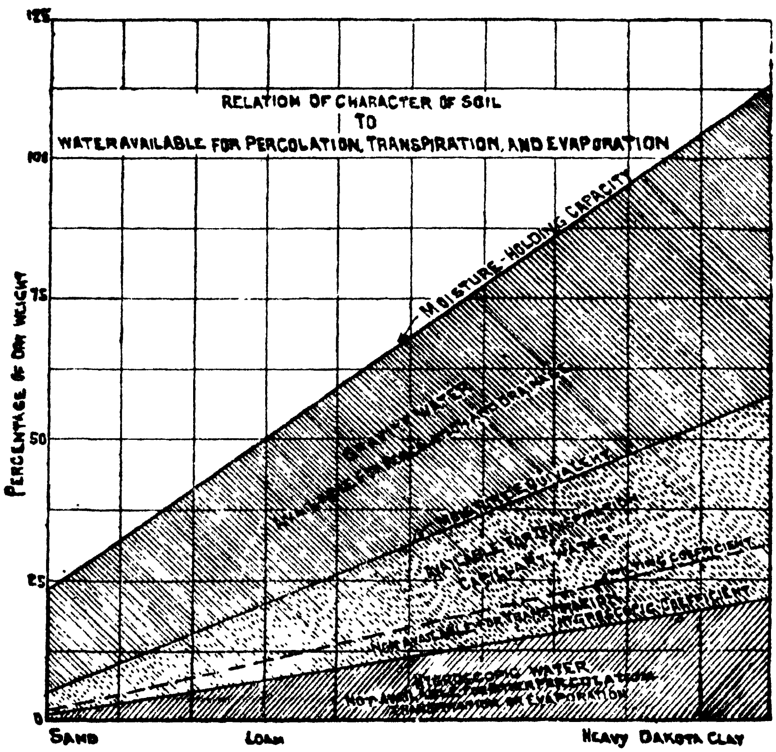


Fig 6

These quantities represent inches depth of water over the entire area occupied by the given from of vegetation. The monthly distribution of this total seasonal transpiration is determined mainly by the monthly mean air temperature as given in Fig. 7. These base values of monthly transpiration must then be modified for deficient or excess precipitation and ground-water supply in the soil occupied by the root system of the given form of vegetation; to ascertain the probable monthly transpiration under the given conditions.

As here considered, transpiration does not include interception, which is treated as one phase of evaporation from the land area.

(*b*) The consuption of water by vegetation, is given by Risler as below
Meadow grass from 0·122 to 0.287 inch daily
Oats „ 0·140 „ 0 193 „
Lucern „ 0·134 „ 0·267 „
Clover from 0·140 to 0·200 inch daily

Indian corn 0·110 „ 0·157 „
Vines 0·035 „ 0·031 „
Potatoes 0·038 „ 0·055 „
Wheat 0·106 „ 0·110 „
Rye 0·091 „ „
Oak trees 0·038 „ 3·035 „
Firs 0 020 „ 0·043 „

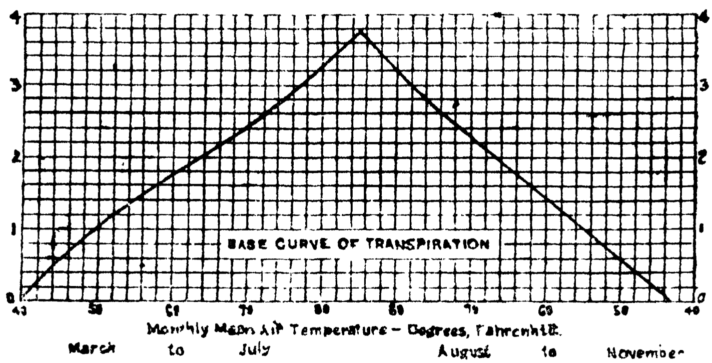


Fig. 7

(*c*) An estimate of transpiration water by *kharif* crops in the Punjab by Dr. E. Mckenzie Taylor in an investigation of the rise of water table in the Upper Chenab Canal Area is given below :

TABLE No. 2

Crops	Water transpired expressed as inches of rainfall or iraigation.
Cotton	12
Sugarcane	38
Rice	22
Maize	9
Millet	8
Fodder	14

PART V
GROUND WATER ENGINEERING
CHAPTER III
Ground Water Reservoir Movements

1. Introduction.

The source of all water is rainfall which in favourable circumstances (Chapter II) is absorbed in the soil, permeates underground and replenishes the ground water (Gravitational water). The ground water is held thereby the presence of the impervious starta below the underground reservoir. The retentive substratum is nothing but the sodium clay hardened by the heat in the interior of the earth globe which increases towards the centre of the globe. This chapter deals with the dynamics of the water table movements of the said water reaservoir. The conception of the problem as described in this chapter is essentially original as far as known. The experimental work was carried out by the author and written up in the author's technical papers, the list of which is given in the beginning of this book.

2. General Configuration of the Ground Water Reservoir.

If we consider the general configuration of the whole water table of the Punjab as presented by a contour-plan prepared from any one of the periodical observations of well levels, and the inference to be drawn therefrom, the first point to notice is the absence of any closed contours. If (as has already been done in the Punjab Irrigation Research Institute for the Rechna Doab), we make a three dimensional model of the water table, the surface so produced gives a clear picture of the direction of flow of ground water at the surface of the water-table at all points. The direction and the rate of flow would be clearly indicated by the movement of a drop of fluid applied at any point of the model. It is particularly to be observed that nowhere on the model would fluid drops applied in this manner collect in pools. All fluid would ultimately flow off the surface of the model leaving it clear of fluid. The inference safely to be drawn from this fact is that everywhere the water table is in a state of flow.

If the observation wells (supplping the data for shaping our model) were sufficiently numerous, a meticulously shaped model would reveal the existance of a network of elevated ridges, which would be found to correspond exactly with the alignments of irrigation channels, thereby furnishing a vivid proof of the fact that the ground water stream is fed and nourished by seepage losses from canals. Along the alignments of rivers, our model would exhibit both ridges and troughs. On each river above a certain point would be found a continuous troughs or valley ; below this point we should observe a continuous ridge, as for canals, but far more pronounced. The model would clearly indicate how, in their upper reaches our rivers are generally draining the subsoil stream and in their lower reaches, near the outfall, are feeding it.

A still more illuminating picture of ground water conditions would be presented if we moulded our model from a transparent and highly viscous fluid placed in an open ended tray of considerable depth compared with the surface irregularities of the model. In order to maintain the initial surface configuration of such a model, it would be found necessary, not only to introduce and maintain a steady stream of fluid at the upstream end of the tray, but also continuously to add fluid along the ridges, and to abstract it along the troughs. Furthe-, to avoid any alteration of the shape of the surface not only would all additions and abstractions have to be steadily maintained, but their distribution all over the model would repuire to be uniquely adjusted. Once this complex adjustment of "sources" and "sinks" was effected, the flow throughout the whole bulk of the model would (in the technical sense of the world) become steady. In particular, the surface of the model would present a fixed appearance in spite of the fact that all the fluid of which the model is composed is in a state of flow. Briefly stated, the water table would be exactly stabilized. The nature of the flow could be studied in detail by colouring the fluid and observing the rate and direction of flow at any chosen point. An important and interesting disclosure would be the placed uniform nature of the flow deep down in the bulk of fluid. It would be seen that, whereas at the surface the flow is very varied in magnitude and direction, below the surface such variations would become less and less marked, untill, at no very great depth such variations would defy detection, and the flow would be sensibly uniform both in magnitude and direction, the latter being that of the general surface slope of the model below that depth. In the hydrodynamic sense, the pressure at any point in a mass of fluid is indicated by the level to which fluid would rise a manometer (*i. e.*, open ended pipe) inserted into the fluid and tapping it at the point under consederations. Taking of pressure in this sense, uniformity of flow connotes a corresponding uniformity of pressure. In a viscous fluid at rest, the pressure is the same throughout the fluid ; with uniform flow, the pressure is the same on any vertical plane normal to the direction of flow It follows that, in our model, in the vicinity of a ridge or 'source, the pressure down a vertical falls rapidly near the surface, then more and more slowly as we descend, until, when the region of uniform flow is reached, there is no further change of pressure. Near a trough or 'sink' the pressure changes in a similar manner, but in the reverse direction, rising inst ead of falling.

The above perssure picture applies equally to the subsoil water stream of a *doab* near a canal or river, the manometric pressure registered by a strainer-tipped pipe gradually sunk into the porous soil, changes rapidly at first and then more and more slowly until a constant basic pressure is recorded which does not change with further sinking of the pipe. This behaviour is of great practical importance in investigating absorption losses or river regeneration. The constant basic pressure is designated to be Basic Subsoil Pressure, or more briefly the B.S.P. For practical purposes much will depend upon the depth of the sinking required to attain constancy of pressure.

The importance of the B.S.P. lies in the fact that the difference in level between the water surface of a canal or river and the B.S.P represents the head, (H) say, promoting percolation flow from the channel into the subsoil in the saturated phase or *vice versa* and that, other things being the same, the percolation flow is directly proportional to this head.

3. Definitions.

(*a*) Phreatic surface.

Phreatic surface is defined to be the water surface at atmospheric pressure in the subsoil ground water. This is analogous to the commonly called spring level (level recorded in a pit or in an open shallow well) but in a soil crust, this surface is not open to the atmosphere in the true sense of the word 'Phreatic' as there is capillary water above it.

(*b*) Basic sub-soil pressure ; B.S.P.

B S.P. is defined in practice to be the monometric pressure registerd by a strainer-tipped pipe which has been progressively sunk along side a very deep standard pipe in the same or directly connected stratum, till it records a reading not appreciably different from that in the standard. The level, thus recorded, is almost influenced by the effect of the distoration of stream lines caused by the presence of a source or a sink. In rectilinear flow with no upward and downward movement of the water table the B.S.P. level in a B. S. P. pipe and phreatic, surface of the ground water reservoir are the same. A, B.S.P. Pipe responds to the water table,

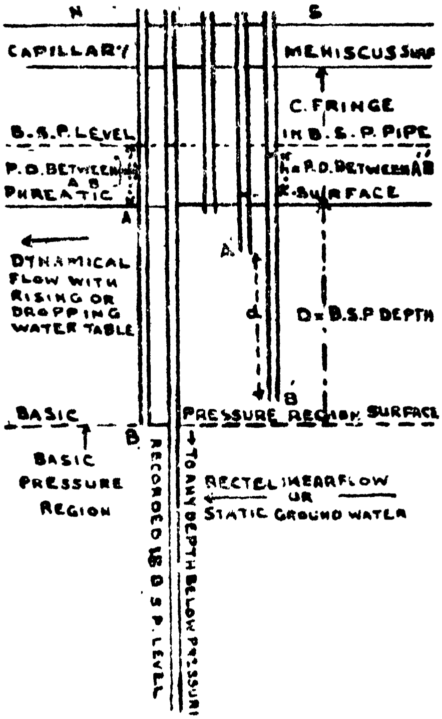


Fig. 1

changes unaffected by the pressure differences usually recorded up to B.S.P. depth in the ground water reservoir.

(*c*) **B. S. P. depth.**

B. S. P. Depth is defined to be the depth below the phreatic surface of the ground water reservoir where the flow is dynomically affected by the sources and sinks causing rising and dropping of the water table respectively and below which depth any two pressure tapping points how so ever vertically apart in a directly connected sratum record the same manometric pressure at any instance, see Fig. 1.

(*d*) **Capillary fringe.**

Capillary fring is defined to be the pressure transmitting zone above the phreatic surface. At any point in this zone, the absolute pressure is negative, that is, below atmospheric and is equal (for the static conditions of the water table) to the height of the point above the phreatic surface. The soil pores in the fringe are effectively saturated.

(*e*) **Capillary meniscus surface.**

Maniscus surface is defined to be the upper boundary of the capillary fringe. Immediately above it there is no pressure transmitting water. Above it are air atmospheric pressure and Funicular or Pendular water, held by the menisci round the points of contact of the soil particle.

(*f*) **Pressure difference (P. D.)**

In working with a U-tube manometer (with suction or otherwise) it is given by the difference between the readings of the limbs which are connected to two pressure tapping points below the phreatic surface up to B. S. P. depth. In the case of dropping water table or source, (canal contributing to subsoil) P. D. recorded is convertionally called positive and in the case of rising water-table or (a sink a seepage drain) it is designated negative.

Note : P. D. recorded by tapping points below the phreatic surface should not be confused with pressure deficiency (defined in paragraph 5 (C), chapter II of this part) which is also negative pressure below atmospheric and in partially saturated soil between capillary meniscus surface and the ground surface.

These terms should be taken to convey the meaning as defined above. These are practical Engineer's definitions although different soil physicist have varied conceptions of capillarity and the allied problems.

4. Experimental Evidence of B.S.P. Conception.

(A) **Procedure in observations.**

The procedure as adopted by the author in the case of observations at R. D. 299,000 Upper Chenab Canal is described below and same was followed by other observers.

A standard pipe of 1½ inch diameter and 45 feet in length, with a five feet strainer was sunk both on the berm and in the centre of the canal at R. D. 299,000, Upper Chenab Canal. These were tested for responsiveness by filling in water up to their top The time of sinkage of water by one foot was noted. Time recorded was less than 15 seconds for head of 7 feet and 2·45 feet respectively in the case of berm and centre pipes. The variable pipe was a 3" diameter W. I. pipe. It was sunk down to the water level in the standard pipe to start with. Water level in the variable pipes rose higher than that in the standard pipe, 72 hours were allowed for the water level in the variable pipes to adjust itself. The variable pipe recorded 0·55 feet and ·75 ft. higher than the manometric levels in the standard pipes respectively, in the case of the berm and the centre experiment. The variable pipe was then sunk a foot every day after the morning observations and 24 hours were allowed for the water in the pipe to adjust itself.

It is necessary to have a standard pipe for comparison because there is considerable variation of the B.S.P. in the standard pipe during the cou^rse of an experiment, due to natural causes such as rain etc.

It was found necessary to observe the following precaution while sinking the variable pipe every day :—

(*a*) While dredging earth or sand from inside the pipe, the water level in the pipe should not be allowed to drop appreciably. If the head into the variable pipe is allowed to increase to say more than a foot, sand may rush in. It will not only cause cavities around the pipe but also a great difficulty will be experienced is its sinking.

(*b*) Dredging is bound to disturb the water level in the variable pipe. After the pipe has been sunk the water level in it should be made up to the morning observations.

(*c*) Generally it will be enough to allow 24 hours for the water level to adjust itself in the variable pipe. If the pipe be in clay soil, more time should be allowed, say 48 hours, or even 72 hours.

The temperatures were recorded by means of a maximum and minimum thermometers at the bed of the canal, which is in this case, a "source".

Statement showing B.S.P. observations **Table 1.**

S. No.	Name of place	B. S. P. Level on standard pipe	Base of standard pipe.	Bottom of variable pipe	P. D. on 1st day.	Depth of progressive sinking of variable pipe to record same as B. S. P.	Observer	Remarks.
1.	299,000 U.C.C. Berm	666·41	628·50	666·41	Plus ·55	666·41−653·2=13·21 ft.	K.R. Sharma Esq.	
2.	RD. 299,000 U C.C. (middle of 200 feet) wide and bed=663·25	66·77	624·23	61·75	Plus ·75	661·75−647 75=14 ft. or 19ft. from B.S.P.	do	
3.	Merala Rest House.	796·2	—	794·2	Plus 1·0	794·2−780·0=14·2 ft.	Ram Lal Overseer	
4.	Tawan Rest House.	528·80	—	528·17	minus 0·17	528·7−514·7=14 ft.	Budh Singh Overseer	
5.	Sangwali Rest House.	735·7	—	735 7	Plus 1·0	735·7−707·7=28 ft.	Ram Lal Overseer	
6.	Khambarwal Rest House.	682·4	—	682·4	Plus 1·7	682·4−559·5=22·9 ft.	Khambarwala Overseer	

(B) Table 1, gives the results of experiments carried out to test the B.S.P. conceptions and observations are ploted in Fig. 2.

B. S P. Centre Experiment
R. D. 299,000 U. C. C.

B. S. P. Berm Experiment
R. D. 299,000 U. C. C.

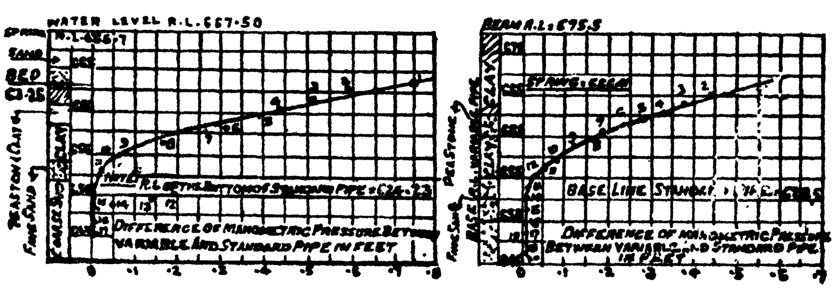
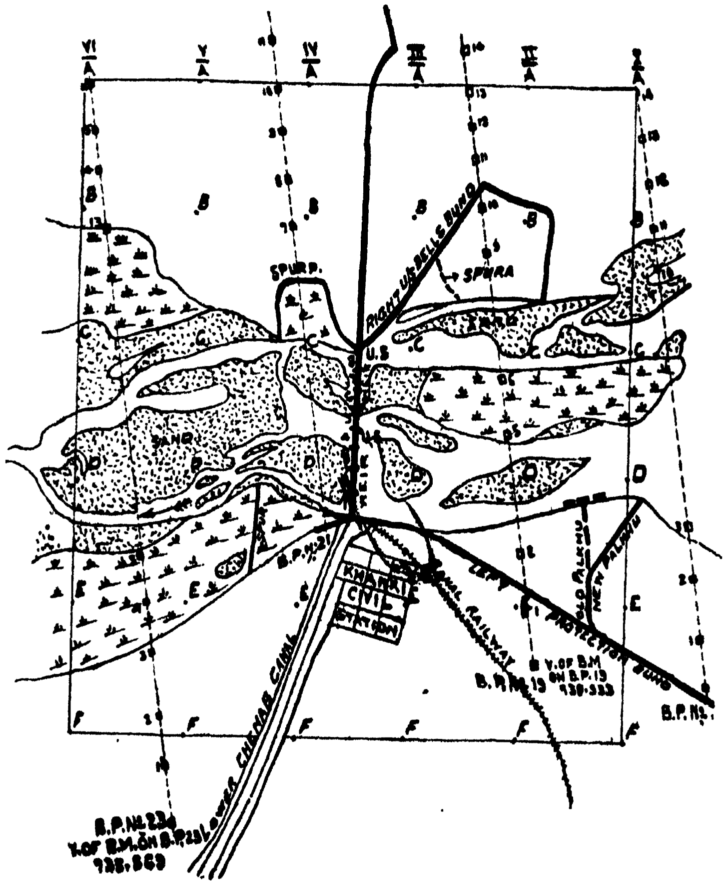


Fig. 2 (a) and (b)



054 (A)

Fig. 3

B. S. P. Contours in River Bed at Khanki

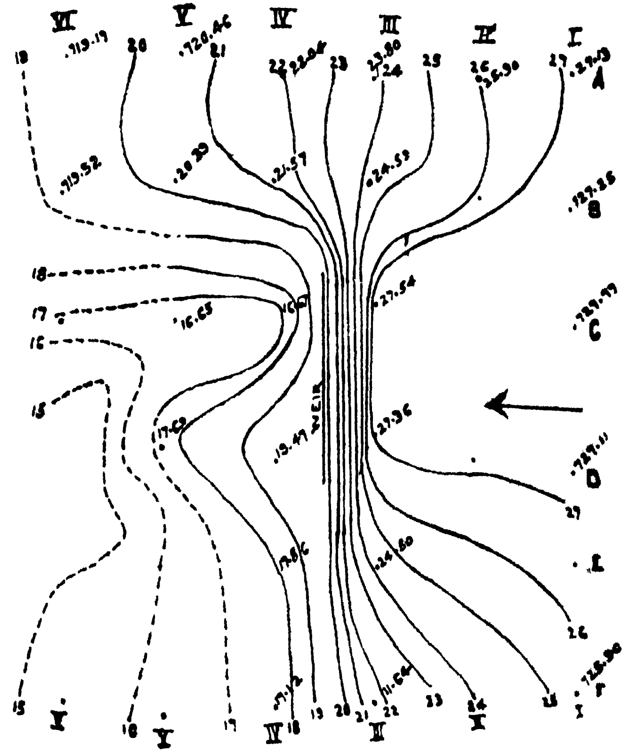


Fig. 4

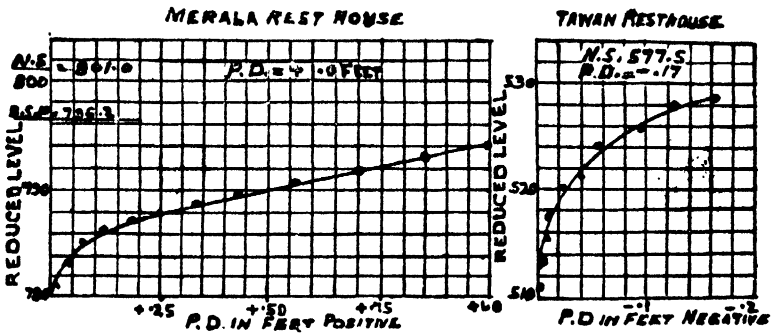


Fig 2 (c) and (d)

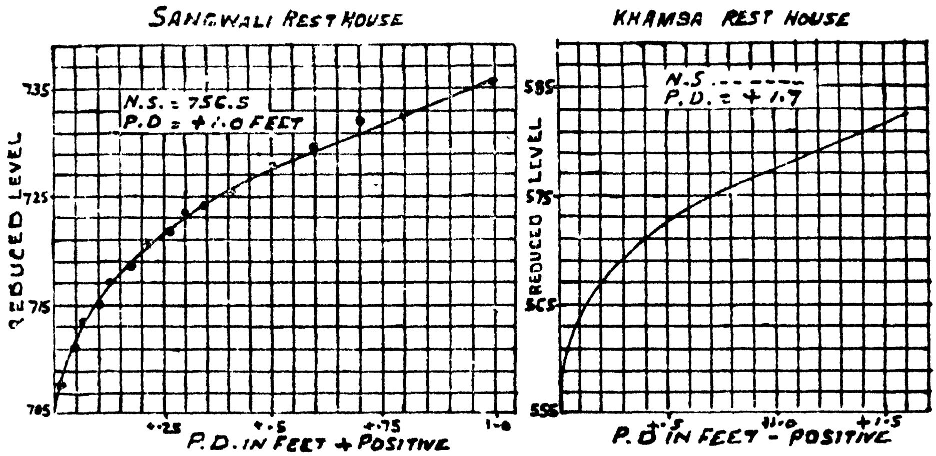


Fig. 2 (e) and (f)

The experiment No. 5 is not reliable as the overseer incharge simply went down to the bottom of the standard pipe. The author considered, that in the normal cases a depth of the sinking 20 ft. below B. S. P. level is enough to record the correct Basic Subsoil Pressure. This also shows that it is only the 20 feet depth of water of the ground water reservoir which is affected by the nearest sources of sinks. In the proximity of large canals the sinking should be about 25 feet.

The negative reading of P. D. on the first day in the case of the 4th experiment clearly shows that the water table was dropping under the effect of soil evaporation and the nearest canals and irrigation were not appreciably contributing to the water table.

(C) (*i*) Investigations were also carried out at Khanki Headworks which is a very complicated case of two different sources and a sink with the following objects :—

(*a*) In an area (the headworks vicinity at Khanki) where the conditions of infiltration and exfiltration are complicated by the great irrigularity in the shape of the water surface as seen in plan, and by there being three bodies of water at different surface levels to ascertain to what extent B. S. P. underlying the area is affected by these irregularities.

(*b*) At selected sites in the area to determine by progressive sinking of a well point, the pressure gradient of subsoil flow and to determine B. S. P. depth in each case.

(*c*) **To observe the B. S. P. contours crossing a river.** Fig. 3 shows the observation points and three sheets of water in plan at different levels, the pound upstream Khanki weir, river downstream of the weir and the Main Line L. C. C. and the B. S. P. contours given in Fig. 4. The first of these is a source to the water table, 2nd a sink or a drain causing regeneration in the river while the last appears to feed the substream during high supplies and to drain it in low surfaces.

(*ii*) **Progressive sinking tests upstream of the weir.**

The result of the observations at these sites are stated in table 2.

TABLE 2.

Experiment No.	Site	P. Max :	Depth of sinking to give Zero P. D.
1	II D	+ 2·24 ft.	736·4— 688·0 = 47·6 ft.
2	II E	+ 2 67 ft.	723·7—692·5 = 31·2 ft.
3	III D	+ 0·57 ft.	717·0—705·0 = 12·0 ft.

Pond loss at 285 level	1 32 cusecs 10^6 sq ft.
Pond area	–40 × 10^6 sq ft
Net loss	40 1 32 52 3 cusecs
Meximum pond level	32 5
Loss rate	1·94 cusecs/10^6 sq. feet
Pond area with level 32 5	90 × 10^6 sq ft
Loss at 32 5 pond level	90 × 1 94 174·6 cusecs
Increase in loss	174 6 – 52 8 121 8 cus

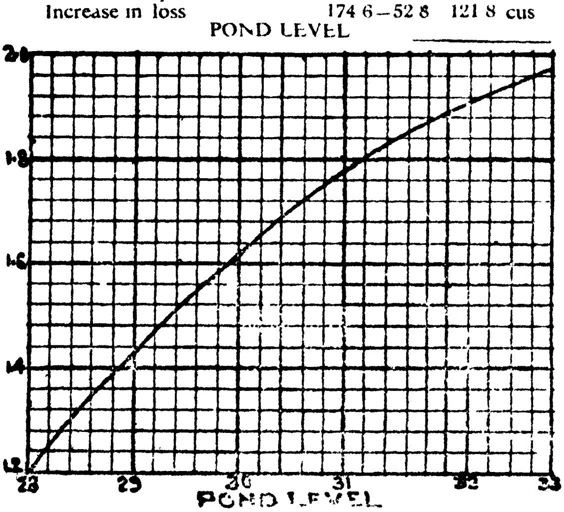


Fig 5 Observed loss intansity in cusecs/million sq. feet.

The positive pressure differences show that the Khanki Pond is a strong source feeding the sub soil. An average seepage loss intensity curve is plotted against the pond level in Fig. 5. This curve is based on the point method observations carried out at site as described in the author's paper No. 20. This curve gives the loss intensity at 20°C per million square feet at different pond levels This loss intensity multiplied by the surface pond area at different levels gives the repective losses in cusecs. The recovery of P. D in the beginning of the experiment is due to the soil evaporation at the site of the experiment, the effect of which will be to reduce the positive value near the surface of the water table.

(*iii*) **Progressive sinking tests downstream.**

The results of the observations showing the maximum pressure differences are tabulated below :—

TABLE 3.

Exp: No.	Site	P. D. (Max:)	Depth of sinking to record Zero P. D.
1	IV D	—0·86 ft.	705·5 – 691 = 14·5 ft.
2	V C	—0·18 ft.	716·0—706 = 10·0 ft.
3	VI E	—3·18 ft	707·5 – 683·5 = 24·0 ft.

In the last case the P. D. did not drop below—0·14 ft. and then there was discontinuity on account of impervious clay strata. The site VI E near the left side which shows that there was strong inflow from the sides. Near the middle, the inflow is light. Clay strata in the downstream sub-soil is known to be pretty stiff. The B. S. P. conception is only applicable to directly connected stratum ; the results of the last experiment were very much upset by the different layers of the clay strata.

Conclusions.

These experiments clearly confirm the B. S. P. conception and show that the phreatic surface of the water table is lower than the B. S. P. level in the case of an upward flow and is higher than the B. S. P. level in the case of a source of downstream of the weir was a strong sink. Negative pressure difference was recorded as low as – 3 17 ft as the site of the pipe VIF. The river upstream of the weir was a strong source causing downward flow. Positive pressure difference was recorded as high as + 2 24 at site of pipe II F.

These experiments clearly show how the B. S. P. contours cross a river. They do not cross straight but are influenced by the behaviour of the river as a source or a sink In the reaches of a river where river level is higher than the B S. P. level *i. e.*, where the river is acting as a source, the contours bulge downstream as shown in Fig 4 upstream of the weir. In reaches of a river where it is acting as a drain, the B. S. P level is higher than the river water-level and the contours bulge upwards as shown in Fig. 4 downstream of the weir.

(D) The hydrodynamic pressure observations described in chapter III, Part VI clearly proved the B. S. P. conception. They were carried with more accurate instruments such as Chatock Micro Manometer and the Benzine Differential Manometer.

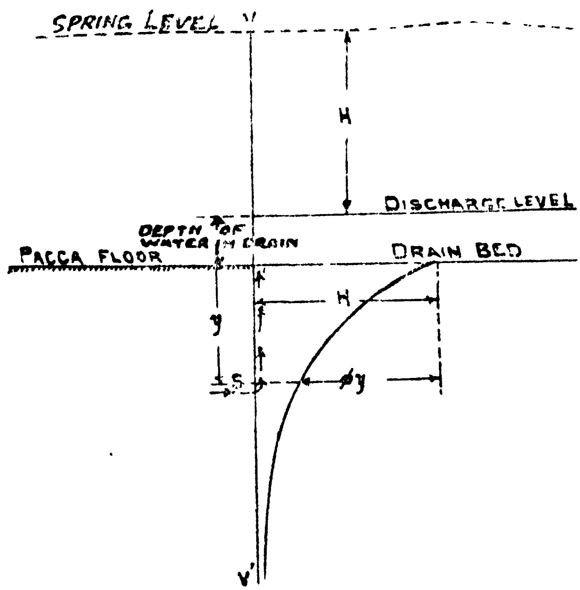


Fig. 6

(E) Khosla's observation.

From pressure observations at Jaurian siphon, R. B. A. N Khosla I. S. E. got a negative pressure curve causing upward flow as sketched in Fig. 4 Paper No. 138, Punjab Engineering Congress Lahore 1930. The sketch is reproduced in Fig. 6. In the terminology used in this book, the spring level shown there is the B. S. P. level and the water level in the drain is the phreatic surface. The pressure head (P. D.) difference between B. S. P. level and the drain water level causing flow is asymptotic to the vertical at a certain depth below the bed of the drain (sink). B. S. P. level is the manometric pressure level at the point of the water table where the pressure head (causing upward flow) curve tends to become asymptotic, Khosla's observations clearly confirm the B. S. P. conception.

5. Well Observations.

(*a*) Procedure in well observations.

A detailed list of the wells under observation in the Punjab is given in the list of wells maintained in each circle and the position of each is shown on the Well plan of the canal. Each well is distinguished by a Roman numeral or capital letter indicating the line of cross section to which it belong and below the Roman numeral or capital letter by a number (in Arabic numerals) showing its serial No. in the line of cross section. A cast iron plate is permanently fixed in the masonry. of each well, as near as possible, to the average natural surface level in the vicinity, bearing on it in raised figures both the cross section and the serial number of the well, and having a projection from which all measurements of the depth to the surface are to be made. These plates are of a standard pattern and are supplied on indent by the Superintendent, Central Workshops Division. After the plates have been fixed, the reduced levels of the projecting measurement points should be ascertained by levelling from standard bench marks.

Instructions. Report of well measurements by subordinates.

The depth of the surface of water in each well below the measuring point is measured twice a year, during the first week in June and the first week in October, by the sudordinate in whose section the well is situated In the case of village drinking wells the measurements are made either at daybreak, or in the ofternoon, that is, in each case, before the well is used or sufficient time after its working. In the case of wells used for irrigation the observer should endeavour, as far as possible, to take the measurement when the well is not being worked and at as long an interval as possible after the last working. If the well is not working at the time of measurement but has been worked during the previous 24 hours the interval which elapsed since it was last worked should be noted If the well be working or has only ceased working for a few hour the actual depth below measuring point should be recorded, but the observer should endeavour to ascertain by local enquiry what reduction in this depth would occur if the well were not worked and should separately propose for acceptance this modified depth.

Register of measurements.

A register is maintained in the Divisional and Superintending Engineers' offices in the prescribed form The reduced levels of the water surface in the well are worked out. The wells are checked by the Sub-Divisional Officer as underlined.

(*b*) **Well lines.**

The wells observed in the Punjab are shown in Fig. 7. They are on provincial well lines. In addition there are lot of local well observations made to watch the effect of anti-waterlogging measures such as drains and pumping schemes. A section across provincial line No. 1 is given in Fig. 8 and longitudinal section of the wells observed in *Rechna Doab* along the line shown in Fig. 7 is given in Fig. 9. The cross section shows the well observations from 1908 and the subsequent rise the opening of the canals in the various *doabs*. In some cases water surface in the wells is still rising and in some, in the water-logged areas they have become stablished. The subject is discussed in detail in the Chapter on water-logging.

(*c*) **Difference maps.**

A convenient method of exhibiting the well observations in the form of difference maps is explained below. A typical map for the Upper Chenab Canal Area is shown in Fig. 10. These maps show the rise in well observations as distributed over the tract, how far it extends and how it is spread over the various periods. The Fig. 10 relates to the period from 1911 to 1935. The Upper Chenab Canal was opened in 1912.

(*d*) **Water surface in wells.**

The well observations described above are taken in open wells 10 to 12 feet diameter which are sunk 10 ft below normal spring level. They are supposed to record the spring level. Since the wells are open and shallow they cansafely be considered to record the phreatic surface of the ground water reservoir. A reference to Fig. 11 and 12 of this chapter will show, that the phreatic surface is not a fixed entity. It changes with the conditions of flow in the water. If the water table is rising, it is below the Basic Subsoil Pressure level and if the water table is dropping, it is above the basic soil pressure by an amount which represents the pressure difference (P.D.). Head lost in this change can be as much as couple of feet in clayey soils. This adjustment of phreatic surface level takes place by mere pressure adjustment without any outside additions and subtractions of water. The actual spring level does actually vary in a pit dug at the same place according to dynamic movements of the water table. Commonly, water table is understood to mean the water surface recorded in the open wells. Phreatic surface or spring level cannot be correctly measured by digging pit in the soil crust on account of soil evaporation.

(*e*) **Water equivalent of well rise.**

A foot rise in the well level does not mean that a foot depth of water has been added to the subsoil reservoir. The actual amount of water needed depends on the effective soil porosity which should be estimated according to the crust thickness, the rainfall and the constitution of the soil. The rase of rise of water table multiplied by the effective soil porosity gives the water equivalent for the well rise if the soil is already wet upto hygroscopic moisture content (scientists) wilting coefficient (agriculture engineers), the actual amount of water required shall be further reduced.

Let porosity be 40 percent by volume and let 15 percent by volume be the hygroscopic moisture content, these are the normal values in the soils crust of the Punjab. A well rise of one foot is then $\frac{40 \cdot 15}{100} = 0{\cdot}25$ ft. $= 3$ inches of water.

6. Subsoil Flow (Darcy and Hazen).

(*a*) Passage of water through a layer of sand or fine gravel is effected by capillary flow through the small irregular tubes that are formed by the void spaces in the sand. It is, therefore, necessary to consider the laws of capillary flow.

Critical velocities in Reynolds experiments being

First, $v_c = \frac{0{\cdot}0388P}{D}$ (A) Second, $v_d = \frac{0{\cdot}2458P}{D}$ (B)

The capillary motion of water is essentially the motion of water through pipes at velocities, which are less than the critical velocity (stream line flow).

Let h be the pressure in ft. of water producing such a flow through a pipe l_1, feet long and d_1 feet in diameter, the velocity of the water in feet per second is given by Darcy's formula.

Velocity of flow $=$ K $\times$ Slope of subsoil water surfrce

$V_1 = 52{,}100\ fd_1^2\ (h/l_1)$ feet per second. (C)

The flow is inversely proportional to the viscosity (Poiseuile's Ratio) with the temperature change.

Poiseuille's Ratio.

$P = 1/(1 + 0{\cdot}0337T + 0{\cdot}000221T^2)$; where T is degrees centigrade

but $f = 1/P = 1 + 0{\cdot}0337T + 0{\cdot}000221T^2$; where T is degrees centigrade.

or $f = 0{\cdot}474 + 0{\cdot}0147t + 0{\cdot}0000682t^2$; where t is degrees fahrenheit

V_1 must not exceed $\frac{0{\cdot}039}{fd_1}$ feet per second, or the flow may cease to be capillary.

Now, in sand or gravel the length l_1 cannot be measured, but on the average, it is a certain multiple of the length of the path of percolation through the sand. Similarly d_1 cannot be measured, but bears, on the average, a certain ratio to the mean diameter of the grains of sand. Consequently, the expression of the quantities contained in the above formula in terms of quantities which are easily measureable, can only be effected by certain assumptions concerning the average values of the ratio l/l_1 and d/d_1 ; where l represent the length of the path through which percolation occurs, and d represents some measureable quantity (say the mean diameter of the sand grains) which is proportional to d_1, the average diameter of the interestital passages through the sand.

The percolation properties of sand or gravel are best defined by the quantities known as the effective size and the uniformity co-efficient. The sand can be separated into grades by sifting through sieves and if the sizes of the holes in the successive sieves are sufficiently close together the diameter of all the particles in a grade will be approximetely equal.

The effective size is defined as the mean diameter of the grain such that 10 percent (by weight) of the sand is composed of smaller particles, and 90 percent of larger particles.

The uniformity co-efficient is the ratio which the mean diameter of a grain such that 60 percent (by weight) of the sand is composed of smaller particles, bears to the effective size of the sand.

It is plain that the effective size is an approximate measure of the mean size of the smaller grains of the sand, and that the uniformity co-efficient is an indication of the ratio between the sizes of the larger and smaller particles of the sand.

It is also plain that these two quantities cannot be regarded as rigidly specifying the properties of the sand, and that their pratical importance is solely due to the fact that sands and gravels as they occur in nature are approximately similar substances, so that a coarse sand may be regarded either as a magnified small sand, or as a fine gravel on a diminished scale.

Subject to these remarks, Hazen (Filtration of water) has found experimentally that the effective velocity of percolation is given by ;

$$v = Cd^2 \frac{(t+10)}{60} \cdot \frac{h}{l} = \frac{kh}{l}; \text{ where } K = Cd^2 \frac{(t+10)}{60} \qquad \text{(D)}$$

where v is the equivalent velocity at which the water passes through the sand, *i. e.*, v is not the velocity of the water in the pores of the sand (which is denoted by v_l) but is the velocity of a solid column of water of the same area as that through which the percolation occurs which delivers the quantity of water which actually percolates through the sand.

l is the length of the path along which the percolation occurs and h is the head producing percolation measured in feet.

d is the effective size of the sand.

The expression $(t+10)/60$ is approximate representation of the factor denoted by f theoretically more accurate expression for which has already been given; C is a constant depending on the units employed, if v is expressed in feet per 24 hours, and d in millimeters, C=3280. If d be expressed in hundredths of an inch, C=210. v could also be expressed in feet per second, but C would then become an inconveniently small fraction. The equation is subject to exceptions. For example, if p be the percentage of voids in sand, it is plain that $v = 100 (v_l/p)$ where v_l depends on d and (h/l). The porosity in sands usually varies from 25·6 to 44·6. Thus we may infer that the general form of the grains alone may cause v to vary as much as 20 percent either way. In practice, however, the equation is found to apply with fair accuracy to sands occurring in nature, in which d lies between 0·10 mm. (0·004 inch) and 3·00 mm. (0·12") and with a uniformity co-efficient less than 5. The equation also applies with equal accuracy in gravels up to 5 or 7 mm. (0·20 to 0·28 inch) effective size, as long as $v_l = 100 (v/p)$ is not too large. In this last case the limit at which the equation ceases to hold is fairly accurately ascertained by estimating the critical velocity, v_c in a tube of a diameter equal to 4d/7.

In most cases we can take t=50° F, and then the bracketed expressions becomes unity.

We can also express this formula in terms of d_m the mean diameter of the sand grains, with a very small degree of error, by changing the value of C in the ratio $(d/d_m)^2$ in ordinary sand Since $d_m = d\sqrt{3}$, we have $C_m = C/3$ and actual values as given bo Seelheim, and Krober, are as follows :—

d_m in millimeteres 0·16, 0·23, 0·28, 0·41, 0·54, 0·78, 0·70, 0·90, 1·35, 2·1.

C_m in feet per 20 hours 1066 1047 1016 1076 1205 1063 1158 1395 1030 1165.

All these experiments were conducted on clean sand, such as is used in filters. The following results show the influence of a small quantity of clayey or dirty matter in diminishing percolation. The experiments were recorded in terms of the effective size.

TABLE 4.

d in milli-meters	Value of v in feet per day as ascertained experimentally when $h/l=1$.	Value of K for sand of effective Size according to Hazens rule	Percentage of flow with dirty instead of clean sand
0 55	758	991	76
0 46	154	695	22
0 45	30	663	5
0·45	92	663	14
0 40	131	525	25
0 38	49	472	10
0·57	36	449	8

Athough the information is incomplete, it seems as well to record the values of Cd^2 or $C_m \times d_m^2$, obtained when $h/l = 0\ 037$ in the alluvial deposits at :—

TABLE 5.

Lyons	Strassburg	Gladbach	Augsburg	Vienna	Bucharest
545	1511	701	1180	273	403

In Hezen's formula the conception for temperature is probably somewhat faulty and the Poiseuille ratio used in investigations of critical velocities in pipes is a more accurate method of allowing for the influence of temperature. Hazen's results are approximately correct for a range of 50 to 70 degrees Fahr.

TABLE 6.

	Effective size of the sand grain in Millimeters.							
h/l	0·10	0·20	0 30	0 35	0 40	0·50	1·00	3·00
0 001	0 033	0 13	0 0	0 41	0 524	0·82	3 28	29 5
0 005	0 164	0 66	1 48	2 06	2 62	4 10	16·40	147·6
0 010	0 328	1 31	2·96	4·12	5 24	8 2	32 8	295 0
0 050	1 64	6·56	14 8	20 6	26·2	41 4	164·4	—
0·100	3 28	13 1	29 5	41 2	52·5	82 0	328·0	—
1 000	32 80	131 2	295·2	411·8	524 8	820 0	-	-

This table gives the value of K in the for nula $v = K\ (h/l)$ for $t = 50°F$

so that $K = Cd^2 \frac{(t + 10)}{60}$.

A table of values of K when $t = 0°F$ in terms of the number of meshes per lineal inch of a sieve that retains 10 percent by weight of the sand (*i. e.*, Cd^2 expressed in terms of the mesh of the sieve) is given below :—

TABLE 7.

Number of meshes per linear inch.	K in feet per day.	Effective size in mm.	Number of meshes per linear inch.	K in feet per day.	Effective size in mm.
6	50000	3 9	55	430	0 32
8	32000	—	60	333	0 24
10	13500	2 04	70	190	0·22
12	10500	1·52	80	160	0 20
16	5800	1 10	90	130	0 18
20	3000	0 96	100	105	0·155
24	2500	—	120	80	0·135
30	1650	0 70	140	60	
40	700	0·46	150	55	
50	500	0·39	200	40	

7. Subsoil Flow (Authour's Conception).

(*a*) In the preceding paragraph some attempts have been desribed to predict the effective area of capillary viscous flow in sands, based either on the porosity or the size of the sand particles. Very little experimental work exist which would ensure the application of these flow equations to an actual soil crust containing clay admixtures. Hazen's work was very useful to predict the flow in the tube-wells which have their strainers in sand-bearing strata of the ground water reservoir but the problem is much more complicated for the Engineers confronted with the waterlogging of a soil crust which is fertile and productive when it contains a decent mixture of clay. The usual attempts to pick up soil samples and then to determine the transmission constants in laboratory by artificially packing the soil in a tube are not only very erroneous but also tedious This method can never give the correct transmission constant of the actual soil crust The mathematical attempts evolving three demensional mathematics (Dr. Bose Memoir Volume II, No. 1) supposing an impervious boundary layer at the bottom have not yet been able to measure the actual flow in the soil crust. B.S.P. conception enables one to measure in the field the actual flow in a soil crust as explained on next page.

(*b*) The student is now in a position to appreciate that in the rising water table, the phreatic surface is below the B.S.P. (Fig 11) by the head causing upward flow. The pressure difference causing upward flow at the point "A" is H and let it be "h" at a point B. D. ft. below A. The method of pressure difference observations by successive lowering of the standard pipe has already been explained in paragraph (4) above. Let the rate of virtual flow in the water table be say q cft in 24 hours and H and h being the average of, say hourly P D. readings for the same period. If flow the considered taking place through unit area having porosity "e" percent then $e.q = \sigma(H - h)$

or $\sigma = e.q/(H - h)$

If $e = 40\%$, $\sigma = \left(\frac{5}{3}\right)^3 \frac{q}{H-h}$ cusecs per million square feet.

If $h = 0, \sigma = \left(\frac{5}{3}\right)^3 \frac{q}{H}$ cusecs per million sq. ft. (E)

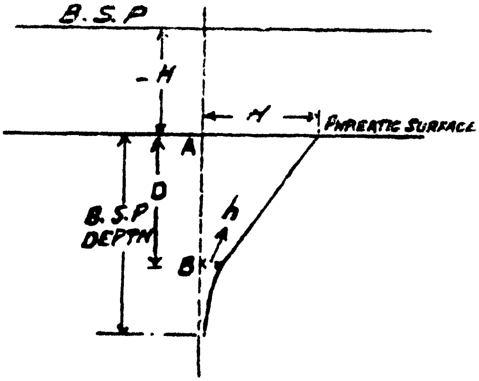


Fig. 11.

In the case of B.S.P. pipe observations, q represents the rise in pipe in 24 hours considering flow through unit area. σ is called percolation intensity coefficient (P.I C) of the soil crust between points A and B. Its value will be affected inversely as the viscosity and therefore, the observation of the temperature of ground water should always be taken in the B.S P. pipe by suspending a maximum and minimum thermometer and its values corrected to 20°C as the standard mean temperature using Poiseuille's Ratio Table No. 1 of next chapter or the graph in Plate XXI. It is expressed in cusecs per million square feet at 20° C. Darcy's transmission constant could also be calculated from these observations as below :—

$v = KS = K(H - h)/D$; Discharge $= v.A$, let A be unit square feet,

Discharge $= v \times 1 = q \times e$ (if 40 percent pore space). $\therefore K = qD/[60^3 (H - h)]$ (F)

where q = rise in ft. in 24 hours.

In an actual observation taken in the Water-logging Investigation Division office, Lahore, it was observed by the author.

B.S.P. change in 24 hours average = 0·01 ft. ; The average P.D. = (H—h) = 0·103 ft. B.S.P Depth = 15 feet ; Pore space = 40%.

then the percolation intensity co-efficient = 4·63 × (0·01/0·103) = 0·45 cusecs per million sq. ft. The transmission constant ; $K = \frac{0·01 \times 15}{60^3 \times 0·103} = 0·67 \times 10^{-5}$

In the case of a droping water table ; the phreatic surface will be higher than the B.S.P., there will be downward pressure gradient as shown in Fig. 12.

The percolation intensity coefficient and the transmission coefficients could be calculated as before.

(*c*) The flow picture of the ground water down the *doab* can now be visualised (Rising water table) Fig 13.

(*i*) Let there be two sections $A_1B_1C_1D_1$ and $A_2B_2C_2D_2$ separated by a distance equal to l. At the first section the distance A_1B_1 represents the pressure difference. (P.D.) causing upward flow from C_1 to B_1 resulting in a certain rise of water table phreatic surface. Below C_1, the manometric pressure that recorded is constant which means that equipotential lines are very nearly vertical. Similar conditions are available at the second section $A_2B_2C_2D_2$. Below

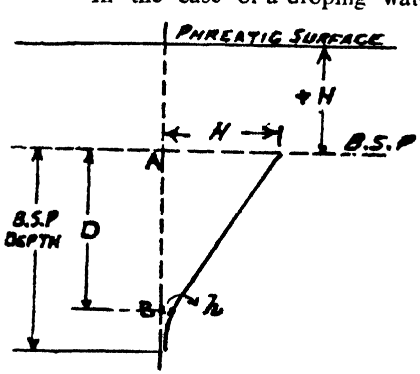


Fig. 12

the line C_1C_2, there is no pressure difference vertically which means there is no change of pressure. In a viscous mass of liquid at rest, the manometric pressure at any point along the full depth is the same. Also in uniform viscous flow, the pressure along the line orthogonal to the stream lines is constant. In uniform viscous flow below the line C_1C_2 the stream lines will be parallel to C_1C_2 and the equipotential lines will be at right angles to C_1C_2. If slope of the line C_1C_2 is small, the equipotential lines will be very nearly vertical. The constant pressure observation below C_1C_2 indicate that the flow below is either uniform viscous flow or the liquid is at rest. If the slope between the points C_1 and C_2 is zero, then the water table below C_1C_2 is at rest Usually there is a slope from C_1 to C_2; which show that there is uniform viscous flow below C_1C_2 and it extends to an unknown depth but the discharge q reaching the first section below C_1 is the same as that leaving the second section below C_2. Whatever be the discharge passing below C_1C_2 it does not effect sub-soil water inventory of a waterlogging Engineer. All additions or subtractions of water to the water table shall be shown and accounted for by the flow above C_1C_2. A water logging Engineer is least interested in the ground water flow below C_1C_2 but the flow there is all important to a water supply Engineer who is out to investigate the strata which shall provide enough yield for his wells.

(*ii*) Now the student will be able to appreciate the flow above the line C_1C_2; it is non-uniform viscous flow. The author prefers to call it differential flow. If the water table is rising, the water is being used up in the way to the extant of this rise, the discharge Q_1 at the first section will be greater than Q_2. If T seconds be the time taken in flow from the first to the second section, the volume of water available to raise the water table is $T(Q_1-Q_2)$. If water is falling then Q_1 will be similarly less than Q_2.

In the case of a rising water table as sketched in Fig. 13, the phreatic surface slope at the first section shall be steeper than that along C_1C_2.

$$\text{Slope } B_1B_2 = \frac{x+(P_2-P_1)}{l}\ ;\text{Slope of line } C_1C_2 = \frac{x}{l}$$

$$\text{Average slope} = \tfrac{1}{2}\left\{\frac{x}{l}+\frac{x+(P_2-P_1)}{l}\right\} = \frac{2x+(P_2-P_1)}{2l}$$

According to Darcy's formula

$$v = KS = K\,\frac{2x+(P_2-P_1)}{2l}$$

GROUND LEVEL

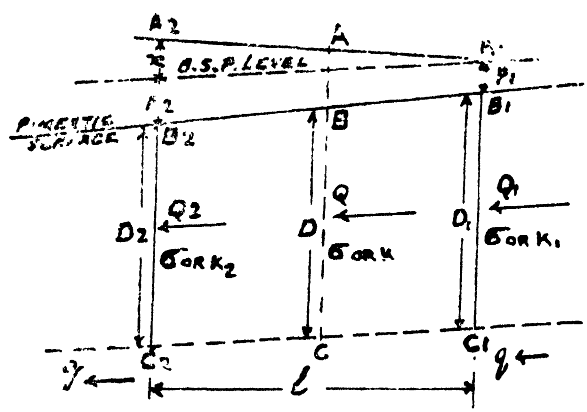


Fig. 13.

Discharge $= Av$; Area $= D\times l = D$ sq. feet (considering unit width)

$$Q = K\,\frac{2x+(P_2-P_1)}{2l}\,D = K\,\frac{D}{2l}\left\{2x+(H_3-h_1)\right\} \qquad \text{(G)}$$

where $K = \frac{K_1 + K_2}{2}$ transimission contant ; $D = \frac{D_1 + D_2}{2}$ = B. S. P. depth

x is the Difference between B. S. P. levels ; P_1 the P. D. at the first section ; P_2 the P. D. at second section.

This can be put in still simple from according to the author's conception of the percolation intensity co-efficient. The average Head causing flow $H ; = \frac{1}{2} [2x + (P_2 - P_1)]$

If $\sigma = \frac{\sigma 1 + \sigma 2}{2}$ = the percolation intensity co-efficient, then $Q = \sigma H$ cusecs per feet width of the *doab*. (H)

Thus by simple pressure observation of the ground water in the field in the actual soil crust, one is not only able to calulate the vertical flow but also the horizontal flow down a tract as affected by the various sources tending to raise the water table and the various sinks tending to lower the water table. The effect of soil evaporation is to cause upward flow in the ground water and to abstract the water thereby lowering the water table This will tend to water, the positive P. D. observed in the of accretion resulting in the rise of the water table and in the case of dropping water table due to decretion in the ground water flow it shall incerase the negative pressure differences observed. If soil evaporation is not effective, the pressure differences observed are simply due to accretion and decretion in the ground water reservoir flow and if effective ; the soil evaporation can be quantitively evaluated from the observed pressure difference. The method of calculations is given in Chapter III, Part VI.

8. Other Methods of Determination of Permeability Coefficient.

(A) Theim method.

This method, properly executed, permits the determination of the co-efficient of permeability of the underground in the field and thus possesses certain advantages over laboratory methods where the permeability is determined on a small sample and the results often later applied over an extensive area.

The method consists sinking a well casing into or through the pervious stratum for which it is desired to determine the co-efficient of permeability, inserting therein a well pump and operating the pump until the undrground water surface becomes practically constant and then taking the measurements required by the formula.

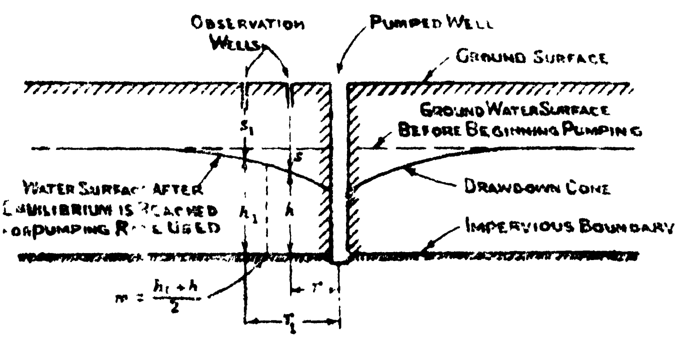


Fig 14 Factors in Thiem Formula for Denterminiing Co-efficient of Permeability from Field Pumping Tests

The observation wells generally consist of small-diameter (say 1¼ in.) perforated pipe or well points. In general, observation wells would be for enough from the pump well to avoid the steeper portion of the drawndown curves, which are the hydraulic gradients of the various filaments of flow. Usually a considerable number of these observion wells (10 to 50) are sent on various radii from the pumped well because with a number of such wells a better picture of the average permeability of the underground may be obtained.

The pump well casing should seldom be less than 12 in. in diameter, and larger sizes up to 24 inch may be advantageous. When pumping is started the water table draws down around the pump casing is the form of an inverted cone (see Fig. 14 . Eventually, unless the pumping capacity is excessive in relation to the permeablity of the underground, with steady pumping this cone become stable and the flow to the casing is then at the same rate as the pumping. It is at this point that the measurements should be taken. It is obvious that continuous operation of the pump is extremely desirable for accuracy. Therefore, a duplicate installation of pumping equipment is desirable, so that if something goes wrong with one pumping out it the other can be started It is not desirable to depend on suction pumps because the drawdown may possibly be enaugh to cause trouble with the suction before the cone reaches a condition of stability. Well pumps or centrifugal pumps which fit inside of the pump casing are suitable.

The Thiem formula modified to utilize units consistent with those used in this book is

$$k = \frac{q \log_{10} (r_1/r)}{20{\cdot}4m s - s_1} \qquad (1)$$

where k the permeability co-efficient in feet per minute (strictly cubic feet per minute per square foot on a 1 to 1 gradient) at a temperature of 54 F.

q the rate of pumping in gallons per minute. r_1 and r are the distances of two observation wells from the pumped well in feet.

m the average vertical thickness of the saturated portion of the water bearing bed between the two observation wells and as indicated in Fig. 14, is equal to $h_1 h$ 2.

s and s_1 drawdowns at the two observation wells in feet Fig 14, indicates the various quantities and measurements With a single installation the value of k may be determined between 20 or more pairs of observation wells.

(B) **Electrolytic determination of permeability co efficient.**

C. S. Slichter in his paper No. 140 published in U. S. Geol. survey water supply on "Field measurements of Rate of movement of underground water" described the method of determining permeability co-efficient in the field condition by determinations of the differences in electrical resistivity between the ground water in its natural condition and the same water when charged with heavy salt solution. W. J. Turnbull described the use of this method in determing the permeability of the functions of Kingsley Dam, on North Platte River, Nebraska as given below : -

For making the field tests, a master well consisting of an 8 ft. section of 4 inch diameter, perforated galvanized pipe was down into the sand gravel at Station 26 on the dam center line. Twelve observation wells were placed on the downstream (East) side of the master well ; one well being placed at a distance of 2·5 ft. six wells being placed along a circle of 5 ft. radius with the master well as the center, and the remaining five wells being spaced regularly along a circle of 10 ft. radius.

The bottom of 4 ft of the master well was perforated so as to alow free movement of water into and out of it. Each of the observation wells consisted of a 1¼ in. sand point and a section of galvanized iron pipe driven to a depth of 8 ft. The field setup of instruments was similar to that employed by Mr. Slichter. This setup consisted of a 6 volt storage bettery and a direct current ammeter connected in series with the master well casing and the casing of any one observation well. The ground then formed the closing link in the series circuit. Then as the movement of the ground water carried the electrolyte from the master well towards the observation wells, the resistance of the ground circuit decreased resulting in an increase to the ammeter current reading. The wire connecting the ammeter to the observation well casing was fixed so that it could be easily connected to any desired observation well. In addition to taking readings between the master well casing and the observation wells, the battery and ammeter were arranged so that they could be connected in series with the casing of any observation well and a center electrode placed in that well. This center electrode consisted of a 3 ft. brass rod with four wooden spools spaced evenly along its length to insulate it from the well casing. This electrode was also fixed so that it could be easily moved to any desired observation well. The reason for taking current readings within the observation well was to check the first procedure mentioned. In this latter procedure the time when the salt reached the well was indicated by a sudden increase in the ammeter reading. In the first procedure, the increase of the ammeter reading was slower, reaching a maximum when the maximum concentration of the salt reached the downstream well. For charging the master well, a cloth sack approximately 5 ft long and 3 inch in diameter and filled with dry salt was employed. The sack held approximately 10 lb. of dry salt. This salt charge was a 50-50 mixture of common salt ($NaCl$) and ammonium chloride (NH_4Cl). After filling the sack, it was tied at approximately every 6 inch point to prevent the salt from settling to the bottom of the sack as the salt dissolved thus keeping a uniform concentration moving out from the well for the full length of the perforations. New sacks of salt where introduced into the well as the previous one became empty. The test was concluded after 40 hours

9. Well Versus B. S. P. Observations.

(*i*) Well observations approximately record the phreatic surface as the tapping point is not at a considerable distance below the water table. They indicate the rise and drop of the water table without showing the causes which contribute towards it.

(*ii*) B. S. P. observations have the following advantages.

(*a*) The B. S P. observation coupled with the phreatic surface observations enable one to measure the effect of the various sources and sinks on the water table.

(*b*) They can be used to evaluate soil evaporation.

(*c*) They can also be used to evaluate the flow in the ground water reservoir above the B. S. P. level.

(*d*) These observations are very useful to determine the soil characteristics such as Darcy's transmission constant or the percolation intensity co-efficient of the soil crust in the field without taking soil samples and then determining the transmission constant by artificially packing them in tubes. Such evaluation can never be correct as naturally deposited soil crust can never be reproduced in a laboratory.

(*e*) The B. S. P. observations are very useful to evaluate the lossess from the canals in the saturated phase. The loss is then simply $q=\sigma H$; where σ is the percolation intensity co-efficient and H is the percolation Head representing the difference between the water level in the canal and the B, S. P. level

(*f*) The B S. P. observations are very essential to measure the capillary fringe above the ground water resevoir (Chapter III, Part VI)

(*g*) The discharges of seepage drains can easily be calculated along their length Paragraph 15, Chapter II, Part IV.

10. Water Table in Clay Soil Crust.

(*a*) Soil crust in the Punjab contains clay from 15 to 30 percent by volume. The average porosity has been measured to be about 40 percent. A soil crust with no pore space is unknown. Any material which is porous is essentially pervious to flow of water. Cement concrete and the hardest rocks like quartz are all pervious. Fuller and Thompson carried out elaborate tests to measure permeability in 18″ concrete slab and one of the their tests for 1 : 3 : 6 concrete is given in Table 8.

TABLE 8

Proportions by weight	Percentage of cement to total dry material	Maximum size of stone	Time in which water appears.	Rate of flow of water in grams per minute at the following pressures per sq. in.			
	%	in	min.	20 lbs	40 lbs.	60 lbs.	80 lbs.
1 : 3 : 6	10·2	2·½	7	1	4	8	12
1 : 3 : 6	10 2	1	26	0	5	10	15
1 : 3 : 6	10·2	1/2	29	0	1	17	20

The porosity in the concrete and the stones connot be reduced to zero and the reduced porosity simply means relatively greater resistance to flow. The flow may be even less than the evaporation which properly is made use of in using concrete as damp proof coarse in buildings. Similarly in soil crust, the porosity could never be made zero. A soil crust cannot. therfore, be impervious so long as it is porous.

(*b*) The student is aware of the specifications of puddle made from clay soil. A brown clay soil containing relatively a larger percentage of clay is selected. It is exposed to the atmosphere for a few hours, then saturated fully and left for a night or say 12 hours. It is then pugged with feet for a couple of hours or passed through a pugg mill. It is only then that clay puddle is prepared which is very nearly water-tight. The exposure to atmosphere is necessary to supply oxygen in the soil to liberate free sodium. The student should remember that all clay soils contain sodium and calcium salts Mere presence of sodium salts in a soil does not make it sodium clay. All sodium salts are soluble and can easily be washed away from the soil crust which is the basic principle of the reclamation of *thur* lands by leaching. To convert a soil into sodium-clay, free sodium must be available and then soil should disperse or be made to disperse by pugging. The sodium then goes in to the form of a coat around the soil particles. The soil particle then behaves like a bladder. It swells absorbing water and tends to choke the pore space (not essentially completely). The soil may then be said to be a good puddle (sodium clay) or very nearly impervious to flow. The question arises whether

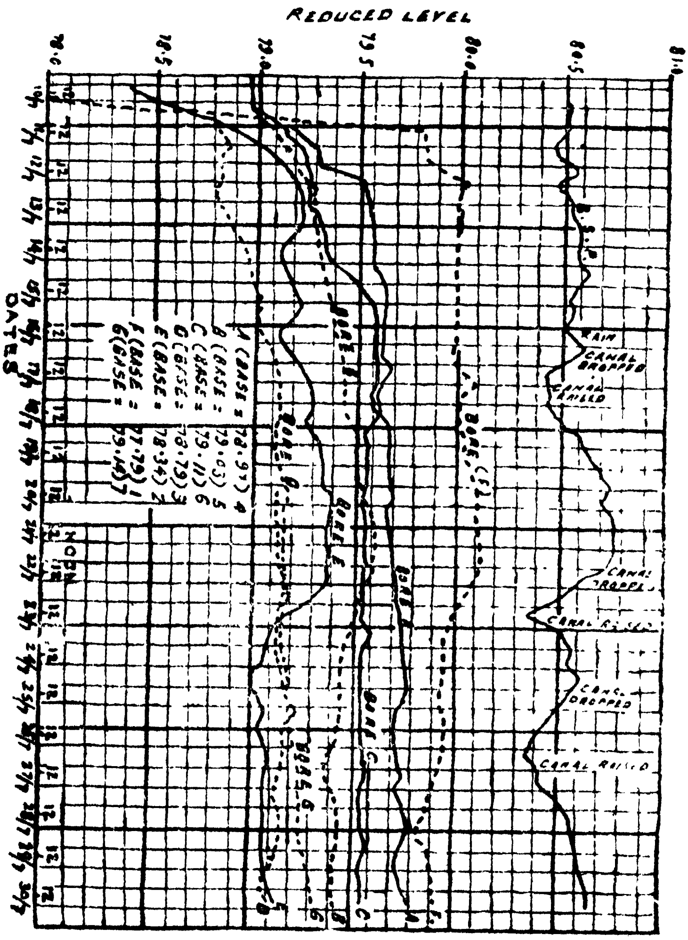


Fig. 15

the soil crust by irrigation over it or by water table rising into it from below will get converted into sodium clay or not. [The author considers that the process will be extremely slow and never completely effective] for want of free sodium, efficient dispersion and effective pugging.

Even if the sodium clay bladders be formed, there can be no guarantee that they shall completely fill all pore space.

(*c*) Another process known to make clay soil very nearly impervious is known as flocculation. The essential feature of this process is that a soil sample should be thoroughly dispersed in water (100 parts for 5 parts of the soil), then vigorously shaken. Then a chemical is added suitable for the calcium or sodium soils which results in base exchange. The soil then falls from the suspension in flocs. The flocculated clay soil then forms very nearly an impervious mass. The whole of this elaborate process is also not possible in a soil crust by simply water table rising into it from below.

(*d*) The Irrigation Research Institute Lahore, dug out pits about 10 feet diameter 6 feet deep in the soil crest on the left side of the Lower Chenab Canal at R. D. 180,000. The thickness of the soil crust is 13·0 ft. and below it is sandy soil. In the holes pierced through the soil, crust, water rose to within 5·0 ft of the ground level while no water appeared in 6·0 ft. deep pit the bed of which had dry appearance after a day of its digging. It was concluded that there was on water table in the soil crust although the head of 8 feet of water pressure was acting upwards below it. (Page 37 Annual Report of the Punjab Research Institute, Lahore for the year ending April 1940). The author carried out a few observations to study the points which were described in detail in the author's paper No. 33 and are briefly summarised below. The technique of these observations is discribed in Chapter III, Part VI.

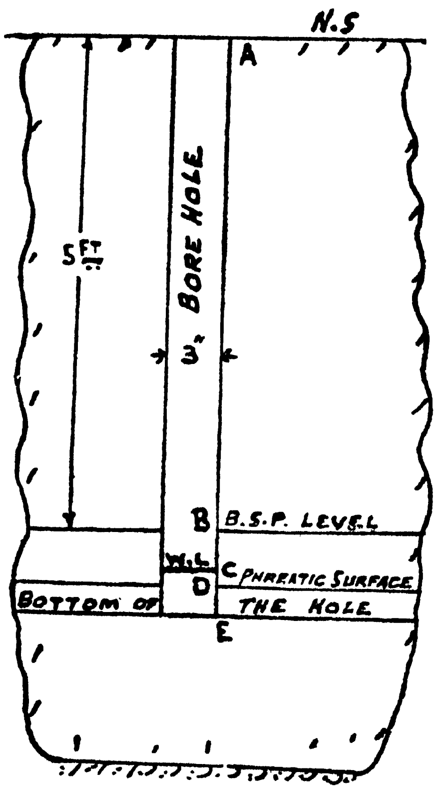


Fig. 16.

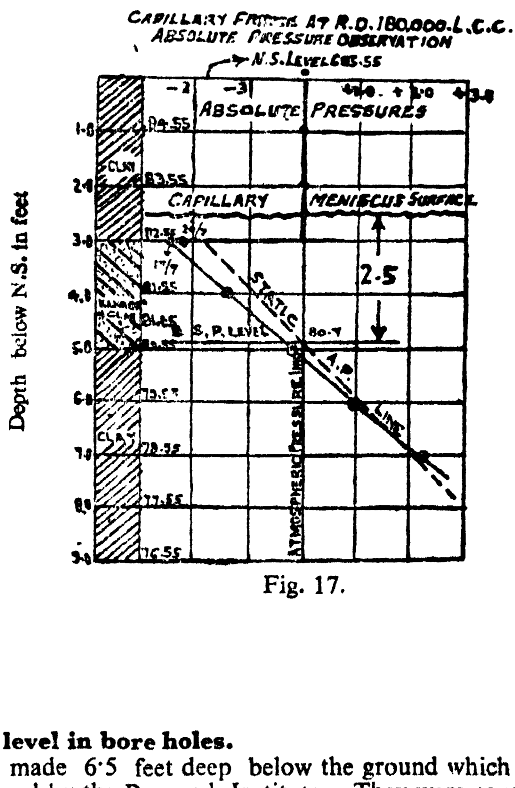


Fig. 17.

(*i*) **Observations of rise of water level in bore holes.**

Six bore holes 3 inches diameter were made 6·5 feet deep below the ground which was slightly uneven within 50 ft. of the pit observed by the Research Institute. They were covered with earthen trays so that the evaporation might be shielded. The water rose in all bore holes and the observed water levels are shown in Fig. 15. The rate was different according to the soil conditions but water levels in all of them became steady below the B.S.P. levels as shown in Fig. 16. This shows that an open bore in soil crust cannot record the correct manometric pressure as observed in a B.S.P. pipe. The explanation is simple.

By covering a hole, the general effect of the soil evaporation on the outside water table cannot be cutoff. The result of the upward evaporation gradient is to depresss the static phreatic surface from B to D (phreatic surface is that at atmospheric pressure). Soil in losing water into the hole from D to E and is capable of taking water from C to D. Water level in the hole, therefore stands at a level C where equilibrium between inflow and outflow is attained, and this depends on the soil conditions at the site of each hole. The fact that the highest water level in all holes was lower that B.S.P. level is a definite proof that, at the site of these experiments, there was a strong upward gardient due to the soil evaporation irrespective of the water table movements.

The main point is that all bores, however stiff the clay, have yeilded water, there by showing that the soil is effectively saturated.

Since the maximum water levels recorded in all cases were below the B.S.P. level, it may be concluded that the holes were not being fed from fissures in the crust. Fissures would have a direct connection to the water table below the crust and in that case the water levels should have risen to B S.P. levels in all holes.

(*ii*) **Capillary fringe observations.**

It was observed by using diaphram manometer that the height of the capillary fringe was about 2·5 feet at this site. The manometric pressure observations as obtained by the author and Blench I.S.E., Executive Engineer are shown in Fig. 17. The actual nagative gradient was greater than the equilibrium gradient showing the upward flow due to soil evaporation. No air rose into the manometer below 3·0 feet of the surface. It is concluded that the soil below 2·5 feet of the ground level was effectively saturated as there was no air below this point. It is quite possible that pore space may be considerably less than 40% and a moisture content observation of about 15 to 20 percent may be enough to saturate it effectively.

(*iii*) **Soil evaporation measurements in a 6 feet deet pit.**

A · feet deep pit 10 feet diameter was dug. It had a wet appearance when progressively dug below 2 ft. of the soil but the moisture dried in 24 hours and for all intents and purposes the side and the bed then looked dry. A pressure tapping pipe with its base 6 inches below the bed was put and connected to U-tube, Benzine differential manometer. A pan of water was put in the bed and pan evaporation was measured. There was no air 6″ below the bed and the pressure difference observation were taken hourly for a fortnight day and night and averaged out along with the averaged B.S.P. change. Then the pit was filled with water 1·0 ft. above the B.S P. level The downward flow through crust 7 0 ft. deep below bed was measured by the actual feeding by an automatic chicken-feet apparatus also by the point method apparatus. The seepage intensity coefficient thus observed is used to calculate the soil evaporotion as given below :—

Observed seepage intensity coefficient $=0{\cdot}103$ cusecs per million sq. feet ; Actual seepage Head $=1{\cdot}0$; Loss in cusecs $q=\sigma H=0{\cdot}103\times1{\cdot}0=0{\cdot}103$cs″; Velocity = Discharge/Area $=q/A$

$$=\frac{0{\cdot}103}{10^6}=0{\cdot}103\times10^{-6}\text{ ft./sec.}$$

This loss is through the soil crust below bed with a head of 1·0 foot

Gradient $=\dfrac{1{\cdot}0}{13-6{\cdot}0}=\frac{1}{7}$; $v=ks$, where k is the transmission constant

$k=\dfrac{0{\cdot}103\times10^{-6}}{\frac{1}{7}}$; $k_1=3{\cdot}72\times10^{-6}$; which shows that the soil was finer than the known finest sands. The fine sands have k down to $5{\cdot}5\times10^{-6}$.

Evaporation.

Actual observations were carried out in a pan put in the pit by using Hook gauges.

On 28-4-1930, the actual pan evaporation was 0 018 ft. in 24 hours.

Average pressure difference on 28-4-1939 $=1{\cdot}0138$ ft.

Base of P. T. Pipe = 678·89 ; Base of soil crust = 672·55

Gradient = 1·0138/6·34 = 0·16 ; observed k in downward flow = $0{\cdot}720 \times 10^{-6}$

v = k s ; Upward water table movement per day due to upward

gradient = $24 \times 3600 \times 0{\cdot}16 \times 0{\cdot}72 \times 10^{-6} \times 0{\cdot}720$ -- 0 00995 ft.

Whereas the pan evaporation in 24 hours was 0·018 ft. Rate of pan evaporation is about twice the supply from the water table due to upward gradient. This means that free water in the pit will be available by lowering the bed to a depth greater than 6 ft. till increased infiltration gradient causes inflow to exceed decreased pan evaporation.

(*iv*) **Downward flow of water through the soil crust.**

The bed 6·0 ft. below ground level B.S.P. 5 ft. below ground and water surface in the pit 1 0 ft. above B.S P. level with 2·0 ft. depth. There was automatic feeding to replenish the water lost in percolation. The observed results were as below ; corrected to 20°C in cusecs per million sq. feet per feet of head :—

Date	σ	Date	σ
2-5-39	0·19	7-5-39	0·076
3-5-39	0·099	8-5-39	0·077
4-5-39	0·104	9-5-39	0·068
5-5-39	0·098	10-5-39	0 036
6-5-39	0·81	11-5-39	0·000

The observations were confirmed to be correct by an independent method known as the point method. Neglecting the first day's observations when the pit was freshly filled with water the next three days very nearly give the constant rate of inflow and then the pit got completely choked by 11-5-39 Water in it without any external disturbance starting eating or caving the sides and it became muddy. The conditions as described in para 9 (c) above were brought about resulting in dispersion and then flocculation of the soil which contained about 30 percent of clay. This showed that the soil crust was pervious for the first 4 days and then gradually choked in downward flow.

(*v*) **The progressive lowering of the bed of the pit.**

(*a*) The experiment was taken in hand to measure flow gradients during progressive lowering of the bed of a pit, and to calculate transmission constants by measuring the actual inflow into the pit by observing the rising water levels in it and actual pan evaporation.

(*b*) Bed appearance in progressive lowering in a pit 10 feet. in diameter near the old one used in (*iii*) and (*ii*) above.

Upto 3 ft. the pit had dry appearance. with 4·0 ft. it had a wet appearance but it dried up in 12 hours. With 5 ft. depth (bed nearly at spring level) it had a wet appearance and dried up in two days. With 6·0 ft. depth, it was wet and slushly in patches. Water was flowing out in patches. It dried up in patches and flow stopped in two days. On further lowering to 6·25 ft. below N.S. it was wet and slushy in patches. Water was flowing out in some patches. At night on 3-6-39 and 4-6-39 water nearly covered half the bed of the pit Even at noon, on, 9-6-39 bed soil had nowhere a dry appearance. On further lowering to 6·6 feet the condition of the bed was the same as that with depth 6·25 ft. It was further lowered to 7·3 feet below N. S. on 8-6-39. The bed was slushly everywhere and water was oozing out at all places During the night between 8-6-39 and 9-6-39, bed of the pit was covered with water everywhere It did not dry up during day on 9-6-39 ; from 9-6-39 ; it steadily rose up to about 2·0 ft. depth.

(*c*) Rise of water level in the pit plus pan evaporation gives the daily inflow into the pit. This is due to negative pressure difference actually recorded by the manometer. The rate of flow per foot seepage head is then worked. The average rate of inflow in this period is wrked out to be 0·1162.

Table 9.

Rate of Seepage into the pit at R. D 180,000 L. C. C

Date. 1	Daily B.S.P average. 2	Change in B.S P. + Fall rise 3	Daily P D. average 4	Daily rise of W.L. in pit from 9 a.m. to 9 a.m. 5	Evaporation in 24 hours. 6	Total of evapo-ration and rise. 7	Column 7 / Column 4 8
12-6-39	80 40	0 05	– 0 972	·051	0·027	·078	·0804
13-6-39	80 37	+ 0 03	0 4424	·056	0 021	·077	·0832
14-6-39	80 34	+ 0 03	– 0 857	·066	0 020	·076	0885
15-6-39	80 27	0·07	-0 0815	·054	Not observed	—	
16-6-39	80·25	+ 0·02	–0 667	·120	0 027	—	Rain
17-6-39	80·25	0 00	– 0 622	·053	0·020	·073	·1172
18-6-39	80 18	, 0·07	0 619	·289	0 034		Rain
19-6-39	80·22	- 0·04	- 0 532	·059	0 015	·074	·1282
20-6-39	80·34	–0 12	0 581	·052	0·025	·077	·1326
21-6-39	80 37	0 03	- 0 563	·046	0 025	·071	1261
22-6-39	80 09	+ 0·28	–0 431	·018	0 041	·059	·1366
23-6-39	80·07	+ 0·02	–0·412	·024	0 028	·052	·1261
24-6-39	80 12	–0 05	–0 409	·034	0 017	·051	·1248
25-6-39	80 12	+0 00	–0 372	·020	0 028	·048	·1290
26-6-39	80 15	– 0 03	–0 384	·027	0 025	·052	Local shower unrecorded
27-6-39	80·15	+ 0 00	0 401	·029	0 017	·046	·1150
28-6-39	80 15	+ 0 00	–0 349	·026	0 018	·044	·1261
29-6-39	80 23	0 08	–0 366	114	0 005	-	Rain
30-6-39	80 26	–0 03	–0 356	017	0 023	040	113
1-7-39	80 21	–0·05	–0 323	071	0 028	—	Rain
2-7-39	80 17	+ 0 04	–0 264			Average	·1162

TABLE 10.

Pit at R. D 180,000 L. C. C. ; N. S. = 685·55.

Pressure difference with different depths of pit.

Date	Average B.S P level 24 hours observation.	Change in B.S P. 24 hours observations	Daily P. D. average	Remarks
12-5-39	80 640	. .	...	
13-5-39	80 607	+ 0 033	+ 0 475	
14-5-39	80 633	0 026	+ 0 445	
15-5-39	80 667	–0·034	+ 0·438	
16-5-39	80 670	–0·003	+ 0 489	
17-5-39	80 640	+ 0 030	+ 0 617	
	Pit bed 1 ft. deep below N. S. Dry pit.			
18-5-39	80 644	–0 004	+ 0 252	
19-5-39	80 580	+ 0 064	+ 0 2 2	
20-5-39	80·530	+0·050	...	
	Pit bed 2 ft. deep below N. S. Dry pit.			
21-5-39	80 520	+0·010	+ 0 141	
22-5-39	80·510	+0 010	+ 0 118	
23-5-39	80 470	+ 0 040	...	
24-5-39	80 500	–0 034	Not set correctly	
	Pit bed 3 ft. deep below N. S. Dry pit.			
25-5-39	80·475	+0 025	+ 0 024	
26-5-39	80·460	+ 0·015	- 0 032	
27-5-39	80·460	·000	...	
	Pit bed 4 ft. deep below N. S. Dry pit.			
28-5-39	80·440	+0·020	– 0 204	
29-5-39	80·440	0 000	–0·180	

Pit bed 5 ft. deep below N. S. Dry pit.

30-5-39	80·432	+0·008	0·384
31-5-39	80·410	+0·022	0·303
1-6-39	80·410	0·000	...

Pit bed 6 ft. deep below N. S. Dry pit.

2-6-39	80·480	0·070	...
3-6-39	80·450	+0·030	0·410
4-6-39	80·420	+0·030	0·573

Pit bed 6·5 ft. deep below N. S. Wet pit.

5-6-39	80·350	0·070	1·2
6-6-39	80·3[illegible]0	+0·040	1·4 nearly

Pit bed 6·6 ft. below N. S.

7-6-39	80·300	+0·010	1·6 nearly

Pit bed 7·3 ft. below N. S. free water in it.

8-6-39	80·310	0·010	Negative, beyond
9-6-39	80·350	0·040	the range of the
10-6-39	80·350	0·000	instrument.

Note :– Negative P. D. even when B. S. P. change is positive shows strong upward gradient due to soil evaporation.

This rate of seepage flow gives the following average transmission constant of the soil.

$k = 5 \cdot 5 \times 10^{-6}$ for 4′ layer between P. T. Point and bottom of the soil crust.

$k = 2 \cdot 3 \times 10^{-6}$ for 1·7′ layer from P. T. Point to the bed of pit.

This finest sand investigated in the Irrigation Research Institute, Lahore had $k = 5 \cdot 0 \times 10^{-6}$.

In the pit in steady conditions when water level rose to the maximum, the pressure difference was 0·25 ft.

Assuming parallel stream line of the upward flow and upward "velocity" $= 0 \cdot 015/86400$ ft. per sec; 0·15′ was the observed evaporation in 24 hours.

Manometric gradient in soil crust below bed of pit $= 0 \cdot 25/5 \cdot 70 = 0 \cdot 0439$.

$$\therefore \text{ Transmission constant; } k = \frac{V}{S} = \frac{0 \cdot 015}{86400} \times \frac{1}{0 \cdot 0439} = 4 \cdot 0 \times 10^{-6}$$

The rate of seepage flow per unit head is worked out in Col. No. 8 of Table No. 9. This shows that the departure from the average value of 0·1162 is not much. For practical purposes, it is concluded that the rate of seepage flow was constant during the period of the observations. The low value recorded during the first three days is probable because the labourers, moving about and digging to consolidate to some extent the surface of the bed of a pit. This superficial consolidation of the bed surface was opened out by the upward seepage flow by 15-6-39. Results from 16-6-39 to 2-7-39 are very nearly constant. Water in the pit had became muddy on account of dispersion but it did not exercise its adverse effect to choke the pit completely. It is concluded that though the flocculation in the downward flow in (*iv* abave), choked the bed completely yet in the upward flow, the rate of seepage flow remained sensibly constant in a pit inspite of dispersion which made water muddy.

(*vi*) **Conclusions.**

(*a*) Tenacious soil crust 13 ft. thick at R. D. 180,000 L. C. C. was effectively saturated up to the capillary fringe which was 2·5 ft. below N. S. The B. S. P. level was about 5·0 ft. below N. S that is, 8 ft. above the bottom of the soil crust.

(*b*) There was water table every where in the soil crust. The soil pores were effectively saturated, capable of transmitting the hydrodynamic soil pressures.

(*c*) In spite of 2·5 ft. of soil above the capillary surface above the fringe being in the partially saturated conditions the evaporation was effective at all times to extract water from the water table below as shown by the changes in the P. D. observations on the process of the progressive lowering of the pit.

(d) With a pit 6·0 ft. deep, pan evaporation was double the yield from the bed : but with a pit 7·3 ft. deep, the the seepage flow into the pit was more than the pan evaporation.

(e) The rate of the upward seepage flow for unit head was sensibly constant, unaffected by the dispersion of the soil of pit bed and its sides which consisted of a tenacious clay.

This leads to a very important conclusion that dispersion with the consequent deflocculation affects only the downward flow in a soil crust but not the upward flow.

(f) The impervious sodium clay soil is the most unfertile soil. Nothing can grow in it. All along the tract, there is good cultivation. Even in the uncultivated areas, there are salts on the surface or vegetation in the form of grass or shrubs. The surface of sodium soil is simply *banjer* or barren without a blade of grass and no salts on the surface.

11. Capillary Fringe in a Soil Crust.

In laboratory observations of sands, the capillary fringe is generally small. The normal height in sand is considered to be about 6 inches.

The height of capillary fringe as defined in paragraph 2, can be easily determined in a crust in the field by the use of the hydrodynamic pressure observations, the technique of which, is described in Chapter III of Part VI.

There is definitely a height of water above the phreatic surface where soil pores are effectively saturated with water and there is no free air. The field observation is very simple. The pressure tapping point is progressively lowered from the ground and the hydrostatic pressure is observed. At any point above the fringe there is partially saturated zone of funicular and pendular water. The hydrostatic pressure cannot be measured in this region because of the presence of air at atmospheric pressure but below the capillary meniscus surface, there is no free air and hydrostatic pressure can be observed by a manometer. For the static

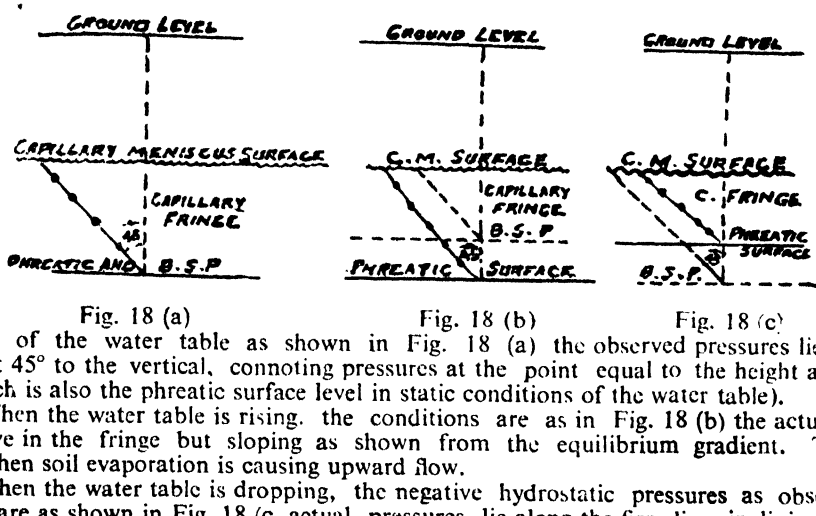


Fig. 18 (a) Fig. 18 (b) Fig. 18 (c)

conditions of the water table as shown in Fig. 18 (a) the observed pressures lie on a plane inclined at 45° to the vertical, connoting pressures at the point equal to the height above B.S.P. level (which is also the phreatic surface level in static conditions of the water table).

When the water table is rising, the conditions are as in Fig. 18 (b) the actual pressures are negative in the fringe but sloping as shown from the equilibrium gradient. This is also the case when soil evaporation is causing upward flow.

When the water table is dropping, the negative hydrostatic pressures as observed in the the fringe are as shown in Fig 18 (c, actual pressures lie along the firm line inclining away from the vertical in the case if soil evaporation is also effective.

Actual heights of capillary fringe observed at sites in the soil crust are shown in table No. 1, chapter II, Part V, varying from 2·0 to 4·0 ft. generally and 11·6 ft in one case. In actual practice capillary meniscus surface is not a straight line but rises and falls as shown in Fig. 18 (c) but this surface is continuous separating the funicular water containing the free air from effectively saturated zone below. The moisture content is very nearly constant in the capillary fringe varying only from 23 to 25 percent by weight down to the phreatic surface and soil pores are effectively saturated. There may be lodged pockets of air. Above the meniscus surface the moisture content reduces form 23 percent to about 3% by weight when the pore space is about 40% by volume.

12. Fluctuation of Shallow Wells in Water-Logged Areas.

Larsden, a Dutch Agriculturist, while studying the fluctuations in the Dutch Wells (waterlogged areas below sea level behind the embankments) reported in his paper that rises of 25 to 60 times the rainfall were recorded. Larsden gave an explanation in terms of the air pressure on the water table due to the sucking in of the rainwater by capillary action. It seems that although this effect does occure, the air would soon leak out or the roof of wet soil would have to blow itself off long befor the air pressure reached the amount required to explain 60 times exaggeration.

An explanation is some times given that the menisci of the capillary fringe above the water table are practically at soil surface level in water-logged tract and a little rainfall soon blots them out and raises the water table by the whole fringe height. There is ample evidence that capillary fringe in alluvial plains are often of the order of 10 feet high, and nct infrequently about 20 ft. In more open soil, fringe height some two or three feet high may be normal. It is the existence of these fringes that prevents water appearing at the surface except locally in water-logged tracts, for evaporation from their top balances the well-rise that was the predecessor of the water-logged conditions. A homely analogy of how this evaporation occurs is given by the oil lamp with a wick. The capillary fringe of oil in the wick provides the flow of oil that is burnt. The oil pressure in the wick is less than atmospheric and differs from what is found hydrostatically by the amount required to overcome resistance to flow The capillary pull has its seat in the menisci at the top of the wick, and would cease immediately the menisci were destroyed. Such a means of destruction would a device such as shown in Fig. 19, where a second lamp is inverted over the first with the wicks meeting each other. The lamp is extinguished and the menisci bringing up the oil in Fig. 19 are destroyed. Oil flow would then proceed downwards, since the menisci would no longer exist. Likewise, a shower of rain on a soil surface with the capillary menisci close to it would result in meniscal destruction.

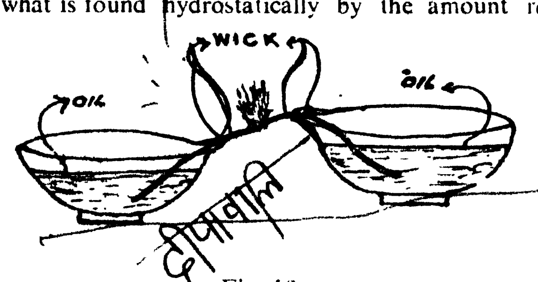


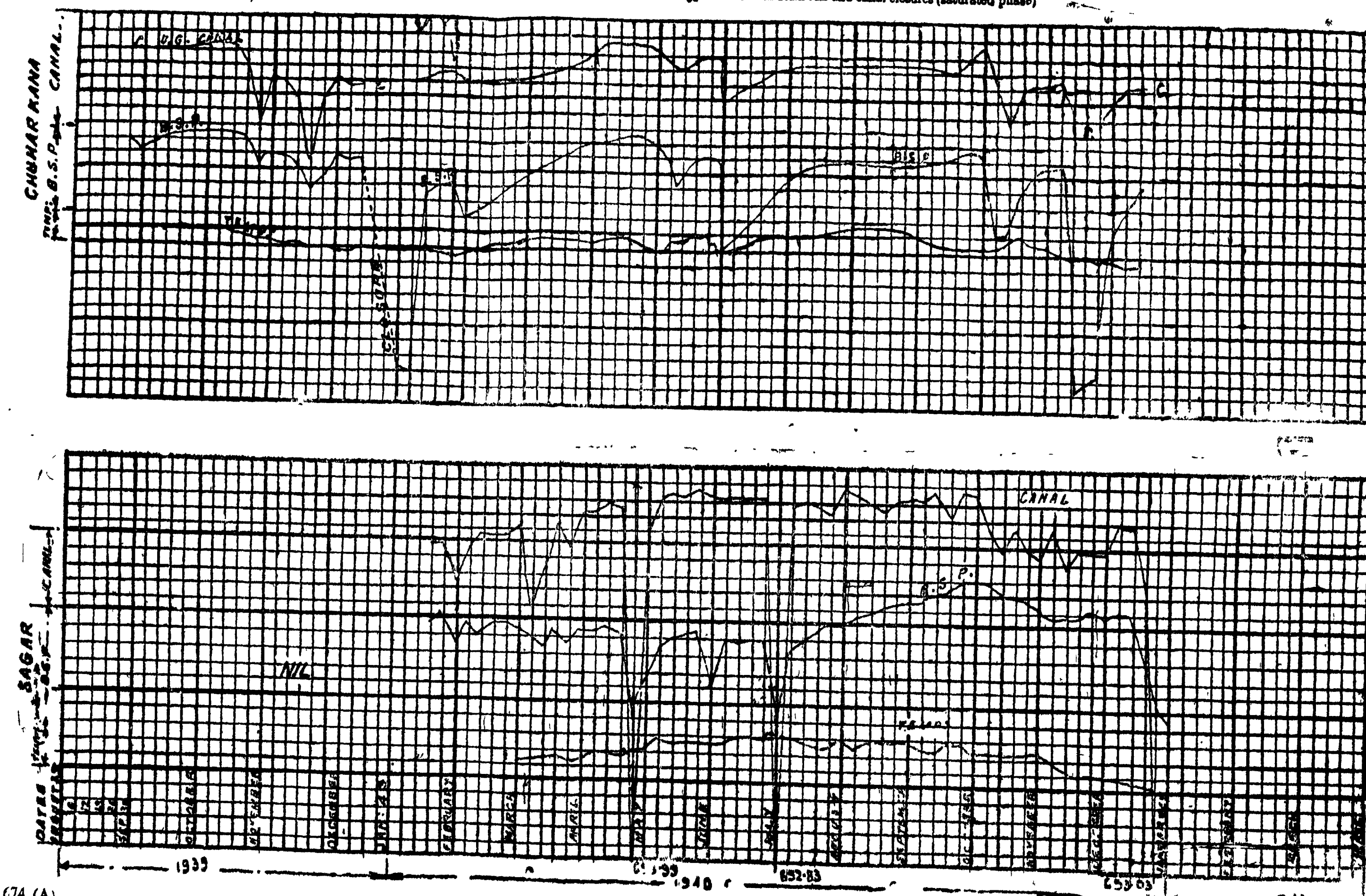
Fig. 19

In the proceedings of the Indian Academy of Sciences. Volume IX, No. 4, April 1939. M. Afzal and V. I. Vaidhianathan explain the phenomenon as below :—

"The reason for the rise of the level in the tube is this. Water is drawn up by capillrity through the fine pores of the sand and when it reaches the surface of the sand it begins to evaporate. A large unmber of concave menisci are formed in the interstices near the surface and these exert a negative pull. The water in the side tube is thus held down by this pull of the menisci in the sand. As soon as water is sprinkled on the surface of the sand, free surface are formed and the concave meanisci are flattened, finally of course forming a continuous flat surface of water. The negative pressure decreases and causes the water in the side tubes to rise by a height which is many times that in the main tank. The action in the wells is also similar. The rise in a well in the field is thus not an indication of what is really taking place within the subsoil, unless the other factors in the surrounding field are known. It is remarkable that a phenomenon as surface tension should play such an important role in sub-soil observation"

The author considers that none of the above mentioned explanations is correct because in no case has it been proved that the spontaneous rise of manometric pressure after sprinking water was approximately equal to the capillary hight. The capillary meniscus surface above the capillary fringe is not doubt destroyed by say even half inch rain, when it is near the surface. In water-logged area it has been measured to be within one foot. Rain simply reverses the upward flow due to soil evaporation to the downward flow caused by the rain. The wells record the phreatic surface which is below the B. S. P. level in upward flow but above the B. S. P. level in downward flow as shown in Fig. 20. The soil above the capillary meniscus surface with funicular water is also pretty saturated. The water is already there effectively saturating the soil very nearly up to the groumd surface. Before the rain, a well records the phreatic surface AB which will be say distance—x below the B. S. P. level

Yearly cycle of B. S. P. Fluctuations in water logged areas with Rain fall and canal closures (saturated phase)



674 (A)

Yearly cycle of B. S. P. Fluctuations with Rainfall and nearest canal closures in unwater logged areas (unsaturated phase)

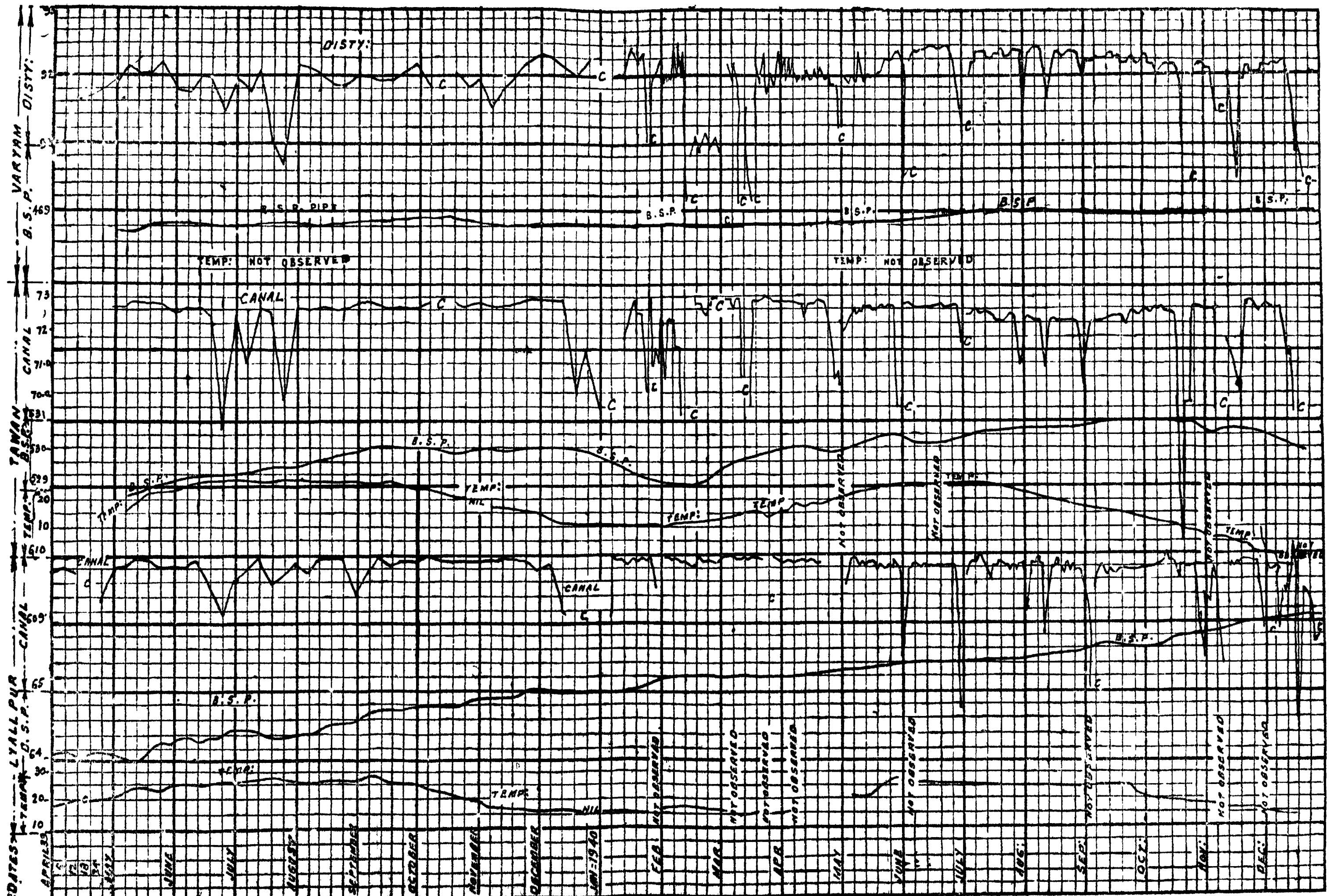


Fig. 22

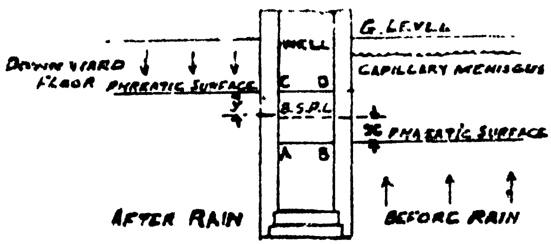


Fig. 20

representing the pressure difference causing upward flow to feed the soil evaporation. After the rain, the capillary water along with the rain moves downwards when the meniscus surface is destroyed by full saturation above it. The well must jump up to C. D. with positive pressure difference – y above B. S. P. level. Let the rise in the well be (y + x). The value of x was measured to be 2·24 ft. of negative P. D. in the case of soil crust 6 ft. deep at R. D. 188,000 L. C. C. experiment to yield evaporation equal to 0·18" of water in 24 hours. Rise in a well would easily be of the order of 4 to 5·0 feet in a soil crust of the type at R. D. 180,000 L. C C. This does not at all represent the amount of water added to the water table which will be shown by any change in the B. S. P. level. This is just a measure of manometric pressure adjustment caused by the reversal of flow affording an easily demonstrable proof of the B. S. P. conception.

13. Effect of Barometric Pressure on B.S.P. Level.

The increase in barometric pressure means inflow of soil atmosphere which depresses the capillary meniscus surface. The B S. P. level was also depressed but the effect was too small to the measured. The effect of the decrease in Barometric was *vice versa*.

Humidity changes had a great effect on soil evaporation and consequently the pressure difference showed measurable change. The increase in humidity reduced the negative P. D. and *vice versa*

14. Yearly Cycle of Rise and Fall of Ground Water Reservoir.

(*a*) At 36 places B. S P. pipes were observed daily. A year's record was ploted by the author. The observations of rainfall, temperature and the water levels of all nearest sources were recorded and plotted daily on the same sheet. The behaviour of daily observation stations in the water logged areas where the additions of seepage from the canal and rainfall was in the saturated phase was similar in all cases. Two of them are shown in Fig. 21.

(*i*) Rainfall had immediate effect on the B. S. P. level The rise reflected the addition to the water-table, but it was not directly proportional to the rain. One inch of rain would normally fill the soil up to nearly 4 inches depth but the rise depended on the gradient head which was necessary to clear out the addition to ground water flow towards the nearest sink. It was observed at Marala that a rain about 1·35 inches caused a rise on B. S. P. level by 1·92 ft. while 0·8 inch rainfall showed a rise of 1·1 ft. when other factors were steady.

(*ii*) The nearest canals exercise great effect on B. S. P levels. In closures the B. S. P. levels dropped by 40 to 50 ft. and then again rose to the original ones rapidly to a state of equilibrium when the canal opened.

(*iii*) Similarly the nearest irrigation also affected B. S. P. levels. The effect of rise cultivation in the water-logged areas was especially marked in raising the B. S. P. levels in water-logged area with flow from the field in saturated phase.

(*b*) The stations away from the water-logged areas behaved absolutely differently.
(*i*) The opening and closure of canals nearest to them had no effect on B. S. P. levels.
(*ii*) The rainfall had absolutely no effect.
(*iii*) The irrigation in the surrouding areas had no effect.

The rise and fall in them simply depended upon the underground flow. Two of such stations, Lyallpur and Tawan are plotted in Fig. 22. There is a yearly cycle of rise and fall in the B. S. P. levels. At Vryam there is no annual wave of rise and the B. S. P. level is steady throughout the year unaffected by the nearest canals, rain and irrigation.

(*c*) In the yearly cycle of the B. S. P. levels of the stations in the un-water-logged areas, the highest peak did not occur in the month of August or September (synchronising with rainfall and rice cultivation) in all cases but varried according to the distance from the water-logged areas (where the additions were in the saturated phase) as given on the next page.

Month	**Daily B. P. S. sites.**
August	Khanki. Hinduana, Khai (river effect)
September	Chuharkana, Sagar, Buchiana, Pacca Dalla
October	Kot Khundayar
November	Lyallpur and Tarkhani
December	Tawan, Lakhlana, Muridwala Bachherianwala
January	Magneja

These observation clearly show that there was no addition to the water table in the unsatureted phase and that the B. S. P. levels in such areas influenced only by the underground flow

15. Hump in Water Table Under the Canals.

(*a*) This is a subject which has not yet been investigated in detail. No definite knowledge about this problem is available. All sources such as canals, rivers, ponds and rainfall which contribute to the subsoil reservoir, are bound to affect the normal flow of the subsoil water table. They are likely to cause some configuration in the spring levels.

Observations were started to investigate this phase of the problem in 1937 on the Upper Jhelum Canal at R. 257,000 and 263,000 by putting three observation pipes across the sections. These observations were exteneded by putting 5 observation pipes across the sections at R. D. 299,000 and 350,000 U. C. C. Lower. The arrangement of the pipes at these sites is shown in Fig. 23.

The conception of B. S. P. does not preclude formation of a hump under such a big source as a canal.

R. D. 350,000 U. C C Lower

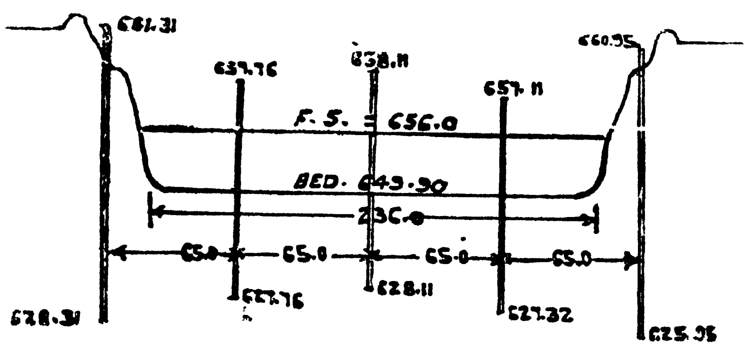


Fig. 23

B. S. P. pipe cuts of the wharping of stream lines due to the presence of a source. It cuts off the vertical components of flow and, therefore, records the correct water level of the subsoil surface gradient. It cannot record higher or lower than the actual surface gradient unless there be some other disturbing factors such as an artesian effect or a region of low pressure at its tapping point. Let AB represent the normal sub-soil gradient before the construction of a canal. For convenience, the flow of the sub-soil in Fig. 24 is considered orthogonal to the alignment of the canal Seepage losses from a canal will constitute a considerable contribution to the water-table. These additions shall need a steeper slope down a *doab* as represented by the line B'C. The effect of the presence of a canal will, therefore, be the flattening of the gradient upstream and its steepening downstream as represented by the line A'CB' in Fig. 24. Point C cannot be higer than A'. Let R. S. and T be three B. S. P pipes. They will respectively record water levels M, C and N on the sub-soil surface gradient. The difference between the averrge of M and N and the level C will give the height of the hump. The fully saturated percolation cone was observed in all cases as shown in Fig. 24 above. There existed a manometric gradient in the pressure observed in the cone downward flow. The percolation cone although recording manometric pressure is not a part of the ground water reservoir.

(*b*) At R. D. 299,000 and 350,000 hump did never exceed more than 0·1 ft. There was no hump forming when the rate of infiltration was steady. The B. S. P. levels adjusted by the pressure gradient required undergound flow to remove the additions. Similarly the site at R. D 257,000 showed no hump.

The site at R. D 263,000 U. J. C., showed a peculiar behaviour which is described below. There was a positive hump 1·76 ft. central B. S P higher than average level of berm ones. There was a positive hump 1·76 ft. on 9-11-37 with seepage head 10 5 feet and it decreased to 0·08 ft. positive when the seepage head dropped to 5·8 feet. This showed that height of the hump was related to the seepage head. A similar phenomenon was recorded from 8-1-38

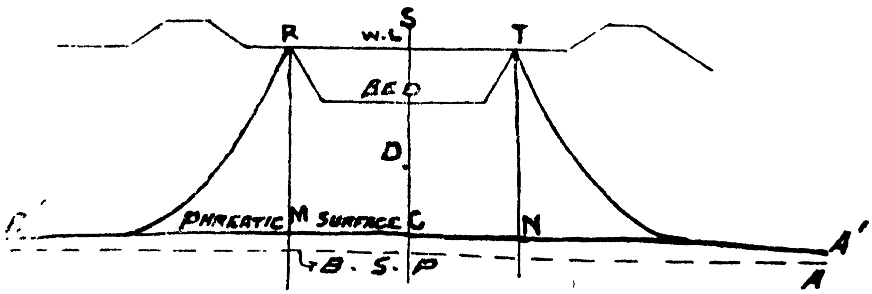


Fig. 24

to ′-2-38 when a positive hump decreased from 0·91 to 0·04 ft. with the drop in the seepage head from 10 95 to 5·62 ft. Similarly hump decreased from +0 86 ft. to +0·2 ft. from 23-2-38 to 14-3-38 when the seepage head reduced from 7·83 to 6·4 ft.

From 14-6-38 to 15-9-38, there was no hump worth considering. In this period, the changes in the seepage heads were not sudden. Seepage heads remained between 8 and 9 5 ft. The record of a continuous long period of three months shows that there was no hump formation in the stable conditions of the seepage losses.

No observations are yet available in which hump in the B. S. P. levels below a canal losing water in the unsaturated phase has been investigated.

PART V

GROUND WATER ENGINEERING

CHAPTER IV

Theory and Physics of Seepage Flow from Canals

1. Introduction.

Of all the problems in Hydraulics that have faced engineers from the earliest times, the one relating to seepage losses in irrigation channels is one of the must complicated and that, perhaps, explains why so little advance has been made therein up to the present day. Laboratory tests while tending themselves to other cases, seen to fail when applied to seepage losses, because the most important factor – nature of sub-soil strata—cannot be reproduced in the laboratory. The author carried out detailed experiments on the subject in the field under natural condition of the sub-soil. The results were published in the proceedings of the Punjab Engineering Congress, Lahore, in papers Nos. 209 and 231. A brief out line of this complex subject is given here. The losses from the canals can now be evaluated with precision, which is of the utmost importance for the Punjab province because the construction of canals in different *doabs* of the Punjab has invariably resulted in a rapid rise of spring levels causing water-logging.

2. History of the Development of this Subject.

Seepage losses from the canals into the sub-soil reservoir used commonly to be called absortion losses. The original experimenters, such as Colonel Dyas (1863) and Higham (1874) were content to know that the absorption losses in the Main Line of the Upper Bari Doab Canal were 20% and $12\frac{1}{2}$% repectively of the Main Line discharges. Kennedy (1883) worked out absorption losses as different rates of sinkage per hour for main line, branch canal distributaries and water coures. His results when reduced to cusecs per million square feet, were 9·75, 2·2, 3·3 and 9·4 respectively Woods tried to be more scientific and produced the formula $q = C\ a\ d$; where q, is absorption in cusecs per million sq. feet, C is a constant varying from 1·2 to 1·33, a, is the reduced wetted perimeter of the channel section and d, is its depth. The absorption in this case varied with wetted perimeter and depth. The work done upto that time did not make any reference to how water passed through the soil forming the boundary of of the channel and how the position of the spring level below the bed affected the results. Bresford (1875) threw the first hint indicating that there were two distinct methods in which water was lost from the canal to the sub-soil reservoir. It is in fact due to Wilsdon that we get a real start in the scientific investigation of this subject. His Lyallpur experiments were published in the Punjab Engineering Congress, preceedings, 1923. He has given a clear picture of what actually happens in the soil when water is lost from a canal by absorption through unsaturated soil (which has a moisture content of less than 23%) and by percolation through saturated soil (which has a moisture content of more than 23%). The term seepage losses stood both for absorption and percolation.

3. Wilsdon's Experiments.

(*a*) Absorption.

Wilsdon's experiments as described in the punjab Engineering Congress, proceedings, of 1923 consisted of the determination of the moisture concentration in the soil below a small irrigation channel. From these observations he determined the contours of equal moisture content as sketched in Fig. 1

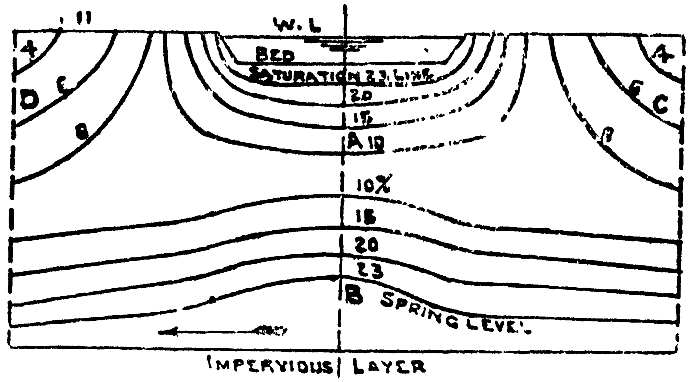


Fig. 1

The contours of moisture concentration are semi-ellipses below an irrigation channel. The moisture content of the soil decreases as one goes down. It is minimum at a certain point between the groups of curves A and B. Again the moisture content rises in group B down to the spring level. The groups of curves C and D are due to the evaporation from the adjoining land. These are also semi-ellipses. They have been drawn here assuming that there is no irrigation in close proximity to the channel. These ellipses are practically parallel to the slope of the sub-soil water table at the point of their exit at C and D. The hump in group B is less pronounced if there is a steep slope in the sub-soil water. It is a maximum if the impervious boundary is quite near to the spring level or the slope of the sub-soil flow is very flat.

This is a diagramatic representation of the losses from canals by absorption Wilsdon explains that absorption depended on three factors *viz.*, gravity, capillarity and chemical attraction. He took chemical attraction as being proportional to capillarity and his test showed that gravity played little or no part in the case of absorption. He concluded that the absorption did not vary directly with depth under any conditions, as in Wood's formula.

(*b*) **Percolation.**

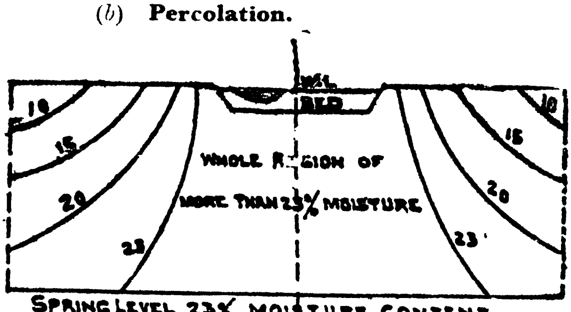


Fig. 2

The case of percolation is very simple The soil is saturated from the bed of the canal to the sub-soil water table. At every place the moisture content is more than 23%. All the pore spaces of the soil are already filled in with water There is no capillarity or chemical attraction. Water simply percolates in the stream tubes around the soil particles under the head available between the free surface of the canal and the sub soil spring level. as influenced by the transmission constant of the soil. The moisture contours are as sketched in Fig. 2.

4. Author's Experiments.

The author's experiments were carried out in a tank opposite R.D. 4000 of 14 R of Upper Jhelum Canal. Wilsdon's work was merely qualitative showing the saturation line below the bed of a channel at a short distance below the bed and the moisture content decreased to about 3% The author's observations supplied both the quantitative results and verified the qualitative picture below the bed of a canal. A line diagram of the plan of the observed tank is given in Fig. 3.

Losses were measured in a 50 ft. long tank with a constant 4 ft. depth and bed width 20 feet and side slopes 1 to 1. The side tanks 100 ft. long were also maintained at the same level so that there was no end effect. The losses were measured by installing automatic chicken feed apparatus which maintained automatically a constant level in the tank. At the site of the experiments due to the proximity of the Chenab river, there was a natural swing in the B. S. P. levels from obout 0·5 above bed to 6·5 feet below bed. The observations were carried for two years daily. The losses corrected to 20° C temperature were as shown in Fig. 4. The

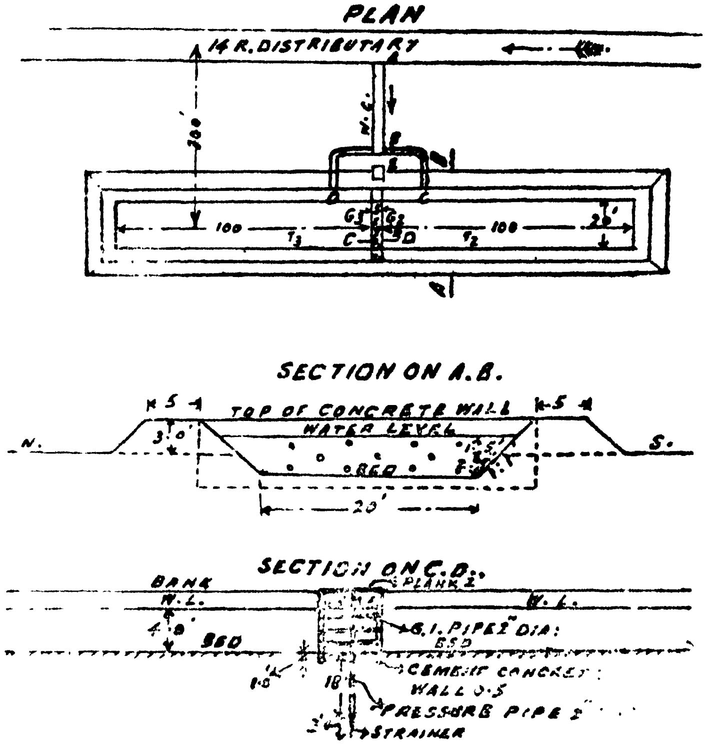
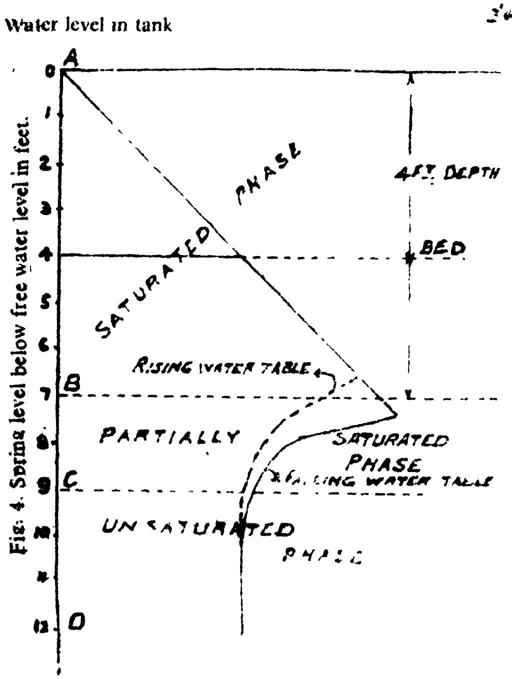


Fig. 73.



results clearly show the phases of losses, (a) the saturated phase, (percolation) (b) partially saturated (phase percolation and absorption) (c) unsaturated phase (absorption). The different phases of losses were verified to be correct in Central Hydrodynamic Research Institute Poona, Bombay by C.C. Inglis, as published in the Punjab Engineering Congress Paper of 1938.

5. Temperature Correction.

It has been seen from the temperature observations that there is a variation of temperature from 50°C. to 35°C. The variation of temperature causes a variation in the viscosity of water. It is natural to expect that water of flow viscosity will pass quicker though the stream tubes of the soil under the same prevailing conditions of head and pore space than water of high viscosity. Absorption is directly proportion to the velocity of the water, as it enters the boundray of bed of the experimental tank. It is, therefore, clear that the absorption is inversely proportional to viscosity.

Referring to page 11 of "Hydraulics by A. H. Gibson, the variation of the viscosity with temperature is expressed on the next page.

$$\mu = \frac{\cdot 00003716}{1 + \cdot 03368T + \cdot 000221T^2} \qquad (A)$$

where μ is the viscosity and T is the temperature in centigrade But velocity of flow V varies inversely viscosity $\therefore V \propto (1 + \cdot 03368T + \cdot 000221T^2)$

But absorption A is directly proportion to velocity.

$\therefore A \propto (1 + \cdot 03368T + \cdot 000221T^2)$ (B)

A diagram is given in plate XXII, giving the percentage correction considering the standard temperature of water to be 20°C. For temperatures above 20°C the correction factor is negative and temperatures below 20°C the correction factor is positive. Table No. 1 is also given for temperature correction

It is clear from this diagram that the temperature effect is so very pronounced that on account of the temperature alone the seepage from canals will drop in winter for a change from 30°C to 5 C by about 66%.

TABLE 1.

Table for use in the correction of absorption losses for temperature, calculated from the viscosity formula :—

Temperature in degrees Centigrade	Rate of flow relative to that at 20 C	C 1 to reduce loss at T to loss at 20°C.	Difference in Col (3) for one degree
1	2	3	4
0	5675	+ 76 21 %	5 76
1	5867	70 45 „	5 49
2	6062	+ 64 96 „	5 22
3	6260	59 74 „	4 94
4	6460	+ 54 80 „	4 70
5	6662	+ 50 10 „	4 48
6	6867	45 62 „	4 28
7	7065	+ 41 34 „	4 08
8	7285	+ 37 26 „	3 88
9	7497	+ 33 38 „	3 71
10	7712	29 67 „	3 57
11	7930	26 10 „	3 40
12	8150	22 70 „	3 26
13	8372	19 44 „	3 13
14	8597	16 31 „	2 99
15	8825	13 32 „	2 88
16	9055	10 44 „	2 76
17	9287	7 68 „	2 66
18	9522	5 02 „	2 56
19	9760	+ 2 46 „	2 46
20	1 0000	Nil „	2 37
21	1 0243	2 37 „	2 27
22	1 0487	4 64 „	2 21
23	1 0735	6 85 „	2 12
24	1 0985	8 97 „	2 04
25	1 1238	−11 01 „	1 98
26	1 1493	−12 99 „	1 90
27	1 1750	14 89 „	1 85
28	1 2010	16 74 „	1 78
29	1 2273	18 52 „	—
30	1 2538	22 24 „	1 72
31	1 2806	21 91 „	1 67
32	1 3076	−23 32 „	1 61
33	1 3349	−25·09 „	1 57
34	1 3624	26 60 „	1 51
35	1 3902	28 07 „	1 47
36	1 4182	−29 49 „	1 42
37	1 4465	30 87 „	1 38
38	1 4750	−32 20 „	1 33
39	1 5038	33 49 „	1 29
40	1 5328	−34 76 „	1 27

6. Moisture Va iat:on Below Canal Bed in the Three Phases.

A picture of the conditions under varying spring levels (B. S. P.) is shown in Fig. 5,(A to G).

In the unsaturated phase Fig 5 (A and G) the whole of the head due to depth of 4·0 ft. is lost within 0·5 ft. of the bed. The moisture content then decreases below upto 10 percent (Wilsdon recorded 3 percent). It again increases upto the top of the capillary meniscus surface above the phreatic surface. The capillary height was measured to be 2 5 ft. by using diaphram manometer and a negative pressure of 2·5 ft. above the phreatic surface was recorded and it was found that there was no air up to this height above the phreatic surface in the soil pores.

In the partially saturated phase Fig. 5 (B and F) there are saturated tube connections in which the pressure is every where atmospheric connoting that the gradient of flow was unity. While outside the saturated tubes, there was free air. The tubes were located by suction tests. The top of a tapping pipe was connected to a U - tube water manometer as shown in Fig. 6 The left limb was sucked up 1·0 ft. showing a difference of 2·0 ft. in both, then suction pump was disconnected. If there was air or free water at bottom of the pipe the limbs equalised in no time but in the case of free water, the water rose in the pipe and if soil pores were filled with water but not free, then the two limbs did not equalise.

In the case of the saturated phase Fig. 5 (C. D and E) all pipes recorded manometric pressures from bed of the channels down to the phreatic surface. There was a positive pressure drop from the bottom of the one to the bottom of the next lower pipe connoting downward flow as shown in Fig 9.

Figs. 4 and 5 clearly show that in the saturated phase, the head causing flow is the difference between the free water surface in the canal and the B. S. P. level. The distance below bed when the saturated flow stopped was about 3·5 ft. (actual 3·42 and 3·6 in two sets of observations) when the B. S. P. level was dropping and in the case of the rising B. S. P. levels, the saturated connection was made when the B. S. P. level rose to within about 2·5 ft. (actual 2 42 and 2·54) below bed. Actually the complete saturation connection is made when the capillary meniscus surface meets the saturation line below the bed of a canal, the difference in both cases is simply due to the fact that in one case the phreatic surface above B.S.P. level about 0·5 ft. and in the other case it was below it by about the same amount. The head causing flow in both cases is $H=[D+D_s+h\,(C.F.)+h(P.D.)]$; where H is the Percolation Head difference between the canal water level and B.S.P. level).
D the depth in the channel ; D_s the The depth of saturation line below bed.
h(C.F.) the Capillary fringe height ; h(P D.) the Pressure difference causing upward and downward movement in the water table.

All the above mentiond terms are always positive but the last h (P D.) is negative in the case of rising B S.P. level and positive in the case of dropping B.S.P levels. This important phenomenon observed in these experiments is a very distinct proof of the B.S.P. conception outlined in this book In the saturated cone, the stream-lines are very nearly confocal-ellipses and the equipotential lines are orthogonal to them. An interesting experiment was carried out by Dr. H. L. Uppal of the Punjab Research Institute, wherein the sub-soil flow was controlled to lie on one side. The streams lines as observed by Haleshaw method are shown in Fig 7 and equipotential lines have been added by the author in dotted. If a channel crosses at *doab* like the U. C. C. and U. J. C. this kind of flow is likely. Fig. 8 shows the stream lines if a drain is collecting away seepage water instead of a canal adding water to the water table. Both in the case of a channel, acting as a source or a sink, the observed stream lines are similar and identical.

7. Pressure Gradient Below Bed in Saturated Phase.

In the experiments at 14-R site, U.J.C. the daily manometric pressures were observed at 0 5, 1·0, 2·0. 3·0, 4·0 and 5·0 ft. below bed of the tank and published in paper No. 231 punjab Enginearing Congress. and pressure head loss is tabulated below:—

14 R TANK EXPERIMENTS U. J. C.

Physics of Seepage Losses in Irrigation Chennals V. Scale=1/50

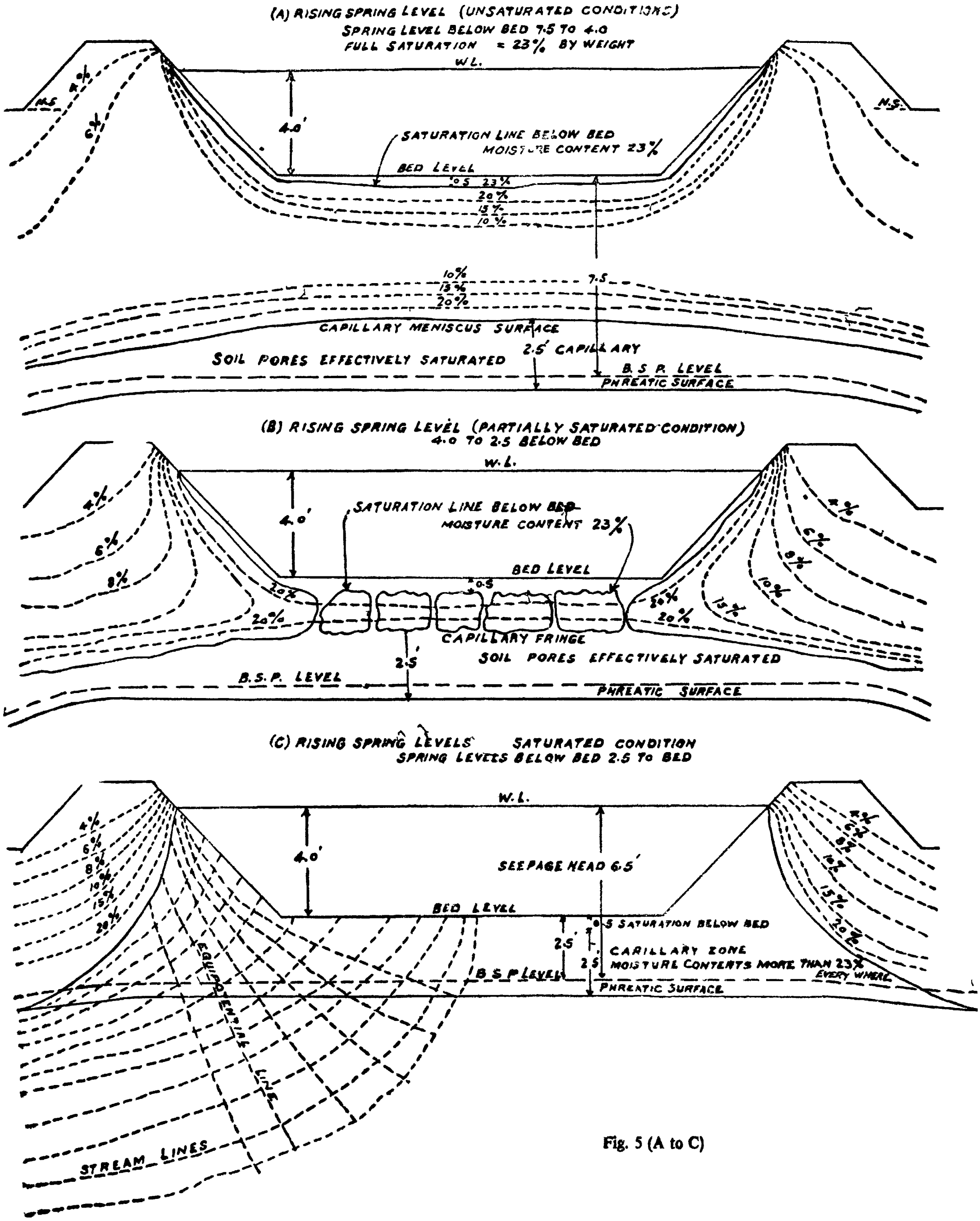


Fig. 5 (A to C)

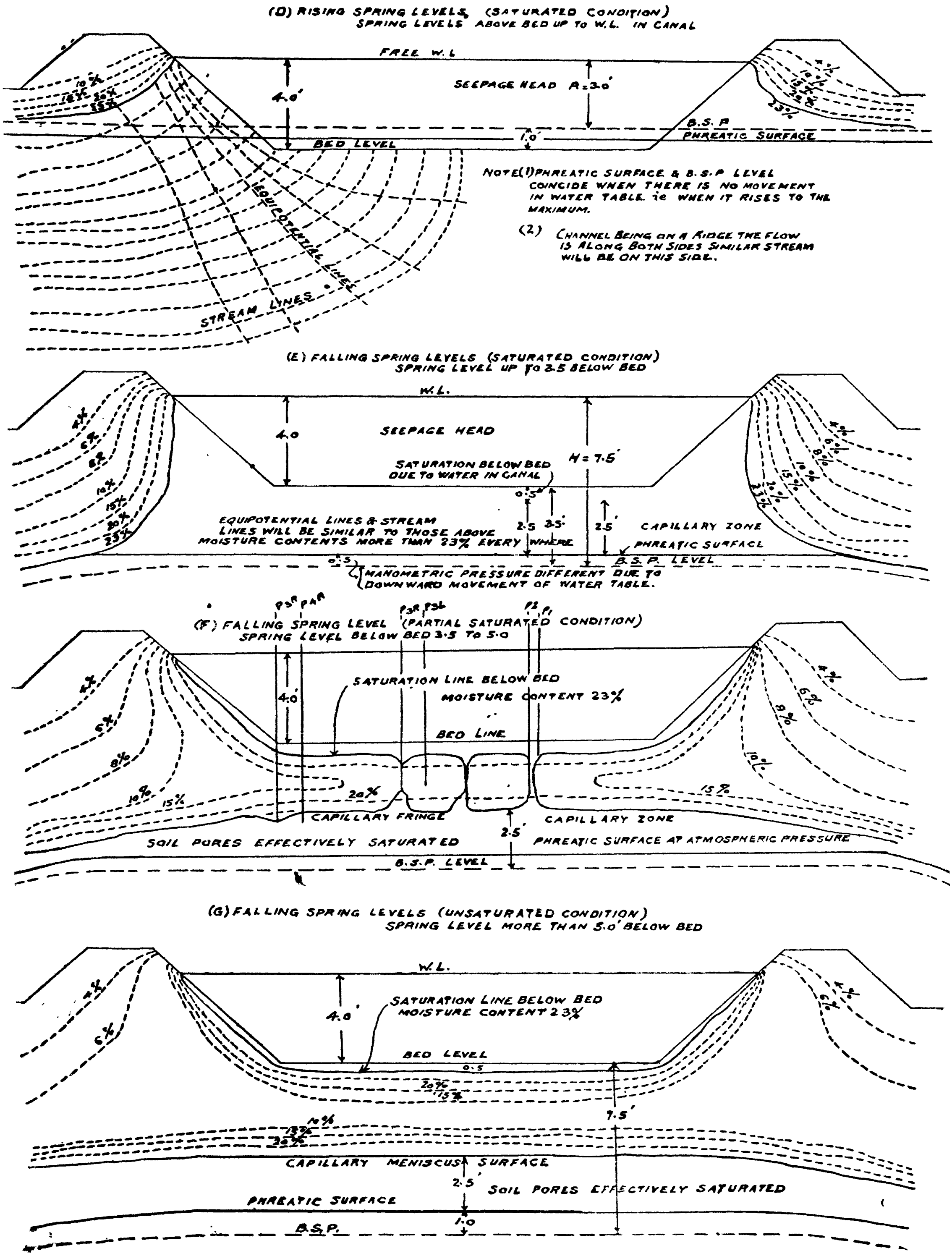

(D) RISING SPRING LEVELS (SATURATED CONDITION)
SPRING LEVELS ABOVE BED UP TO W.L. IN CANAL
FREE W.L
SEEPAGE HEAD H=3.0'
4.0'
B.S.P
PHREATIC SURFACE
BED LEVEL
1.0'
EQUIPOTENTIAL LINES
STREAM LINES
NOTE (1) PHREATIC SURFACE & B.S.P LEVEL COINCIDE WHEN THERE IS NO MOVEMENT IN WATER TABLE. ie WHEN IT RISES TO THE MAXIMUM.
(2) CHANNEL BEING ON A RIDGE THE FLOW IS ALONG BOTH SIDES SIMILAR STREAM WILL BE ON THIS SIDE.
(E) FALLING SPRING LEVELS (SATURATED CONDITION)
SPRING LEVEL UP TO 3.5 BELOW BED
W.L.
SEEPAGE HEAD
4.0
H=7.5'
SATURATION BELOW BED DUE TO WATER IN CANAL
CAPILLARY ZONE
EQUIPOTENTIAL LINES & STREAM LINES WILL BE SIMILAR TO THOSE ABOVE MOISTURE CONTENTS MORE THAN 23% EVERY WHERE
PHREATIC SURFACE
B.S.P. LEVEL
MANOMETRIC PRESSURE DIFFERENT DUE TO DOWNWARD MOVEMENT OF WATER TABLE.
(F) FALLING SPRING LEVEL (PARTIAL SATURATED CONDITION)
SPRING LEVEL BELOW BED 3.5 TO 5.0
SATURATION LINE BELOW BED
MOISTURE CONTENT 23%
BED LINE
CAPILLARY FRINGE
CAPILLARY ZONE
SOIL PORES EFFECTIVELY SATURATED
PHREATIC SURFACE AT ATMOSPHERIC PRESSURE
B.S.P. LEVEL
(G) FALLING SPRING LEVELS (UNSATURATED CONDITION)
SPRING LEVEL MORE THAN 5.0' BELOW BED
W.L.
SATURATION LINE BELOW BED
MOISTURE CONTENT 23%
4.0'
BED LEVEL
7.5'
CAPILLARY MENISCUS SURFACE
2.5'
SOIL PORES EFFECTIVELY SATURATED
PHREATIC SURFACE
B.S.P.
1.0

Fig. 5 (D to G)

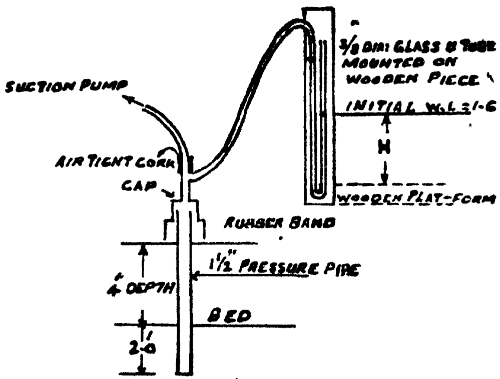


Fig. 6

FLOW OUT OF A TRANSVERSE CANAL

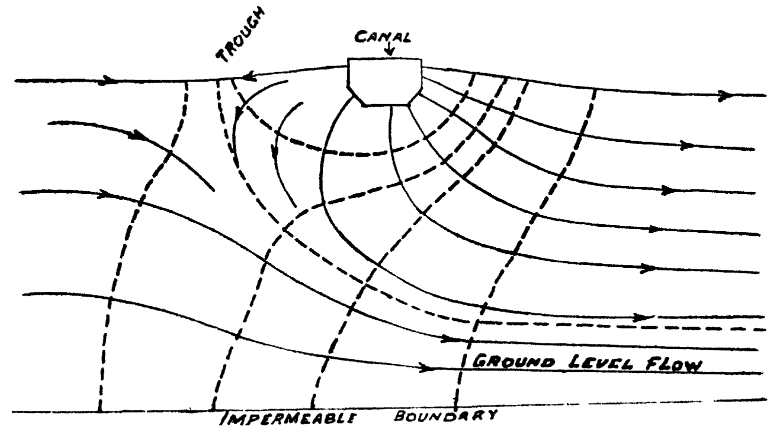


Fig. 7

Absolute Pressure Diagram in saturated phase

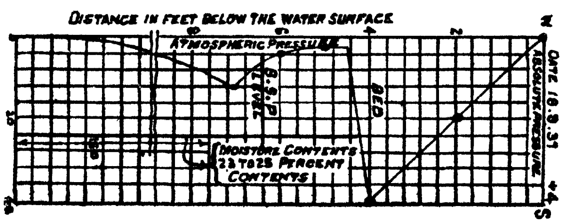


Fig. 10

Moisture Contents

FLOW INTO TRANSVERSE DRAIN

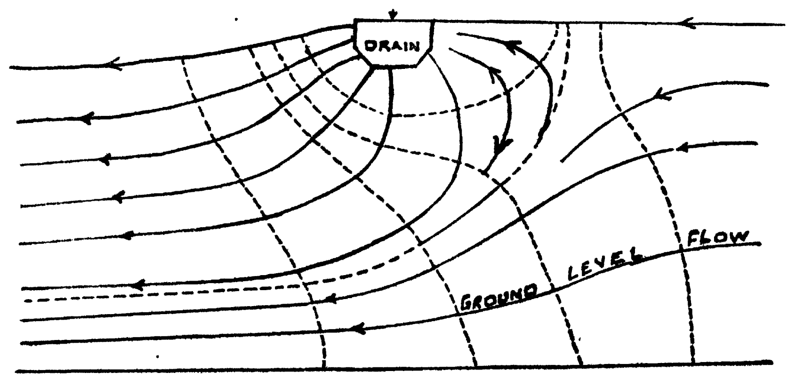


Fig. 8

Water Level in Pressure Pipes and Spring Level in feet below free W S. in Tank

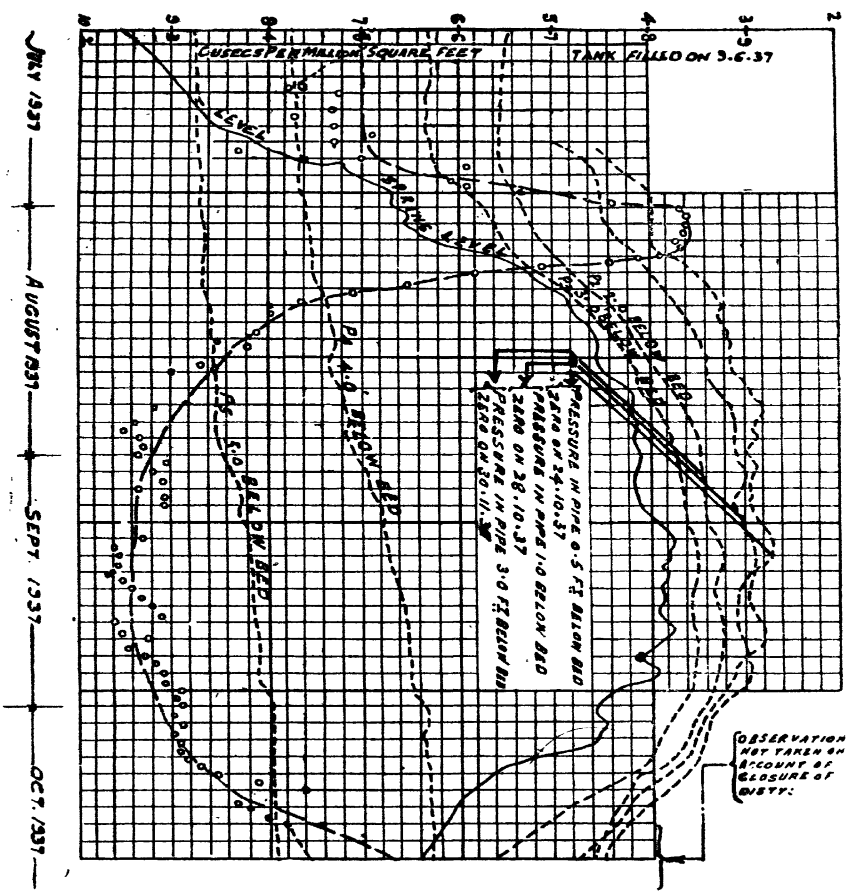


Fig. 9

Distance in feet below free water surface

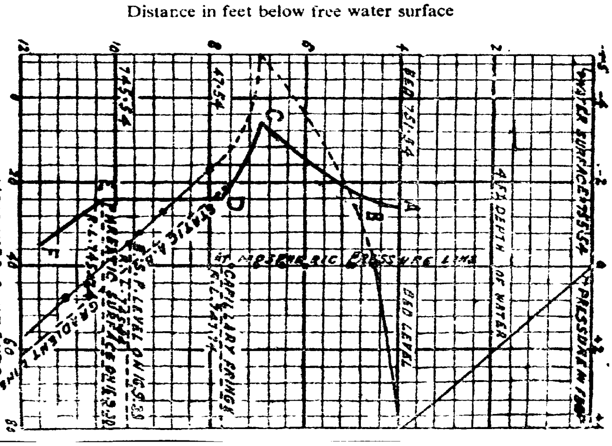


Fig. 11

Theory and Physics of Seepage losses 14 R Tank Experiment U J Canal absolute pressure diagram

TABLE 1.

Manometric pressure Gradient below bed 14-R Experiments Saturated Phase, Rising water table B. S. P.=2·5 below bed 26-7-37. Percolation loss 8·5 cusecs per million sq. ft. 20°C. $\sigma=8{\cdot}5/8{\cdot}5=1{\cdot}0$.

Serial No	Region below bed	Pressure change	Pressure drop	Gravity	Distance	Total drop	Gradient loss in ft per foot
1	2	3	4	5	6	7	8
1	0 to ·5	4 to ·1	3·9	0·5	0·5	+4·4	+8·8
2	0·5 to 1·0	·1 to ·2	−0·1	1·0	1·0	+0·9	+0·9
3	1 to 2	·2 to ·45	−0·25	1·0	1·0	+0·75	+0·75
4	2 to 3	·45 to ·80	−0·35	1·0	1·0	+0·65	+0·65
5	3 ft. to Bottom of B S P.	·80 to 0	+0·80	+0·5	15	+1·3	+·09

TABLE 2.

Date 18-9-37. Saturated Phase B.S.P. Steady height level 0·2 ft. above bed, B.S.P. bottom 18 ft. below bed, percolation loss 2·3 cusecs per million sq. ft. at 20°C and $\sigma=0{\cdot}87$.

1	0 to 0·5	4 to 1·38	2·22	0·5	0·5	+2·72	+5·44
2	0·5 to 1·0	1·38 to 2·10	−0·32	1·0	1·0	+0·68	+0·68
3	1·0 to 2·0	2·15 to 2·78	0·63	1·0	1·0	+0·37	+0·37
4	2·0 to 3·0	2·78 to 3·50	0·72	1·0	1·0	+0·28	+0·28
5	3 to bottom of B.S.P.	3·50 to 0	+3·5	−3·2	15·0	+0·3	+0·02

TABLE 3.

Date 18-10-37. Dropping water table B.S.P. 2·4 ft. below bed, percolation loss 5·9 cusecs per million sq. ft. at 20°C and $\sigma=0{\cdot}92$.

1	0 to 0·5	4 to 0·03	+3·93	0·5	0·5	+4·43	+8·86
2	0·5 to 1·0	0·03 to 0·35	−0·2	1·0	1·0	+0·63	+0·63
3	1·0 to 2·0	0·4 to 1·0	0·6	1·0	1·0	+0·4	+0·4
4	2·0 to 3·0	1·0 to 1·4	−0·4	1·0	1·0	+0·8	+0·8
5	3 to bottom of B.S.P.	1·4 to 0	+1·4	−6	15·0	+0·8	+0·053

Absolute pressure Diagram for the above observation is shown in Fig. 10 for saturated phase.

In the saturated phase, there is positive manometric pressure gradient from the bed of the channel down to the bottom of B.S.P. pipe.

8. Pressure Gardient Below Bed in Unsaturated Phase.

The table and pressure diagram are reproduced below from author's paper No. 231 of the Punjab Engineering Congress, 1940.

TABLE 4.

Date 16-9-39, B.S.P. level below bed=5·7 ft. unsaturated phase, loss=4·1 cusecs per million sq. feet at 20°C.

1	0 to ·5	+4·0 to 0	+4·0	+0·5	+4·5	0·5	+9·0
2	·5 to 1·0	0 to −1·6	+1·6	+0·5	+2·1	0·5	+4·2
3	1 to 2·0	−1·6 to −3·5	+1·9	+1·0	+2·9	1·0	+2·9
4	2 to 3·0	−3·5 to −4·5	+1·0	+1·0	+2·0	1·0	+2·0
5	3 to 4·0	−4·5 to −2·3	−1·2	+1·0	−0·2	1·0	−0·2
6	4 to 5	−2·3 to −1·25	−1·05	+1·0	0·05	1·0	−0·05
7	5 to 6	−1·25 to −2	−1·05	+1·0	−0·05	1·0	−0·05

The observed points are shown in circles in Fig 11. The pressure observations as tabulated and plotted clearly show that even taking gravity in to account there is no manometric pressure gradient positive connoting downward flow. The pressure diagram was completed in the partial satu·ated regions accepting Wilsdon's observations that saturation varied as log of the distance from the full saturation line. The results are tabulated above.

Column 8 should be positive ev.rywhere if any precolotion or any addition to the water table.

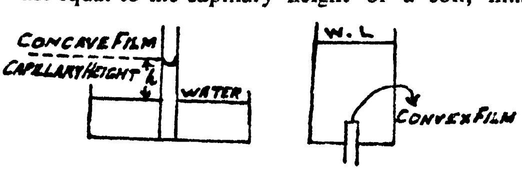
The essential condition to establish percolation or addition to the water table was that the pressure gradient should be throughout positive from the bed of a source down to the water table. This essential condition was also laid down by Wilsdon on page 22-a Punjab Engineering Congress, 1923 "Water movement would be determined by the factors governing absorption when the moisture content was below saturation, while percolation was only met with when the soil was saturated or the hydrostatic pressures positive, establishing positive gradient."

9. Pressure Gradient in Partially Saturated Phase.

On a vertical line through the center of the channel, Wilsdon showed a moisture content declining to a minimum value of 3% by weight and then increasing upto the full saturation of the water-table as shown in Fig 11. As regards the flow down this vertical. Wilsdon's view appears to have been that the moisture potential was more important than that of gravity. If so, the author would point out that a minimum moisture content implies the absence at that point of any moisture gradient potential. At this point, at least, flow must be due to gravity alone, and this consideration makes it difficult to accept. Wilsdon's picture as applicable to a state of steady flow using transmission constants applicable to saturated flow. In the case of a wide applicatoin of irrigation water, it is usually assured that water moves down by first bringing the soil to a field moisure capacity and thereafter moves down through this *uniformly saturated* zone by intermittent quantum flow, such as rainflows down a pane of glass. The author is disposed to believe that it is this mode of flow that obtains sometimes below the bed of a large channel when water table is far below.

It seems possible also that to some extent, downward flow under the full potential of gravity (*i. e.* s = 1·0 in Darcy's law with unity gradient) may take place in isolated columns of soil that are, fully saturated, and not merely at field moisture-capacity. For this also, the flow would be constant for a fixed system of saturated columns. In the face of reducing moisture contours, the steady unsaturated flow is an impossibility but quantum-flow is possible when the moisture content is more than the field capacity *i. e.*, when the soil water is funicular and not pendicular which is disconnected. Moreover, even for the quantum flow there must be positive manomentric gradient down to the water table. It was observed to exist in the saturated tubes in the partially saturat.d phase. The pores were effectively full of water but at atmospheric pressure every where from full saturation below bed and down to the B S.P. level.

The quantum flow or saturated flow in isolated columns with unity gradient cannot always take place. There are limitations and strong resistances which must be overcome before this type of flow is established In a vessel if a capillary tube be fixed as shown in Fig. 12 a film is formed. There is no downward flow till the pressure due to the depth in the vessel is strong enough to break the film The resistancs of a convex film to downward flow is likely to be more than the upward pull of a concave meniscus. The depth of water, at least equal to the capillary height of a soil, must be piled up before the film could break. It is evident that flow with unity gradient could only take place in a soil actually dry containing air in the pores if the capillary height of the soil is less than one foot. Only pure sand and coarse silt have capillary height less than one foot. If there is even 10 percent clay in a soil the the capillary height is more than a foot. In the normal soil crust in the Punjab inspite of 40 percent pore space, the quantum-flow or saturated flow in isolated columns is an impossibility under the meager depth of about 3·0 inches in irrigation waterings.



(A) Fig. 12. (B)

Water from the vessel sketch (A) shall no doubt continue to be lost by the process of evaporation inspite of the films. It would perhaps interest students to know that a mystic brass vessel (*Sorahi*) exists with the Raos, of Raikot, Distt. Ludhiana, Punjab, (supposed to be presented by Guru Gobind Singh ji) which holds water and keeps it cool by evaporation but a string or wire can be passed through the holes right across in all directions. There is no flow out but all water can be lost by evaporation. An earthen picher has similar properties though holes are not visible.

10. Conditions of Losses in the Three Phases.

(*a*) Saturated phase (Percolation).

1. Losses take place according to the formula $Q=\sigma H$ (C)

where Q is the Loss in cusecs in million sq. ft. at 20°C average temperature.

H the Seepage Head *i. e.*, difference between the free water surface in the canal and B.S.P level.

σ the Constant depending on the nature of the soil traversed by the seepage flow.

Various methods have been developed to determine σ in the actual field conditions. Their description is given in subsequent paragraphs. Its variation is given in Plate No. XXI, Vol. III.

(*i*) The losses are indenpendent of the depth of water in the canal.

(*ii*) There is saturated connection from a canal to the water table and losses are directly added to the water table.

(*iii*) All the pressure pipes inserted below the bed of the canal will record pressures and there will be definite manometric gradient for the base of one pipes to that of another.

(*iv*) The head due to the depth of water in the canal was lost within couple of inches below the bed due to flocculation of the silt particles.

(*v*) There was a definite position of B.S.P. level below the bed of a canal where losses will be the maximum. Its position varied according to the rising or falling water table and depended on the height of the capillary zone peculiar to the actual local soil conditions.

(*vi*) Major portion of the additions to the water table from the irrigation channels are in the saturated phase. It has been actually measured that losses. in certain reaches of U.C C., I .C.C. and U J.C. Main Lines are of the order of 30 to 40 cusecs per million sq. ft. On this account, in the case of such canals, as have seepage losses in the saturated phase, selective lining is the best remedy as an anti-waterlogging measure.

(*vii*) In sands and silts there shall always be saturated phase whatever the position of B.S P. because the resistance to downward flow is likely to be less than the gravity gradient.

(*b*) **Partially saturated phase (Percolation).**

1 Losses in this phase rapidly decline from the critical position giving the maximum losses in the saturated phase.

2 No mathematical solution has been attempted, though one could be found from the curves given in Fig 4. This phase is not of much practical importance as it will always be of a limited range (2 to 3 ft.) depending on the soil conditions.

3 There are fully saturated tube connections from the canal to the water table below. Free air at atmospheric pressure in the enclosed spaces between the tubes and free water at atmospheric pressure in the saturated tubes are available in the same plane between the saturation line below the bed of a canâl and continuous capillary meniscus surface above the water table

4. Major portion of the losses are directly added to the water table through the saturated tubes and a part of them is coutributed by the soil evaporation from the area adjoining the channel.

(*c*) **Unsaturated phase (absorption).**

1. The quantitative losses in this phase will be of low order unlike the saturated phase.
2. The variation of losses in this phase will deptend on the following considerations.

(*a*) Loss depends on the percentage of clay in the soil crust. A graph giving variation with the clay content of soil crust is given in plate XXI, Vol. III.

(*b*) There will be more loss from the sides than through the bed.

(*c*) The channels in ebmankment will have relatively higher loss, because of the exposures of other sides to soil evaporation.

(*d*) The depth in a channel does not directly affect the losses except that it will tend to increase the area of sides and consequently the area exposed to evaporation.

(e) The depth will also tend to lower the saturation lines below the bed of a canal and thereby increase the film surface from where soil evaporation will take place. The saturation line will be approximately proportional to wetted perimeter of the channel

3. The rate of loss is constant and independent of the position of the B S P. level.

4. There is no saturated connections from a channel to the water table below it. Free air at atmospheric pressure is every where available below the bed of a canal causing soil evaporation on account of temperature and vapour pressure gradient.

5. Losses in this phase are not directly added to the the water table and are all lost in soil evaporation.

6. The losses in the saturated phase do indirectly contribute to the water-logging of a tract because they cut off the soil evaporation which would have otherwise taken place.

7. This phase of loss is only possible where the clay content in the soil crust exceeded about 10% and B.S.P. level is away from the bed more than the critical depth below as required for establishing the saturated phase.

11. Haigh's Absorption Loss Formula.

Investigations by the author and Wilsdon have clearly shown that Wood's formula (K=C a d) was not correct. The absorption loss was not at all directly proportional to depth. The author's investigations, however, clearly showed (Paragraph 10) that the depth in the channel did affect the absorption losses by increasing the side exposed to soil evaporation as well as by depressing the saturation line below bed when the depth was increased.

Assuming an absorption loss of 6 cusecs per million for a channel of 20 cusecs and 8 cusecs per million for a channel of 2000 cusecs. F.F. Haigh I. S E., Punjab Irrigation has suggested a formula for K as below ;—

$$K = 5{\cdot}0\ Q^{\cdot 06{\cdot}5} \qquad \text{(D)}$$

where K represents the absorption loss per million sq. feet of the wetted perimeter and Q the discharge in any reach of a channel. As $q = KD^{5/3}$ in the expotential formula of flow of channels, this formula acrues to allow for the effect of depth in the absorption losses.

The values of K are tabulated below and plotted in fig. 13.

Q	K	Q	K
1	5·0	500	7·3
10	5·7	1 000	7 7
20	6 0	2,000	8·0
50	6 4	5,000	8·5
100	6 6	10,000	8·9
200	6·9		

Combining equation (D) with Lacy's formula $P_w = 2{\cdot}67\ Q^{\cdot 5}$

The absorption loss Q_a for channels in regime works out

$$Q_a = {\cdot}0133\ LQ^{\cdot 5625} \qquad \text{(E)}$$

in which L is length in thousand feet. The values of Q_a for different values of Q for every thousand feet length are given and tabulated below and plotted Fig. 13.

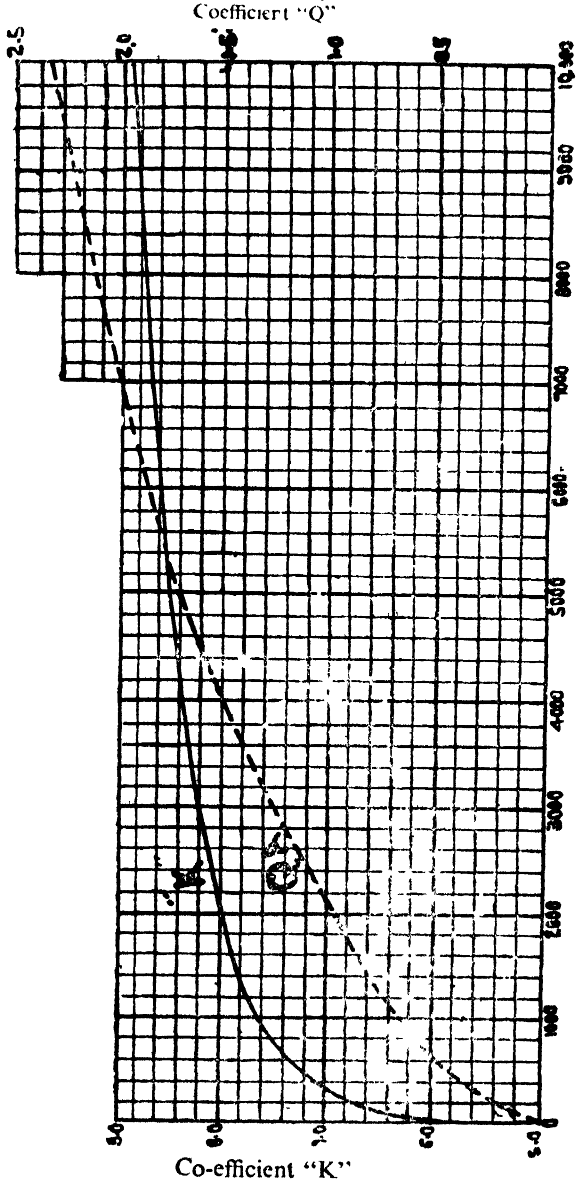
Q	Q_a	Q	Q_a
1	·013	500	·439
10	·049	1,000	·648
20	·072	2,000	·957
50	·120	5 000	1·602
100	·177	10,000	2·365
200	·262		

The value of constant in formula (D) has been assumed to be 5·0 but if one interprets the author's results of the 14R experiments (paper 209 and 231 Punjab Engineering Congress) the value works out to be about 4·0 for normal soil crust of the Punjab containing about 15% clay. For the standard average temperature of 20°C., the final formula reduces to the form $Q = {\cdot}01Q^{\cdot 65}$ and is plotted dotted in Fig. 13.

This formula and the graphs apply only to the unsaturated phase.

12. Reasons for Unsaturated Phase not Causing any Additions to the Water Table.

The seepage loss (absorption) in the unsaturated phase does not reach the water table and therefore, does not affect the water-logging. This is very important point and its full realization will cut short the cost of anti water-logging measures by about 75 percent. There will then be no need to waste money on lining water courses and channels and no adverse



Graph A

Shows Values of K in the Formula

$K = 5{\cdot}0\ Q^{0.625}$

Discharge (cusecs)

Fig. 13.

effect could be expected from rainfall or irrigation in areas where the losses are in the unsaturated phases. Fortunately such channels and such areas are about 75 percent in the Punjab. There are five arguments put forward in the paper written by the author which lead to this important conclusion:

(*i*) The absorption loss is constant, quantitatively. The constant value shows that the position of water table or gravity has no influence when the unsaturated phase is established. This simply means that there was some other constant factor such as soil evaporation which accounted for the loss, and that no addition was being made to the water-table.

(*ii*) The reducing moisture contours as observad by Wilsdon and shown in Fig. 5, below a canal and above the water table mean equilibrium but no liquid flow because of ;

(*a*) Resistance to flow being stronger than gravity.

(*b*) Pendular water below 10% moisture content being definitely disconnected as shown in Fig 5, Chapter I, Part V.

(*c*) Flow being possible in Funicular water only if the gradient of flow is positive and more than the equilibrium gradient.

(*iii*) The pressure observations as given in table 4 of this chapter and Fig. 11 clearly show that even taking gravity into account there was no manometric pressure gradient (positive) connoting downward flow.

(*iv*) Daily B.S.P. stations in the unwater-logged areas. where the losses were in the unsaturated phase (West of Sangla ahd Buchiana) were not affected by the nearest sources, such as canals, distributaries, irrigation and rainfall. The closures of even Branch canals had no effect.

(*v*) In the yearly cycle of the B S P levels of the stations in the un-waterlogged areas. the highest peak did not occur in the month of August or September (synchronizing with rainfall and rice cultivation) in all cases but varied according to the distace from the water-logged areas (where the additions were in the saturated phase) as given below :

Month	Daily B.S.P. sites
August	Khanki, Hinduana, Khai (River effect)
September	Chuharkana, Sagar, Buchiana. Pacca Dalla
October	Kot Khudayar
November	Lyallpur and Tarkhani
December	Tawan, Lakhlana, Murdiwala, Bachherianwala
January	Magneja

These observations cleary showed that there was no addition to the water table in the unsaturated phase and that the B. S P levels in such areas were influenced only by the under-ground flow which accounts for rise or drop of the B S. P. levels.

13. Method of Assessing Seepage Losses.

The various methods used by the author in the water-logging Investigation Division are detailed below :—

1. **By current meter or velocity rod discharge observation** :—

This method was used to evaluate losses on the Bikaner Canal.

2. **By constructing standard design modular meter flumes** and then getting the discharges from the gauge reading between different reaches. This was applied on Mangtanwala feeder.

3 **By tank observations**. This was used on the 14-R U. J. C. and Udeyra U. C. C. experimental sites.

4 **The evaluations of losses from seepage observation in closures**. This was used in the case of U. C. C. and L. C. C. Main Lines.

5. **Point Method.**

(*a*) **By using point method apparatus.** This was used in case of all distributaries.

(*b*) **By trough method.** This was used in case of U. J. C. R. D. 25500 and 26000 U. J. C.

6. **By statistical methods.** This was used in the case of L. C. C. Main Line and Branches.

7. By observation of the sub-soil water profile from a canal, Dr. Vaidianathan method, Irrigation Research Institute, Lahore, Research Publication Vol. V, No. 7 ; 1938.

14. Discharge Observation Method.

Normally, current meter or velocity rod discharges are likely to have an error of about 5% which is usully much more than the seepage losses occuring in the canals. In lined canals the loss is still less because it takes place by evaporation and through settlement or temperature cracks. This method was used on the Bikaner canal lined reach and special precautions were taken to attain the best accuracy. The channel of about 2000 cusecs is lined with 6 inches thick *kankar* lime concrete These observations were statistically analysed by T. Blench I. S. E. Executive Engineer.

The Bikaner Main Line percolation rate was found from the difference of head and tail discharge (300,000 ft apart) taken daily for 3 months. Observers and instruments were interchanged every ten days to eliminate instrument and personal bias. Sites were chosen to be identical The standard Discharge Division methods and instruments were used. The current meters used were checked fortnightly. The temperatures ranged from 14° to 26°C giving a velocity range of 1·33 times against a probable 1·7 for the whole year.

The 47 best results in periods of steady supplies were selected and analysed assuming (*i*) percolation proportional to perimeter (*ii*) percolation inversely proportional to viscosity (*iii*) personal puls instrumental bias proportional to discharge. The principle of Least Squares was used in analysis

It was found that the value of percolation at 26°C was : 1.65 ± 0.28 (s. d) cusecs/million sq. ft.

15. Meter Flume Method.

(*a*) This method is superior to the first one because a current meter record less velocity in cold water and more in warm water. The correction necessary is unkonwn. Velocity rod observations are, therefore, sometimes considered more accurate than current meter ones. Meter flume results are relatively more reliable because in the case of the long crested meter they are neither affected by the temperature of the water nor by the amount or quality of the silt carried in water.

The following measures are necessary for greatest accuracy in meter flume observations :—

(*i*) Confining observation to periods of steady supply or, failing this, to minimise the effect of changing supply by allowing for a time-lag between the pair of observation, based upon actual observation of the rate of travel of fluctuations from the upper to the lower site.

Time-lag curve for U. J. Canal from Rashidpur to tail fall from

R. D. 107,180 to R. D 418,500

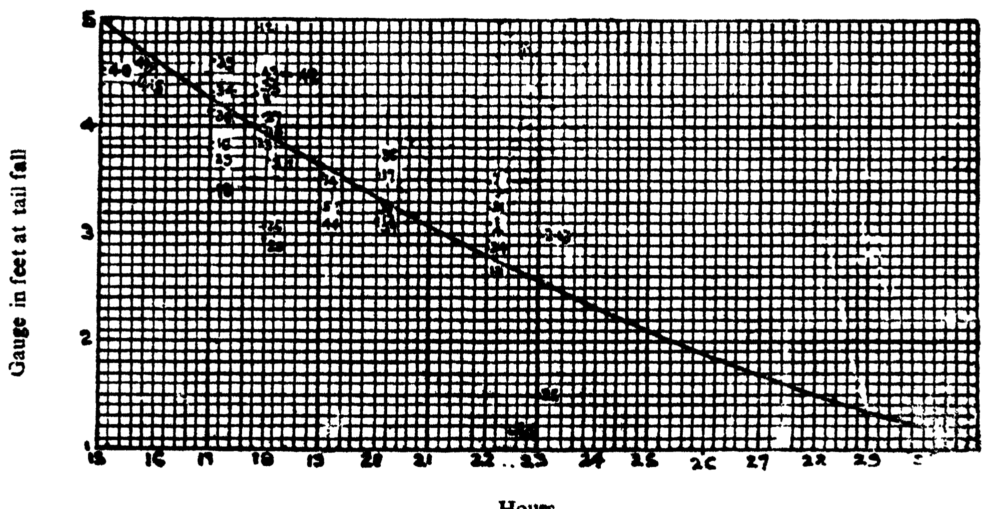


Fig. 14

(*ii*) The provision of automatic recorders to facilitate the detection of time-lag or, periods of steady supply.

(*iii*) The adoption of semi-modular falls or B.C. meters having a unique discharge gauge relationship, in order to eliminate the relatively large errors of observation that cannot be avoided when discharges are observed directly by current meter or velocity rods.

(*iv*) The adoption, where possible; of B C. meters of identical design, in order to eliminate the constant errors which arise when the falls or meters used are of different design and have therefore, to be calibrated independently by a series of special discharge-observations.

(*v*) The provision of double gauge-wells one on each side of the fall or meter to minimise the distributing affect arising from the tendency, in all silted channels, for the main current to sawing from side to side

(*vi*) The observation of water temperature and correction for viscosity.

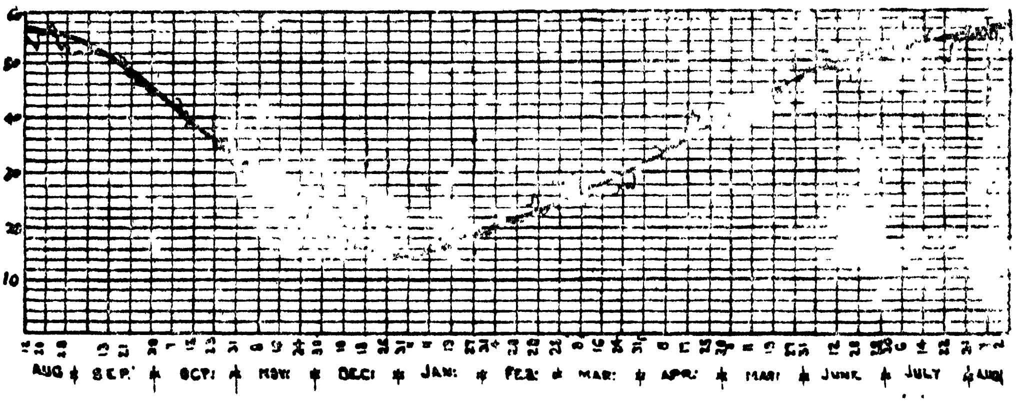


Fig. 15

Even when all such measures are employed, and the most elaborate arrangements are made to ensure accurate gauging, the author has found it impossible to obtain individual results which are fully consistent. Errors arise from causes such as wind, silt movement, and unavoidable fluctuation of discharge, and these are large enough to necessitate the averaging or statistical treatment of the individual.

In the case of Mangtanwala feeder meter flumes of identical standard design were especially constructed. A theoretical discharge table can rightly apply to all meters of identical discharges. The observations were continued for four months and steady periods of supply were analysed statistically by the author.

The results as obtained are given below :

No :	Reach.	σ at 20°C in cusecs Million square feet.
1	284000 to 309000 U.C.C.	$1{\cdot}6\pm0{\cdot}19$
2.	309000 to 328000 U.C.C.	$0{\cdot}47\pm0{\cdot}04$
3.	328000 to 347000 U.C.C.	$0{\cdot}22+0{\cdot}047$

(*b*) **Upper Jhelum Canal losses by meter observations.**

(*i*) **Description of the method.**

Two broad-crested meter flumes were constructed at Rashidpur and tail fall in 1932. Their design is identical in all important details. They have the same width and a common discharge table. Gauging sections are similar in design and have got masonry floors higher than the upstream bed levels. The upstream floor levels have been so designed that they remain without any silt even down to ⅓ of the full supply in the canal. There can be, thus, no error in the velocity of approach head. These flumes were designed by E. S. Crump, Esquire, C.I E., Superintending Engineer, U. J. C. Circle in 1932 and discharge tables worked out by him have been adopted in this analysis.

Both of these meter flumes were fitted with Legget Recorders and the recorder charts for both flumes are available from summer 1933. Continuous records of the discharges passed over the two meter flumes at Rashidpur and tail fall is available. Their difference, allowing for the time-lag, gives the water lost in the canal in this reach plus the discharge of the Gujrat Branch and the Main Line Distributares. There is a meter flume at about R. D. 1000 of Gujrat Branch. The gauges of this flume have been read hourly and the hourly plot of the corresponding discharges is available from 1933. The total discharge of mainline distributaries is only 350 cusecs, and the actual daily discharges in them have been taken from the gauge registers. It is, therefore, clear that the losses in the canal can be worked out daily with a fair degree of accuracy.

(*ii*) The results of analysis of three years record are shown in Fig 14 showing the time-lag between Rashidpur and Tail Fall, Fig. 15 showing the temperatures of water in this canal and Fig. 16 showing the losses.

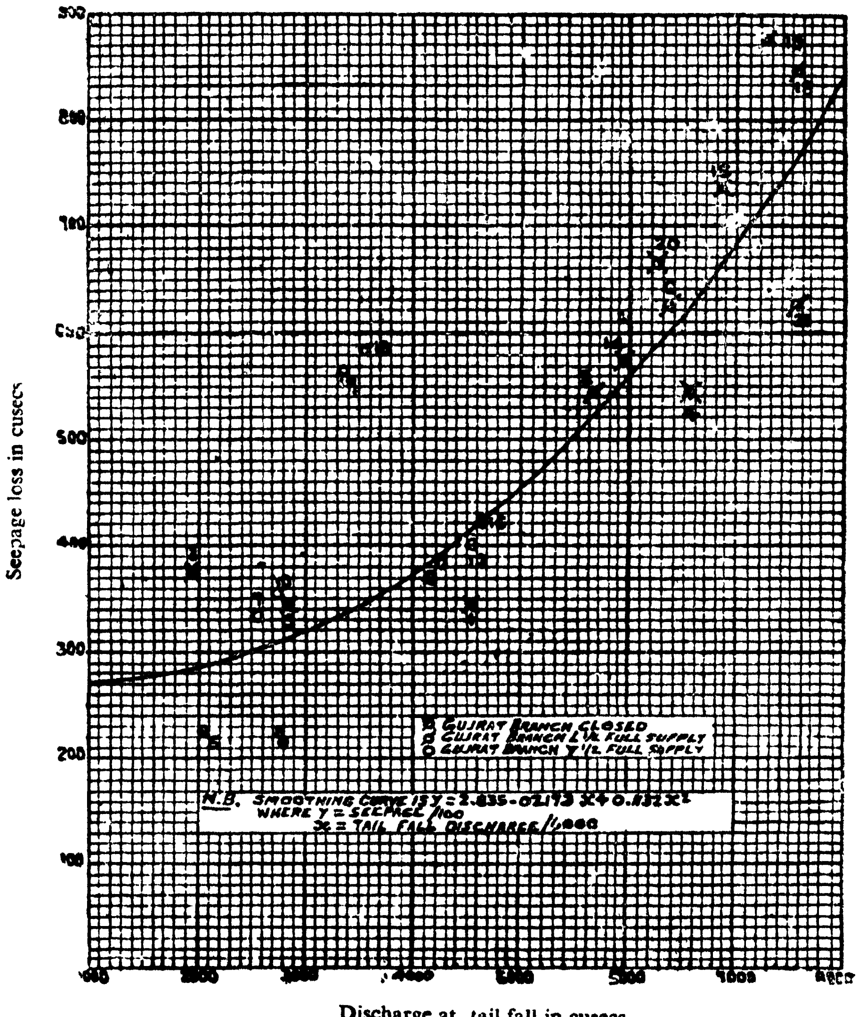


Fig. 16

(*iii*) Use of the diagram to get losses at any time from Rashidpur to Tail Fall is given below : —

Let discharge at tail fall	=6,000 cusecs
Date	= 15th January
Loss for 6000 as supply from Fig. 16 at 20°C.	=540 cusecs

Temperature on 15th January form Fig. 15	=8°C
Correction Factor 8°C from plate No. XXII	=+37%
Actual loss at the prevailing temperature	$=\frac{540}{1 \cdot 37}=394$ cusecs
Similarly let the supply at Tail Fall	=4,000 cusecs
Date	=15th June
Loss for 4,000 cusecs at Tail Fall from Fig. 16 at 20°C	=348 cusecs
Temperature on 15th June from Fig. 15	=25°C
Correction factor for 25°C from Plate XXII	=−11·3%
Actual loss at the prevailing temperature	=384/0·887=434 cusecs

16. Tank Observation.

The procedure of the tank observation is described in detail in author's paper Nos. 209 and 231 Punjab Engineering Congress, Lahore, and the results of the experiments at 14-R site have been discribed in this book. The percolation intensity coefficient was about unity in this with a soil crust with clay percentage 12 to 15%.

One mile long tank in the reach of the main line Upper Chenab Canal opposite Udeyra Rest House was filled with water with arrangements to maintain steady water levels by feeding over a standard meter flume. The seepage loss observations were taken to measure the seepage rate from the rate of feeding as well as by the point method at these places. The results as obtained by both methods with varying depths are shown in Fig. 17. They are plotted against the percolation head (the difference of the water level in the tank and the difference of average B.S P. levels). These results clearly prove that the loss from a canal was proportional to percolation head in the saturated phase with varying depths.

Tank Experiment at R D. 348,000 U.C.C.

Seepage Intensity Co-efficient σ

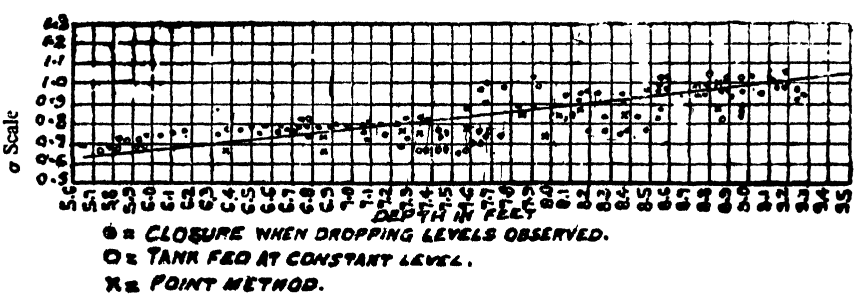


Fig. 17

The percolation intensity co-efficient (σ) in this reach was 0·11 cusec at 20°C per million sq. ft. (soil of the bed containing about 35% clay). The rate of loss was very low but still the abandining of reach was very successful anti-waterlogging measure, because in the tenacious clay soil of the tract the runoff was also very slow resulting in great heading up of the water table. The spring levels had dropped by about 4·0 ft. by abandoning this reach of the canal.

17. Evaluation of σ (P.I.C.) from Seepage Discharges in Closures.

(*a*) **Introduction.** E S. Crump, Esquire, C.I.E., I.S.E., Officer on special duty, Water-logging Investigation suggested that for channels which ran as drains during closures the well-known method of predicting yields be applied, by observing the actual seepage discharge in short reaches and by determining in those reaches the seepage influx heads. This method was tried by the author for the first time and applied on the upper Chenab Canal from head to R.D. 40,000 U.C.C. Lower in the December 1948 closure.

TABLE 5.

Valuation of losses on 31-12-1938 in Upper Chenab Canal.

No 1	Reach in thousand ft. 2	Head 3	σ at 20°C 4	Perimeter in mil. sft. 5	Actual temperature 6	Temperature C F. 7	Loss or gain in cs. 8	Remarks 9
1	0—10	+ ·82	25·5	2 67	7 2C	·60	+34·8	71·9 × ·5 = + ·36 gains in head-reach ar: halved as the river is quite near on the right and effective heads will be half nearly—165 6
2	10—20	+1·00	10·3	2·85	7·2 ,,	·60	+17·6	
3	20—30	+ ·96	8·9	2·85	7·7 ,,	·615	+14 9	
4	30—40	+ ·5	5·42	2 75	7 7 ,,	·615	+ 4 6	
5	40—50	—1·73	1 47	2 82	8 3 ,,	·64	— 4 6	
6	50—60	—1·5	2·15	2 79	9 4 ,,	·68	— 6 1	
7	60—70	—0 88	3·41	2·84	10·0 ,,	·7	— 6 0	
8	70—80	—1 42	2·80	2·85	10 0 ,,	·7	— 8·0	
9	80—90	—2 41	2·21	2 83	10 0 ,,	·70	—10·5	
10	90—100	—3 55	3·56	2 82	11 0 ,,	·735	—25 0	
11	100—110	—4·80	1·60	2·80	11·0 ,,	·735	—15 9	
12	110—120	—2 40	1·78	2 84	11·5 ,,	·76	— 0 25	
13	120—130·5	—2 51	4 45	2 84	11·5 ,,	·76	—32 8	
14	0—10	—2 24	6·48	2 60	12 0 ,,	·775	— 3·2	
15	10—20	—1·1	5 82	2 60	12 2 ,,	·790	—13 2	
16	20—30	—4 80	1·60	2 60	13 0 ,,	·815	—16 3	
17	30—40 5	—12 87	0·52	2 70	13 0 ,,	·815	—14·75	
	Net loss = 165 6—36=129·6 say 13 susecs							

TABLE 6.

Valuation of seepage losses on 18-1-1939, L.C.C.

No 1	Reach 2	σ at 20°C 3	Perimeter miliion sq. ft. 4	Head 5	Actual Temp. 6	Temp. C. F. 7	Loss or gain 8	Remarks 9
1	0-10 L.C.C.	1 90	2·63	—0·34	10 C	0·71	1·75	
2	10-20 ,,	1·95	2·76	—0·34	10 ,,	0·71	1·47	
3	20-30 ,,	2·91	2·77	—0·99	10 ,,	0.71	5·70	
4	30-40 ,,	3·13	2·86	—0·89	10 ,,	0·71	5 60	
5	40-50 ,,	0·80	2·72	—0·87	10 ,,	0·71	1·35	
6	50-60 ,,	11·45	2·89	—0 87	10 ,,	0·71	1·35	
7	60-70 ,,	8·85	2·95	—0·49	10 ,,	0 71	7·95	
8	70-80 ,,	2·12	2 90	—1·00	10 ,,	0·71	4·30	
9	80-90 ,,	0 70	2·91	—1·20	10 ,,	0 71	1·60	
10	90-100 ,,	0·90	2·92	—1 40	10 ,,	0·71	2 60	
11	100-110 ,,	18·00	2·88	—1·50	10 ,,	0·71	55·00	
12	110-120 ,,	0·65	2 89	—2 00	10 ,,	0·71	40·00	
13	120-130 ,,	1·28	2·66	—3·50	10 ,,	0 71	8·50	
14	130-140 ,,	1·33	2·70	—2 40	10 ,,	0·71	6·20	
15	140-150 ,,	1·04	2·52	—5·37	10 6 ,,	0·725	10·20	
16	150-160 ,,	0·292	2·71	—9·64	10·6 ,,	0·725	16·10	
17	160-170 ,,	0·256	2·56	—9 50	10·6 ,,	0·725	4·50	
18	170-180 ,,	0·31	2·65	—8·58	10·6 ,,	0·725	5·20	
19	180-190 ,,	0·45	2 79	—7·80	10·6 ,,	0 722	7·10	
20	190-200 ,,	1·10	2·62	—7·85	10·6 ,,	0·725	16·30	
						Total	221·17	
21	0-10 Jhang	1·22	1·72	— 7·35		·725	11·20	
22	10-20 ,,	0 65	1·53	— 7·86		·75	6 40	
23	20-30 ,,	0 314	1·54	—10 03		·75	3·75	
24	30-40 ,,	0 346	1·45	—11·56		·75	5·90	
25	40-50 ,,	0 584	1 46	— 9·71		·75	6·20	
26	50-60 ,,	0·20	1·32	—16·86		·75	3·40	
27	60-70 ,,	0·236	1 30	—14·85		·75	3·50	
28	70-80 ,,	0·62	1·43	— 9 81		·75	5 50	
29	80-90 ,,	0·56	1·45	—11·40		·75	7·00	
30	90-100 ,,	0 57	1·61	—11·20		·75	7 80	
31	100-110 ,,	0 54	1 51	—13·26		·75	8·10	
32	110-120 ,,	0 50	1·55	—14·09		·75	8 20	
33	120-130 ,,	0·52	1·58	—18·87		·75	11·60	
						Total	85 55	

(*b*) **Description of the method.**

This method simply consists of measurements of seepage water and their extinction to a dead end in the various reaches of a canal. The head regulator of the Upper Chenab Canal was tamped so that no water escaped into the canal throughout the 24 days closure. The channel was divided into small reaches as shown in tables Nos. 5 and 6 of this chapter. A bridge or a fall was selected to be the terminal point of every reach where accurate discharge observations could be taken. The seepage water was run through a single bay at every discharge site. This ensures silt free and definite sections on the floors of the bridges. The errors of discharge observations were very much reduced by thus providing definite and accurately measureable sections. The velocities were taken under the bridges by using velocity rods and their values were corrected applying Bellasis coefficients for rod observations In the case of free falls, the discharges were calculated by using the broad-crested weir formula corrected for friction. A greater accuracy of discharge observations was thus ensured than is possible in earthen irrigation channels. The difference of the water level in the B. S P. pipes and the seepage water level in the canal gave the seepage head at the place In the table No. 5 and 6 the infiltration head is shown as positive and the exfiltration head is shown as negative.

The seepage loss or gain in a particular reach is worked out as the difference between the discharges observed at its beginning and its end. Let it be denoted by Q cusecs. Average head has been worked out by taking the arithmetical mean of the seepage heads measured at every mile. Let it be denoted by H feet. Actual water surface widths were measured every thousand feet and the wetted perimeter of the reach was computed. Let it be denoted by P. Darcy's seepage law in the fully saturated conditions can be written in the simplified from

$$\frac{Q}{P} = \sigma H \therefore \sigma = \frac{Q}{PH}; \text{ where } \sigma \text{ (P. I. C.) at } 20^{\circ}C \text{ per foot head.} \qquad (1)$$

This straight line relation of seepage losses has been proved in the field experiments carried out by tanks at R. D 384,000 of Upper Chenab Canal Lower and at 14-R. Distributary experimental site on Upper Jhelum Canal.

(*c*) **Limitations of the use of this method.**

This method of determination of the seepage losses is applicable only to the reaches where the losses are in fully saturated conditions. In unsaturated conditions the law of the seepage losses is not a straight line one and the losses are almost independent of the head.

There is one point in this method which needs special condsideration In the head reach of this canal up to R. D. 80,000 the seepage intensity coefficient has been worked out from the seepage gains and will be applied to the case of losses in full supply conditions. One may question if the equation (1) is upset by a change in the geometry of the stream lines from the percolation to the infiltration conditions. The qualitative work to investigate this phase of the problem has already been done in the Irrigation Research institute, Lahore by Hele-Shaw methods. The stream lines were determined using a trapezoidal section, both as a sink (that is a drain) and a source (that is a canal). The stream lines in both cases were exactly similar. One may easily mistake the one for the other. Two tracings from photographs are given in Fig. 7 (canal) and Fig. 8 (drain). There seems to be no doubt that the equation of flow both in the cases of gains and losses is the same.

(*d*) **Results.**

The metods of calculating the discharge are indicated in tables 5 and 6 which also give the actually observed percolation intensity co-efficients.

(*e*) This method of calculations was applied for the whole year from December 1938 to December 1939. Monthly average value of losses corrected to 20°C temperature is plotted in Fig. 18 for the U.C.C. Main Line. The daily discharge observations were also being taken in this period. The results of losses got from these are also shown therein in dotted. The variation of loss with different supplies is shown in Fig. 19. The calculated results by the above-described method

Calculated loss at 0°C = ●
Actual loss from discharge observation = ×
Calculated from n = O

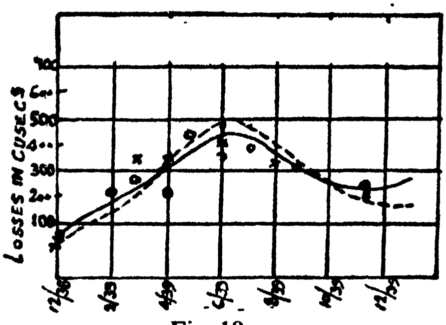


Fig. 18

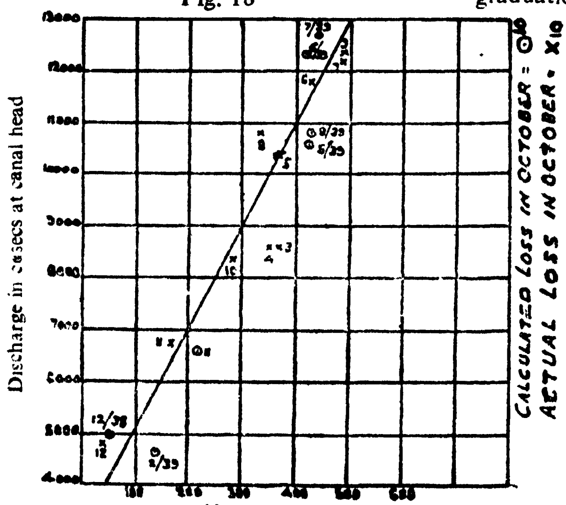


Fig 19 Actual losses in cusecs.

agree with those observed by discharge observations, Fig. 20. shows the temperature variation for this canal.

18. Point Method.

(a) Point method apparatus.

(1) Description of the apparatus.

The apparatus used is sketched in Plate No. ɪɪ It consists of a 10 inch dia pipe cylinder. This pipe is sunk at least a foot below bed and it projects about half a foot above the water level in a channel. The Pipe is provided inside with a welded flange $1\frac{1}{2}''$ wide 12" below its top. Six 3/8" dia. bolts are fixed water tight to this ring flange. Conical Plate P is secured water tight to the flange F by using $\frac{1}{4}''$ thick rubber washer, shown as E in Fig 21. Special tightening wrench has been devised to tighten the nuts. D is a brass knob and forms the apex of the conical plate. A is a special tube of $\frac{1}{4}''$ diameter which has graduations showing one tenth of a cubic centimeter. The tube is provided with a metallic base H which is secured water-tight to the knob D by screwing and a rubber washer is provided in between. The arrangement above is suitable to drive out all air from the cylinder.

B is a plain glass tube of $\frac{3}{4}''$ diameter. N is a brass nipple screwed to the pipe with its outside surface flush with the pipe. B is connected to the nipple N by means of a rubber tube T. Similarly C is a plain tube of $\frac{1}{4}''$ diameter and is connected to another glass tube G of $\frac{1}{4}''$ diameter by means of a rubber tube connection T. The tube G has a finely cut end at M and is secured about an inch away to the pipe by means of the clamp K The tubes A, B and C are clamped together so that the menisci in them are in the same plane.

The pipe is driven in the bed of a channel by using a light monkey weighing about 2 maunds. The driving arrangement is sketched in Fig. 21.

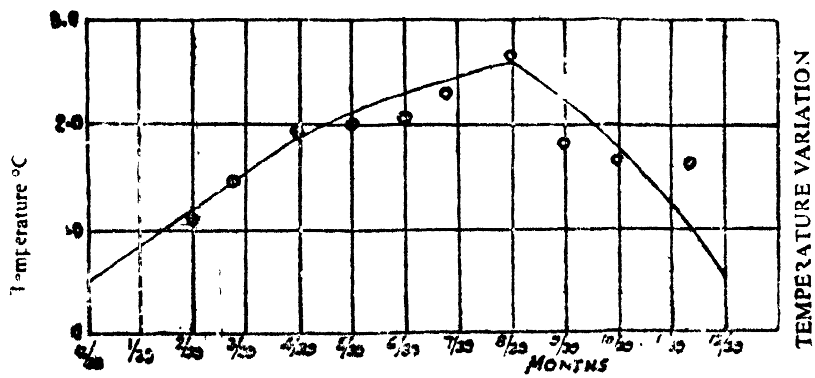


Fig 20

The pipe is driven in the middle of the channel and a temporary planked platform is erected from the berm of the channel to the pipe. The platform is made up of easily detachable units and is portable.

Procedure in the observations.

A temporary platform and the observation pipe are erected as explaind before. The conical covering plate and the graduated tube are fitted as shown in Fig. 21. Water above the cover Plate is then removed. The pipe A is filled with water to test leakage through any of the joints. All the joints have got rubber washers and are easily made water-tight.

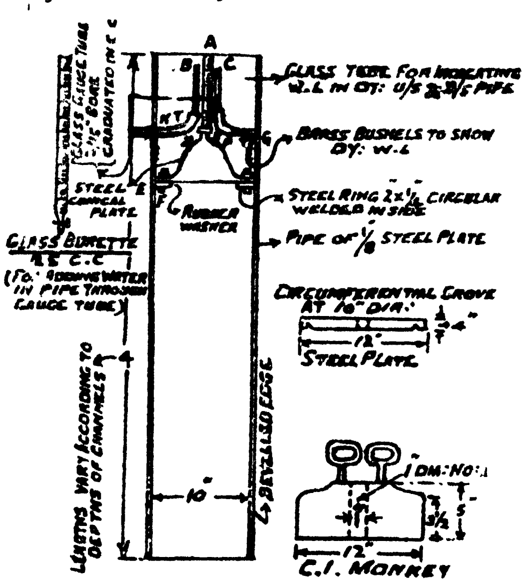


Fig. 21

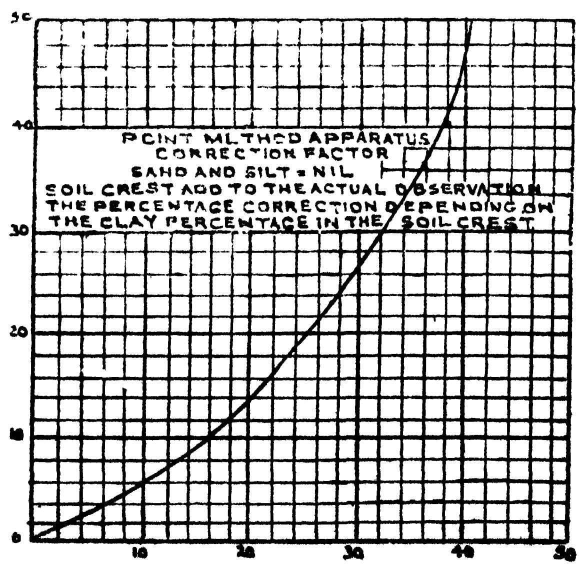
The tubes B and C are then fitted. Tube B records the water level in the channel downstream of the pipe. The tube C gives the water level upstream of the observation cylinder. The mouth M of the pipe G is kept about 6 inches away from the cylinder so that it is not affected by the impact against the pipe because of the velocity of approach. The gauge readings as read from the inscribed gauge on tube A are recorded and the average of the two readings is considered to be the correct average water level outside the observation cylinder.

The readings of the water meniscus are taken by putting an inclined looking glass in the cylinder on the over plate. The reflection of the scale is magnified according to the inclination of the looking glass In practice, a magnification of the reflection to double the scale is possible which is enough to help in the precise regulation of constant level in the pipe A.

The observation is taken by feeding the pipe A from a graduated burrette. The readings of the water level in the

Clay Percentage

Fig. 22



Percentage correction factor

burrette are recorded before and after the observations and the time is recorded by means of a stop watch. Generally the observation is taken for one minute and the water lost is measured from the burrette in cubic centimeters.

Loss in cusecs per million sq. ft. is cubic centimeters lost per minute divided by six. The calculations of this conversion factor are given in table No. 7 of this chapter.

The temperature of the canal water is recorded by suspending a maximum and minimum thermometer to the bed of the channel. The thermometer is allowed to lie on the bed for five minutes and the minimum reading recorded there by the movement of the pointer is taken. The results are reduced to 20°C by the use of the temperature correction diagram plate XXI, Volume III.

(*iii*) **Correction factor.**

When the apparatus is sunk obout 1·5 ft. below bed there is bound to be some vibration which shall result in compacting the soil to some extent. The apparatus results were checked against other methods at the Udeyra tank and at 180,000 L. C. C. site. The observations were also compared with the method described in paragraph 16 by observed inflow in closure. These results were absolutely correct in the case of sandy silt ded, in the case of soil crust containing clay the correction was necessary. The correction is shown in Fig. 22 depending on the percentage of clay in the soil crust.

TABLE 7.

Losses by point method from Mangtanwala Branch.

1	2	3	4	5	6
No.	R. D.	Loss in cusecs million sq. ft.	Temperature in°C	Loss at 20°C	Remarks.
1	1000	0·72	25·0	0 63	All observations taken in middle of the channel
2	3000	1·44	23·9	1·32	
3	5000	0·96	23 1	0 892	
4	7000	1·01	22 8	0 957	
5	9000	2·4	18 3	2·50	
6	11000	1·5	17·2	1 605	
7	13000	1·23	15 6	1·37	
			Average ..	1 328	

Calculations of the conversion factor.

Pipe diameter = 10″

Sectional area = ·5454 sq. ft.

Loss per minute = C cubic centimeter.

1 cubic centimeter = ·0000054 cubic sq. feet at 20°C.

Loss in cusecs per million sq. feet = $C \times 10^6 \times \cdot 0000054 \times \frac{1}{\cdot 5}$ = C/6 nearly.

TABLE 8.

Statement showing Seepage losses observations by Point Method on Buchiana Distributary from 0 to tail (40,000).

R. D	Seepage losses observations in cusecs per million sq. ft.	Temperature data in °F	Absorption losses at 20°C average temperature	Remarks
1	2	3	4	5
500	2 45	63·9	2·63	
2500	1 66	63·5	1 77	
5000	2·25	64·0	2 38	
7500	1·29	64·0	1·36	
10000	1·29	62·0	1·41	
12500	1·13	62 0	1·23	
15000	1·05	63·0	1·13	
17500	0 40	64 0	0·42	

(Cont.)

1	2	3	4	5
20000	1 68	60 0	1 91	
22500	1 16	60 0	1 31	
25000	0 97	60 0	1 10	
27500	0 15	62 5	0 16	
30000	0 38	62 0	0 41	
32500	0 31	64 0	0 33	
37500	0 42	60 0	0 48	
40000	0 009	62 0	0 01	
42500	0 007	63 0	0 008	
45000	0 006	64 0	0 007	
47500	0 006	64 0	0 007	
		Average	0 930	

TABLE 9.

Statement showing Pt. Method Seepage Observations on Paulinni Disty. of L. Gugera Divin.

Date	R D of observations	Centre of the bed of the Disty: Absorption in cusecs per million sq ft	Temp: to °C	Loss at 20°C
1	2	3	5	5
17-2-39	200	5 52	12 2	08
,,	2500	6 63	12 3	8 50
18-2-39	5000	1 49	13 1	1 85
18-2-39	7500	2 85	13 2	3 52
19-2-39	10000	2 89	13 2	3 57
19-2-39	12300	1 82	13 2	2 22
1-3-39	13000	1 2	14 4	1 5
,,	17500	0 23	15 6	0 26
7-3-39	22500	1 64	15 8	1 84
,,	25000	4 22	16·4	4 64
,,	27500	1 98	17 1	2 14
,,	30000	1 63	17 2	1 75
8-3-39	35200	1 72	16 9	1 87
,,	35000	1 79	17 2	1 92
,,	37500	1 74	17 2	1 87
,,	40000	3 54	17·2	3 61
			Average	3 01

*Very serverely cold wind blowing.

Note:—There was heavy rains on 27-2-39 and 28-2-39 following by hail storm and also on 16-2-39

TABLE 10.

Statement of Seepage Losses in Jaranwala Distributary by Point Method.

R. D	Temperature in °C	Seepage losses in cusecs per million sq ft.	Seepage loss corrected to 20 °C
1	2	3	4
2000	16 9	0 91	1 00
2500	17 8	1 22	1 29
5000	17 8	0 64	0 68
7500	19 2	1 66	1 70
10,000	20 0	0 56	0 56
12,500	20 6	1 62	1 65
15,000	19 2	0 67	0 69
17,500	20 0	0 35	0 35
20,000	20 6	0 15	0 16
22,500	20 0	0 42	0 42
25,000	21 1	0 09	0 10
27,500	22 2	0 17	0 18
30,000	20 0	0 09	0 09
32,500	22 5	0 22	0·23
35,000	20 0	0 18	0 18
37,500	20 6	0·14	0·15
40,000	22·8	0 10	0·11
		Average	0·56

(*b*) **Trough method.**

(*i*) The suggestion about the method of observation originated from E. S. Crump, Esquire, C. I. E. on the eve of his retirement in 1937. It was proposed to construct water-tight inverted troughs accross the canal section and to measure the losses from the measured additions in a known time to keep the water level constant and equal to the canal level in area enclosed by the trough. This is a from of the Point Method observations of seepage losses in a canal and is supposed to have been tried for the first time. The design of the actual structure was very carefully devised by Nand Gopal and the author, and the troughs were successfully constructed.

(*ii*) **Design of the inverted troughs.**

The design of the troughs at both sites was similar and is given in Fig. 23. An arrangement was provided at R. D. 257,000 so that the side could be cut off and the losses could be measured from the bed alone. However, no observations were taken by omitting the side on account of the transfer of the author. Three B. S. P. pipes were put in, to record the spring levels at each site, one in the centre and the other two on the berms, one on either side. At R. D. 263,000 four additional pipes were put in, two upstream and two downstream to measure water table gradients along the length of the canal. The pipes at R. D. 257,000 were driven 40 ft. below the bed and were provided with 30 ft. filter. The pipes at R. D 263,000 were driven 30 ft. below bed and had 3·0 ft. filters. All pipes were considered to be sufficiently deep to record B. S. P. levels. The B. S. P. levels observations on these $1\frac{1}{2}''$ diameter pipes were recorded daily by using Khosla's sounder.

Results at R. D. 257,000 U. J. C.

The results obtained at this site are plotted and tabulated in table 11. Daily observed loss in cusecs per million square feet corrected to 20° temperature, the seepage head and the depth are plotted therein.

The losses were of a relatively higher order to begin with from 11-4-38 to 27-4-38 when

Flume at R. D. 256,000 of main line U. J. C
Section of Flume Scale=1/200

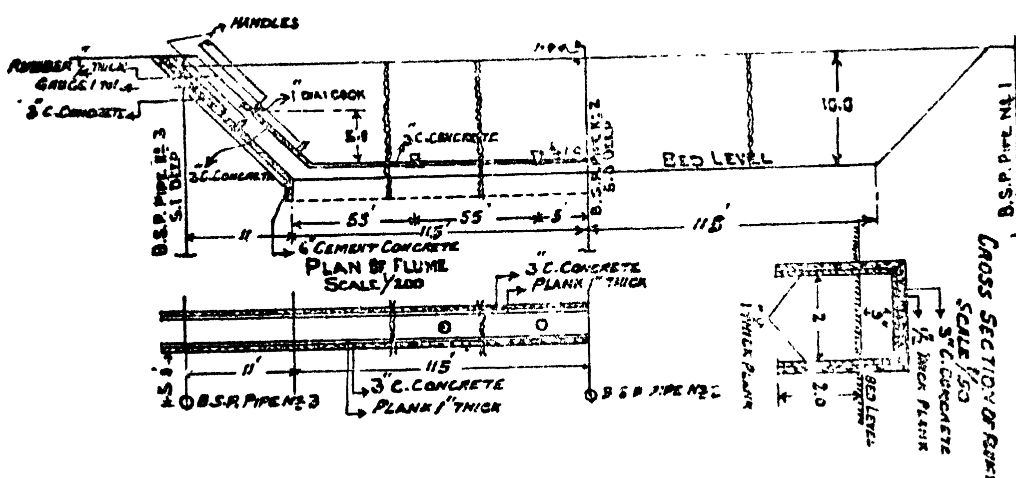


Fig. 23

the trough flumes started working. The reduction in the beginning may be due to initial staunching. Normal losses of steady nature about 6 cusecs per million sq. feet were recorded from 28-4-38 to 19-5-38. In this period, depth was about 8·0 feet and seepage head 21 feet that is, spring level was 13·0 ft. below bed. There was a closure from 30-5-38 to 13-6-38. For the first few days after the closure, the losses were high which is likely. The soil pore below bed which dried up in closure filled up to the saturation line below bed. The seepage head was 27·2 feet and depth was very nearly steady 6 ft. from 14-6-38 to 29-8-38, and the losses in this period were very nearly steady about 4·5 cusecs millions sft. The seepage head in this period dropped from 27·2 ft. to 22·20 ft. by about 5·0 ft. This means that the water table rose from 21·2 below bet to 16 0 ft, below bed with no change in points.

The results of the period form 30-8 38 to 20-10-38 are very peculiar. The depth in the canal rose from 6 ft. to 9 7 ft. and then fell to 5·6 ft. The losses also increased to 9·5 cusecs per million sq. ft. when the depth was the maximum and then fell to about 5 cusecs per million sq. ft. The seepage head in this period steadily dropped from 22·2 ft. to 17 2 ft. which means that spring level rose from 16·2 ft. below bed to 11·6 below bed.

(*iii*) **Discussion of results at R. D. 257,000 U. J. C.**

It is apparent from these observations that during the course of the experiment spring level remained far below the bed from 11·6 to 21·2 ft. The losses were in the unsaturated phase. This is definitely clear that the losses in the unsaturated phase were not effected by the position of the water table below bed and its movements.

These experiments have shown that the losses in the partially saturated phase are influenced by the depth in the canal. These losses increase with the depth. Roughly speaking, the loss in cusecs per million sq. feet at this site was the same as the depth in feet in the canal. This should be considered as a local peculiarity corresponding to the available soil conditions. There are not enough data to predict for all cases the increase in losses in the partially saturated phase due to an increase in the depth.

(*iv*) **Results at R. D. 263,000 U. J. C.**

The results at this site are available from 1-8-37 to 20-10-38. The spring level changed in this period from 3·0′ below bed to about 4·5 ft. above bed. The losses were definitely in the saturated phase, throughout the course of observation.

The losses at this site were measured to be as high as 35 cusecs per million sq. ft. and as low as about 3 cusecs per million sq. ft. The soil at this site was all sandy upto 30 ft. below bed. A study of the high peaks and the plot of spring levels in the Long Section gives a clue about the cause of the sudden jump in the losses. High peaks always occur when the water-table is rising and suddenly drops down when it is falling. This means that when the hump was being filled up, the losses were abnormally higher and when it was being depleted, they were relatively lower than their normal value. Three pipes across the canal were fixed to begin with and 4 more pipes were put in along the length of the canal from November 1937. Vector slope of the water table at this site was also observed. This is the resultant of the cross slope and that along the length of the canal. The resultant slope was always towards Rasul and has been shown in feet per thousand feet. The peaks in the losses are situated where the vector slope was high, but sometimes the effect was felt after a variable time-lag.

TABLE 11.

Trough Experiment at R. D. 263,000 U. J. Canal.

No.	Period	Average seepage in cusecs at 20°C	Average head in feet	Percolation at 20°C	Remarks
1	2	3	4	5	6
1	27-11-37 to 13-12-37	5·43	5 54	0 972	
2	19-1-38 to 31-1-38	6·80	6 77	1·0	
3	26-2-38 to 28-3-38	5·56	5·5	1·008	
4	21-4-38 to 30-4-38	5 42	6 01	0 904	Average σ= 0 955
5	15-10-38 to 27-10-38	4 21	4·58	0 910	
6	23-8-38 to 9-9-38	5·35	5 86	0·912	
7	23-9-38 to 10-10-38	7·10	7 3	0 974	

The abnormalities as explained above are therefore a special peculiarity of this site due to a very steep slope towards the river and due to the spring level being very much above bed. For the purpose of the analysis of the data, they have been ignored. The analysis of results is restricted to the periods of sufficient duration when the vector slope was constant, that is, when the condition of the hump were stable. Seven such periods are tabulated in Table No. 11. Daily seepage intensity co-efficients have been worked out in Col. No. 5 according to the following formula :—

$q = \sigma H$; q is the Loss in cusecs at 20°C ; H the Seepage Head.

σ the Soil constant (P. I. C.) Percolation Intensity Co-efficient Head.

Average Percolation intensity coefficient per foot seepage head is 0 254. The formula above was confirmed to be correct. The depth had no effect and the seepage varied with H only.

19. Statistical Methods.

20 years recorded discharge at the head and the tail of main and branch canals Lower Chenab Canal system were statistically analysed by T. Blench I. S. E. The off-takes discharges were also taken as recorded.

The method of averaging adopted was as follows :—Twenty results for each month were averaged vertically to give monthly averages. The latter averages were then averaged horizontally to give the grand yearly average of all months and all years. From the results it is required to know :—

(*i*) The raliability of individual results.
(*ii*) ,, ,, ,, monthly averages.
(*iii*) ,, ,, ,, the grand averages.

Difference between individual and average values is known as "deviation". The λ or "standerd deviation" for individnal results is merely the root-mean-square of individual "deviations", and represents the average error (or deviation from the average) to which individual results are liable. The "probable error" is $\frac{2}{3}\lambda$ (expressed as percentage of the average value).

The results obtained for avarage losses in a year were as below :

TABLE 12.

Channel					Average for 20 years	Average for last three years
Main Line	...	...	...	...	347	288
Upper Gugera Branch	...	...	...	...	231	246
Lower Gugera Branch	...	...	...	...	279	343
Burala Branch	...	...	..	...	128	161
Jhang Branch Upper	...	...	..	...	270	346
Jhang Branch Lower	...	...	...	...	111	164
Rakh Branch	...	...	...	...	55	104
			Total		1421	1552

The above figures represent losses under full supply conditions ; and in calculating all-the-year round additions to the water table, due allowance must be made for closures. The variation of the loss throughout the year by months is given in Figs. 24 to 29 for various branch canals of L. C. C.

20. By Observing Sub-Soil Water Table Profile Away from the Canal.

This is described in detail in Research Publication Volume V, No. 7 1938. The observation required is the ground water level as shown in Fig. 30.

The flux or the loss is then calculated from the equation

$$F = vd_1\left(\frac{1}{\cos\theta} + \frac{\sin\theta}{\theta}\right)$$: F is the Flux or rate of loss from the canal

θ is the Inclination of the stream line with the critical ; d_1 the Horizontal distance of the point of observation on the outermost stream ; v the Velocity in feet per hours along the stream line

The losses abserved along Jhang Branch were as below :—

(1) R D. 3337, 15·8 cusecs per million sq feet.
(2) R. D. 7260, 11·5 cusecs per million sq. feet.
(3) R. D. 1260, 12·8 cusecs per million sq feet.

This method is applicable to the losses in the saturated phase only. It presupposes that whole or major portion of the loss flows away along the sides and very little from bed. The author had occasion to check this method at R. D. 263,000 U.J.C. when the B.S.P. was about 3·0 ft. above bed and depth in canal 10 feet. This method showed very little loss while actual loss by trough method is given in Table No. 11.

21. Examination Questions.

(1) (*a*) Define the following terms applied to running canals :—
(1) Absorption. (2) Peroclation. (3) Seepage.
(*b*) State the laws of the seepage losses from the canal in the different phase of the saturation of the soil below the bed of a canal. (*P U. 1940*)
(2) State the various methods of seepage loss measurements in canals.
(3) Describe the point Method apparatus used for obsorption measurements of losses in Distributaries
(4) (*a*) Is seepage loss more from the bed or sides ?
(*b*) What is the effect of depth both in the case of absorption and percolation ?
(5) Assuming that brick lining saves 75% of the absorption losses of an earthen channel and that the average value of a causec capacity is Rs 1800/- per annum, what expenditure would be justified on a lining project for a Branch canal 30 miles long ? The lined channel has the following particulars :—
Bed width=60 feet ; Depth=12 feet; side slope=1 in 2 feet and Q=3500 cusecs.
Absorption loss in the lined channel
$K=1{\cdot}25Q^{\cdot 056}$ where K=absorption per million sq. ft. of wetted percolation ;
Q=Discharge of channel in cusecs,
Note :—A canal mile may be taken 5000 ft. long and rate of interest 4 percent per annum. (*P.U. 1944*)
(6) (*a*) What is the most practicable method of measuring seepage from canal ?
(*b*) By what method is it economically possible io ascertain whether there is heavy seepage even in short length of canal ?
(*c*) Is there evidence to show that general seepage is greater through the bed of a canal than through the banks ?
(7) Is there anything to show that seepage depends on (*i*) supply level in the channel (*ii*) the difference between the supply level and the sub-soil water level (*iii*) the age of the channel ?

PART V

GROUND WATER ENGINEERING

CHAPTER V

Water-logging and Anti-water-logging Measures

1. Introduction.

A land is said to be water-logged when the soil pores within the root zone of the normal crops are effectively saturated to cut off the normal circulation of air. The yield is then reduced below the normal. The position of the water-table below the ground surface depends on the height of the capillary fringe, the soil is capable of supporting. In the Punjab soil crust. the normal height of the capillary fringe is about 3·0 ft. When the capillary meniscus surface above the capillary fringe is within 2·0 ft. of the ground surface the crop yield is affected. Usually the land is said to be water-logged when the water-table is 5·0 ft below the ground surface. In cases, where the capillary fringe has been measured to be 11 ft. high, the ground shall be water-logged with the water-table 13 0 ft. below.

The destruction of crops by flooding should be taken to be quite distinct from that by water-logging. In that case water standing for a long time kills the crops, by completely filling the soil pores by its percolation into the soil. The term water-logging is reserved for the deterioration of land by the rise of the water-table. In southern India. the term damaged land is used when the crop yield is affected and water-logged lands are said to be those when the ground water rises to the ground surfase.

In the Punjab. the term *sem* land is used when the full saturation of the soil is at ground surface on accouut of rise of water table and nothing can grow in this case. This happens when the water-table is 2 to 3 feet below the normal soil crust or at the ground surface in the case of the sandy soils.

The depth of the water-table below the ground surface which will reduce the yield of the crop depends on the various crops sown as determined by the depth of roots going into the soil crust, when the water table is within the following depths, the yield of the various crops will be affected :—

Wheat 3′ to 4′ ; Cotton 5′ to 6′ ; Rice about 2 ft;. Sugarcane 3·0 ft. Fodder crops 4·0 ft. except Lucrene yield of which shall be affected when the water table is 7′ to 8′.

2. Factors Responsible for the Infertility of Water-Logged Lands.

Dr. Mackenzie Taylor gave a very lucid reply to this question as pulished in the questionnaire on water-logging by the Central Board of Irrigation. India, Delhi and it is reproduced here for the convenience of the students.

The following are the main causes of infertility in water-logged soils :-

(*i*) The anaerobic conditions in the soil.

(*ii*) The difficulty of carrying out cultivation operations.

(*iii*) The competition between the crop and the natural flora of water-logged soils.

(*iv*) The possibility of a high concentration of sodium salts in the surface layer which in themselves may be toxic or lead to the formation of alkaline conditions.

With reference to (*i*) the growth of normal cultivated crops is dependent upon an adequate supply of nitrogen in the form of nitrates. The process of nitrification, as it is called, is carried out by bactera which require for their activity the supply of oxygen. If the supply of oxygen in the soil is reduced due to the presence of excess water, then the nitrification process does not take place. Instead, there appears to be a loss of nitrogen from such soils. The anaerobic conditions in the soil, therefore, are only indirectly felt by the crop, since the main effect appears to be on the micro-biological activities in such a soil. Drainge, by keeping the water-table in motion, brings air into the soil to replace the water abstracted by the drains. It is on account of the increased aeration that crop growth is so successful after drainage.

With regard to No. (*ii*) the difficulty in carrying out efficient cultivation is mainly of two kinds :—

(*a*) The limited period during which cultivation operations can be performed owing to the wet condition of the soil.

(*b*) The difficulty which is always experienced in obtaining a good tilth on soils in which excessive moisture is present. Drainage improves the condition under which cultivation takes place and also allows for the alternate drying and wetting of the soil which is so essential for the production of a good tilth.

With regard to No. (*iii*) water-logged lands have a characteristic flora which usually makes luxuriant growth. One of the difficulties in establishing a cultivated crop is the competition which is experienced between the seedings of the cultivated crop and the natural flora of the area. Unless water-logged land is frequently hoed to keep down the natural flora, it is impossible to produce a paying crop. The expense connected with the control of weeds is one of the main difficulties in keeping land of this nature under crop.

Regard No. (iv) the presence of the salts is not a general characteristic of water-logged soils. When, however, salts are present their concentration may be such that cultivated crops cannot grow The second effect of salts is the reaction which takes place between the soil and the salts resulting in the production of an alkaline soil, and consequently the anaerobic conditions, already discussed, may be intensified.

3. Water Requirements of Crops Reduced in Water-Logged Areas.

The nearer the water table is to the surface the more can the crop draw upon the water present in the sub-soil and hence, the less that need be given in the form of irrigation. No definite experiments have been laid down to determine the amount of water which would be given to a crop with various depths of water table. Experience at Chakanwali has shown that with crops such as sugarcane, cotton, wheat, with a water-table about 2½ feet below the surface, 30 percent less water is required to mature the crop than in areas where the water table is situated below 10 feet. Successful *Barani* sugarcane crops have been raised in the Chakanwali farm (water logged area) in the Punjab. It follows that when the water table is within 10 feet of the surface, then water removed by the crop may be wholly or partly

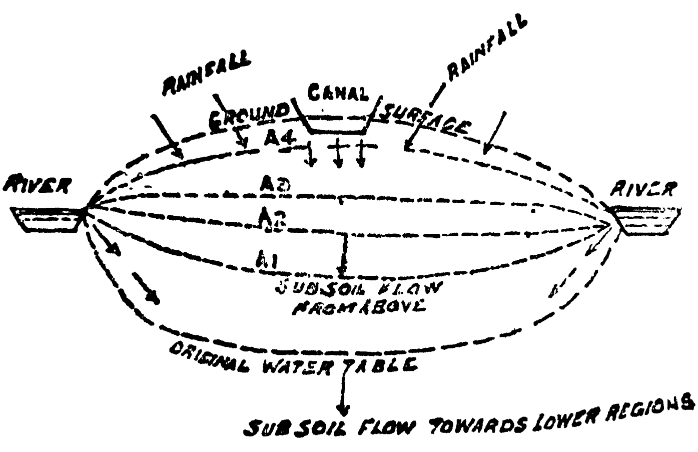


Fig. 1

replaced by that rising from the sub soil water table. When the water tables is situated below 20 ft. from the surface, it will have little influence on the water content in the surface soil as the water table approaches the surface. No evidence recording the amount of water required when the water table is only two feet from the surface is available. So far as evidence in the Punjab goes at present, when the water table is beyound 10 feet from the surface, then the whole of the water required by the crop must be supplied by irrigation in the absence of rainfall.

4. Causes of Water-logging.

R. B. Natha Singh's analysis in paper No. 197 Punjab Engineering Congress 1947 is utilised and modified according to the author's experience and research work.

It may be safely assumed that in the pre-canal period the water table was in a state of equilibrium. There is no doubt that its level fluctuated with the amount of rainfall from year to year and some times very violently, but there is nothing to show that it was continuously rising. There would be a sudden rise in a year of heavy rainfall but it would adjust itself in a subsequent dry year. If obse vations were available for a large number of years for the pre-canal period, it would be found that after centuries of fluctuations, the sub soil water table had attained a state of equilibrium. If it were continuously rising the whole country would have been water logged long before the advent of canals. On the other hand if it were continuously falling the water table would have gone down indefinitely.

Before the canals are constructed in a *doab* there are the following factors tending to raise the level of the water table

(*i*) Infiltration from rivers towards bottom of the trough.
(*ii*) Percolation of rainfall in saturated phase.
(*iii*) Percolation from well or *sailab* irrigation in saturated phase.
(*iv*) Sub-soil flow from upper regions of the *doab*.

All these may be called the inflow.

The only factors tending to lower the water-table were as below, They may be called the "out flow".

(*i*) Sub-soil flow towards the lower regions.
(*ii*) Soil evaporation from the water table surface.

Fig 1, illustrates what is actually happening.

As the water table in the pre-canal period was in a state of equilibrium, it can be safely deduced that the inflow and outflow must be equal *i.e.*, the sub soil daringe and soil evaporation were sufficient to cope with the inflow from all the various sources enumerated above. A canal is then constructed in the *doab* in the position shown in Fig. 1 For the sake of convenience we may omit the branches and distributaries from consideration and assume that the collective seepage of all the irrigation channels takes place from the canal. By the construction of the canal a new source of constant inflow into the sub-soil is introduced. The sub-soil drainage and soil evaporation were just sufficient to deal the sources of in flow previously operating, but are incapable of dealing with the additional inflow caused by the seepage from the canal. This naturally results in a continuous rise of water-table. The rise, however, is not uniform every year In a year of heavy monsoons the rise would be abnormal, which would have gone down to its original level, had there been no canal, but owing to the continuous inflow from the canal, the original would never he attained and some of the rise would become permanent.

As the water table rises, the inflow from the rivers decreases owing to the reduction in yield, while the outflow, *i.e.*, the sub-soil flow towards the lower reaches of the *doab* increases owing to the steepening of the gradient. This explains why the rise in water-table is rapid in the early years of the opening of the canal. The inflow from both the rivers and the canal would be obviously maximum when the bottom of the trough is lowest. A_1, A_2, A_3, A_4, in the sketch represent the levels of the water table in the various stages of the rise, subsequent to the opening of the canal. When this level rises to the same leve as the rivers, the inflow from the rivers ceases while the inflow from other sources con=nues practically unabated. As soon as the level of the water-table rises above the level of the rivers, outflow starts from the water-table towards the rivers. Soil evaporatien also becomes relatively more active as the distance of the water table from the ground level is reduced. In these conditions the sources of inflow are :—

(*i*) Infiltration from rainfall in saturated phase.
(*ii*) Sub soil flow from upper regions.
(*iii*) Seepage from canals in saturated phase.
(*iv*) Infiltration from canals irrigation in saturated phase.

While the sources of outflow are :—

(*i*) Infiltration into the rivers.

(*ii*) Sub-soil flow to the lower regions of the *doab*.
(*iii*) Increased soil evaporation.
(*iv*) Transpiration by the plants.
(*v*) Seepage drains.

The volume of outflow from these sources will continue increasing as the water table rises, until a stage is reached when the outflow becomes equal to the inflow. This means that in every *doab* the rise of spring level must come to a stop at a certain level when the water table attains a state of equilibrium. What this level below ground surface will be, depends on the height of the water table above the rivers, and the slope of the ground towards the rivers. The greater the height and the steeper the slope, the greater the depth below ground at which equilibrium is attained. If the outflow towards the river is small due to poor slope, the equilibrium level is then by soil evaporation and the transpiration by the plants. The level in such cases rises up to about within 5·0 ft. below natural surface.

5. Actual Rise of Water-Table in Various *Doabs* in the Punjab.

The above explaination of the causes of water-logging is amply supported by the actual behaviour of the water table in the different *doabs* of the Punjab as shown in cross section across the Punjab in Fig 8, Chapter III of this part and longitudinal section in Fig. 9, Chapter III of this along the length of the *Rechna doab*.

Fig. 9, Chapter III of this part shows that the shape of the water-table in *Rechna Doab* in 1908 *i. e.*, before the advent of canals was in the form of a trough and below the level of two rivers. The canal was opened in 1915 and the water-table for the year 1920 shows a sudden rise which pushed up its level above that of the surrounding rivers. After this year the rise is slow and the levels of 1935 are not very much higher than those of 1928 and 1932. Water table in the Upper Chenab Canal area had practically reached a condition of equilibrium since 1926. After this year, there has been no permanent rise, but only fluctuations due to rainfall.

How the sub-soil flow from upper regions of the *doab* affects the rise of water-table is clear from Fig. 9, Chapter III of this part. Lyallpur is situated above 2/3rd the distance down the *doab*. The rise here has been about 50 feet since the opening of the lower Chenab Canal and about 40 feet after the opening of the Upper Chenab. It is clear that if the water-table is lowered by some means in the upper reaches of the *doab*, it will automatically go down in the tract near Lyallpur.

The other significant phenomena in connection with the rise of water-table that are notiocable from Fig. 9 of Chapter III of this part are :—

(*i*) That no rise of water-table has taken place in the upper reaches of the *doab* obove Raya Branch canal owing to absence of any irrigation channels and steepness of the country.

(*ii*) That the maximum rise has taken place under Upper Gugera Branch which is the oldest large sized channel in the tract. The slope of the ground changes at the crossing of this channel.

(*iii*) That no rise has taken place a few miles above the confluence of the rivers Ravi and Chenab. The spring level lines for all the years join here and come out at the confluence.

Now let us examine the conditions in the various *doab* of the province as exhibited in the Section along Provincial Well line No 1, Fig 8 of chapter III of this part. It will be noticed that a state of equilibrium in the water table has already been reached several years back in certain tracts of the western Jumna canal, on the whole of the Sirhind and Upper Bari Doab canals and seems to be nearing it in the *Chaj Doab*.

It is noteworthy that no elaborate preventive measures against water-logging have ever been taken on the Sirhind Canal uplands and yet no water logging has occurred and water-table is in a state of equilibrium. The section through the *doab* clearly shows that this is due to the the enormous height of the water-table above the two rivers on each side of the *doab* and therefore, the outflow being sufficiently great to balance the inflow from canal, rain irrigation as the soil crust is such that the losses are mostly in the unsaturated phase.

In the *Bari Doab* the water table is also at a great height above the river on the Beas side and the water table on this side is consequently in a state of equilibrium. On the Ravi side the height is comparatively less and the rise of water table is still noticeable in the area between Main Canal and Lahore Branch.

On the Western Jumna Canal there is practically no rise in the area between the main line and Jumna river, but between the Main line and Sirsa Branch the water-table is still rising, as it is the shape is of a trough and consequently thereis no outflow from it to the sides. The comparatively slow rise due to the fact that inflow is small being only from the distributaries of the Westren Jumna Canal. The trough expands laterally in its travel down as the rivers diverge.

The shape of the water table under the Western Jumna Canal is significant. It does not follow the N. S. contour on the left of the main canal. This is a definite indication that this crest is due to canal only.

Now consider the conditions in the *Bist Doab i.e.*, between Sutlej and Beas Rivers. In this *doab* the water-table has actually fallen since 1913-14 and its level is lower than that of the two surrounding rivers. The shape of this *doab* and its levels represent the same conditions as existed in the *Chaj* and *Rechna Doab* before the introduction of canal irrigation. The fall is obviously due to the sinking of excessive number of irrigation wells during recent years which means that the outflow became greater than the inflow, the only source of which is the rainfall. If canals were introduced in this area, there is no doubt at all that water-table would rise and there might even be waterlogging.

6. Cure of Water-logging.

It is obvious from what has been stated above under "Causes of water logging" that the water table can be definitely lowered. if the amount of inflow into the sub soil can be reduced. The water-table being in a state of equilibrium, outflow and inflow balance each other. Reduce the inflow and the outflow will get reduced automatically so as to maintain equilibrium. For a reduced outflow, the head above the rivers must become less, which means a fall in the level of the water-table. Therefore, the problem before us is now to reduce the amount of inflow. Out of the factors which constitute tne sources of inflow some are given below :—

(*i*) Percolation from canals.
(*ii*) Subsoil flow in the ground water reservoir
(*iii*) Percolation from irrigation in fields.
(*iv*) Percolation of rainfall.

7. Anti-Water-Logging Measures.

The anti-water-logging measures usually adopted are detailed below :—

(A) **Reduce percolation from canals and Irrigation channels.**

The student should refer to paragraph 12, chapter IV of this part. The seepage losses from irrigation channels are not added to the ground water reservoir every where but only when the seepage loss is in the form of percolation in the saturated phase. The remendies given below need only be applied in the case of irrigation channels losing water by way of percolation :—

(*i*) Lining canals
(*ii*) Lowering canals
(*iii*) Intercepting seepage Drains

(*i*) The first one has already been discussed in detail in chapter No. IX, Part II of this book on lining of channels. Lining of channels losing water even in the saturated phase is not necessary throughout their length. Leaky reaches along the various canals in the *Rechna* and *Chaj Doabs* have shown in the observations of the percolation (seepage) intensity coefficients given in Table I, Chapter IV of this part. Where the percolation intensity coefficient falls below 0·5 cusec per million square feet, the channel need not be lined to begin with. The selective lining can thus cure most of the trouble and can be a financial success. Moreover, head reaches of main lines Head to Chenawan L.C.C, Marala to R D. 5000 and Mangla to Khokhara U.J.C. can safely, be left out as the losses from these reaches do not contribute to the ground water of the *daab* but flow away into the adjoing rivers.

(*ii*) The lowering of existing high channels is advantageous when the soil crust is not cut through. Lowering shall reduce the seepage head in the formula $q=\sigma H$ but if the soil crust is removed, the value of σ will increase tremendously (Plate XXI). When the losses in the irrigation channel are simply of absorption, the lowering is definitely injurious because it shall change the absorption losses of the low order into the percolation losses.

(*iii*) The principle of intercepting drains running parallel to the canal as sketched in Fig. 2 is definitely sound. They can lower the water table below the lands as shown in the dotted lines up to the seepage level in the drain. If they are very near the toe of the bank, they will serve their purpose alright but shall also increase the percolation loss on the side by

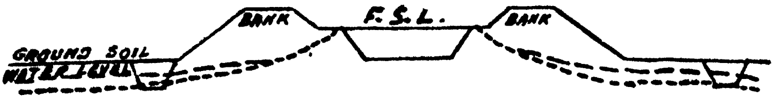


Fig. 2

steepening the gradient as shown dotted. If they are away as shown on the right side by surface profile will be depressed as shown dotted. In such a case there is no effect on the seepage flow from the canal. In the normal soil crust of the Punjab, they need only be about 100 feet away from the toe of the canal.

(B) **Sub-soil flow of the ground water reservoir.**

The measures under this head take the following forms :

(*i*) Artificial seepage drains.

(*ii*) Natural seepage drains such as rivers.

(*iii*) Sub-soil flow down the *doab*.

(*iv*) Pumping of sub-soil water.

(*i*) Seepage drains have been dealt with in detail in Chapter III, Part IV. They are useful both as open drains and underground drains in the form of tile or mole drains to remove the local trouble due to high water-table. They are very expensive and can only be construc·d where gravity outfalls into rivers (natural drains) are available. The total amout of seepage water removed in all drains in the *Rechna Doab* is hardly 200 cusecs which is very trifling compared to the additions made.

(*ii*) When the water-table is above the level of the winter levels in rivers they do collect seepage flow from the adjoining lands. S. H. Bigsby, Chief Engineer, gave the peak figures of regeneration for November averaged out for the 16 years in the rivers bordering *Rechna Doab* (page 349, Punjab Engineering Congress, 1937) as below :—

Chenab river (Marala to Jhang) = 1032 cusecs.

Ravi river, Sidhnai = 176 cusecs.

Evidently, this water is comming into the river from both sides. It is maximum rate of regeneration into the river which will be nil in monsoons but in fact the rivers do feed the general water-table when they are in floods It is likely that they very nearly take in winter what they nomally contributes in floods to the sub-soil reservoir. In the experiments at 14 R distributary U. J. C. at distance of 5 miles, the natural swing of 7′ in the water table was due to the Chenab river. The net effect of river to remove water-logging of a tract is very trifling. We cannot control this factor by artificial means and is out of question as an anti-water-logging measure.

(*iii*) The longitudinal section of *Rechna Doab* shown in Fig. 9, chapter III, of this part shows that although the water table has risen in places by more than 50 feet, there is no rise or increase of gradient of flow near the end of the *Rechna Doab*. The sub-soil flow down the *doab* at the exit has not been materially changed. It can only be increased by lowering the transmission constant of the soil crust or the steepening of the gradient. Neither of them can be artificially changed.

(*iv*) **Pumping from the sub-soil.**

This has not yet been tried in the Punjab on an appreciable scale. The amount of water pumped shall be the net subtraction of the ground water reservoir and therefore, must be effective to lower the water table. The major factor which is contributing to the lowering of the water-table in the *Bist Doab* is the irrigation from wells. In the water-logged areas, the replacement of canal irrigation by tube well irrigation is a very effective anti-water-logging measure.

(C) **Percolation from irrgation in fields.**

Usual measures adopted are detailed below :—

(*i*) Restriction of irrigation.

(*ii*) Economical use of water.

(*iii*) Lining water courses.

In water-logged areas, where the saturated phase of losses below an irrigated field can be established, the irrigation should be restricted by reducing permissible irrigated area, that is by reducing intensity of irrigation and by converting perenniel channels into *kharif* channels. In summer evaporation opportunity both from the surface and from the soil is very great and is enough to cope with any additions made then to the water table by irrigation. There is no idea in restricting irrigation in areas below the Sangla, Buchiana, Mangtanwala line in the *Rechna Doab* where the water table is away and there are no additions to the water table from the irrigation because all water allowed to flow there in a field is used up in the transpiration and in surface and soil evaporation. Whatever water is absorbed, when it is standing in the field, is brought up again by capillary action and is used in transpiration and soil evaporation in the case of the soil crust. Sandy soil near ground surface does not exist anywhere and is usually barren and no irrigation is done.

(*ii*) **Economical use of water.**

This is very desirable as an anti-water-logging measure in water-logged areas. This will increase irrigation and revenue in the unwaterlogged areas. The *khal kiari* system of irrigation should be resorted to, Fig 9.

(*iii*) **Lining water courses.**

This is a very good anti-water-logging measure in the water-logged areas where any losses from these shall be added to the water table but it would be a waste of money and energy to resort to this methed in the unwater-logged areas except to save the absorption loss which is usually of a low order.

(D) **Rainfall.**

The quick disposal of rainwater in excess of the crop requirements is always good for the public from the health point of view. It usually takes the form of surface drains described in chapter III, (Part IV) or pumping water from depressions for use in irrigation locally or for delivery into canals so that it may be use where required.

The efficiency of surface drains as an anti-water-logging measure is very nearly nil in areas where rain water does not percolate down to the water table but is only absorbed and again brought up by capillary action to be used up in soil evaporation.

In water-logged areas (capillary fringe within a foot or two of the ground surface) the rain water percolates and becomes ground water. In such cases rain is the predominent factor to cause water-logging. In the Upper Chenab Canal area which is mostly water-logged, the rain water is worked out to contribute 93% to the water-logging (Punjab Irrigation Research Memorandum No. 3, 1937) The quick removal of surplus rain water is then a very effective anti-water-logging measure. In fact any rain water removed in such cases is a net saving of the additions to the water table.

The auther concludes that the major and the most affective anti-water-logging measures are lining of channels and the surface drains in the water logged areas as water saved thereby is a net reduction of the additions to the water table. Tube well pumping is every where an effective anti-water-logging and a preventive measure in subtracting water from the ground water reservoir.

The construction of surface drains, lining channels and water courses and restriction of irrigation in areas near Lyallpur, Sergodha and Montgomery have no place even as preventive measures. The surface drains are no doubt everywhere usefull from the sanitary and health point of view.

In the case of new canals, the surface drains should be provided from the very beginning and channels should be lined where the seepage losses from them are likely to be in the saturated phase.

8. Lowering of Water Table in *Bist Doab* **(Between Sutlej and Beas.)**

(*i*) As is clear from Fig. 8, Chapter III of this part the water table in this *doab* has fallen 30 to 50 feet during the last 30 years which shows that the inflow to the ground water reservoir was less then the outflow. The water table is lower than the water levels in the adjoining rivers. Factors contributing to this lowering are :—

(*i*) The increase in the well irrigation (there are no canals in this area).

(*ii*) Reduction in the inflow from the Sutlej river which is very nearly dry for six months in winter as river water is utilized at Ruper in the Sirhind Canal.

(*ii*) Deforestation in the Swalik hills bordering North-East of this tract. The perennial streams (white and black *Binse*) and *Choas* (rainy season streams) have all very nearly dried up which were replenishing the water-table. Now rain water from the hills flows away in a day or so and then the streams are dry for the rest of the year.

(*iii*) The forest areas in the tract and the gardens have also disappeared which have resulted in reduction of the average annual rainfall to some extent.

(*b*) **Measures necessary to raise the water-table in** *Bist Doab.*

The irrigation wells are now 70 to 80 feet deep at places. The lifting of water for irrigation from such depth is not only uneconomical but a hardship to the cattle. The following measures are necessary to recover the lost water depth of the ground water reservoir.

(*i*) This is a very fertile tract of land in the Punjab and was never in need of artificial irrigation (canals) from times immemorial. Well irrigation is still giving about double the yield as compared with the canal irrigated tracts in the Punjab. *Barani* cultivation has become precarious. It is necessary to extend canal irrigation to the *barani* areas. The soil crust is about 15 to 20 feet deep, the irrigation channels are not likely to contribute to the warer-table in the saturated phase but their losses will be in the saturated phase and they will reduce the soil evaporation to some extent and add to the wealth of the tract.

(*ii*) It is only near the foot of the hills that the soil is sandy and suitable for addition to water-table by way of percolation from the irrigated fields and the irrigation channels but no irrigation is required there. If the *choas* which passed out near the foot of hills and lost all their water in percolation, had not dried out, then the trouble would have never arisen. The addition to the water-table can only be made in the hills or near the foot of the hills. The surest remedy to recover the lost ground water is to construct small dams in the Swalik hills and then to utilize their water in small irrigation channels to irrigate the *barani* areas in whole *doab.*

(*iii*) The forestation of the hills bordering this *doab* by stopping grazing and by planting trees shall also help to relieve the trouble very much.

(*iv*) Wholsale replacement of well irrigation by canal irrigation shall be ruinous. The author born and bred in this tract, is well aware of the topography and nature of the soil conditions. The very name of the District (Jullundur) means underwater. This was the last piece of earth in the Punjab to appear as land when the sea receded. The tract will be watter-logged very soon by large channels cutting the crust and then it can never be reclaimed as it is very low as compared with the adjoining *doabs.* Fig. 8, Chapter III, of this part.

(*v*) *Watt Bundi.*

This is very useful near the foot of the hills to cause rain water to stand for some time and then percolate underground. In the Jullundur, Nakodar and Phillaur Tehsils water in wells is 50 to 80 feet deep and no cultivator can affored to waste a drop of rain water. Rain water is utilized for crops by constructing strong *bunds* (*Watts*) around the fields. This *Watt Bundi* helps indirectly by reducing the water lifted from wells.

9. Stable Water-table Near the End of the *Rechna Doab.*

There is nothing to show that the so-called ridge of rocks underground due to soil crust wharping running across the *Rechna Doab* near Sangla is water tight. The sub-soil water is flowing across it without a sudden drop in its surface. The steepening of slope simply means increased resistance to flow, that is, somewhat reduced transmission constant. The water-logging is not exactly up to Sangla but about 20 miles upstream of it

The water table down the *doab* leaving aside the water-logged areas is mainly fed by the underground flow as shown by the daily B. S. P. observations Figs. 20 and 21, Chapter III, of the part. The rise and fall in the water table levels are absolutely unaffected by the nearest sources such as canals, irrigation or rainfall. The cycle of yearly fluctuations is explained in Fig. 3.

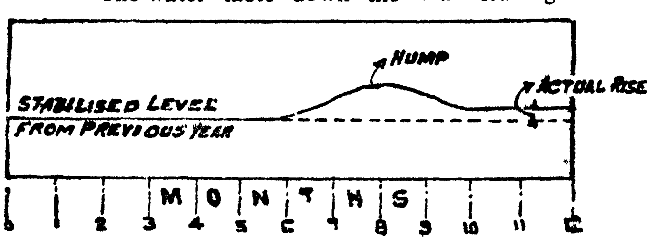


Fig. 3

When increments in flow are received from above, in B. S. P.

rises, then drops, but the trough is left higher than the previous year's level. The crest of the wave subsides after the increment in the sub-soil flow has passed the point. The sub-soil flow is being used up in the way to raise the water-table and there is soil evaporation extracting water throughout. The yearly rise in the trough dries out by the time it reaches near Janiwala. Negative pressure difference showed that there was evaporation subtraction at *Tawan* even though the water-table in the observed period was stable about 50 feet below the ground. The wave is of short durations of about a month or so. The water table is stable in the remaining months of the year although the intensity of irrigation is maximum in this areas. No such waves reach the end of the *doab* and whole of the sub-soil flow is used in soil evaporation. Even 20% soil evaporation is equal to about 3 inches depth of water, taking, Lyallpur figure of surface evaporation which is 123 inches in a year. This is equivalent to 12 inches of the water table allowing 40 percent pore space. (Paragraph 5 (E), chapter III of this part) 2 percent soil evaporation in arid areas of Jhang Disirict is strong enough to lower water-table by about 12 inches in a year.

10. Soil Water Inventory of *Rechna Daab.*

The figures of the table given below are taken from Appendix (C) of R. B. Natha Singh's paper 197 Punjab Engineering Congress 1937, as got out from 1933-34 statistics of working out distribution in the Punjab.

TABLE 1.

Addition to the sub-soil water table in the *Rechna Doab* due to canals and rainfall.

Tract	Gross area.	Discharge utilized at disty: heads.	Absorption losses in mainline and branches.	Total absorption from canal.	Equivalent depth over gross area on account of absorption from canals.	Mean Rainfall	Total depth of water from Canals and rainfall.
Region I. Marala, Gujranwala & Raya Divns.	1192000	4126	724	4850	35 3	22 25	57 55
Region II. Sheikhupura and Upper Gugera Divns.	993562	2773	680	3453	31 0	14 52	45 52
Region III. Hafizabad Division.	436011	1236	668	1904	38 0	18 75	56 75
Region IV. Lower Gugera, Burala, Lyallpur and Jhang Divisions.	1864967	6286	536	6822	31 8	10·27	42·07
				Total mean absorption			50 5"

Total water used in canals and rainfall is equivalent to a depth of 50·5 inches spread over the whole area. The transpiration is only about 10 inches depth of water for the normal crops grown. The average rise of water-table is about a foot in the *doab* which is equivalent to 3 inches depth of water. The remaining 37 5 inches depth of water cannot be explained. Naturally some of this is evaporation from the water surface when water is applied in the field and soil evaporation when crop is on, and the rest is the soil evaporation when crop is cut and land is lying vacant. The correct water account cannot be prepared unless the actual investigations are carried out to differentiate the areas where the canals and the irrigation add to the water-table by percolation (saturated phase) and also those areas where it is simply a case of absorption lost in soil evaporation.

The regeneration into rivers is only 58) cusecs at the the peak in November which is equivalent to 1·0 inch depth of water on the whole area and is, therefore, trifling. The average discharge of all the seepage and storm water drains has been calculated to be only 5 percent of the annual mean rainfall which is equivalent to one inch depth of water. Even allowing these, the soil evaporation and surface evaporation come to be 35·5 inches depth of water. At least one third of this may be taken to be the surface evaporation from the rain water and irrigation water standing in the fields. The remaining 24 inches depth of water is equal to 18% of the normal pan evaporatian at Lyallpur. From plate XX, Vol. III, it is clear, that if the water-table be on the everage 14 ft. below ground, it can loss this much in soil evapration.

Soil evaporation is the major factor to consume and account for the water utilized in irrigation. The panacea of all ills and errors of omission and commission in not lining channels or not providing enough surface drains but is the soil evaporation which generally keeps the water table steady 5 or 6 ft. below the ground surface enabling thus normal crops.

11. Examination Questions.

(1) What are the known causes and remedies of water-logging ? (*P.I.B. 1939*)

(2) Why is the water-table in the *Bist Doab* of the Punjab going down and down, and what remedies do you suggest to arrest the tendency ? (*P.U. 1943*)

(3) (*a*) What are the principle causes of water-logging in a canal irrigated tract.
(*b*) What precautions will you observe in constructing a new canal system ?
(*c*) What steps will you take to improve an already water-logged tract ?

(4) What is the definition of water-logging and what the factors are responsible for the infertility of water logged soils.

(5) What is the minimum depth from the surface to which the level must rise before the tract can be considered in danger of water-logging ?

(6) What is the best method of recording sub-soil water level ?

(7) (*a*) What are the causes of water logging ?
(*b*) Give the relative extent to which these causes are responsible for water-logging ?

(8) (*a*) Is the lowering of the Full Supply Level of a channel an effective method of preventing water-logging ?
(*b*) What is the effect on the sub-soil Water Table of opening irrigation system ?

(9) Is local rise in the Water Table in the vicinity of irrigation channel distinct from general rise in water table of the Irrigation tract ?

(10) (*a*) What is the maximum depth of the sub-soil from which the moisture can be brought to the surface by the soil evaporation ?
(*b*) How do the soil characteristics effect this problem ?

(11) (*a*) Does the introduction of Rice irrigation tend to cause a rise of Water Table everywhere ?
(*b*) When does the water used for agricultural crops such as wheat, cotton, maize, etc., tends to cause rise of the water-table ?
(*c*) How is it caused to prevent excessive use or waste of water ?

(12) What are the known causes and remedies of water logging ? Why is the water table in the *Bist Doab* of the Punjab is going down ? What remedies do you suggest to arrest this tendency (*P.U. 1951*)

(13) How is the water logging caused ? What are the remedies and what degree of success may be expected in various types of soil. (*P.U. 1943*)

(14) Explain the significance of water logging in irrigation.

(15) What is meant by waterlogging ? What are the factors responsible for the infertility of water logged lands ? What are the causes of water logging. Describe the anti-water logging measure ? (*P.U. 1958*)

PART V

GROUND WATER ENGINEERING

CHAPTER VI

Reclamation of 'Thur' and 'Sem' Lands

1. Introduction.

The term reclamation as used in the Punjab may be defined at the process of restoring to cultivation lands which were once fertile but have since deteriorated to such an extent as to make cultivation uneconomical or impossible. Very exhaustive research work on the reclamation of *thur* lands has been done by M. L Mehta, Reclamation Officer Punjab and Dr E. McKenzie Taylor, Director Irrigation Research Institute, Lahore. The results were published in paper No. 235 Punjab Engineering Congress 1940. The author has simply attempted to give a brief summary of the subject as available in their writings.

Reclamation of the water-logged (*sem*) land is essentially a subject for Engineers concerned with the irrigation problems. Both aspects of the reclamation for very much allied and are taken up together in this chapter. It is considered desirable to initiate the students to the elemantary principles of the subject although from the examination point of view, the subject is beyound their scope.

2. Presence of Salt in the Soil Crust.

The soil in the Punjab may be described as stiff containing large percentage of clay, from 15 to 40 percent. An exact scientific classification would be tedious : but a good picture can be obtained from such facts as the generally good crops, the almost universal existance of clay suitable for bricks and mud plaster, and the practice of the digging wells through the crust without any shuttering. Beneath the crust, whose average thickness is about 10 to 12 ft. with extremes up to 50 ft. are layers of clay, sand, *kankar*, gravel, and mixtures, to a great depth. Gravel is only common near the foothills.

The crust contains soluble salts, and there are areas of clay rendered unirrigable by them. The common salts are the sulphate, chloride and carbonate of sodium, (all soluble). Sodium carbonate may act on the clay to form a highly impermeable and uncaltivable soil. Free salts when near the surface, may concentrate during dry period as a white deposit known locally as *kallar*, (if of long standing), and *thur* if brought up since the start of irrigation.

In general distribution salts favour the tracts of poor slope, relatively impervious crust, and low rainfall. They are almost absent in the vicinity of the 25" rainfall zone. In local distribution they are patchy. Salty fields exist in the midst of good irrigation and large areas of salt border abruptly on areas that are free.

The general explanation of the initial visible occurence of salts is based on their original existence throughout the crust and the motion of soil water acting as a vehicle. Clay is deposited from still water and not only is it likely that the clay itself contained the source of salts, but the evaporation of the still water could also cause salts concentration. *e g.*, the dead sea or the bitter lakes. Given a salt bearing crust and a deep water-table *i.e* , pre-water-logged conditions, a reasonable explanation for surface salts is that rainfall on a poorly pervious crust sinks in for a short distance during the brief precipitation period and dissolves the salts that exist in its saturation zone. When rain ceases the water moves upward to evaporate and leaves the salts at or near the surface This is not the only explanation. Water can move upwards when night temperature or low barometric pressure causes the included air of the soil to expand and squeeze water out of the pores to the upward direction is the easier one for movement as the soil is more open towards the surface. Yet another explanation is for sodium sulphate. Water near the soil surface may be expected to have roughly the yearly temperature range of canal water *i. e*, 5° to 35° C and sodium sulphate is about twice as soluble at the higher temperature. as it is at the lower. Water lower down has very slight range of temperature. It appears therefore, that during summer, sodium sulphate can move, by diffusion, from the colder water to the warmer water at the surface. Thus the salt can move without the water moving at all. This effect would be expected to occur during a period embracing the monsoons. It need not be reversed fully during the rest of the year. Probably all these explanations are valid.

During rainfall or irrigation the movement of water near the surface is downward, the salts will temporarily vanish. The application of adequate waterings may keep salts from damaging crops. The reduction of waterings may allow them to become dangerous again. This effect might be expected where areas were fairly salty before irrigation.

A special cause of saltation occurs when, as in water-logging, the water table rises into the soil crust. The crust, because of the fineness of its pores, exerts a powerful capillary (blotting paper) pull and can draw water up to some 5 to 15 ft. as a common figure. The resistance of the crust to this pull is great, but it seems that it can allow a lift of some 3 to 5 feet in the first year. The slow advance of this capillary water gives it ample time to dissolve all the salts of the soil crust in its advancing front and bring them into the crop root-zone, which is the zone of irrigation water penetration, where a complicated series of action move them up and down to the detriment of agricultuae. Normally the salts concentrate at the surface during by periods, become anhydrous and are visible as *thur*. Settlement Reports comment on the improvment of crops during the two or three yeary before *thur* becomes visible and destroys them. This is explicable by the slow advance of the capillary water ; for a small doze of salt is beneficial and such a small doze is given when the salty water just reaches the bottom of the root-zone.

There are secondary causes for *thur* being found in certain localities. Once it has arisen from the fundamental causes it may be moved by water or even wind. Thus it may be washed from higher lands to lower during heavy rain, especially if the soil is highly impervious. The practice of irrigating one field from the over-flow from another may damage the second if the first is bad. Even sub-soil water may carry damage. Thus if an area of thin crust, originally free from salt, be surrounded by thick crust highly salt-charged, the rise of water table into the thick crust may be expected to bring by diffusion or flow, the salinity of soil water under the thin crust to such a high figure that *thur* will appear due to soil evaporation.

The salt appears most in winter when humidity and temperature conditions at night are suitable for moisture absorption and in the day time for evaporation. In the summer either humidity or temperature or both are unsuitable for moisture absorption by sodium sulphate and, therefore, the auto-movement of this salt does not take place in the *kharif* season.

Mehta summarises his conception of salt movements as below in his paper No. 235 Punjab Engineering Congress, Lahore :—

To summarise the dicussion in the preceding paragraph it may be stated that, before commencement of irrigation, salt is distributed throughout the depth of the soil crust and in the absence of irrigation water no movement takes place. With the introduction of irrigation the salts accumulate in the form of a solution at some depth below the surface forming a zone of accumlation. The subsequent position of this zone of accumulation in the soil crust depends on the intensity of irrigation and the agriculture, which means the use made of irrigation water. If the agriculture is of such a type that the amount of water used for irrigation is sufficient to counter balance that lost by transpiration, then the zone of accumlation will tend to remain stationary or to move to greater depth. If, however, the quantity of irrigation water used is insufficient to balance the loss due to transpiration and evoporation and the zone of accumulation is within 10 feet of the surface, then the tendency will be for the zone of accumulation to move upwards.

Definition.

Soil crust suffering from excess of soluble salt at the surface is known as *kallar* or *thur* land.

3. Formation of *Rukkar* Soils by Base Exchange in the Salts.

The presence of large quantities of certain soluble salts change the entire physical and chemical condition of the soil. Chemical reactions take place between the salt and the clay in the soil and the stage to which soil deterioration has taken place is determined by the extent to which the chemical reactions have advanced. On the physical side by the continued action of the salt, the structure or the tilth of the soil is affected, colloidal substances are altered and changes in the moisture relationship occur. The clay in a normal soil is mainly present in the form of calcium clay. A soil containing calcium clay is premeable to water and air, the conditions in it are aerobic and the methods to be adopted in the cultivation of such a soil are known to the Punjab farmers. When a solution of sodium sulphate or sodium chloride is brought in contect with a calcium clay soil, base exchange takes place, the sodium replacing the calcium in the clay, resulting in the formation of sodium clay. The reaction is known in the following equation ; –

$Na_2SO_4 + \text{Ca-clay} = Na_2\text{-clay (alkaline)} + CaSO_4$

It will be seen that there are two stages in soil deterioration, the first stage is the accumulation of salts which is relatively harmless ; the second stage is the base exchange reaction given by the above equation, which renders the soil conditions much more difficult to ameliorate. The development of alkalinity is a gradual process and it follows that after the first stage of *thur* appearance has been reached, the longer the land is left before reclamation the greater will be the approach to the second stage of alkalinity. As the alkalinity increases reclamation becomes more and more difficult and finally economically impossible.

In the Punjab the main salt present is sodium sulphate. Owing to alternate dry and wet periods and the consequent movements of moisture and salts in the soil profile the reaction between sodium sulphate and calcium clay has taken place repeatedly. The result is that the Punjab plain soils tend to be sodium clays and are alkaline. In a very alkaline soil it becomes deflocculared (Dispersed) and the soil is impermeable to water. This soil presents difficulties in leaching and is unsuitable for the growth of normal crops. Thus the main limiting factors, other than water, in crop production in the Punjab are the salt content and the alkalinity. From the point of view of reclamation it is essential to know the limits of both salts and alkalinity in the soil at which yields of crops begin to decline, the limits at which crop growth becomes impossible and also the limits when reclamation cannot economically be carried out. With a view to ascertain there limits two types of investigations have been carried out. Firstly soil surveys of wheat and rice growing tracts have been made and, secondly, the crops have been grown at Chakanwali Reclamation Farm under controlled conditions on fields having different salt contents and varying degrees of alkalinity. McKenzi Taylor, Puri and Asghar have shown that, provided a salt content is low, the yield of wheat is not affected until the pH value rises above 8·5. The general characteristic is the lack of growth on the surface (absolute infertility) and impreviousness. Usually no white salts are on the surface.

4. Soil Classification for Purposes of Reclamation.

Mehta in his paper No. 236 Punjab Engineering Congress, Lahore, gives the classification as below :—

Type (*i*) Good land capable of carrying the normal crops and likely to give normal yields. Such soil are characterised by a soluble salt content of 0·2% or below and pH value not higher than 8·5.

Type (*ii*) Soils which have some limiting factor such as salts or alkalinity which will tend to reduce the yields of crops below normal. The main charcteristics of these soils are a salt content below 0·2 percent a pH value ranging between 8·5 and 9·0 Given proper cultivation and subsequent interculture, etc., it is possible to obtain a yield of 18-20 maunds of rice in soils of this type. The rotation advocated for this type is, rice followed by *berseem* or *senji* followed by sugarcane, followed by wheat or cotton. After cotton rice can be eliminated from the rotation.

Type (*iii*) Soils in which the influeuce of some liming factor is very marked so that they are only suited for a limited type of cropping in the initial stage of irrigation.

The salt content of this type is almost always less then 0·5 percent and the pH value ranges between 9·0 and 9·5. These soil belong to what is locally known as the 'Mild Rakkar' type. Soils of this type are unfit for general cropping and need to be reclaimed. Reclamation experiments show that two rice crops enable this land to be reclaimed and made fit for normal cultivation.

Type (*iv*) Salt soil that can be economically reclaimed :—

In soils of this type the salt content is above 0·2 percent while the pH value is below 9·0. Soils of this type are locally known as *thur* soil. Usually one rice crop is sufficent to reclaim soils of this type. After the first rice crop it is possible to introduce gram *berseem, senji* in the *rabi* and sugarcane or cotton in the following *kharif*.

Type (*v*) Salt and alkaline soils that are difficult and at the same time expensive to reclaim. These soils may or may not have a high salt content but their pH value is always higher than 9·5. These soils are locally known as *Rakkar* soils. The reclamation of this type takes a much longer period than is the case with type (*iii*) and (*iv*). The reclamation of the *Rakkar* type is not a commercial proposition. It is advisable to exclude soils of this type from reclamation.

From the soil classification given above it will be seen that type (*ii*) *i. e.*, soils containing salt content less than 0·2 percent and pH value ranging between 8·5 and 9·0 can be reclaimed with one rice crop only. Types (*iii*) and (*iv*) sometimes require more than one rice crop to be grown before they are fit for growing normal crops.

5. *Telia Kallar* lands.

In a few cases, the ground surface becomes oily dark, brown in winter. This is known as *Telia Kallar*. Such lands are absolutely infertile. *Telia Kallar* is due to the presence of Potassium Nitrate and Sodium Carbonates. The former is soluble and can be washed down by flooding while the trouble due to the latter cannot easily be cured because Na_2CO_3 goes to form a sodium clay after base exchange and then make the soil impervious.

6. Reclamation Methods.

(*i*) Leaching and suitable cropping. (*ii*) Flooding and washing into drains (Natures Methods). (*iii*) Addition on chemicals.

(*i*) Leaching of fields with a depth of about 6 to 9 inches will no doubt suppress the salt zone and even wash out salts down the soil crust when the clay content of the soil is low. In the Punjab, the soluble salts are mostly of sodium, there is danger of the soil srust being left as sodium clay after it is cleared of salts Moreover it will be utter waste of water if no advantage is taken when it is kept standing in the field. Simple and pure leaching is uneconomical and impracticable for want of surplus water.

Naturally the method of reclamation by washing down soluble salts with water is coupled with suitable cropping to make the process a financial success. The rotation of crops is rise in summer when water is available for leaching due to floods in rivers and then followed by a leguminous crop such as *berseem*, *senji* or gram which require very little water in winter, when water is very scarce in the Punjab as every drop available in winter has already been utilized. The rice cultivation while leaching is not only necessary because it can grow in standing water but also rice is relatively greater producer of carbon dioxide near its roots which go relatively deeper as compared with other crops such as wheat, *bajra* or maize (except cotton and sugarcane). The carbon dioxide thus liberated is useful to remove the alkalinity of the sodium soils and thus makes them pervious for downward flow of salt zone. Leguminous crops after rice is useful as a covering to the soil crust to reduce soil evaporation in winter and yields good income using very little water. Grams can grow even in wadwater (*Barani* on the moisture left after the rice is cut).

When the alkalinity of soil is low, that is, pH value less than 9·0 and the soluble salt content (sodium salt) is not much in excess of 0·2 percent, the land can be reclaimed by one leaching. Similarly soil containing soluble salt of calcium such as calcium chloride and calcium carbonate can be easily reclaimed by one leaching and one rice crop. *Rakkar* soils usually need two leachings along with rice crops before the normal crops of the Punjab as wheat maize, cotton etc., can be sown.

Kallar lands also exist in the Jhang District on Lower Chenab Canal where the water table is still 40 to 50 feet below ground level and the climate is very dry. *Jhona* rice cannot normally grow in arid areas on account of low humidity but this difficulty has been got over by Mehta by growing *Sathar* variety of rice which can give normal yeilds even in arid areas having low humidity.

Where do the salts go in the reclamation by leaching ? There are three possible answers to the question :—

(*a*) They are washed down into water. This is likely when the water table is high within 5 or 6 feet of the ground surface and also when the soil crust contains clay content less than 10 to 15 percent but they will contaminate the ground water with the possibility of their appearing again lower down the *doab*.

(*b*) They are washed into the field drains when such gravity seepage drains are possible and provided around the reclaimed field.

(*c*) The salt zone is depressed by leaching. If it can be depressed below 10 ft. it is then supposed that salts won't return to the surface. In a soil crust having 30 to 35 percent clay content, the depression is not likely because it is capable of supporting a capillary fringe of height 4 to 5 feet with a similar depth funicular or pendular water. The salt would come up again when soil evaporation has its chance after rice cultivation. In such a soil, the complete satutation shall not travel below the surface more than a couple of inches with 6 inches depth in the field and the partial saturation with decreasing moisture contours shall travel only 3 to 5 feet below it.

(*ii*) This is Nature's method to reclaim soils. Salts are dissolved in rain water and are taken away into the sea. There is one significant fact in the Punjab that there are no *kallar* lands when the rainfal is over 25 inches. A couple of rains over 3 inches intensity in a day's are enough to wash away the annual return of soluble salts from the soil crust to the surface by soil evaporation. The rain water should have uninterupted flow from the *kallar* lands by constructing surface drains and then leguminous crops should be sown to reduce the alkalinity of the soil by liberation of carbon dioxide. They will also serve to reduce soil evaporation in winter and the upward flow of the salt zone. If early heavy rainfall is available in April to June to surface wash the soil crust then rice could be cultivated to depress the salts and to remove the alkalinity of the soil. It could be followed by a leguminous crop. There is so enormous a quantity of soluble salts available in the soil crust in the Punjab, that but for this Nature's method of reclamation, all lands in the Punjab would have been *kallar* lands.

(*iii*) The use of chemicals to neutralize the salts is impracticable and prohibitively expensive. However in the Punjab the lands have been successfully reclaimed by the use of gypsum (calcium sulphate)

However, gypsum does not act in presence of excess of sodium salts which abound in *thur* soil and thus gypsum can not act to bring the desired base exchange converting sodium carbonate into sodium sulphate. When the salts have been reduced by leaching and by growth of rice, the addition of gypsum is helpful in reducing the residual alkalinity. The methods to determine the gypsum requirements of the soil are not yet fully developed.

7. Reclamation Operations.

(*i*) **Preliminary**. Before undertaking reclamation, it is necessary to frame a programme for the whole of the canal system in general and for specific channels in particular. For this the following information must be available :—

(*a*) The total additional water-supply that can be made available during the summer (no additional supplies ordinarily are available in the winter), the exact period for which it will be available and the manner in which it can be distributed amongst the different chanels on which reclamation is to be carried out.

(*b*) The areas that require treatment.

There are many factors which determine the aiddtional supplies of water that can be made available for reclamation, and Chief Engineer issues instructions every year allotting reclamation supplies to different canals.

As regards areas that require treatment the most reliable method is a scientific soil survey of the area, for which soil samples are taken and examined in laboratory. A field method of carring out a soil survey has, however, been developed which gives an indication of the stage of deterioration of soil. Such surveys are known as *thur girdawris*. Orders exist on many canals for the Irrigation Branch *patwaris* to do the *thur girdawris* during the course of their normal work. Detailed instructions for recording all types of land deterioration have been issued. *Zilladars*, deputy Collector and the land reclamation officers are required to check the *thur girdawris* carried out by the Irrigation Branch *patwaris*. From the results of the *thur girdawris*, areas that require treatment are selected.

Having selected the areas for reclamation, the next step is to carry out a detailed soil survey of the area. Soil samples should be taken by the land reclamation officers and sent to the laboratories. From the results of analysis it is possible to determine the number of rice crops required for the complete reclamation of the different fields.

In canal colony areas where cultivated field are going out of cultivation due to the preference. For such areas a programme of reclamation for the whole of a distributary should be framed so as to finish reclamation of such areas on the distributary in not more than ten years

(*ii*) **Selection of fields for reclamation.** Fields for reclamation in any particular area should be selected in proportion to the total *thur* area on the outlet and if possible in each square. In preparing schemes, efforts should be made to start reclamation on all outelts from head to tail of a distributary. This will avoid remodelling of individual outlets

The *Zamindars* are required to apply for reclamation supplies on a standard application form.

(*iii*) **Additional water supplies required for reclamation.** Having determined the area and the period of reclamation, the extra discharge required should be worked out on the basis of 45 acre of rice per cusec of extra water supply. The Executive Engineer should ascertain if the distributary is capable of taking the extra discharge during *kharif* without heavy remodelling or upsetting the regime. If the distributary cannot take the extra supply, proposals for remodelling the distributary, raising its banks and alterations in control points should be forwarded to the Superintending Engineer before the first of May. This will give sufficient time to the Superintending Engineer to frame proposals and have them sanctioned so that the remodelling can be carried out during the winter season.

There will always be some increase in the water level in a distributary on account of the increased water supply during the reclamation period. Overdrawals by the existing outlets should be worked out. The balance of the extra discharge is to be supplied to the area concerned through additional shoots or pipes made of wood or iron. From a series of experiments carried out in the Irrigation Research Institute, charts showing the relationship between the working head and the dicharge obtained both under free fall and submerged conditions, have been prepared for two different sizes of barrel type outlets. These should be freely used.

(*iv*) **Short period** *warabandis*. On most of the canal colony *warabandis* are framed on seven to twelve day periods. For efficient leaching and successful reclamation it is necessary to maintain saturated conditions in the soil crust. This requires more frequent waterings. *Warabandis* are, therefore, reduced to five day periods. These *warabandis* remain in force from the date the extra water supply is given, up to the date it is withdrawn. During the winter season *zamindars* must revert to their normal *warabandis*.

The normal *waries* on an outlet ware proportional to the culturable commanded area owned by each cultivator. For the reclamation of *thur* land additional supply is given in proportion to the *thur* area. For calculating reclamation *warabandis*, the following method may be used.

Let the number of days in the *wari* be $=$N (here 5).

Let C C.A. $=$ A acres, with full supply factor$=$f,

Let *Thur* be $=$ T acres, with full supply factor$=$t.

Then the *wari* for C. C. A., $$W = \frac{1440N}{A+(T/f.)T} \text{ minutes per acre.}$$

Therefore the *wari* for *thur* $$W_t = \frac{1440N}{T+(T/f.)A} \text{ minutes per acre}$$

As the framing of short period *warabandis* requires a large number of calculations facilities for such calculations have been provided in the Irrigation Research Institute. The Executive Engineers and the land reclamation officers are advised to make use of them.

The land reclamation officer and, in certain cases, the assistant land reclamation officers have been empowered under the Canal and Drainage Act of 1873 to sanction and announce *warabandis* for the reclamation period. These *warabandis* remain in force for the period the extra water supply is given. All reclamation *warabandis* must be announced before April 1st.

(*v*) **Layout of fields and watercourses.** The lay-out of fields and water-courses will depend upon the depth of the water-table, variations in water-table the texture of the soil and the cropping that will be most suitable after reclamation. In areas where the water table is at a depth of more then ten feet from the natural surface, each acre field should be divided into four sub-plots the watercourse running in the middle of the field. This will enable each quarter plot to be irrigated independently from the water-course. Field dowels should be formation of *thur*, areas which require treatment for two *kharif* seasons should be given strengthened and made one foot high For high water-table areas, advice must be obtained from the Director of Land Reclamtion, Punjab.

(*vi*) **Irrigation.** The reclamation *patwari* must see that the *thur* fields get their turn of water after every five days according to the sanctioned *warabandi*. It is the duty of the land reclamation staff to see that the extra water alloted for reclamation is on no account used by *zamindar's* for irrigating, or increasing the area under normal crops If on account of the *zamindar*, neglect to carry out the reclamation process according to instructions, the rice crop fails in a fie'd, no *kharaba* should be given The refusal of *kharaba* should be supported by evidence that the shareholder has not watered the field according to the special *warabandi* in force supplemented by weekly or monthly reports of the *patwari* on record in the office of the reclamation officer.

If the reclamation supply is used in fields other than the fields for which it was alloted, the irrigation done should be treated as unauthorised irrigation, and suitable *tawan* levied by the land reclamation officer on the area on which the supply has been used.

(*vii*) **The rice crop.** As soon as the fields have been laid out in accordance with paragraph (*v*) and water supply becomes available leaching should be started. During the process of leaching, weeds and grass should be allowed to grow. The should not be cut or grazed

The period for the sowing of rice nurseries and transplanting of rice depends upon the dates between which extra water supplies are available and the condition of field during the leaching period. On no account should two rice crops - one early and the other late—be sown on the same water supply in one *kharif* season. The early crop of rice affords favourable conditions for the breeding of the pest known as "rice borer" which attacks the late crop The damage resulting from attack of the ' rice borer" is sometimes as high as 80 percent.

(*viii*) **Leguminous *rabi* crops.** In order to restore the nitrogen balance of the soil which is upset by leaching and the growth of rice during the *kharif* season, it is necessary to sow leguminous *rabi* crop after rice The most suitable crops are gram, *senji* and *berseem*. The two latter crops require irrigation during the *rabi* season. The area under these crops is, therefore, limited to the water supply that is available in *rabi* and also by the quantity of fodder that can be consumed. Gram crop sown in the *wadh-wattar* of rice, without sub-sequent irrigation, has been a remarkable success. With reasonable care, *zamindars* have been able to secure a yield of as much as fifteen maunds of gram per acre.

(*ix*) **Examinatian of soil after a rice crop.** After a reclamation rice crop has been cut, soil samples should again be taken to determine the depth to which the soil salts have been washed out and the extent to which the alkalinity of the soil has been reduced. Further programme of reclamation should be based on the results of these soil analyses.

8. Permanency of Reclamation by Leaching.

It is permanent reclamation if the salts are washed down to water table but both by aggregating the water-logging trouble in having made huge water additions to the ground water and by making the ground water reservoir contaminated with salts which will appear again lower down the *doab*, where soil evaporation opportunity exists.

If the salt zone is simply depressed 3 to 4 feet below the ground surface by leaching, the process shall have to be repeated say after 3 or 4 years. In fact, the crust will again be covered with salts if it is allowed to rest even for one winter or irrigation load is reduced

If the salts are washed down into the open seepage drains which cannot be more than 3·0 feet deep in the Punjab soil, it means that about 2 to 3 feet of soil crust is reclaimed The salts from the crust below 3 feet depth shall come up by soil evaporation. This method is definitely superior to leaching in which additions are made to the water-table, because the salts washed out in drain flow away into rivers.

9. Why no *kallar* Trouble in Well-Irrigated Areas ?

This question is aptly replied by Mehta in his paper No. 235 Punjab Engineering Congress ; 1940.

"A question that may be asked is that in the case of well irrigation, if there is a salt bearing layer in the soil crust below the natural surface, why is it that no deterioration due to *thur* formation takes place. In order to examine soil conditions under well irrigation pits were dug in recently irrigated fields under *bajra* and cotton. In the case of *bajra*, it was found that the maximum depth of penetration of the irrigation water was 6·5 inches. Between this moist surface soil and the salt bearing layer 2·5 feet of completely dry soil existed. In the case of cotton the depth of the penetration of irrigation water was about 20 inches since cotton receives twice the amount of water given to *bajra*. The moist soil under cotton was again separated from the salt bearing stratum by completely dry earth Figs. 1 and 2. The examination of these profiles has shown that under well irrigation no contact of the salt layer with the surface soil is established and. therefore, no rise of salts under well irrigation can take place.

Penetration of Well Irrigation in a *bajra* field in the *thal* area.

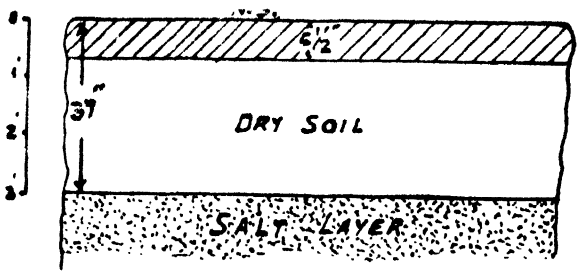


Fig 1.

Peneteration of well irrigation in a Cotton Field in the *thal* area ;

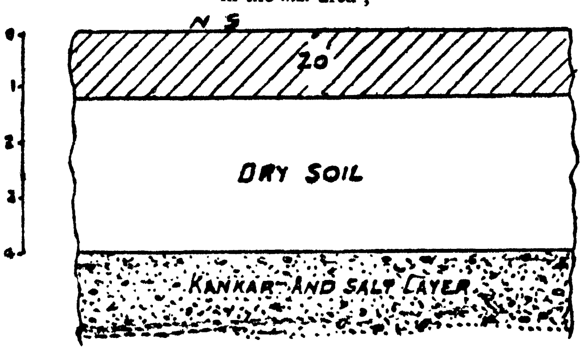


Fig. 2.

Under canal irrigation much larger quantities of water are used which establish the connection between the surface soil and the salt bearing layer in the soil crust and help in the movements up and down of the moisture and salts in the soil crust ; hence the necessity for taking more precautions to avoid deterioration of land under canal irrigation than under well irrigation.

10. Reclamation of *Sem* Lands.

If there are no salts in the soil, the rise of the water table is not accompanied by accumulation or afflorescene of salt at the surface of the soil crust. The yield of the crops is reduced when capillary fringe rises to within the root zones of the crops. Root zones of sum of the crops are shown in Fig. 3.

There is thus no free supply of air containing oxygen available to the roots, they cannot freely produce carbon dioxide. The lands are said to be water-logged.

If the water-table rises further they become *sem* lands when full saturation is at the ground surface, that is, when the capillary fringe reaches the ground level. (Pheratic surface of the ground may still be within a couple of feet below ground level).

The only remedy in such cases is to drop the water-table to reclaim the *sem* lands by seepage drains, by sub-drainage in the form of tile or mole drains and by pumping. The subject of open seepage drains and tile sub-surface drains has been dealt with in detail in part IV of this book.

11. Adjoining Fields of *Thur* and Good Culturable Lands.

Investigations were carried out by the author while working as the Waterlogging Investigating Officer 1938, by digging pits in adjoining fields of *thur* and good culturable lands near Mangtanwala. The conditions found are given in Fig. 4. Five sites were observed and similar conditions were found every where. If the water table is in the sand below the soil crust Fig. 4 (b), then the capillary water does not rise in the soil crust. If the water table is in the soil crust Fig. 4 (a) then the capillary meniscus surface is near the ground and soil evaporation is effective in causing concentration of the salts near the ground surface. The presence of deep clay crust in the water-logged tracts (where spring level is high) is, therefore, not a desirable thing. One may not find enough water in clay crust below spring level for tube-well yields but such lands shall deteriorate by salt concentration at the surface due to soil evaporation. In tracts where the water table touches the clay crust or rises in the clay crust, the crops yields shall fall and the fields will become unculturable in course of time.

(*a*) Sand-Grass; (*b*) Sand-sage; (*c*) Bunch-Grass;
(*d*) Big Bluestem; (*e*) Bush Morning-Glory;
(*f*) Wire-Grass; (*g*) Black Grama or Short Grass.

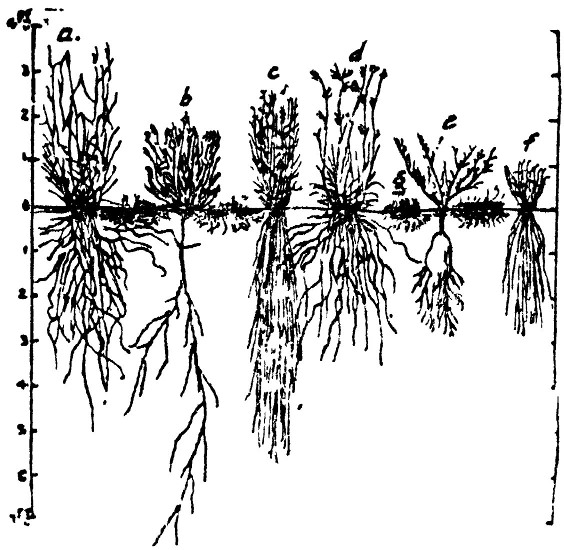


Fig. 3.

It is therefore, concluded that it is the characteristic of the *thur* lands that the clay crust extends upto or beyond the spring level. One may infer from this fact that the proper

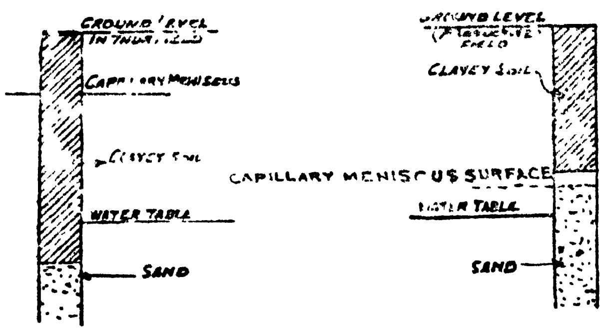
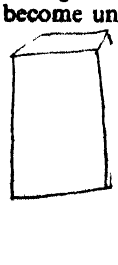


Fig. 4

and permanent remedy to reclaim *thur* lands is to drop the water-table by tube-well irrigation so that it falls a couple of feet below the bottom of the clay crust. The flooding of such fields resulting in the depression of the salt is only a temporary remedy .to reclaim them. So long as water is standing, the upward evaporation gradient is cut off. When a crop is cut, the cycle is reversed. The evaporation comes into play and the salt comes up again. If there is not enough water or if the fields have to be left over for a year or so for rest, then they will again become unculturable, by the rise of the salt zone.



PART VI

GENERAL SECTION

CHAPTER I

Soil Stabilization

1. Introduction.

(*a*) Soil is the oldest road-making material in use and yet also unfortunately the one which is the least understood of all in its performance under traffic loads in changing weather conditions. Having been lost sight of once, the subject remained in the back-ground, till repeated and mysterious failures of well constructed pavements over defective ground drew the attention of engineers to the ultimate foundation of the road, namely the sub-grade.

(*b*) America was the first country to start the study of soils, but it was not till as late as 1924, that a research station in soil mechanics in relation to roads was set up there. The reason for setting up of soil research in America was also the fact that it was becoming more and more evident that unless a far cheaper method of road making than the one in vogue was discovered for the very large mileage of roads required in agricultural areas, it would not be possible to afford an efficient system of roads in that vast country. Energies were, therefore, directed towards developing a method of economically improving the soil as such and using it for the making of roads, fit to take up the light agricultural traffic in chaning weather conditions.

(*c*) The problem of soil stabilization is very important to an Irrigation Engineer, since all roads in his charge are earthen and vast economies are likely to be effected by the construction of stabilized soil banks to resist percolation. Moreover, the failure of Haveli Lining has shown that inadequate compaction, of earthfill behind the lining results in subsequent settlement and the formation of a plane of segregation behind the lining.

2. Properties of Soil.

The student is already familiar with the classification of soils, the physical properties of the soil, the soil characteristic and the soil moisture as described in Chapter I of Part V. The methods of soil analysis have also been described in that chapter.

3. Ideal Soil for Best Stabilization.

(*a*) The stability of a soil depends on the internal friction of its component parts, cohesion between particles, capillary preserve elasticity and liability of change of state under external conditions. Clays are highly cohesive, sands are high in frictional resistance, mica or diatom and organic matters are elastic and rebound when pressure on them is removed but they cannot be permanently compacted and therefore make bad sub-grades and compacted banks. Cohesion and internal friction depend also on the state of the soil, whether wet or dry. Shrinkage limit defines the limiting shrinkage of the wet soil by throwing of water by dry soil grains.

(*b*) The correct conception of an ideal soil for best soil stabilization is that soil should be like concerete containing the required quantity of coarse material, its filler and its binder.

The coarse material in the form of sand provides internal friction and hardness. The filler in the form of silt provides embedment of sand grains and the binder in the form of clay coats the surface of silt and sand and provides cohesion. This natural stickness in the clay and this property is further augmented by surface tension of the film of moisture covering the various particles.

Ideal proportions are	clay	5 to 10 percent
	silt	10 to20 percent
	sand	70 to 85 percent

(c) The clay should be increased in dry areas and reduced in wet tracts. Clays change more in volume than silt while sands remain more or less constant. The finer the soil praticles, the greater will be the change in volume. In the arid regions in the hot season, the sub-grade will dry out and shrink leaving the hard metalled road surface without a support. In addition to the surface waves produced by a roller, the drying out of the sub-grade has a great deal to do with the corrogations and also heaving of metalled roads.

4. Clay Percentage.

It is very essential to determine the percentage of clay in a soil, before any remedial measures can be adopted to improve it. The usual methods adopted by the scientists, *viz.*, (*a*) Pipette or sedimentation method. (*b* Hydrometer method and (*c*) Optical siltometer (photographic) method, are too elaborate to be of any use to a practical engineer. The liquid limit and the plastic limit can be determined by simpler methods, the difference of these limits gives the plasticity Index (measure of clay proportion).

Lipuid Limit. 40 gms. of powdered soil sieved through No. 40 sieve (A.S.T.M.) is dried in 4 inches porcelain dish and then a measured quantity of water is added till it becomes pasty. Then it is grooved with a grooving tool of standard size and given 10 taps with finger. If it joins, the moisture is right.

$$\textit{Liquid limit} = \frac{\text{weight of wet soil—weight of dry soil}}{\text{weight of dry soil}} \times 100 \qquad \text{(A)}$$

Plastic Limit. 15 grams of powdered soil is dried and weighed and then water is added in a measured quantity. The soil ball is rolled into 1/8" diameter thread of soil crucible and then more water is added in the next attempt.

$$\textit{Plastic limit} = \frac{\text{weight of wet soil—weight of dry soil}}{\text{weight of dry soil}} \times 100 \qquad \text{(B)}$$

5. Optimum Moisture Content.

The optimum moirture is defined to be the moisture needed for maximum densification in the soil compaction. Mr. Proctar proved in his work that if the soil was compacted at the optimum moisture, it has a maximum dry weight per unit volume of the compacted soil.

(A) The following simple method was employed by S. R. Mehra of the Punjab P.W.D. to determine the optimum moisture in case of soil as described by him in Indian Roads Congress, 1939, Vol. (V).

The hygroscopic moisture content of the soil was first determined by drying it in the oven. A quantity of soil passing the number 10 sieve was then taken, such that the total hygroscopic moisture contained in it would be a round figure, say 5 grams. A measured quantity of water was then added to the soil from a graduated burette and well mixed.

The wet soil was compacted in a metal container 2 inches cube, in three layers, the compaction being done by dropping a hammer from a fixed height, say ten times. The top was struct off and the cube weighed. The dry weight of the soil in the cube was determined as follows ;—

Let total weight of soil plus water	$=W$	
Weight of cube	$=W_1$	
Total weight of water in soil	$=W_2$	
Then weight of water in soil cube	$=W_2 \times W_1/W$	
and therefore weight of dry soil in cube	$=W_1—(W_2 \times W_1/W)$	(C)

The experiment was repeated after adding more water each time, till the dry weight of the soil in the cube after rising to the highest figure began to fall. The moisture content at the highest figure gave the optimum moisture. The figure for water to be actually added in the field was, of course, obtained after deducting the hygroscopic moisture from the total. In the actual execution of work it was found that this quantity had to be slightly increased during the hot afternoons due to excessive evaporation.

(B) A still simpler method has been used and described by K.B.S.I. Mahbub in paper No. 257, Punjab Engineering Congress, to determine the optimum moisture content.

The sand content in sample was first determined by drying the specimen after heating it to 110°. It was then lightly pulvarised and then passed through sieve No. 270 using copious water. The material left on the sieve was considered to be sand which was weighed and percentage worked out and denoted by S. After plotting a lot of observations, he determined an empirical relation as below to give the optimum moisture content.

$$W = 24 - 0{\cdot}14S \qquad \text{(D)}$$

where W is the optimum moisture by weight expressed as percentage.
S the sand percentage in the sample

The optimum moisture content for maximum compaction was found by him to be 24 for clay, 22 for silt clay, 15 for loam and 11 for sandy soil. The relation between moisture content by weight W and moisture content by volume W′ is given by the equation which was first obtained and used by George, Washington University U.S.A.

$$W' = \frac{W}{W + 100/G} \times 100 \qquad \text{(E)}$$

where G is the specific gravity of the soil (average value 2·65).

6. Density of Soil.

(A) The density of a soil is its weight per unit volume. As 1 c.c. of water weighs 1 gram, the density of water is 1 in the C.G.S. (centimeter-gram-second) system. A soil which consists of solids and pores have two densities, that of the mass termed 'bulk' density and that of the solids termed absolute density

If voids ratio, *i.e.*, $\frac{\text{volume of pores } (V_e)}{\text{volume of solids } (V_s)} = e$

then volume of soil bulk per unit of soil volume, $V = 1 + e$;
Porosity E, expressed as a percentage $= [e/(1+e)] \times 100$;

Moisture content required to fill the pores expressed as a percentage of the weight of dried solids $(W) = \frac{100\,e}{G}$ (where G is the specific gravity of solids)

When the pores contain no water.

Bulk specific gravity $G_o = \frac{V_s \times G}{V}$ = Bulk Density D_b in the C.G.S. system.

When the pores are filled with water

$$G_o = \frac{V_s \times G + V_e \times 1}{V} = D_b \text{ in the C.G.S. system}$$

When the moisture content is w%

$$D_b = \frac{V_s \times G\,(1+W)/100}{V}$$

The dry bulk density $\frac{V_s \times G}{V} = \frac{\text{weight}}{V}$ is thus a true measure of compaction, as its value will increase proportionately with V_s in a unit volume of soil.

(B) **Determination of Density.** The following method has been tried and used by K.B.S.I. Mahbub as described in paper No. 257, Punjab Engineering Congress, 1942.

To compare the compaction (dry bulk density) obtained with varying strokes in the compaction apparatus with that of the natural soil, it was necessary to note the volume of the specimen of natural soil in its undisturbed condition. This was done as follows.

The surface of the soil was cut away to give as accurate a flat level as possible. The soil specimen was taken in the shape of wedge but slightly larger than the standard box used with its top level with the ground surface. This box which was in the form of inverted wedge 6 inches square base and 8 inches height was then placed in the hole in position and was secured by nailing two wooden strips on the top projecting from the sides to rest on the sides of the hole. The space between the box and the sides of the hole was then filled with dry sand from a graduated cylinder so that its volume could be ascertained. The volume of the hole was thus the volume of the box plus that of the sand. The cylinder containing sand as well as the wedge was well vibrated so that the degree of compaction obtained in the two was the same. The whole of the specimen was then carefully dried and weighed and the density calculated.

It was noticed that while compacting with a $4\frac{1}{2}$ lbs. weight with a 2 inch stroke, the density exceeded that of the natural soil even after one or two strokes. As the densities normally expected to be obtained in the field with rolling, were more or less of the same order as that of natural soil, it was felt that the results thus obtained would not be comparable with those in the field. A weight of 1 lb., with 1 foot stroke was accordingly adopted. The size of the cylinder in the compaction apparatus was also increased from $2\frac{3}{4}$ inches to $4\frac{1}{4}$ inches diameter and a 2 lb., weight used in this case.

Penetrometer. A penetrometer was devised by Haigh for gauging the bearing value of soils of different types as shown in Fig 1.

In this A is graduated glass tube 15 inches long held in a vertical position by a wooden support.

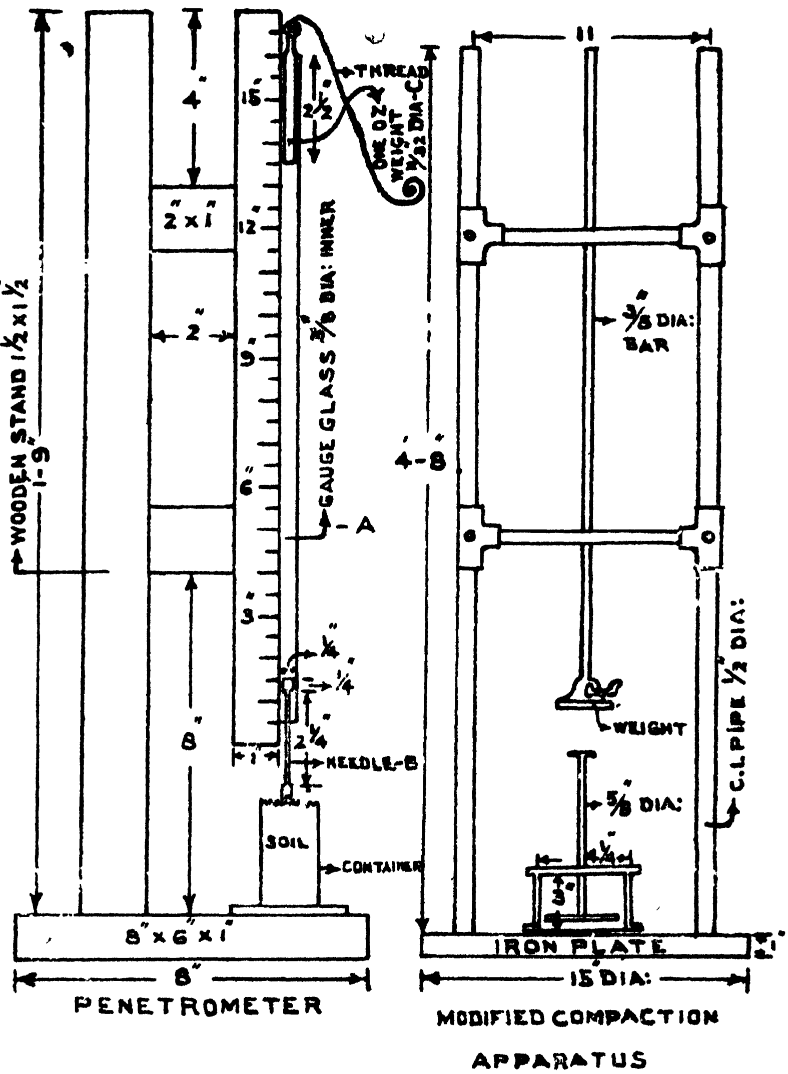


Fig. 1

B is a needle 3 inches long fitted with a guide at the top so that it is free to move up and down. The end of the needle is cylinderical in form with a flat end. It is of full thickness for a length of $\frac{1}{4}$ inch only from the bottom, the stem above this being thinner to obviate friction between the sides of the needle and the hole.

C is a weight in the form of a cylinder fitting loosely in the tube with a thread attached to it for lifting it out. The test comprises determiention of the number of blows of the weight, dropped from a height of one foot, required to cause the needle to pentrate one inch.

This experiment was tried with different specimens and it was found that there is a change in slope in the penetration curve at the optimum moisture content and the same can thus also be determined by the penetrometer.

A typical graph obtained with a sample which was compacted with 1 lb., 2 lbs., and 4 lbs. weights varying the moisture content from 4 to 20 percent. is also shown in Fig 2.

This indicates that the bearing value of soils, which depends on the extent of compaction, decreases very rapidly with increase in the moisture content. If may be noted here that for every density each soil has a particular stability, as determined by its resistance to penetration.

Compaction and penetration test on artifical mixes. To get some idea of the change in compaction and penetration in the range of soils likely to be met with generally, the main components of the soil, *viz.*, clay, silt and sand, were separated artificially in the laboratory and mixed in 32 different proportions indicated by the encircled points in the trilateral chart shown in Fig. 3.

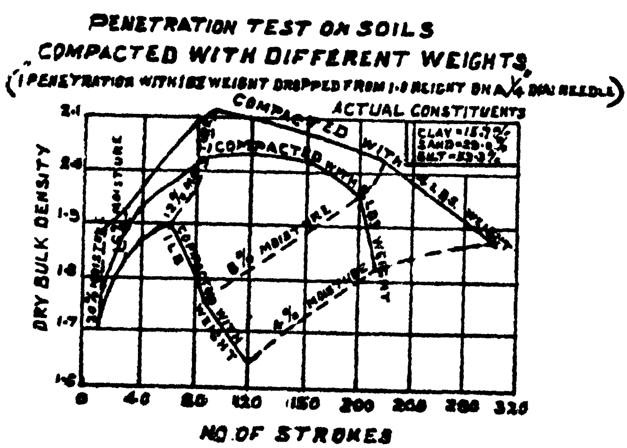


Fig. 2

This chart was developed originally by A.C. Rose of the U S. Bureau of Public Roads and shows the mechanical grading of the types classified according to texture. The range of soils thus obtained covered different proportions of clay from 5 to 30 percent and sand from 30 to 90 percent.

As these artifical soils were compacted with a 2 lbs. weight in a $4\frac{1}{4}$ inches diameter cylinder to different densities with varying moisture contents, so as to find out the compaction obtained with different number of strokes These were also tested with a penetrometer with a view to find out the strokes required with 1 oz, weight on $\frac{1}{4}$ inch needle dropped from 1 foot height to penetrate 1 inch in the soil. Over 500 readings were thus taken each for compaction and penetration and the following formulae were deduced :

(*a*) Compaction—

$$\log N = \left(\frac{K D}{W^{1/6}} \right)^4$$

where N is the number of strokes with a 2 lbs. weight dropped from 1 ft. height in the compaction apparatus having $4\frac{1}{4}$ inches cylinder.

K the constant for the soil depending on its constituents.

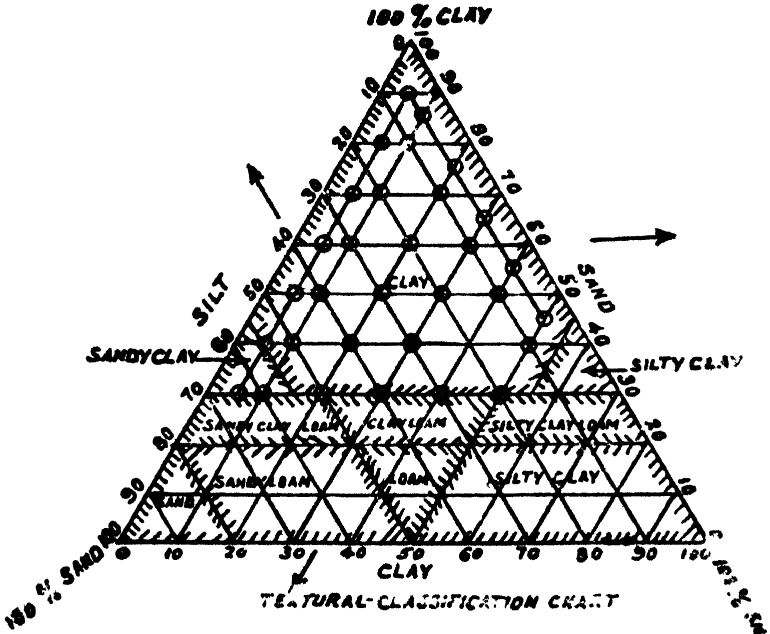
D the D.B. density
W the moisture perecentage by weight
(*b*) Penetration :—

$$\log N' = \left(\frac{K \cdot \sqrt{D}}{W^{1/8}}\right)^4$$

where N′ is the number of strokes with 1 oz. weight on a ¼ inch needle dropped from 1 foot height to penetrate 1 inch.
K the constant depending on the constituents of the soil. Its value is, however, other than that in the case of the compaction formula.
A graph based on the above formula is given in Fig. 4.

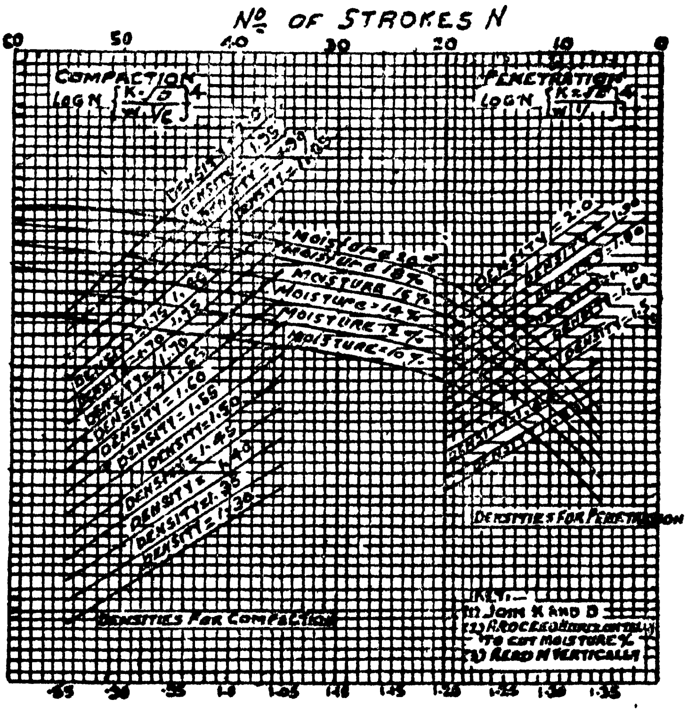
These relationships apply up to the limit of optimum moisture content only. Densities corresponding to higher moisture contents can be easily determined by assuming that the rate of decrease in density beyound the optimum moisture is the same as the increase up to this limit. In other words, if the optimum moisture for a soil is 18 percent, the density obtainable with 20 percent moisture would be approximately the same as that with 16 percent.

It may also be noted that the above results and formulae can be taken as accurate only within the extreme limits attained in the experiments. The results would naturally vary if the



CLASS	% SAND	% SILT	% CLAY
SAND	80-100	0-20	0-20
SANDY LOAM	50-80	0-50	0-20
LOAM	30-50	30-50	0-20
SILTY LOAM	0-50	50-100	0-20
SANDYCLAYLOAM	50-80	0-30	20-30
CLAY LOAM	20-50	20-50	20-30
SILTY CLAY LOAM	0-30	50-80	20-30
SANDY CLAY	65-70	0-15	30-45
CLAY	0-55	0-65	30-100
SILTY CLAY	0-15	55-70	30-45

Fig. 3.



GRAPH SHOWING

COMPACTION AND PENETRATION

OBTAINED IN THE LABORATORY

AREA OF CYLINDER 89 SQ C.M.

WEIGHT OF SAMPLE 600 GMS.

Valve of K.

		30	40	50	60	65	70	75	80	85	90
5	K_1	·98	·97	·89	·85		·96		·91		1·00
	K_2	1·295	1·32	1·296	1·27		1·22		1·22		1·27
10	K_1	1·00	·93	·95	·91		·97		·92	·92	
	K_2	1·30	1·225	1·26	1·26		1·25		1·26	1·29	
20	K_1	1·01	·93	·98	·96		·91	·91			
	K_2	1·36	1·26	1·25	1·24		1·23	1·24			
	K_1	1·04	·96	·95	·93	·97					
	K_2	1·31	1·29	1·27	1·25	[illegible]					

D = DRY BULK DENSITY
W = MOISTURE %
N = FOR COMPACTION:—
No. OF STROKES WITH 2 LB WT. LENGTH = 1·0
N FOR PENETRATION:—
No. OF STROKES WITH ONE OZ WT. DROPPED FROM 1·0 HT. TO PENETRATE 1" BY A 1/4" DIA. NEEDLE

constituents in the soil have a different grading. The presence of salts in the soil would also affect these results considerably.

7. Soil Compaction at Optimum Moisture on Large Works.

Compaction methods.

It is definitely established by the experiments that we can get the same compaction as that of natural soil by ramming as well as rolling by bullocks, if the moisture content is properly controlled. In cases where a work is to be done on a large scale, rolling would obviously seem to be a better proposition, as this would ensure a more uniform compaction at a lower cost. The real efficiency of the various types of rollers as well as the methods of controlling the moistnre content could however be best tested only if the work was undertaken on a large scale in the field. The actual process as used by K.B.S.I. Mahbub is described below : (Paper No. 257, P E.C.. Lahore).

The depth of layers to be consolidated was restricted to 6 inches, as it was found that this should not exceed double the height of the teeth to avoid undesirables stratification.

A sheep's foot and a tamping roller with knobs 4½ inches high and a toothed roller with 3 inches teeth were at first tried in soils where the sand content varied from 30 to 60 percent and clay from 15 to 30 percent. The results are shown in dotted lines' in Fig 6, which indicate that a tamping roller would give the best compaction in the above range of contituents.

The sheep's foot, roller Fig. 5., though better than the toothed roller, could not be of much partical untility, however, when rolling had to be done by bullocks, as two pairs simply refused to move it even for a short distance and considerable difficulty was also experienced in turning it at the end of a run The turning was essential as all the projections were inclined

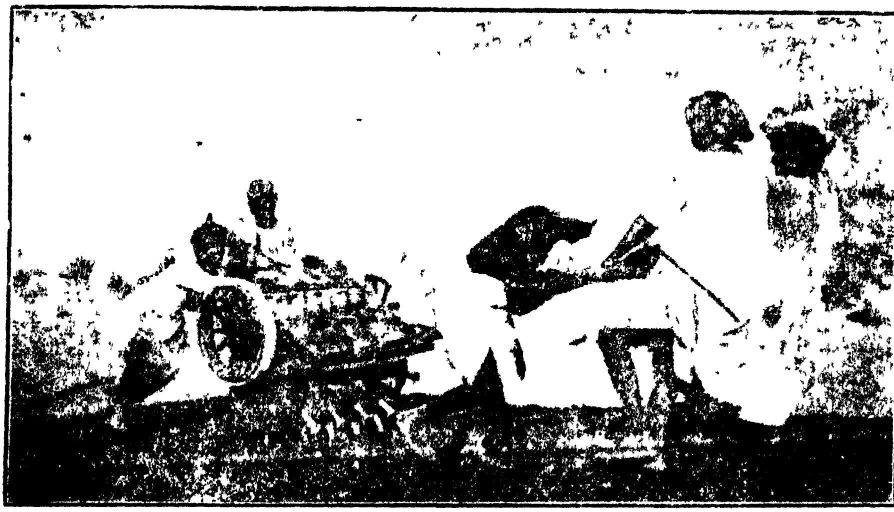


Fig. 5

to one side and the soil was rippedup instead of being compacted, if the roller moved in the wrong direction.

Two pairs of bullocks also found it difficult to pull a tamping roller with 4½ inch knobs and so their length was reduced to 3 inches, which was good enough for a 6 inch layer.

The origal toothed roller was also modified. The size of the teeth, which was originally 9 inches diameter at bottom and 6 inches at top, was redneced to 6 inches at bottom and 4 inches on top, the depth being kept the same *viz.*, 3". The spacing of the knobs was also reduced from 1 foot to 9 inches in this case, though the weight, 1·3 tons was kept unchanged.

A combination of the tamping and toothed roller having alternate concrete teeth 5" × 3" diameter and tamping iron knobs 3 inches high was also tried and the average results obtained by rolling in several soils where the sand content varied from 5 to 20 percent and the clay content from 50 to 75 percent is shown in Fig 6.

It was found that a toothed roller was better than a tamping roller in this range of constituents and a toothed-cum-tamping roller was even better.

It was also found that a tamping roller would not be effective in clean sand, as heavy kneading in such soils made up of finely divided particles without any binder (clay), increases the film moisture content, thus keeping the soil particles futrher separated, resulting in less density.

While compacting with this roller in clayey soils, the bottom layers were not sufficiently consolidated to allow it to ride on their surface. This resulted in the knobs continuing to sink even after 10 to 12 rolling. The friction to be overcome thus increased with every rolling and the work become more difficult in the later stages. The layers also developed a pitted surface due to adhesion of the clayey soil to the surface of the tempers in this case. Thus the above results can be abstracted as follows.

Percent content of sand in soil	Type of roller which would give the best compaction.
0 - 20	Toothed
20 - 30	Toothed or tamping
30 – 60	Tamping
60 – 75	Toothed or tamping
·75 - 100	Toothed

On the whole, therefore, taking into account the comparative difficulty of working a tamping roller, a toothed roller with teeth 6"×4" diameter and 3 inches high would be the best where work has to be done by bullocks.

A toothed-cum-tamping roller could also be used with advantage in place of a toothed roller, where facilities for making iron knobs are available, as concrete knobs of this size are liable to be easily chipped.

It has been found that on an average 16 rollings with the toothed roller would give a densiiy of about 1·5. Although this is appreciably less than the denesity obtained in the laboratory, it is in most cases better than the density of the natural soil and may thus be considered good enough to prevent any future settlement of irrigation channel bank.

ROLLING 6" LAYER WITH DIFFERENT ROLLERS

Sand 30 to 60% –Clay 15 to 30%
Sand 5 to 20%—Clay 50 to 75%

Where absissa is, No. of rollings and ordinate is, Dry bulk Denstiy

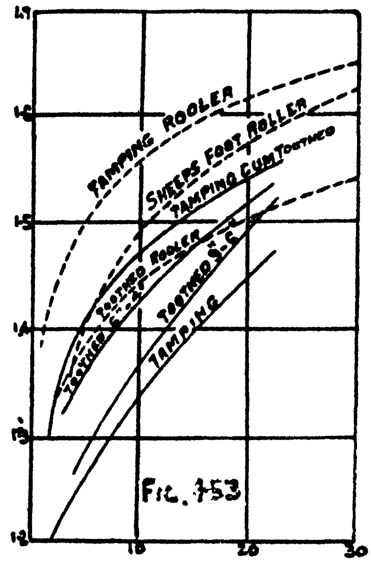


Fig. 6.

As a further check on the densities actually being obtained, the total quantitity of earthworks in the banks, as well as the excavation in the borrow pits was msasured and it was found that the earthwork in banks was 6 percent less than the quantity obtained from the borrow-pits, which indicated that the average compaction obtained in the bank was 6 percent more than that of the natural soil.

While working with these rollers, it was noticed that the end 2 feet of the bank could never be rolled properly, as the bullocks shied when taken too uear the edge of the bank.

This portion had thus to be compacted by rammers, preferably having knobs. To avoid this ramming a toothed roller was constructed in two parts, placed 5 feet apart with a connecting shaft. The roller wheels thus go outside the space over which the bullocks have to walk, and it becomes possible to roll very nearly up to the extreme edge.

The extent of compaction was generally measured by the wedge method, the centre of gravity of the wedge being kept at the centre of the layer of which the density was to be ascertained. This could also be determined by measuring the resistance of penetration of the soil layer in place, and comparing the values with the readings obtained when the same soil was compacted in the standard manner in the cylinder, as the penetrometer readings depend directly on the density. This test was, however, not tried much, as the direct measurement of density was also considered to be quite simple. It may be noted though, that the top 1 inch or 2 inches layer which gets dried up and has thus a lower moisture content than the optimum must be removed to get a true idea of the penetration in the field.

It was noticed in the reaches where the clay content was over 30 percent, that the surface of the kank showed cracks about 1/8 inch wide on drying, after 8 to 10 days. To overcome this, mixing of sand in varying proportions was tried by spreading a uniform thickness of sand on the borrow-pit area before excavation. The mixing, however, could not be uniform as the sand simply stuck outside big lumps of clay. This had accordingly to be discarded.

Consolidation at a lower moisture content than the optimum was also considered but was rejected as this would have meant a decrease in the dinsity. Spreading a layer of sand about 1 inch thick on the compacted bank was then tried. The sand was removed after a couple of days, when the cracks had got mostly filled up. This was found to be quite usefull in cases, where the compected earth could not be covered by another layer within three or four days, resulting in the development of cracks.

Control of moisture

After determining the optimum moisture content, the first point to be settled on taking up the work is the method to be adopted for the addition of water. This would be done either by sprinkling water over the loose earth spread on the bank, or by watering the area under borrow-pits beforehand and allowing sufficient time to elapse till the average moisture content in the top one foot was equivalent to the optimum moisture content. Sprinkling was tried in a short reach to start with, but was not found to be very successful, as the mixing was not thorough and the moisture content was thus not uniform throughout the layer. Moreover, it involved a lot of extra expenditure.

Some experiments were also carried out to determine the quantity of water needed for watering the borrow-pits, to get the required moisture content. This distribution would depend, among other things, on the contituents of the soil moisture content existing in the soil, temperature and depth of water-table. As the soil constituents varied considerably at every foot-depth and also from place to place, no general rules could be laid for the quantity of water that should be added in the borrow-pits area.

The moisture content already in the soil was first determined and the additional quantity required to make up the optimum moisture content calculated, after making due allowance for the evaporation losses which were of the following order :

	Percent
August	25
September	17
October	12
November	10
December	8

The time that had to elapse between the watering and the excavation, varied from place to place and required careful regulation. It was found that in some cases the borrow-pits were fit for working after 24 hours, where as in other cases no work could be done till after three days, due to the top layer not being sufficiently dry to be workable The moisture content was checked before taking out the earth and spreading it on the bank. As some moisture was lost by evaporation during consolidation, the moisture content was kept up to the optimum by sprinkling, as necessary, its value being checked both during and on the completion of the rolling.

8. Various Methods of Soils Stabilization.

(*i*) Ideal proportions of the constituents, clay, silt and sand (peragraph 3).
(*ii*) Compaction at optimum mixture.
(*iii*) The use of various admixtures.
(*a*) Solutions of electrolytes to reduce the thickness of adhesive water films.
(*b*) Neutralizers such as limestone dust, slag, etc., to neutralize acid soils.
(*c*) Moisture retentive chemicals as calcium chloride and common salt to provide enough moisture to facilitate compaction by traffic.
(*d*) Water insolude binders such as portland cement and bituminous materials to furnish films more substantial than those of moisture alone and to destroy the colloidal properties of soils.
(*e*) Adhesives such as molasses, calcium silicate, etc.
(*f*) Primes and fillers such as soaps, stones, dust and slag to increase the adhesion between the soil particles and chemical and bituminous admixtures.
(*iv*) Electro-chemical treatment, application of heat, etc.

9. Detrimental Salts in Soil.

The most important thing that has to be guarded against in the construction of stablized soil is the presence of detrimental salts in the harmful quantities which renders perfectly good pavements soft and fluffy in winter.

Very little, if anything, was known about the action of these salts, popularly called "*phulna kallar*" and the lack of this knowledge was a great handicap to many experimenters on soil stabilization in this province at least, as such salts are very common here.

The detrimental salts are sodium sulphate and Sodium carbonate, the former being predominant but the latter very rarely present in detrimental quantities in this province.

The action of Sodium sulphate is, that it hydrates and dehydrates with the atmospheric temperature and humidity alternating above are below 30°C and 80% respectively and in its crystalline state it increases in volume to 1 88 times its volume in the amorphous state. Further, it is soluble in its own water of crystallisation.

Now in winter specially, when the changes of temperature and humidity are very favourable for the hydraction and dehydration of the salt present in a compact layer of soil. The alternating volume changes break up the structure of the soil crust ; and the soil grains pushed out of position on hydration of the salt being unable to fall back into their position when dehydration takes place, the familiar loose and fluffy condition of the soil is brought about, that is, of coarse if the salt is present in sufficiently large quantities for the particular soil. The salt being also soluble in its own water of crystallisation, does not require outside moisture to travel upwards when evaporation takes place at the surface. The breaking up of the crust starts at the top and goes downwards.

The action of Sodium carbonate is more or less the same.

The remedy that suggests itself, to protect the road crust from salt action from underneath, in a salt affected area, is to interpose a layer of cellular material which will stop the rising of salt from below due to its lack of capilarity. Experiments on a small scale has shown that a 6" layer of ordinary pit sand or any other equally sharp sand prevents the rising of salts. It is also evident from this that a soil containing such salts in detrimetanl quantities is unfit for use in stabilizing, the road crust or irrigation channel in banks. Careful local enquiries as to whether any particular area gets soft and fluffy during winter and making a practice of avoiding grassless areas for the making of borrow-pits, will in most cases meet the situation.

10. Use of Soil Stabilization in Road Engineering.

S.R Mehra, Executive Engineer. Punjab P.W.D., after carrying out large scale experiments on use of soil stabilization, contributed papers in the Indian Road Congress and the Punjab Engineering Congress No. 256, 1942. A summary of his conclusions is given below :—

(*a*) **Earth roads.** The economic possibilities of soil stabilization are unlimited. The cheapest all-weather road, *viz.*, a water-bound macadam pavement surfaced with tar, cost about Rs. 10,000 for a 9 feet wide mile.

According to the current practice of road making, therefore, an all-weather road can be provided only if the considerations of traffic and the economic position, justify spending Rs. 10,000 per mile at least, on the pavement alone. It is needless to say that under these conditions the opening up of most rural areas in this agricultural country, will remain an economic impossibility.

With the help of soil stabilization, on the other hand, it is possible to provid a fairly comprehensive range of pavements to suit various intensities of traffic at competable costs. The types shown in Fig 7, give an idea of the kind of pavement, the average cost and the intensity of traffic that each type is considered fit for.

This straightway makes the conversion of numerous miles of fair-weather roads into all-weather roads, an immediate economic possibility.

(*b*) **Berm stabilization.** Taking the case of the unmetalled berms of metalled a few roads, these are a potential source of danger to the modern fast traffic, Except in the case of three small sections of road in the whole province, where the metalled width allows two lanes of traffic, the width of metalling ranges from 9 to 12 feet only with the result that use had to be made of the unmetalled berms, by one or both vehicles, every time one vehicle crosses or overtakes another. In the dry weather there are clouds of dust set up, which cut out all vision in front and in the wet weather there is always the danger of skidding.

The obvious remedy is to widen the metalling. But this is a very expensive affair, not only in the first cost which is about Rs. 100 per foot mile extra every alternate year, and with the amount of money avilable annually for such work, it will not be possible to keep pace with the growth of traffic.

Stabilization of berms, on the other hand, will in most miles, cost not more than about Rs. 900/- for a width of 10 feet, *i. e.*, 5 feet on each side of metalling. The wear on the surface

5" Loose soil compacted with sheep's foot roller with 40% granular material impregnated in the top $1\frac{1}{2}$ inches only. Cost Rs. 750/- per mile.
9' Wide. Traffic upto 100' tons per 24 hours.

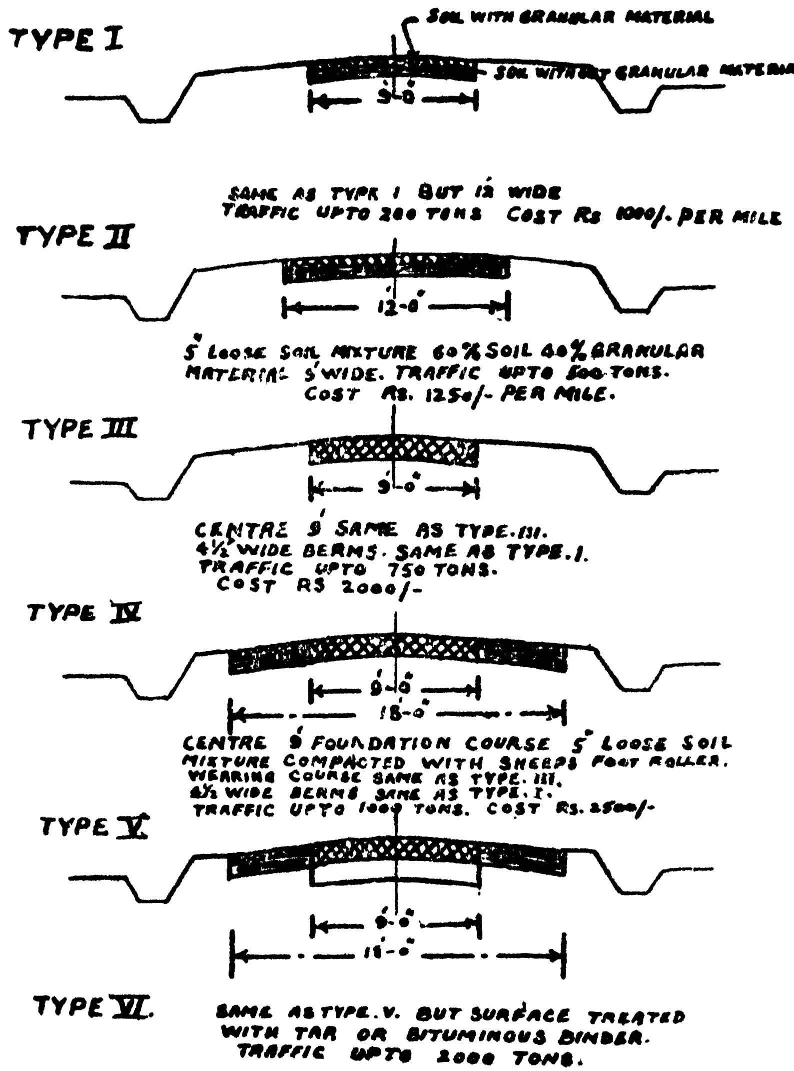


Fig. 7

being of the order of about $\frac{3}{4}$ inches per year, the first cost of such a crest can be spread over about 5 years, *i. e.*, Rs. 180/- per year. Allowing Rs. 70/- per year for maintenance *i. e.* $\frac{1}{4}$th of a cooly per mile (Rs. 36) and Rs. 34/- for patch materials, the cost including maintenance of providing a five-foot wide stabilized berm on each side of the metalling, would thus be about Rs. 250/- per mile per year, as against Rs. 225/- per mile per year, already being spent by the Punjab P. W. D., B & R Branch, for the upkeep of berm in their present dangerous state. So that for an extra Rs. 25/- per mile per year, you will have roads which will be safely negotiable in all kinds of weather. It is noteworthy that after this much width of the berm has been stabilized, the traffic confines itself to this width and leaves the remaining unstabilized part of the berm intect

(c) **Base stabilization.** Considering the use of a stabilized soil foundation for the metalling of roads, the cost of such a foundation being only Rs. 188/- per foot mile, as against Rs. 356/- for a brick on edge and Rs. 237/- for a flat brick foundation the great economy involved is self evident. An additional advantage is, that where as a brick soiling is perfectly useless for the dispersion of wheel loads, a stabilized soil foundation disperses the loads in the same way as the metalling on top, and thus helps in keeping down the intensity of pressure on the subgrade, which in turn adds to the life of the road.

(d) **Stabilized soil pavements.** Now that it has been possible to surface-treat a stabilized soil crust successfully, it is for consideration, whether the orthodox type of crust, *viz.*, the water bound macadam, should not be gradually replaced by the more scientific stabilized soil crust.

The water-bound macadam has one great drawback. It is essentially a cellular mass and and under the combined action of the static load and the fast traffic vibration, the sharp interlocked corners of adjoining pieces of metals, get rubbed off in course of time, causing local settlements in the body of the mass, which are seen on the surface in the shape of ruts and small depressions. The proof lies in the fact that when on old water-bound crust is dug up, the pieces of metal no longer have sharp corners, but are found to be rounded of all over.

A recognised practice now, to counteract this drawback, is to spread a layer of "good clay" under the wearing coat, so that during consolidation with water, the "good clay" should work up towards the surface and fill up the voids. This process certainly minimises the drawback, but does not remove it. There being always an excess of water in water-bound consolidation, when the "good clay" in the crust dries out, a certain percentage of voids are formed again. Besides, more often than not "good clay" is not available at site and whatever stuff is available, is made use of. In any case there is no definition of this "good clay" given anywhere.

If stabilized soil crust were to be used instead, with the usual 40% granular material, the elastic padding provided by the soil round the fragments of granular material will prevent, them from abrasion within the crust. Further the mass having been compacted to the fullest possible extent at "optimum moisture" there will be no further possibility of any more compaction under traffic, with the result that there will be no forming of ruts or small depressions, as in the case of water-bound macadam. It is also to be noted that the stabilized soil crust will be much cheaper than the water-bound macadam crust as the quantity of granular material will be only 40 percent.

11. Use of Stabilized Soil Base For Aeroplane Landing Grounds.

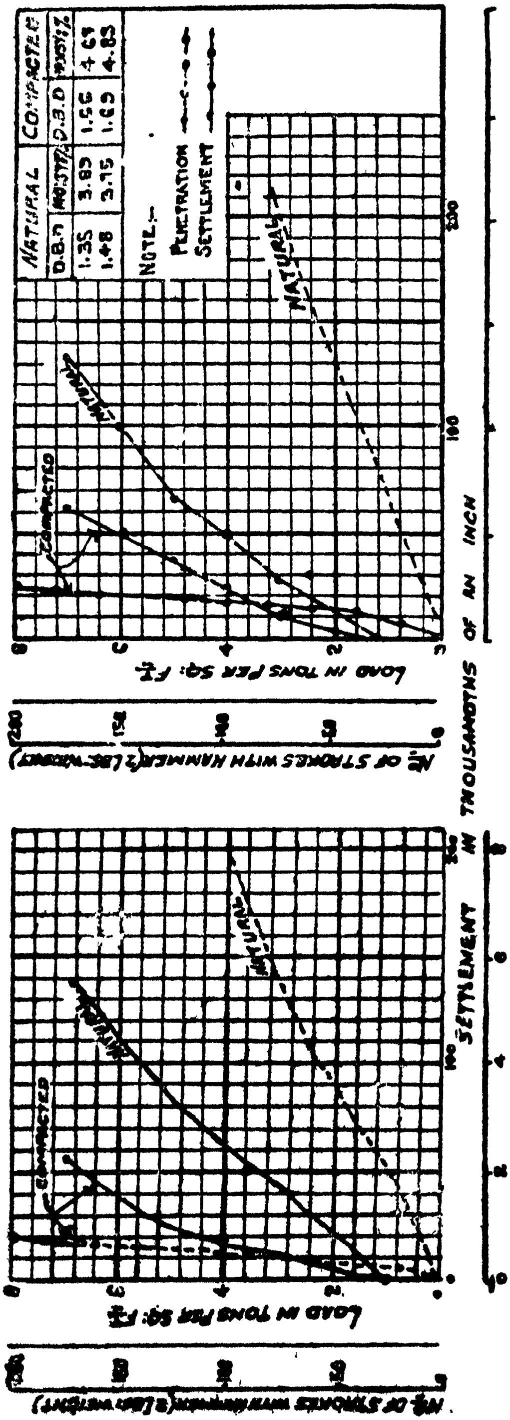
An abridged summary of methods used for stabilization of the base of the aerodrome landing grounds is given from Paper No. 270 P E.C. 1944, by A. R. Khanna, I. S. E.

(a) **Preparation of subgrade.** With the mode of construction adopted for paving the runways, *i. e.*. plain cement concrete slabs 4" or 4½" thick without any soling coat, it has been shown that subgrade plays an important part. Plain slab, without any reinforcement, has to depend on the subgrade for its safety as failure occurs by tension, which increases with sinking of the subgrade. As no soling coat was given, it was very necessary that the subgrade should be compacted to maximum possible density according to latest scientific methods.

The subgrade was ploughed 4" deep and all roots of vegetable growth carefully removed. The formation was made slightly higher than designed levels (to avoid any filling being necessary after compaction had been done) and the area was flooded. This was done most economically by constructing a high water level water course outsides the 50 yards strip which also came in very usefull for flooding the concrete slabs for curing purposes. As soon as the moisture content of the soil was optimum, as determined by the method outlined in Paper No. 257, Punjab Engineering Congress, Lahore, by K. B. S. I. Mahbub Densification of canal bank by S. I. Mahbub, the area was rolled over by toothed rollers to begin with and finally finished with plain power rollers.

The subgrade was kept ready in advance of concrete in a length of two to three chains and just before laying the concrete slabs, final dressing of the surface was done to designed levels. If, while doning this, the subgrade was lowered below the designed levels in some patches, it was not made up to design levels by filling loose earth, but additional quantity of concrete

PENETRATION TESTS ON NATURAL AND COMPACTED SOILS



BEARING TESTS ON CONCRETE SLAB

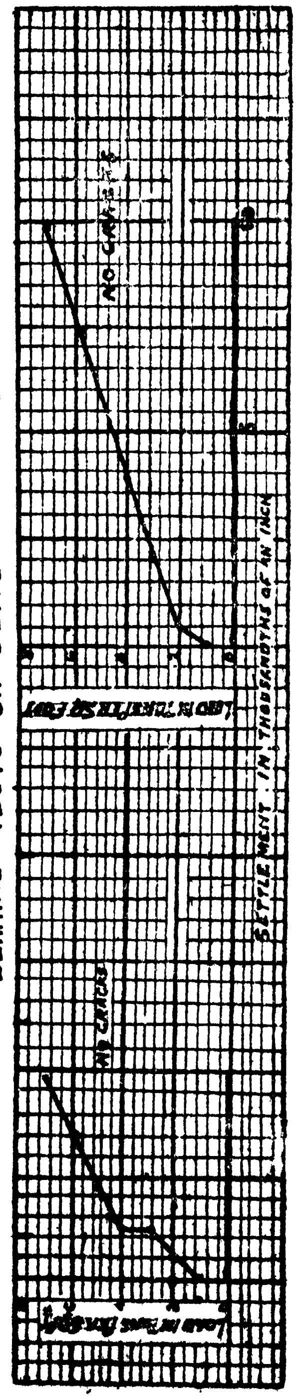


Fig. 8

was poured to make up such inequalities. Before laying concrete, depth of the subgrade was carefully tested by means of suitable templates to see that thickness of concrete would now here be less than 4" or 4½" as required.

(*b*) **Subgrade tests.** In order to check up the results actually obtained by compaction, dry bulk density of natural and compacted soil was obtained every 500 feet apart or closer. For this purpose, field laboratories were set up at each site where, besides determining dry bulk densities and penetration tests, complete mechanical analysis of soil was also carried out. Penetration tests were carried out by means of a Penetrometer.

In addition to penetration tests, actual settlement was also observed on compacted subgrade. The settlement was observed on a standard bearing area 6" × 6" with the help of a micrometer dial gauge reading up to 1/1000 inch. Typical results as actually obtained at one of the sites both for penetration and bearing tests are shown in Fig. 8. The results speak for themselves and do not need any comments. Bearing capacity tests must, for practical reasons, be on a small scale. It is, however, fully appreciated that tests on a small scale do not test the subgrade adequately, as the smaller the bearing area under test, the less is the transmission of load per square foot.

It is, however, felt that for practical purposes, the results given by field tests are fairly representative.

(*c*) **Advantages of compaction.** With the subgrade compacted as above, it will be seen that there is no further necessity to lay any soling coats of bricks or stone, before laying concrete slabs. The subgrade which when fully compacted gives a settlement of 0 04 inch only with a load of 7 tons per square foot does not need any soling coat on it. In all airfields constructed by the Irrigation Branch, 1 : 2 : 4 concrete slabs were laid direct on the compacted subgrade in case of runways. On taxi tracks the soling coat given was of bricks on edge in 1 : 5 cement mortar, laid on 1/4" thick cement plaster on compacted subgrade. This meant saving in large quantities of materials repuired for soling both on runways and taxi tracks. Every inch of thickness saved from a 2,000 yards runway, saves about 4500 tons of material. Taxi and dispersion tracks have equivalent area of about two runways, and most airfields have two runways. So every inch saved meant a saving of about 18,000 tons per landing ground.

12. Soil Stabilization in Water-logged Area.

The experimental work in this connection while constructing landing grounds in water-logged areas in the Punjab was done by C.L. Hands, I.S.E., Executive Engineer and published in paper No. 269 P.E.C. 1944.

At one of the sites, the sub-soil water-level was very high, being within two feet of of natural surface. While rolling the subgrade with 12 tons steam roller, it was noticed that the bearing capacity of the soil was not high enough and the subgrade would be lifted in ripples as the roller passed over it creating boggy conditions. Measures like rolling ballast into the subgrade and increasing the thickness of concrete slabs were considered to overcome the difficulty. Finally it was decided to roll the subgrade by a repeated number of rollings with a light roller. This gave the desired results and the steam roller was altogether discarded at this site. The clue to this solution of soil compaction in water-logged lands was suggest by observing the camal tracks (foot paths). These lightly tamped paths had great bearing capacity while the remaining area was marshy. Rollings with light roller weight of 1·3 tons, gave dry bulk densities of the order of 1·75 which could compare favourably with any other site and slabs of ordinary thickness of 4" were laid on the subgrade thus compacted.

Experimental concrete slabs 4" thick laid under the conditions described above, were later subjected to actual tests It was found that with sub-soil water level only 6" below the bottom of the slab, could take a load of 17 tons per square foot applied on a 6" square base near the corner before it cracked. The same slab with a load of 24 tons per square foot on a 6" square base applied at the centre neither showed any settlement nor any cracks.

The formula of S. I. Mahbub (author of paper 257) was tested by C. L. Handa. It was seen that the formula (W = 34—14 S) could be used only in the case of soils in which there was a predominance of sand (over 50 percent). In the case of soils with a predominance of silt (over 35 percent) the above formula was not only a rough guide but was actually misleading as the variation between moisture percentage actually needed and that given by the formula was so large as to be up to 40 percent.

One obvious fact in this connection is that it is not sand content alone that governs the moisture percentage, but silt also plays an important part. In the case of above and similar compositions, more concrete results have been derived from the following formula :—

$$W = 24 - 0{\cdot}14\ S - 0{\cdot}1\ S_{t} \qquad (A)$$

Where W is the optimum moisture percentage. S the sand percentage. S_t the Silt percentage.

The formula (A) was no doubt a better guide in the case of predominently silty soils as compared with *that* soils, yet this formula also was accepted with considerable reserve as in certain instances the moisture percentage of samples with similar silt content was considerably different.

PART VI

GENERAL SECTION

CHAPTER II

Design of Bridges and Culverts

A brief summary is given below from a paper written by Brijmohan Lal, I.S.E. Indian Roads Congress and Technical No. 4 by R. Trevor Jones, Chief Engineer.

In future all bridges and culverts other than foot bridges shall be designed to carry Indian Roads Congress Standard Loading. This loading has been defined in paragraph B pages 6 and 7 of Indian Roads Congress Bridge Specification as follows :—

"0·34 ton per linear foot of each traffic lane plus a knife edge of 6 tons for computing bending movements, or of 9 tons for computing shears with the limitation that for computing bending moments the total distributed load on loaded lengths of 20 feet and under, shall never be less than 6·8 tons per lane of traffic over the whole loaded length."

The lane of traffic has been defined in Paragraph A_3. "For road bridges it may be assumed that each line of rolling load traffic occupies a width of 10 feet, and it is, therefore recommended that, with the exception of bridges for a single lane of traffic, the width of the carriage-way should, wherever possible, be a multiple of that dimension."

The loaded length is the length of the member loaded in order to produce the most severe stresses. In a freely supported span the loaded length would thus be, (*a*) for bending moment, the full span, (*b*) for shear or support, the full span, and (*c*) for shear at intermediate points, the length of span from this point to the farther support.

For all bridges foot paths and other parts of the floor accessible only to pedestrians and animals, the loading shall be 84 pounds per square foot.

Impact factor. The clause B_4 of the I. R. C. Bridge specifications gives the following formula for impact factor :—

$$I = \frac{1}{2} \times \left(\frac{65}{45 + [L(n+1)]/2} \right)$$

with a maximum value of 0·50 where n is the number of traffic lanes and L is the loaded length of the span giving the maximum stress in the member under consideration.

It is proposed in this chapter to gives designs of bridges of small spans made of reinforced concrete, masonry arches and timber respectively.

Reinforced Concrete Bridges.

The following types of bridges have been considerd.

(A) Reinforced Concrete Slabs for spans upto 20 feet.

(B) Rolled Steel Beams carrying Reinforced Concrete Slabs. For spans 10 to 40 feet.

(C) Reinforced Concrete T-Beams and Slabs. For spans 15 ft. to 40 feet.

The stresses recommended be the I.R.C. Bridge Specification for ordinary grade concrete have been accepted and are given below. The stresses are to be used in future for designs of bridges.

Steel in tension or f_s	18,000	lbs. per sq. inch
Concrete 1 : 2 : 4 in compression or f_c ...	750	,, ,, ,, ,,
Concrete 1 : 2 : 4 in shear or v	75	,, ,, ,, ,,
Maximum shear stress with shear reinforcement is equal to 4 time v	300	,, ,, ,, ,,

Bond stress or u for plain bars	... ...	100 lbs. per sq. inch.
Local maximum bond stress at any point not to exceed twice u	... ...	200 „ „ „ „
n =ratio of modulus of elasticity of steel in tension to modulus of elasticity of concrete in compression	... ...	18

The following symbols used in reinforced concrete design are explained : –

k = Ratio of depth of netural axis to depth d

$j = (1 - 1/3k)$

P = Ratio of steel to concrete

$R = pf_s j = (\frac{1}{2} f_c kj)$ a constant depending upon ratio of steel to concrete and there strengths.

M_r = Moment of resistance per foot width of slab. The value of these constants for stresses adopted are :—

$P = 0{\cdot}0089$

$J = 0{\cdot}857$

R 137

These values can be easily determined from the following formula : —

$$k = \sqrt{2\,p \times n + (p \times n)^2} - p.n. = \frac{1}{1 + [f_s/(nf_c)]}$$

$$p = \frac{\frac{1}{2}}{f_s/f_c\,[(f_s/nf_c) + 1]}$$

Detailed examples of R.C.C. bridge, slab type have been worked out below.

A (*i*) reinforced concrete slabs for spans up to 20 feet :—

Take for example a span of 10 feet, assume total thickness of slab as $10\frac{1}{2}$ inches with effective depth of 9 inches. Effective span = clear span plus effective depth of the slab = 10·75′

Assume 3 inches cement concrete or tar concrete as wearing coat weighing 140 lbs. per cft.

Dead load.

Weight of slab $10\frac{1}{2}$ inch thick at 144 lbs. per cft.	...	= 126 lbs.
Weight of concrete 3 inches thick ...	...	= 35 „
	Total...	161 „
B M. due to dead load = $[161 \times (10{\cdot}75)^2 \times 12]/8$	=	28,000 in lbs.

Live load.

Due to distributed load = $[0{\cdot}68 \times 2240 \times (10{\cdot}75)^2 \times 12]/8$	=	24,500 in lbs.
Due to knife edge load = $[0{\cdot}6 \times 2240 \times 10{\cdot}75 \times 12]/4$	=	43,400 in lbs.
Total B.M.	...	67,900 in lbs.
Add 50 percent for impact, as impact factor for this span is 0·50.		
Therefore total B.M. for live l ad ...	...	101,800 in lbs.
Adding dead load bending moment the total bending moment	...	129,800 in lbs.

Referring to table (1), it will be seen that a slab of total thickness $10\frac{1}{2}$ inches and with 5/8 inches diameter bars at $3\frac{3}{4}$ inches centres, is safe for bending moment. Half the bars may be bent upwards near the supports according to the usual practice.

Now test this slab for shear and bond stresses.

Shear stress. This will be maximum at the supports.

Due to dead load of slab and wearing coat = $(161 \times 10{\cdot}75)/2 = 867$ lbs.

Due to live load with impact of 50% :—

Distributed load $= \frac{\cdot 34 \times 10\ 75 \times 2240 \times 1\cdot 5}{10 \times 2} = 615$ lbs.

Knife edge load $= \frac{9 \times 2240 \times 1\cdot 5}{10} = 340$ lbs.

Hence total shear $= 867 + 615 + 3024 = 4506$ lbs.
Shear stress $= v = [V/(b\ j\ d)]$
$j = 0\cdot 857$, $b = 12''$ and $d = 9\frac{1}{4}$ inches.

$v = \frac{4506}{12 \times 0\ 857 \times 9\frac{1}{4}} = 48\cdot 0$ lbs. per square inch against an allowable stress of 75 lbs. per square inch.

Bond Stress.

Bond stress $= u = V/(j\ d\ \Sigma o)$

where Σo = sum of perimeters of bars in the tensile reinforcement.

The tensile reinforcement provided is 5/8″ rods at ($3\frac{3}{4}$″) centres. Therefore $\Sigma o = \left(11 \times 5/8 \times \frac{12}{3\frac{3}{4}}\right) = 6\cdot 288$ inches. As half the bars are bent at top, the effective tensile steel is 50% only.

Hence effective sum of perimeters of bars $= 6\cdot 288/2 = 3\cdot 144''$

$u = \frac{4506}{3\cdot 144 \times 9\frac{1}{4} \times 0\cdot 857} = 183$ lbs. per square inch which is safe.

Table 1 gives the complete design of slabs form 2 feet to 20 feet spans to carry I R.C. Standard loading. Details of reinforcement, thickness slabs. etc., can be readily determined from the same (arrangement shown in Fig. 1).

Details of bending of bars for all spans simply sported slab.

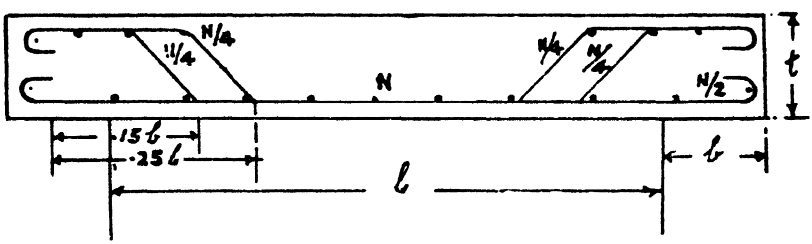


Fig. 1.

Note :—(1) $\frac{1}{4}$ *No. of bars are bent up at 0·25 l distance*
$\frac{1}{4}$ ....*at 0·15 l distance*
$\frac{1}{2}$ *No. of bars run straight*

Note :—(2) *Bearing of slab over abutment* $= b = \frac{l}{2} + \left(\frac{5}{40}\right)$

as per M. E. S. Hand Book Vol. III
or
$b = t$ (*for small spans*)

Note :—(3) *Total number of main Reinforcement Bars* $= N$

To make the slab simply supported, it is necessary that there should be no bond between the abutment and the slab. The top of abutment should be cement-plastered and then covered with two coats of hot bitumen or tar before the slab is laid.

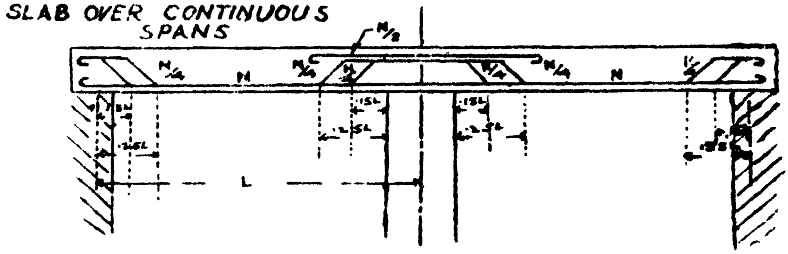
(*ii*) **Slab for continuous spans.** It is frequently necessary to build bridges of multiple spans using continuous slabs.

Table 2, gives bending moments for live loads for intermediate spans and end spans. For purposes of calculations, bending moment for intermediate spans has been taken as

WL/12 and that for end spans as WL/10.

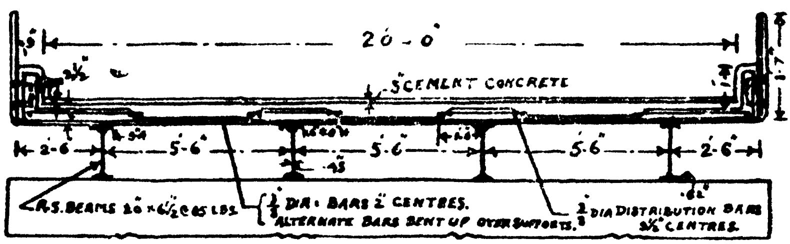
Take for example 10 feet span. Effective span to say 10 feet.

Intermediate span



NOTES : –
In continuous spans N/4 bars are bent up at 0.30 point and N/4 bars at 0·15 point. N/2 bars run straight.
¼ No of short bars are provided on top at the support in addition to bend up bars.
2. N is total number of main reinforcement bars.

Fig. 2.



R.S. BEAMS WITH R.C. SLAB
CLEAR SPAN 25 FEET
ROAD WAY 20 FEET
SCALE 3/16" = 1 FOOT

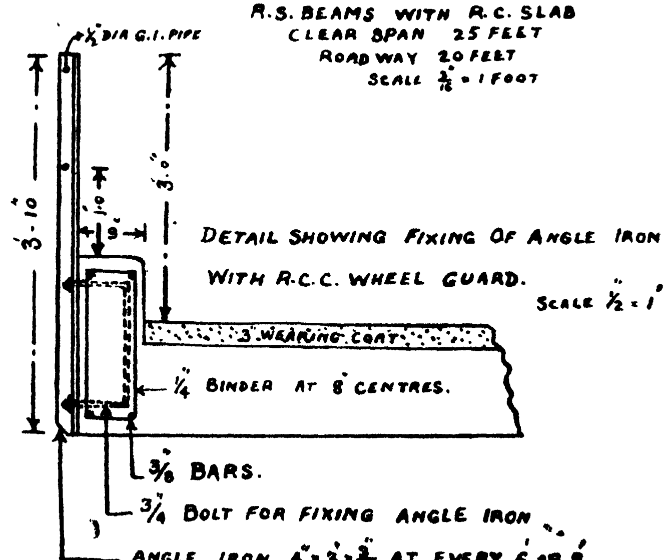


Fig 3

Bending moment for distributed live load excluding 50 percent impact per foot width $= \frac{6 \cdot 8 \times 2240}{10} \times \frac{10 \times 12}{12}$ inch pounds $= 15{,}230$ inch pounds.

Bending moment for knife edge load per foot width $= \frac{6 \times 2240 \times 10 \times 12}{10 \times 6} = 26{,}880$ inch pounds

Total $= 42{,}110$ inch pounds

Adding 50 percent impact this amounts to 63,165 inch pounds.

A reference to Table 1 will show that a slab of 8 inches thickness has a moment of resistance of 80,600 inch pounds.

Assume a slab of 8 inches thickness.

Bending moment for dead load $= \frac{(96+35) \times 10 \times 10 \times 12}{12} = 13{,}100$ inch pounds.

Total bending moment $= 76{,}265$ inch pounds.

Therefore a slab of thickness 8 inches with a resistance moment of 80,600 inch pounds will suit.

For an end span the total bending moment will be $76{,}265 \times \frac{6}{5} = 93{,}518$ inch pounds.

A slab of thickness $8\frac{1}{2}$ inches with a resistance moment of 92,500 inch pounds will therefore suit. Arrangement of bars is shown in Fig. 2.

B. Reinforced Concrete slabs supported on Rolled Steel Beams.

It will be seen from Tables 1 and 2 that simply supported slab bridge up to 15 feet span and continuous slabs up to 20 feet span can be economically constructed. For bigger spans it is necessary to support slabs on beams of steel or reinforced concrete.

The economical spacing of beams for a 20 feet clear roadway is either 5′-6″ centres, in which case 4 beams will be used or 8 feet centres in which only 3 beams will be used.

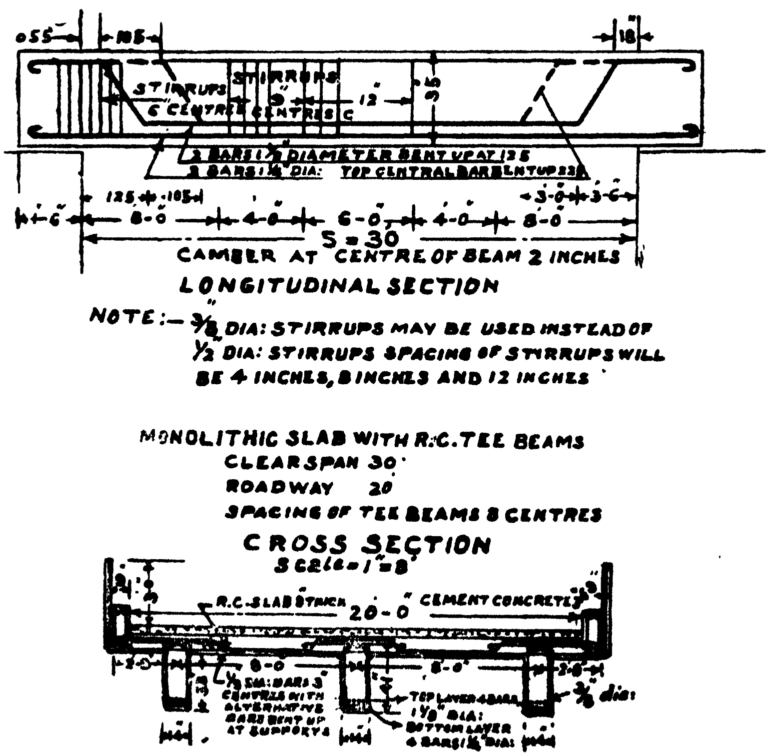


Fig. 4

The roadway slab will be 7 inches thick, with 3/8 inch diameter rods 2 inches centres as main reinforcement in the first case, and 8 inches thick slab with ½ inch diameter rods 3 inches centres as main reinforcement in the second case.

Table 2 gives the design of R.S. Beams placed 5 ft. 6 inches centres for various spans and Fig. 3 shows the arrangement of decking, etc. Similarly Table 3 gives the design of R.S. Beams place 8 feet centres for various spans.

C. Reinforced concrete slabs supported on R.C. Beams.

The reinforced concrete beams will act monolithic with slab. Their detailed design has been worked out according to principles of design of reinforced concrete structures given in various text books.

The size of T-Beams at 5′-6″ spacing are given in Table 4, which also gives quantity of reinforcement. The details of reinforcement of slab and T-Beams for 30 feet span are shown in Fig. 4.

Table 4 (A) gives the sizes of T-Beams when they are spaced at 8 feet centres and Fig. 5 gives details of slab, parapet and T-Beams for a 30 feet span bridge.

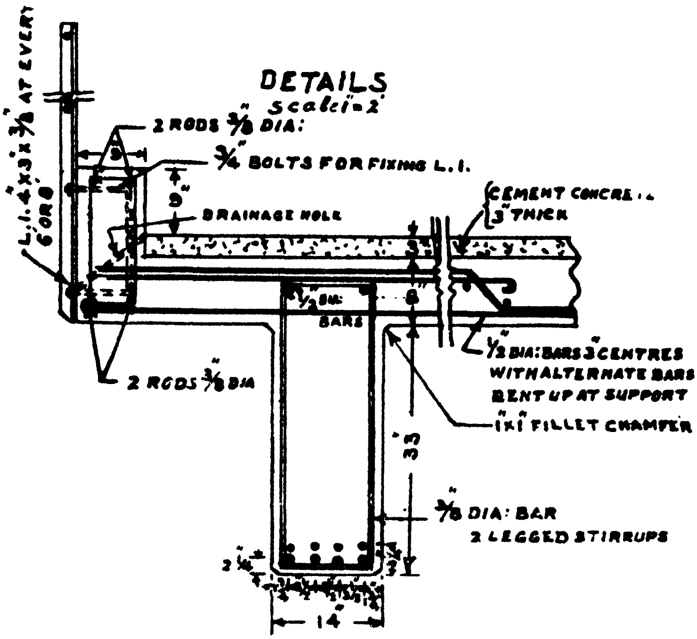


Fig. 5

Design of structure. It is not possible to give standard designs for foundations, as conditions vary to a considerable extent in different soils. Each case must be designed by the Engineer-in-charge according to the general principles and the following points must be considered in design :—

(1) Bearing pressures of foundations.
(2) Type of filling behind the abutment.
(3) Method of support of slab, etc.

A typical design of abutment is shown in Fig. 6. The design of wing walls, piers, etc., as per M E S. practice are shown in Fig. 7. The steps are arranged by projecting not more than ½ brick. The thickness of wall sections for 9″ bricks will be as shown in Fig. 6.

Masonry arches.

The thickness of arch ring for various spans can be determined by various empirical formulae given in different handbooks of Civil Engineering. Rankine gives the following formula

$$t = C\sqrt{S}$$

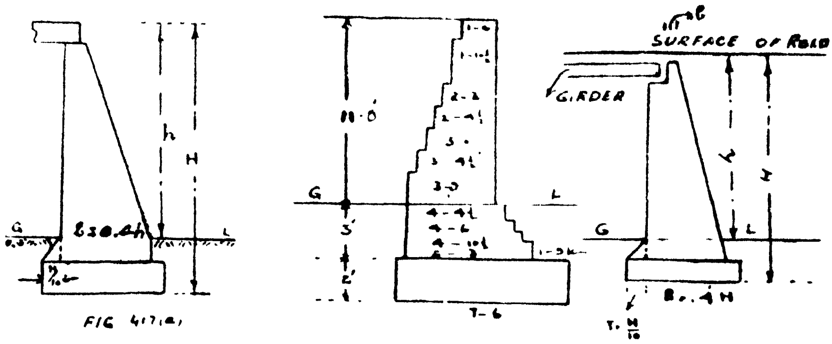


Fig. 6.

where t is the thickness of arch ring at the crown in feet. C is a constant, the value of which depends upon the nature of materials used, span rise ratio, etc., and S is span. Its average value is taken $=0{\cdot}4$.

The French Manual of Civil Engineering gives the following formula :—

$$T=\frac{S}{30}+1{\cdot}1.$$

where t and S have the same meaning as in Rankine's formula. The following rules based on Rankine's formula give more exact results for semi-circular and segmental arches.

The data regarding length of radius in terms of span rise, etc., are also tabulated :—

Type of arch	Arch thickness at crown	Radius in terms of span	Rise
60° Arch ..	$0{\cdot}43\sqrt{S}$ ...	R = S	$r=0{\cdot}134$ S
75° Arch ...	$0{\cdot}40\sqrt{S}$ ...	R = 0·821 S	$r=0{\cdot}170$ S
90° Arch ...	$0{\cdot}38\sqrt{S}$ ..	R = 0·707 S	$r=0{\cdot}207$ S
120° Arch ...	$0{\cdot}36\sqrt{S}$ ...	R = 0·577 S	$r=0{\cdot}288$ S
180° Arch ...	$0{\cdot}36\sqrt{S}$ ...	R = 0·500 S	$r=0{\cdot}500$ S

With a thick cushion of about $1\frac{1}{2}$ feet over the crown of the arch, it is safe to assume that the impact of live load on the arch will be only about 25 percent The I.R.C. Standared loading with 25% impact is practically equivalent to 15 ton Road Roller load with 25 percent impact. As these arches are safe to carry a 15 ton road roller, they may be considered safe for I.R.C. loading.

The thickness of arch ring at springing may be taken the same as at crown for small spans. In case of large spans of over 20 feet, the thickness at springing should be increased by about 20 percent. It is advisable to test the stability of arches of larger spans, by the usual method of stress diagrams.

In case of well-dressed stone arches, the thickness of arch ring may be reduced by about 20 percent. The stone or brick masonry for arches having spans of 5 feet and over, must be in 1:3 cement-sand mortar. As a general rule, a 60° arch should be avoided, as it has tendency to crack at haunches and also is more expensive to build. Its only advantage over other types of arches is that it has a low rise.

Thickness of abutments. The following rules based on Irrigation practice are given for thickness of abutment at springing for various types of arches :—

1. 60° Arch $T=0{\cdot}21\ S+C$
2. 75° Arch $T=0{\cdot}18\ S+C$
3. 90° Arch $T=0{\cdot}16\ S+C$
4. 120° Arch and 180° Arch $T=0{\cdot}15\ S+C$.

Where T is the thickness of abutment at springing and S the span.

C is a constant the value of which is $1\frac{1}{4}$ feet for span under 5 feet.

$1\frac{1}{2}$ feet for spans between 5 feet and 10 feet.

2 feet for spans between 10 feet and 20 feet.

The back slope of abutment shall be 1 in 2 for 60, 75° and 90° arches and 1 in 3 for 120° and 180° arches.

The above formulae apply to spans up to 20 feet and abutments having a depth of 10 feet or less below springing.

Thickness of piers. The dimensions of piers and pier foundations are given in a Table published by the Irrigation Department, which is reproduce for guidance (Table 5).

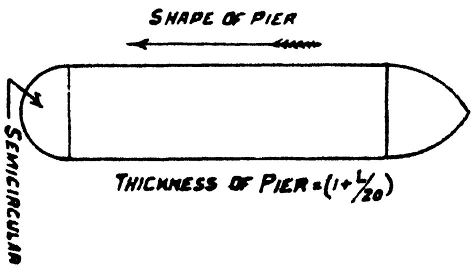
For bridges having spans of 15 feet to 20 feet, the dimensions of piers and foundations should be increased by about 12 percent over those given for 15 feet spans.

The dimensions given for foundations are for alluvial soils of the Punjab. In a hilly area, where the soil may be hard, these dimensions may be reduced suitably.

Wing Walls. The dimensions of wing walls for various heights can be worked out from Fig. 7.

NOTE :—
The shape is not to be used upstream of the pier where boulders or logs of wood flow in the stream.

WING WALL



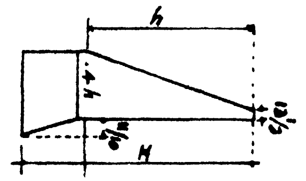


Fig. 7.

Timber Bridges.

In cases of urgency, it may be sometimes necessary to construct a bridge of timber planks over masonry abutments This is especially so where good stone for arch work in masonry is not easily available and where timber is cheap. Timber bridges with spans varying from 2 feet to 15 feet have been designed as per I.R.C. Standard Loading and thickness of timber planks are given for each case in Table VI

Paragraph C_7 of I.R.C Bridge Specification gives a formula for determining the sizes of timber planks and beams :—

"All timber subjected to bending stresses shall be proportioned by the formula :—

$$M = \frac{P}{8} b \times d^2$$

where M is the greatest bending moment in inch pounds, p is a constant varying with the type of timber given in the Table below, b the width of the beam in inches and d the depth of the beam in inches.

Value of p and weights of timber commonly used in the Punjab are given below :

	Weight in lbs. per cft.	P
Teak	43	2467
Sal	56	3107
Shisham	49	2373
Deodar	35	1653
Kail	32	1093
Chir	36	1360

Design of a timber bridge of *deodar* wood of 8 feet span.

Assume thickness of planks = 6 inches

Weight of wearing coat and cushion, etc., about 7″ thick	=75 lbs. (say) per sft.
Weight of planks 6″ thick	=17 lbs. per sft.
Total ...	92 lbs.

Note :—(As the weight of timber is small compared to the weight of wearing coat, etc., slight variation in thickness of planks will not affect bending moments very much. Hence for bridges up to 10 feet span 6″ thickness of planks has been assumed).

Effective span of bridge = 8·5 feet

B.M. due to dead load $= \frac{92 \times 8\cdot5}{8} = 830$ ft. lbs.

Live load : —

B. M. due to distributed load including impact $= \frac{68 \times 2240 \times 1\cdot5 \times 8\cdot5}{8} = 2425$ ft. lbs.

B.M due to knife edge load $= \frac{\cdot6 \times 2240 \times 1\cdot5 \times 8\cdot5}{4} = 4260$ ft. lbs.

Total B. M. $= 830 + 2425 + 4260 = 7515$ ft. lbs.

Now $bd^2 = \frac{8M}{P}$ and $b = 12$ inches

Therefore $d^2 = \frac{8}{P}\left(\frac{M}{12}\right) = \frac{8}{P}(M')$

where M' is the bending moment in ft. lbs.

Hence $d = \sqrt{\frac{8M'}{P}}$

$p = 1653$ for *deodar* wood in 1 lb. per sq. inch.

$d = \sqrt{\frac{M'}{207}}$;or $= \sqrt{\frac{7515}{207}} = 6\cdot16$ inches or say $6\frac{1}{4}$ inches.

Normally planks less than 4 inches in thickness should not be used for a permanent construction, as thinner planks are liable to warp in an exposed condition.

The thickness of planks as given in the Table must be multiplied by appropriate factor for the following timbers :—

Timbers.	*Factors.*
1. Teak	0·82
2. *Sal*	0·70
3. *Shisham*	0·84
4. *Kail*	1·22
5. *Chir*	1·1

Bearing of planks over abutments :—

The bearing of planks should be equal to the thickness of planks, with a minimum of 6 inches.

Other standard loads for bridges.

(*a*) *15 ton roller for metalled roads.*

The following is the standard data for 15 ton Road Roller as taken from the Interim report of P.W.D. committee on Reinforced Concrete Bridge, 1924, and as approved by the Chief Engineer, Irrigation in his No. 3107, dated 27th August, 1924, to all Superintending Engineers :—

Weight of front wheels	13500 lbs.
Weight of back wheels	12,000 lbs.
Wheel base centre to centre	12 feet
Track centre to centre	5 ft. 9 inches
Width of each back wheel	1 ft. 8 inches
Width of front wheel	4 ft. 3 inches

Area covered by the Roller over which there will be no crowed load $= 20' \times 9'$.

Width of contact between Roller wheel and the Road surface $= 3''$.

(h) Cart loads for unmetalled road bridge.

Bridge is designed for standard load for the cart load bridges as approved by the Cheif Engineer, Irrigation Branch, vide No. 296/93 N.I., dated 25th January, 1923, to all Superintending Engineers and published in Interim report by the Buildings and Roads Branch.

(a) Crowd load including impact=40 lbs. per square foot.
(b) Dead load of the fill and the slab .. ..(as per actual thickness).
(c) Live load including impact for the loaded.
(d) Cart and bullocks... ....4,000 lbs. point load each wheel **4·0 ft. apart.**
(e) Load for foot bridges ....80 to 100 lbs. per square foot.

TABLE I.

Serial No.	Clear span in feet	Effective span.	Total thickness of slab.	Effective depth of slab.	Dead load of slab 3" wearing coat.	Main steel reinforcement: Diameter of bars	Main steel reinforcement: Spacing C to C.	Distribution steel reinforcement: Diameter of bars	Distribution steel reinforcement: Spacing C to C	Dead load B.M. inch pounds	Live load B. M. including impact.	Total B. M.	Moment of resistance of slab	Shear stress in lbs.: Shear due to dead load.	Shear stress in lbs.: Shear due to distributed live load.	Shear stress in lbs.: Shear due to knife edge load	Shear stress in lbs.: Total shear.	Shear stress.	Bond stress.	Remarks.*
	Ft.	Ft.	Inches	Inches		Inches	Inches	Inches	Inches											
1	2	2·5	13/2	23/4	113	3/8	2	1/4	5	1060	23700	24760	54400	141	142	3024	3307	55	186*	
2	3	3·5	49/8	23/4	113	3/8	2	1/4	5	2080	33200	35230	54400	198	200	3024	3422	57	196*	
3	4	4·5	7	25/4	114	3/8	2	1/4	9/2	3620	42600	46220	64000	270	258	3024	3552	54	182*	
4	5	5·5	15/2	27/4	125	3/8	2	3/8	19/2	5700	52100	57800	74500	352	319	3024	3695	53	174*	
5	6	6·6	8	7	131	1/2	3	3/8	9	8500	62500	71000	80600	440	380	3024	3844	53	200*	
6	7	7·6	17/2	15/2	137	1/2	3	3/8	8	12000	72000	84000	92500	522	438	3024	3984	51	192	
7	8	8·7	19/2	33/4	149	5/8	4	3/8	7	16900	82500	99400	112000	672	490	3024	4186	50	200	
8	9	9·7	10	35/4	155	5/8	4	3/8	7	21900	91900	113800	125000	783	555	3024	4362	49	194	
9	10	10·75	21/2	37/4	161	5/8	15/4	3/8	13/2	28000	101800	129800	140000	867	615	3024	4506	48	183	
10	11	11·8	11	39/4	167	5/8	7/2	3/8	6	34900	111000	145900	155000	1020	678	3024	4729	47	165	
11	12	12·9	23/2	41/4	173	5/8	13/4	3/8	6	43200	122400	165600	172500	1157	740	3024	4921	47	152	
12	13	13·9	12	43/4	179	5/8	13/4	3/8	6	51700	131900	183600	190000	1300	795	3024	5119	46	151	
13	14	15·0	13	23/2	191	3/4	4	1/2	11/2	64500	142000	206500	216000	1430	850	3000	5280	45	150	
14	15	16·0	27/2	12	197	3/4	4	1/2	17/2	75650	152100	227750	237000	1630	910	2970	5510	45	150	
15	16	17·0	14	25/2	203	3/4	4	1/2	17/2	88000	160000	248000	256000	1750	950	2940	5640	44	149	
16	17	18·1	29/2	13	209	7/8	5	1/2	8	102700	167600	270300	278000	1890	1010	2920	5820	44	158	
17	18	19·2	15	27/2	215	7/8	5	1/2	8	117600	174500	292100	298000	2060	1060	2900	6020	43	159	
18	19	20·3	31/2	14	221	7/8	19/4	1/2	15/2	136000	185000	321000	322000	2248	1105	2890	6243	43	147	
19	20	21·3	16	29/2	227	7/8	9/2	1/2	15/2	153000	194000	347000	648000	2420	1160	2880	6460	43	140	

*In these slabs, depth as well as reinforcement is governed by bound stress.

TABLE 2.

Design of rolled steel beams placed 5 feet 6 inches apart and carrying A.R.C. slab 7 inches thick and a wearing coat of cement concrete 3 inches thick.

Serial No.	Span.	Effective span.	Dead load bending moment in inch tons	Live load bending moment including allowance for impact in inch tons.	Total bending moment in inch tons. M.	Section modulus required about x–axis=M 3	Size of suitable beam in inches from Hand Book of Dorman Long and Co., Ltd., 1924 edition.	Weight per foot run.	Section modulus
1	19	20	191	450	641	80 1	18×6	55	93 5
2	20	21	210	480	690	86 3	18×6	55	93 5
2	22	23 5	272	560	832	104	20×13/2	65	122 6
4	25	27	345	660	1005	126	20×13/2	65	122 6
5	27	29	410	715	1125	141	22×7	75	152·4
6	00	32	500	810	1310	164	24+15/2	90	203 7
7	32	34	580	890	1470	184	24×59/8	90	203 6
8	35	37	682	1000	1682	210	*85/4×10	109	215·4
9	37	39	780	1070	1850	231	*43/2×10	118	257 4
10	40	42	924	1180	2104	263	*87/4×10	127	259·0

*Compound Girders

TABLE 3.

Design of rolled steel beams placed 8 feet apart and carrying an 8 inches thick R.C. slab and a wearing coat of 3 inches thick cement concrete.

1	19	20	306	657	963	120·4	20×13/2	65	122·6
2	20	21	340	708	1048	131·0	22×7	75	152·4
3	22	23·5	427	818	1245	155·6	24×15/2	90	203·6
4	25	27	572	964	1539	192·9	24×15/2	90	203·6
5	27	29	666	1044	1710	213 8	*85/2×10	109·5	215·8
6	30	32	821	1183	2004	250 5	*23 ×12	117·5	252·6
7	32	34	933	1299	2232	279 0	*93/2×12	128	281·7
8	35	37	1082	1460	2542	317·8	*101/4×12	143	340·9
9	37	39	1251	1562	2813	351·9	*51/2×12	153	372·4
10	40	42	1462	1723	3185	398·1	*103/4×12	164	404·1

*Compound Girders

TABLE 4.

Design T. beams placed 5 feet 6 inches centre to centre carrying a 7 inches thick reinforcement concrete slab and wearing coat of 3 inches thick cement concrete safe for Indian Standard Loading.

Span.	Total bending moment in inch pounds.	Depth of stem	Width of stem	MAIN REIN-FORCEMENT: Top layer Number of bars with diameter in inches	MAIN REIN-FORCEMENT: Bottom layer, Number of bars with diameter in inches.	TWO-LEGGED STIRRUPS ½ INCH DIAMETER BARS: Spacing 6 inches.	Spacing 9 inches.	Spacing 12 inches	ae=distance between centres of bottom layer of bars.	b=distance of centre of outer bar 66 from edge.	c=distance from centre line of bottom layer of bars to bottom of beam.	d=cover, i.e., distance from centre between the two layers of bars to bottom.
		In.	In.	Dia Ins.	Dia. Ins	Ft.	Ft.	Ft.	Ins.	Ins.	Ins.	Ins.
15	1,144,000	16	12	2×1	3×1	0 to 4	4 to 6	6 to 15/2	4	2	2	3
20	1,717,000	20	12	3×1	3×1	0 to 5	5 to 8	8 to 10	4	2	2	3
25	2,510,000	25	12	3×1	3×1	0 to 6	6 to 10	10 to 25/2	15/4	9/4	5/2	3
30	3,445,000	28	14	3×9/8	3×5/4	0 to 8	8 to 12	12 to 15	9/2	5/2	2	15/4
35	4,490,000	32	14	3×5/4	3×5/4	0 to 9	9 to 14	14 to 35/2	17/4	11/4	2	3
40	5,770,000	34	16	3×3/2	3×3/2	0 to 10	10 to 16	16 to 20	11/2	5/2	11/4	4

NOTE :—3/8 inch diameter stirrups may be used instead of 1/2 inch Diameter stirsups as they are easy to bend and fix in proper position. The spacing of stirrups will be 4 inches, 8 inches and 12 inches.

TABLE 4. (A)

Design of T. Beams placed 8 feet centre to centre carrying an 8 inch thick reinforced concrete slab and a wearing coat of 3 inches thick cement concrete safe for Indian Standard Loading

		Ins.	Ins.	Ins. dia.	Ins. dia.	Feet	Feet	Feet	Ins.	Ins.	Ins.	Ins.
15	1,700,000	20	12	3×1	3×1	0 to 4	4 to 6	6 to 15/2	4	2	2	3
20	2,500,000	25	12	3×1	3×1	0 to 5	5 to 8	8 to 10	4	2	2	3
25	3,870,000	28	14	3×5/4	3×5/4	0 to 6	6 to 10	10 to 25/2	9/2	5/2	9/4	15/4
30	5,200,000	33	14	4×9/8	4×5/4	0 to 8	8 to 12	12 to 15	7/2	7/4	9/4	15/4
	6,600,000	37	16	4×5/4	5×5/4	0 to 9	9 to 14	14 to 35/2	7/2	2	9/4	15/4
40	8,520,000	40	18	5×5/4	5×5/4	0 to 10	10 to 16	16 to 20	7/2	2	9/4	15/4

NOTE :—In the design of stirrups, it is assumed that part of shear stress is taken up by concrete (*vide* Clause D 24, paragraph (*iii*) of I.R.C. Bridge Specifications.)

TABLE 5

Bridge Dimensions required to take a 15. ton Roller.

(All Dimensions are in feet.)

Width of the foundations.

Spans	Thickness Arch 90°	Thickness Arch 75°	Thickness of piers 90°	Thickness of piers 75°	Depth of pier foundations 90°	Depth of pier foundations 75°	90° Height of piers 1	2	3	4	5	6	7	75° Height of piers 1	2	3	4	5	6	7	*Remarks.
2	·8	·8	1·3	1·3	2·0	2·0	2·0	2·25						2·25	2·50						
3	·8	·8	1·3	1·7	2·0	2·0	2.25	2·75	3·00					2·50	3·00	3·25					
4	·8	1·3	1·7	1·7	2·0	2·0	2·50	3·00	3·25	3·50				2·75	3·25	3·50	3·75				
5	1·3	1·3	1·7	2·1	2·0	2·25	2·75	3·25	3·50	3·75	4·00			2·75	3·50	3·75	4·00	4·25			
6	1·3	1·3	2·1	2·1	2·25	2·25	3·00	3·50	3·75	4·00	4·25	4·50		3·00	3·75	4·00	4·25	4·50	4·75		
7	1·3	1·3	2·1	2·1	2·25	2·50	3·00	3·75	4·00	4·25	4·50	4·75	5·00	3·25	4·00	4·25	4·50	4·75	5·00	5·25	
8	1·3	1·3	2·1	2·5	2·50	2·50	3·25	3·75	4·25	4·50	4·75	5·00	5·25	3·50	4·00	4·50	4·75	5·00	5·25	5·50	
9	1·3	1·7	2·5	2·5	2·50	2·75	3·50	4·00	4·50	4·75	5·00	5·25	5·50	3·50	4·25	4·75	5·00	5·25	5·50	5·75	
10	1·7	1·7	2·5	2·5	2·75	2·75	3·50	4·25	4·50	5·00	5·25	5·50	5·75	3·75	4·50	5·00	5·25	5·50	5·75	6·00	
11	1·7	1·7	2·5	3·0	2·75	3·00	3·75	4·25	4·75	5·00	5·50	5·75	6·00	3.75	4·50	5·00	5·50	5·75	6·00	6·25	
12	1·7	1·7	3·0	3·0	3·00	3·00	3·75	4·50	4·75	5·25	5·50	6·00	6·25	4·00	4·75	5·25	5·50	6·00	6·25	6·50	
13	1·7	1·7	3·0	3·0	3·00	3·25	4·00	4·75	5·00	5·50	5·75	6·00	6·25	4·00	5·00	5·50	5·75	6·00	6·50	6·75	
14	1·7	1·7	3·0	3·0	3·25	3·25	4·00	4·75	5·25	5·50	6·00	6·25	6·50	4·25	5·00	5·50	6·00	6·25	6·50	7·00	
15	1·7	2·1	3·0	3·4	3·25	3·50	4·25	4·75	5·50	5·75	6·25	6·50	6·75	4·50	5·25	5·75	6·25	6·50	6·75	7·25	

*NOTE :—(1) The pier will extend to 0 5′ below floor level and will be stepped 0·8′ in this height.
(2) The earth and metalling over crown of arch should be in all at least 1·5′

PART VI

GENERAL SECTION

CHAPTER III

Technique of Hydrodynamic Sub-Soil Pressure Observations

1. Introduction.

Preliminary observations were started at Lahore in the month of March, 1939 to develop an apparatus capable of recording manomatric pressure difference between two points in the ground below the effective full saturation line in the soil crust. These differences are caused by the upward and downward movements of water-table. Where water-table is receding, there is a downward flow causing corresponding pressure differences. There is a drop in the manometric pressure from the upper point to the lower. The pressure defference in this case is considered positive. A fall in the spring level in this case is also considered positive. In this case of rising spring levels with pressure drop upward, the pressure difference, and the change in the levels is considered negative.

Detailed investigation of the pressure gradients associated with the movement of water in the water-table have not, so far as is known, been carried out elsewhere. The method used in this case is believed to be original.

2. Apparatus and its Working.

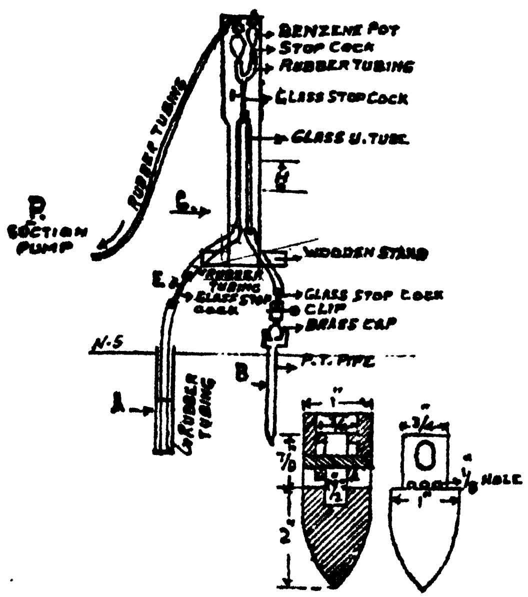


Fig. 1.

The apparatus used in these observations was locally made and is called the "Benzine Differential Manometer." Detail of the apparatus and its layout is shown in Fig. 1. It is essentially an inverted U-tube. One of its limbs is connected to a B.S. Pipe, which records the basic sub-soil pressure level or a pressure tapping point and the other limb is connected to another pressure tapping pipe (P. T. Pipe) which records pressure of the sub-soil water at a point higher than the base of the B. S. Pipe. Water is raised by using a suction pump when P. T. is closed by means of a cock A shown in Fig. 1. Both limbs are filled with water, all air being driven out. Cock A is then opened. Water levels in both limbs adjust themselves to different levels with pressure difference H as shown in Fig. 1. The difference between the water levels in each limb gives the P.D, that is, the pressure difference When this difference is small and a high degree of accuracy in the observation is require. Benzine is introduced and air is driven out from the space above the water levels in both limbs. The pressure difference is hereby magnified by about 9·0 times.

3. Design of Pressure Tapping Point.

The design of the pressure tapping point was adopted as shown in Fig. 1. The pressure tapping pipe is one inch dia. galvanised iron pipe. It has got a pointed shoe The shoe is slung from the pipe, and its top hollow portion fits the inside of the pipe. There is a play of ¼ inch between the bottom of the pipe and the top of the shoe There are 6 holes of 1/8" diameter. These holes are covered by the pipes when it is driven by the hammering to the required position. The pipe is made in pieces of 3 0 ft. length. The pieces are joined by means of 3 inches long screw sockets placed inside the pipe. The outside surface of the pipe is thus throughout plain. The iron cap is screwed to the pipe so that the pipe is not damaged while hammering The P T. pipe can be sunk without any soil getting into the pipe. After the pipe is driven it is filled with water and then a conical metallic cap is put on the pipe as shown in Fig. 2. Then a rubber band is put on making an airtight joint between the cap and the pipe. A piece of pressure tubing with a glass cock is then put to the cap. Water is than put in the cap and all air is driven out up to the cock level. Cock A is then closed. The pipe is then lifted by ¼ inch, so that the holes in the shoe are opened, while cock A remains closed. Water flows out of the holes and makes contact with the soil along the dotted lines shown in Fig. 2. The contact surface is circular ring of 1 inch diameter and of ¼" height. It is not a small hole, which could form a film and cause the trouble that can arise with a porous pot. The set up of the pressure tapping (P.T.) pipe is now complete without an addition of a single drop of water to the soil. Cock A remains closed. Water is put in at atmospheric pressure and it contains dissolved air at atmospheric pressure. After the cock A is closed, water is held suspended under negative pressure The dissolved air comes out and collects below the cock A.

After the manometer is set up as in Fig. 1. (say with water level the same in both limbs) with one limb connected to B.S.P. and the other connected to cock A above the P. T. pipe, the cock A is then opened. The dissolved air moves up into the manometer U-tube and becomes part of the air above the water columns in both limbs. There is no further supply of air from P.T. pipe when its base is below the effective saturation line in the soil crust. Both limbs are then free to adjust themeselves to the actual pressures. The difference between the readings of both columns gives the manometric pressure difference which is negative when the water-table is rising and positive when the water-table is falling.

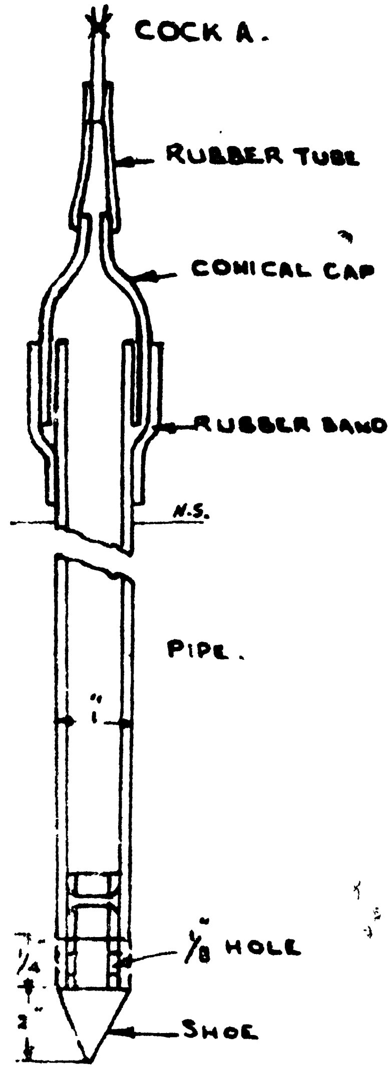


Fig. 2.

4. Theory of Benzine Diffrential Manometer.

(*a*) The inverted U-tube used in this apparatus is sketched in Fig. 3. Water and Benzine are separately marked. The right limb of the U-tube is connected to the pressure tapping pipe and the left to the sub-soil pressure pipe. The bottom of the P.T. pipe is higher than that of the B.S.P. pipe. The sketch represents conditions with a falling water-table. Water is sucked and with Benzine introduced, takes up the position as shown therein.

(*b*) Pressure in both limbs at plane A is the same$=p_o$ say

let ρ & σ be the specific gravities of water and Benzine respectively.

Pressure in the right limb at plane B.

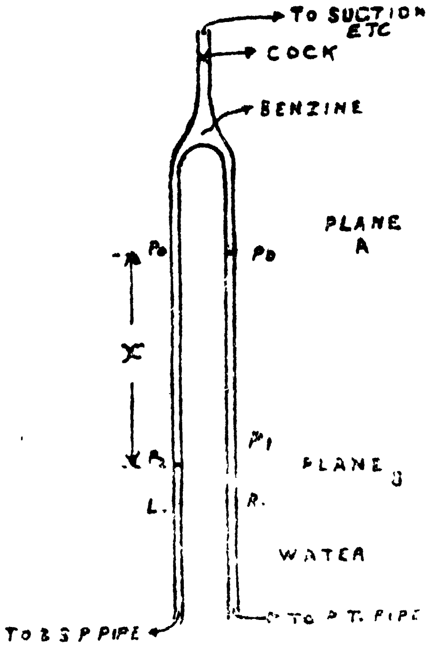


Fig. 3.

$$p_1 = p_0 + g\rho x$$

Pressure in the left limb at plane B

$$p_2 = p_0 + g\sigma x$$

Let H be the equivalent head of water

$$p_1 - p_2 = g\rho H$$

$$g\rho H = p_0 + g\rho x - p_0 - gx\sigma = g(\rho - \sigma)x$$

$$H = \frac{\rho - \sigma}{\rho} x = k.x \qquad \text{(A)}$$

(c) Equivalent water head with Benzine corrected for temperature.

Let observation be taken at t°C

The formula (A) will be written as below :—

$$H_t = \frac{\rho_t - \sigma_t}{\rho_t} x_t; \; H_t = k_t . x_t$$

$$H_t . \rho_t = p_t = (\rho_t - \sigma_t). x_t \qquad (1)$$

Mass is constant and the sectional area of the tube is constant.

$$H_t . \rho_t = H_{4°C} . \rho_{4°C} = H_0 \rho_0 \qquad (2)$$

Let equivalent water head be always reduced to a column of water at 4°C

The specific gravity of water at 4°C $= \rho_{4°C} = 1{\cdot}0$

From equation (1) & (2)

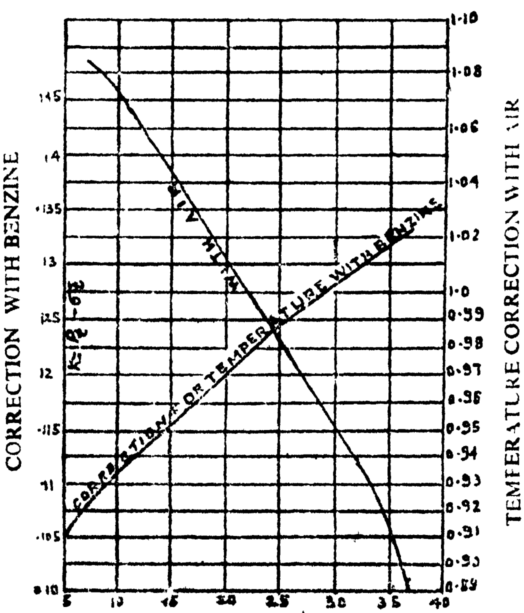
$$H_t . \rho_t = (\rho_t - \sigma_t). x_t = H_{4°C} \rho_{4°C}$$

$$H_{4°C} = \frac{(\rho_t - \sigma_t)}{\rho_{4°C}} . x_t$$

From an observation at t°C., the equivalent water head at standard temperature 4°C.

$$H = (\rho_t - \sigma_t) x_t, \qquad \text{(B)}$$

$$H = k_t . x_t \qquad (3)$$



Absissa is :—
TEMPERATURE °C

Fig. 4.

Table showing variation of k with temperature.

t			k	k Correct to 3 decimals
5° C	1·00000	0 89508	0 10492	0 105
10° C	0 99994	0 83940	0 11054	0 111
15° C	0 99988	0 88390	0·11498	0·115
20° C	0·9983	0·87843	0 11980	0·12
25° C	0·99695	0 87300	0 12395	0 124
30° C	0·99567	0·86760	0 12807	0 128
35° C	0 99393	0·86241	0·13153	0·132

Correction factor k_t is plotted in Fig. 4 and the curve is drawn giving its variation with temperature.

(*d*) Equivalent water head in a water manometer.

From equation (2) in the above paragraph.

$$H_t . \rho_t = H_0 . \rho_0 = H_{4^\circ c} . \rho_{4^\circ c}$$

$$H \text{ at } 4°C = \frac{\rho_t}{\rho_{4^\circ c}} H_t$$

For water manometer, a separate curve in given in Fig. 4 to reduce results to the standard water temperature of 4°C.

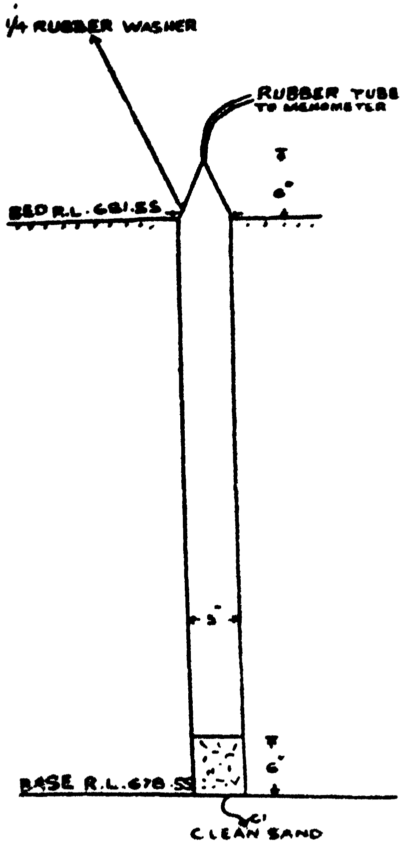


Fig. 5

5. Temperature Errors because of Non-Responsive P. T. pipe and Expansion and Contraction P. T. Limb Liquids.

The calculations in paragraph 4 are based on the assumption that both limbs are connected to responsive pipes The water levels adjust themselves to the correct pressure difference. In practice, it is not the case. The pipe connected to the B. S. P. limb is thrown in water in the B. S. P. pipe and is, therefore, infinitely responsive. P. T. pipe arranges water through small holes at its base and the supply is limited by the transmission constant of the soil.

Let B.S.P. limb be higher than the P.T. limb in the U-tube. Increase in temperature means an expansion of the enclosed volume of air or benzine above the water menisci. This means increase of pressure above the menisci B. S. P. column will be pressed more and P. T. column very little. The increase of temperature will result in the reduction of P. D. The P. D. difference would increase at night and reduce during day time.

Let P. T. limb be higher than the B. S. P. The increase of temperature would mean pressing down of B. S P. limb more than the other. Increase of temperature would mean increase in P. D. In this case, there will be increase during day time and reduction at night.

Moreover, expansion of P. T. limb connection will mean reduction in its reading and *vice versa*. A mathematical treatment of all these changes is impossible. A practical solution is to improve the responsiveness of the P. T. limb by increasing the sectional area at its base.

It has been shown that the effect of the variation of temperature from the average during day and night is in the opposite direction. If one tends to increase the other tends to decrease the P. D. For all practical purposes variation of temperature from its average value may be considered uniform. It is, therefore, concluded that for all practical purposes, the average value for the hourly observations of P. D. in 24 hours is the correct pressure difference at the average temperature during day and night.

6. Design of P. T. Pipe in Stiff Soil Crust.

It was decided that in the case of alkaline clay, the soil at the base of the P. T. pipe should not be in direct contact with the water in it to avoid dispersion or flocculation. The design of the P. T. pipe as sketched in Fig. 5 was adopted. The new P. T. pipe is a 3 inches dia. brass pipe 1/24" thick with a sharp cutting edge at its base. This needs no hammering with a monkey to drive it. It can be pushed in wet soil with a little force. There is thus no appreciable disturbance of the soil strata. The earth from the inside was taken out by means of a scoop. There is no difficulty in removing it. The coarse and clean sand is put in it as shown in Fig. 5. Sand serves as a screen between the clay at its base and the watar in the P T. limb. A special conical cap is provided which is especially designed to drive out air efficiently. The cap is covered with an asbestos sheeting so that the water inside is not affected by the outside temperature. The sectional area of this pipe is about 144 times that of the U-tube limb. This arrangement makes the P. T. limb pretty responsive and minimises the trouble of soil choking at the base by flocculation. This design was first used at R. D. 180,000 L C. C. The author was thoroughly satisfied with its working and would recommend to adopt this design is case of clay soils. We are not likely to meet soil conditions worse than those at R. D. 180,000 L. C. C. The author is convinced that this design of P. T. pipe will meet the requirements in all cases.

7. Method of Recording Correct Pressures, Unaffected by Temperature, etc.

It is evident from paragraph 4 & 5 that temperature changes affect the observations in two ways. Firstly, by changes in the specific gravity of the fluid and secondly, by the expansion, and contraction of the air enclosed in the inverted U-tube. The correction in the first case would be applied according to the graphs in Fig. 4 and the correction in the second case is mathematically a complicated affair and in practice it was worked out as shown by comparing daily average P. D. as observed during day time with that of 24 hours of the same day. These corrections are likely to give an impression that the method of observation was very crude and that the results could only be approximate. This is true to some extent but it is claimed that the results obtained are good enough for practical purposes The pressure differences recorded were occasionaly checked by an independent method described below, which gives results unaffected by the temperatre changes

Referring to Fig. 3 it will be seen that when cock A is closed, the process of pumping or adjustment of columns in the U-tube does not affect or bruise the pressure at the base of P. T. pipe. The observation made on 11-4-1939 is plotted in Fig. 6 as an example. The cortect pressure difference at the time of the observation was 0·108 feet. In the first case, the manometer was set with P. D.+0·863 ft. (P. T. higher than B. S. P.) before the cock A was opened. Naturally the pressure difference would reduce with time. The observations were recorded every five minutes and they are plotted as curve A in Fig. 6. After a few observations cock A was closed. The manometer was set up again in such a way that the P. D. was—0·604 ft. (B. S. P higher than P. T.) The cock A was then opened. The pressure difference reduced with time and the observations are plotted in curve B Fig. 6. Thus we get two curves, one with higher values and the other with the lower values. Both the curves tend to be asymptotic

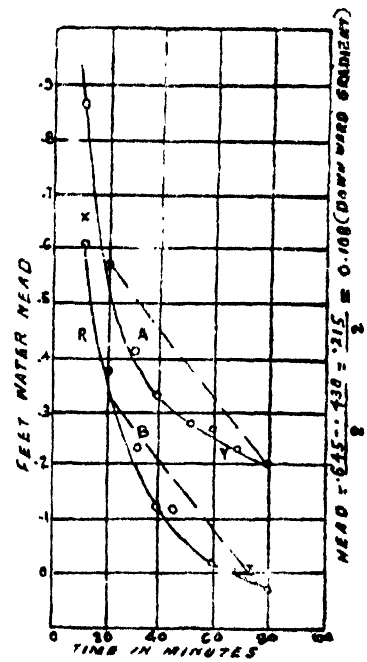


Fig. 6.

to a common line. The distance of each from this common asymptotic line is the correct pressure difference. It could be mathematically calculated by obtaining the equations of these two curves. However, a graphical method is shown in Fig. 6. Draw a line by joining two points on curve A with vertical distance xy. Then draw a line RT parallel to it on curve B so that the vertical distance between these two lines RT and xy is double of the correct pressure differece. The calculations are shown in Fig. 6. This method has been called the method of recording the correct pressure difference odservations by the process of *asomptotic ranging.* This was used as a check for the hourly observations.

8. Diaphragm Manometer.

Diagram of Crump Type Diaphragm Manometer

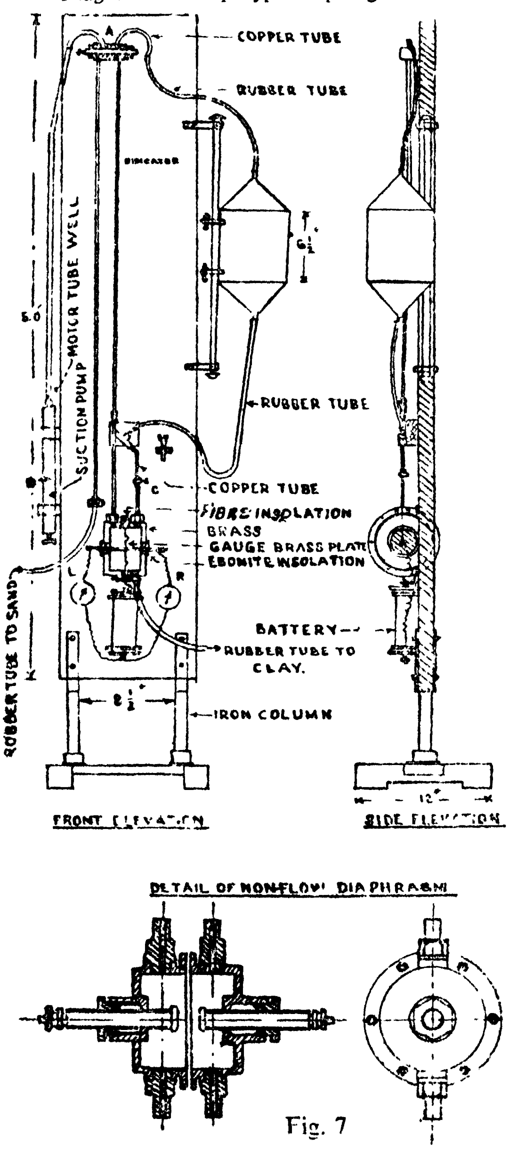


Fig. 7

Benzine differential manometer requires that the B. S. P. pipe and P. T. pipe should be responsive to yield water for the corresponding water level changes in the inverted U-tube. In a stiff soil crust a special P. T. pipe as described in paragraph 6 above was used to make it responsive. However, an instrument called Diaphragm Manometer was devised by Crump and manufactured at the college workshops at Rasul The principle is the same as that of Chattock micromanometer or a Turtle Gauge ensuring no liquid movement while observing pressure. The design is shown in Fig. 7. The diaphragm was of Rubber with metallic strip and bob in the centre. If the flow in the beginning was in right chamber, electric cantact was made on the left side as shown by the volumeter readings and if the liquid flow was on the left from B. S. P., the electric contact was on the right side. The pressure readings were taken when the diaphragm was in the neutral position showing that the soil was neither receiving water nor yielding any water. This was actually used for observations at R D. 150,000 L. C. C. and 14-R of U. J. C. experiments by the author. Its use neads skill and patience. The diaphragm of 6 inches was too big and made the instrument some-what insensitive in readings. The diaphragm design could be improved upon.

9. Use of Chattock Micro-Manometer.

Soil pressure observations below the saturation line were carried out in the office compound at Lahore and R. D. 180,000 and 150 000 L. C. C. using Benzine Differential Manometer. It was found in these observations that the slow responsiveness of the pressure Tapping (P. T.) pipe caused errors in the observations due to temperature and time lag. It was therefore decided to use a Chattock Micro-manometer for pressure observations at the Chichokimallian site. This is rather a complicated instrument but it has got a very important advantage over

over other manometers, in that it requires no fluid movement from the prassure tapping pipe. Correct pressures are communicated to a liquid-sensitive diaphragm The detailed description of the instrument is not necessary and can be read in any standard text book on physics Pressures recorded were of the order that they could be accurately recorded by and water or mercury manometer; but by using this instrument we got over the difficulties arising out of slow responsiveness of P. T. pipe in clay soils The set up is shown in photograph, A for Chattock and B for Benzine Differential Manotmeter Fig. 8.

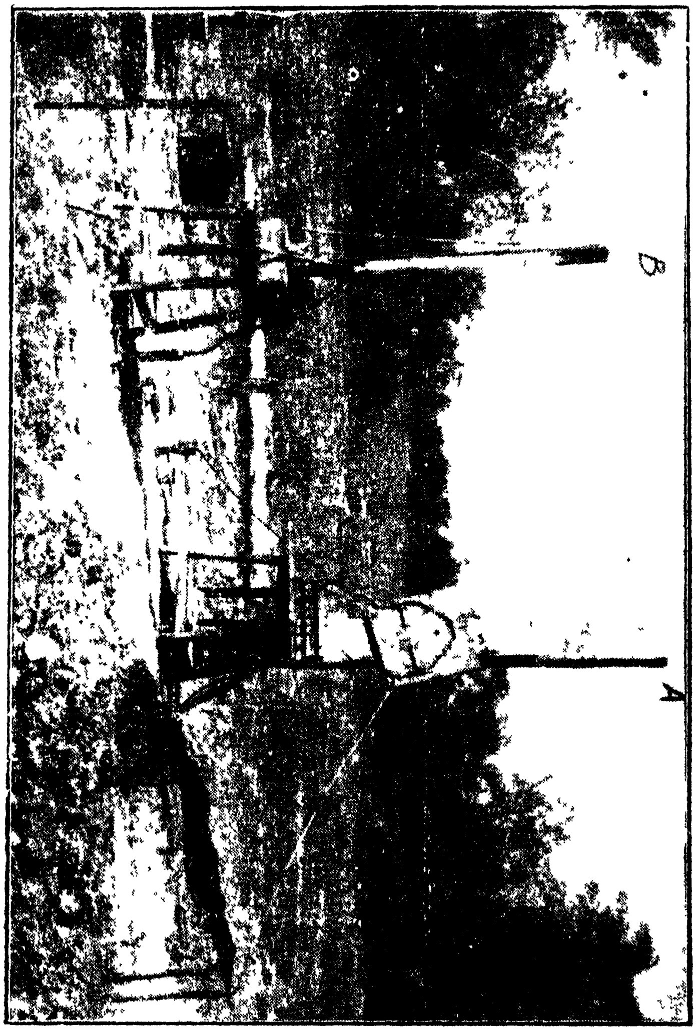


Fig. 8

10. Uses of Hydrodynamic Pressure Observations.

1. To measure the height of the capillary fringe.
2. To measure Transmission constants or P. I. C. (percolation intensity coefficient) of the soil crust as it exists at site.
3. **To measure soil evaporation.**

4. To measure the effect of rainfall or irrigation.
5. To know quantitatively the effect of all sources and sinks on the water-table.
6. To estimate the ground water flow down the *doab*.
7. To measure transpiration.

The method of calculating these factors from the hydrodynamic pressure observations is indicated in the examples below which have been taken from actual observation recorded in Author's Technical Paper No. 33.

11. Height of Capillary Fringe.

The pressure tapping pipes progressively are sunk from the ground surface say half a foot every time, and the pressure reading is taken at every point by easing it so that the holes in the shoe communicate to the soil water. Above the capillary meniscus surface of the capillary,

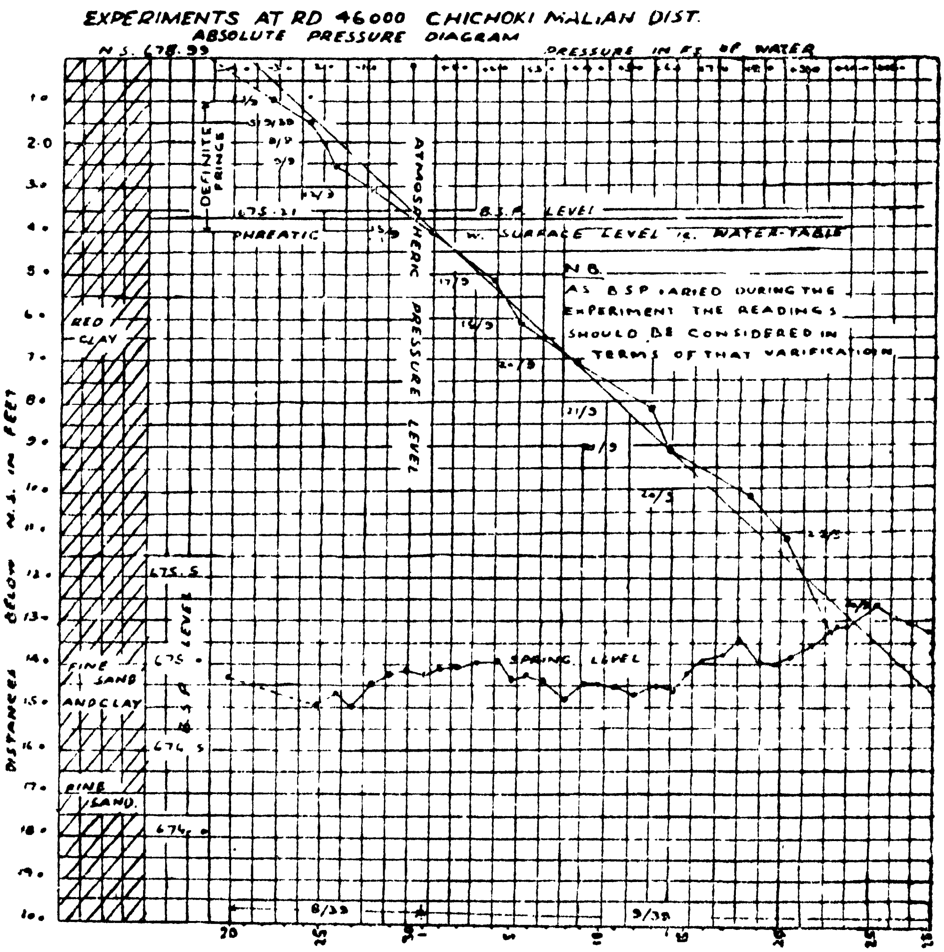


Fig. 9.

free air comes out and pressure reading indicates atmospheric pressure. Below the capillary meniscus no air comes out and the manometric pressure recorded is negative according to the height of the front above the phreatic surface of the ground water. The reading is influenced by the effect of the soil evaporation as already explained in chapter II, Part V. The negative pressure reading recording pressures below the atmospheric can then be taken down to the phreatic surface at every half foot interval and no air comes out. The test of the capillary fringe is that the soil pores are affectively saturated and manometric pressure can be recorded and its negative value equal to the height of the point above the phreatic surface in the static conditions of the water-table Fig. 9, gives the observation of the Chichokimallian experimental site.

12. Measurement of Darcy's 'k' or Author's σ (P.I.C.)

(A) Let the water-table be rising.

Let two points P_1 and P_2 be selected about 10 to 12 feet apart below B. S. P. level. Let two pressure tapping pipes be put with their bottom at P_1 and P_2. Connect them to Benzine Differential Manometer or Chattok set up. P_2 records a pressure higher than P_1. Let it be say 1·0 ft. This is considered negative according to notation used in this book. There is a pressure loss flow from P_2 to P_1 equal to reading (P D.) say 1·0 ft. The measure of the upward flow is the rate of the change of B S.P. level and upward flow caused by soil evaporation. Let B.S.P. level be 7·0 ft. below N S. (Ground Surface). Let daily observation of Pitche's evaprometer be taken and let it be taken for 10 days. Let average daily surface evaporation from these observations be 0·16 inch per day. The effective soil evaporation with B S.P. level 10 ft. below N.S. from Plate XX, is 42%. The daily effect of soil, evaporation $=0\cdot16\times42/100=0\cdot067$ inch per day $=\cdot0056$ ft. per day. If soil porosity is 40% this is equivalent to a height $=\cdot0056\times100/40=\cdot014$ ft. per day in the soil. Similarly we will have B.S.P. level observations for 10 days. Let the rise be ·15 ft. The daily rise will be $0\cdot15/10=\cdot015$ ft. per day. The net daily upward flow is $=\cdot015+\cdot0146=\cdot029$ ft. Let σ be the percolation intensity coefficient of the crust between point P_1 and P_2. Considering unit area through which flow is taking place Porosity 40%.

$$\therefore\ q=\sigma H=\cdot029\times1\times40/100=0\cdot0116 \text{ ft. per day.}$$

As observed P.D. (H) = 1·0 ft.

$$\sigma=q/H=0\cdot0116 \text{ ft per day}$$

$$=\frac{0\cdot0116\times10^6}{60\times60\times24}=0\cdot134 \text{ cusecs per } 10^6 \text{ sft.}$$

Let average temperature of water in B.S.P. pipe during 10 days be=25·2°C.

The correction to reduce the result to 20°C = −10% (Plate XXI A).

$\sigma=0\cdot134\times90/100=0\cdot121$ cusecs at 20°C per million sft. per ft. head.

Using Darcy's law per unit area $q=KS$

$$S=\text{slope}=\text{P. D./Length}=1/10 \qquad \text{Porosity}=40\%$$

$$K=\frac{\cdot029\times0\cdot4}{24\times60\times60}\times10/1=1\cdot34\times10^{-6}$$

(B) In the office compound of the Water-logging Investigation Division with water-table 21 ft. below N.S. (below soil crust) observations were taken for a long period of about 4 months between two points 15·0 ft. apart below the B.S.P. level. The actual effect of soil evaporation was not known. The following method was used to calculate transmission constant and soil evaporation.

Results for 14 periods giving average daily B.S.P. change and the P. D. are plotted in Fig. 10. Numbers against the point indicate the respective periods. Point No 14 is rather badly out. It relates to the month of November, 1939 when temperatures were low and the evaporation was the least. There are three negative and 11 positive P. D. periods. Two lines are drawn between them. A firm line is drawn passing through the positive paints so that their scatter is even about this line. Similarly a dotted line is drawn passing through the negative points These lines clearly show that their is some common factor which tends to increase the negative pressure difference and *vice versa* tends to decrease the positive pressure difference. This factor is the evaporation from the soil which results in an upward gradient from the water-table and consequently affects the pressure differences as stated above.

A line bisecting the angle between firm and dotted lines in Plate VI represents the correct relation between P. D. and the B. S. P. change unaffected by evaporation. This line is drawn chain dotted. The transmission constant of the soil between the P T. Base and the top of the B S. P. Pipe strainer is worked out below :—

Distance between P. T. Points Level—15 ft.
B. S P. Daily change from Plate VI = 0·01 ft.
P. D. from Fig. 10 (chain dotted line) = 0 103 ft.
Soil Porosity = 40%

$$K = \frac{0{\cdot}01}{24 \times 60 \times 60} \times \frac{15{\cdot}0}{0{\cdot}103} \times 0{\cdot}4 = 0{\cdot}67 \times 10^{-5}.$$

This is just the same value as found for the adjoining site near Central Jail.

Using this transmission constant, the evaporation may now be calculated as below :—

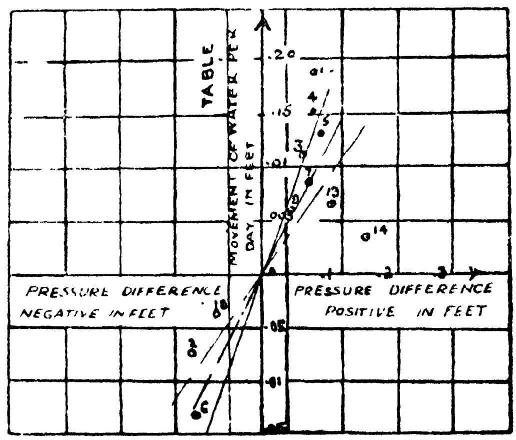
Suppose B. S. P. dropping = 0·01 ft. daily. Evaporation pressure difference from Fig. 10 = 0·136—0·103 = 0·034 ft./day.
$K = 0{\cdot}67 \times 10^{-5}$; when $v = KS$.
Evaporation in feet per day.

$$= 0{\cdot}67 \times 10^{-5} \times \frac{0{\cdot}034}{40} \times 24 \times 60 \times 60 = 0{\cdot}00165 \text{ ft.}$$

per day free water.

$$= 0{\cdot}00165 \times \frac{10}{4} = 0{\cdot}00041 \text{ ft. per day in soil.}$$

HYDRODYNAMIC SUB-SOIL PRESSURE
OBSERVATION AT LAHORE
Pressure diff rence with movements of water-table



No.	TEMP:	HMY.	No.	TEMP	HMY.
1	36·20	·489	8	35·20	·721
2	33·30	·711	9	35·00	·706
3	34·60	·700	10	32.50	·764
4	36·24	·522	11	34·33	·642
5	37·40	·442	12	32.50	·696
6	34·75	·663	13	28·55	·684
7	35·98	·612	14	26·32	·600

Fig. 10

This means that 40% of the drop in water-table at this site was due to the soil evaporation alone.

There is no record of pan evaporation at this site. Let us suppose that it is 1/4" in a day when the water-table is dropping 0·01 ft. per day. Ratio of soil evaporation relative to pan

$$\text{evaporation} = \frac{0{\cdot}00165}{0{\cdot}0202} = 7{\cdot}7\%.$$

This means that at the site of these experiments the soil evaporation was about 1/13th of the pan evaporation.

14. Soil Evaporation.

One method is shown in paragraph 13 B above. A quicker method was used in waterlogged areas to measure soil evaporation. The change of B S. P per day is worked out for a period and the corresponding P. D. between P_1 and P_2 is got from the manometric pressure difference readings.

Then a pit, 10 ft. diameter was dug about 2·0 ft. below B. S. P level. From the rate of rise of water level in the pit plus the pan evaporation gives upword flow and the pressure difference between P_1 and P_2 gives the corresponding P. D. The transmission constant can thus be calculated and the same can be used to calculate the soil evaporation from the observations before the pit is dug.

The methods of tabulation of observations and calculations of soil evaporation are given in tables No. 1 to 4 as actually done for the Shiekhupura experimental site.

NOTE :—It was also tried to measure the transmission constants in a pit by putting water from automatic chicken feed arrangements keeping a constant level with about 1·0 ft. head above B. S. P. level The bed tended to choke itself by dispersion of the soil and flocculation in course of time. But no choking took place in upward flow in half a dozen cases in the soil crust of the Punjab.

15. Rainfall or Irrigation Flow into the Water Table.

When water-table is high as in water-logged areas, the capillary meniscus surface is within a foot or two of the ground surface. If irrigation water or main water is on the ground surface it percolates down to the water-table. Let pressure difference between any two points P_1 and P_2 be observed and the corresponding rate of sinkage of water surface plus surface evaporation will give the transmission constant of the soil between the points P_1 and P_2 which can then be used to calculate the water added to the water-table for the time by the rain or irrigation water.

16. Effect of Sources (Canals) and Sinks (Drains).

This has been discussed in detail for measurements of seepage flow from various canals Chapter III, Part V

The pressure difference between the B. S. P. level and the water surface in a canal is the infilteration head in the case of sources such as canals and the pressure difference between the B. S. P. and the water surface in the drain is the seepage head causing inflow.

17. Ground Water Flow Down the *Doab*.

The percolation intensity coefficient can be determined by the methods described before for the determination of B. S. P. depth below the B S. P. level in a soil crust. Let σ_1 and σ_2 be the percolation intensities of the sites A & B separated three miles apart. The drop in B. S. P. levels = 5·0 ft. (*Continued on page 763*)

TABLE 1.

Progressive lowering of P. T. Pipe.

N. S level – 99·94 B. S. P. = 94·8 Top of datum pipe = 100 64

Date.	Time.	R_o Zero reading	R reading after	Diff. R_o and R	P. D. equivalent W. Head H	Datum pipe reading	W.L. in Datum pipe	h.	A.P = h – H
1	2	3	4	5	6	7	8	9	10
				Base of P T. 1·0′ below N. S.					
8-2-40				Free air coming atmospheric Pressure.					
				Base of P.T. 2 0′ below N.S.					
9-2-40				Free air-atmospheric Pressure.					
				Base of P.T. 3·0′ below N.S 96 94					
11-2-40	1 p.m.	6 20	12·48	6 28	+·163	6·10	94 54	-2·40	-2·568
				Base of P T. 4·0′ below N S 95 94					
18-2-40	12 a.m.	4 09	3·65	-·44	-·012	6·31	94·33	-1·61	-1·598
				Base of P.T. 5·0′ below N S. 94·94					
19-2-40	10 p.m.	10·90	1·83	9·07	-·243	6·42	94 32	-·720	-·477
				Base of P T. 6 0′ below N S. 93·94					
19-2-40	5 p.m.	13 25	1·88	-11·38	-·304	6 41	94·23	+·29	+·594
				Base of P.T pipe 7 ft. below N.S. 92·44					
21-2-4	10-15 a.m	12·56	-1 90	-14·46	-·387	4·80	92·84	1·90	2·287
				Base of P.T. 8 0′ below N.S. 91·94					
21-2-40	5 p.m.	– ·49	-7·70	-7·21	-·193	5 44	95 20	3·26	3·453
				Base of P.T. 9 0′ below N S. 90 94					
22-2-40	12 a m.	1·02	-7·23	-8·25	-·221	5 10	95·54	4·60	4·821
				Base of P.T. 10·0′ below N.S. 89 94					
22-2-40	5 p.m.	1·33	-6·49	-·782	-·269	5·0	95 64	5·70	5.969
				Base of P.T. 11 0′ below N.S. 88·94					
23-2-40	11 a.m.	1 49	-·743	-8·92	-·239	4 88	95·76	6 82	7 059
				Base of P.T. 12·0′ below N.S. 87·94					
23-2-40	5 p.m.	1·61	-8·22	-9·83	-·263	4·77	95·87	7·93	8·193

Note ;—Similar observations for another site plotted in Fig. 9 and show the height of api lary meniscus surface above the B. S. P. which is 2·14 ft. in this case and the departures from the static pressure line.

TABLE II.

Gradients of Inflow into the Pit.

Date	Average B. S. P. level.	W. L. in pit at 10 a. m.	Total Head.	Distance between bed and bottom of crust.	S_1	P. D. between bottom of crust and B. S. P.	P. D. between P. T. Base and bottom of crust.	Distance between base of P. T. and bottom of crust.	S_2.	Distance between bed and base of P. T.	P. D. between bed and base of P. T.	S_3.	$\frac{\text{Column 12.}}{\text{Column 8.}}$
1	2	3	4	5	6	7	8	9	10	11	12	13	14
18th March, 1940	94 572	93·96	0 612	8 0	0·0765	— 06	—·375	6 0	·0625	2 0	—·177	·0885	·472
19th March, 1940	94 578	93·984	0·594	8 0	0 0742	—·029	—·377	6 0	·0628	2 0	—·178	·0890	·472
20th March, 1940	94·576	94 026	0·55	8·0	0 0687	—·045	— 342	6·0	·0570	2 0	—·163	·0815	·477
21st March, 1940	94·559	94·062	0·497	8 0	0 0621	—·037	— 374	6·0	·0625	2 0	— 160	·0800	·426
22nd March, 1940	94·595	94·102	0·493	8 0	0 0616	—·025	—·309	6 0	0516	2 0	— 149	·0745	·482
23rd March, 1940	94 606	94·133	0·473	8·0	0 0591	— 053	—·282	6·0	0470	2 0	—·135	·0675	·478

$$\sigma_1 = \cdot092 \text{ and } \sigma_2 = \cdot086$$

$$\text{average } \sigma = \frac{\cdot092+\cdot086}{2} = \cdot088 \text{ million sft. head at } 20°C$$

Flow in cusecs $= \sigma$ H $= \cdot088 \times 5 = \cdot440$ cusecs per foot width.

This is the flow for the B.S P. depth of say 20 ft. in the soil crust per foot width of the *doab*. The value of Chichokimallian and the Sheikhupura sites have been taken. If similar observations were available every mile apart, the difference of the flow between the consecutive miles shall give the additions and the subtractions as the case may be.

18. Transpiration.

The measurements of transpiration by the crops could be made according to the method indicated in paragraphs 14 and 15 but the author had no opportunity to use the method for this purpose. The author considers this method relatively more accurate and reliable to measure transpiration.

TABLE III.
Sheikhupura Experimental Site.
DAILY INFLOW INTO THE PIT.

Date.	Daily average B. S. P.	Change in B S. P.	Daily P. D. average between B. S. P. and P. T.	Daily rise of W. L. in pit at 10 a. m.	Evaporation in 24 hours in pan.	Total of E+rise in pit.	Column 7/Column 4	REMARKS
1	2	3	4	5	6	7	8	9
18th March, 1940	94·572	—·006	·375		·0282			
19th March, 1940	94·578	—·006	·377	+·030	·018	·049	·130	
20th March, 1940	94·576	+·002	·342	+·030	—·197	·0497	·145	
21st March, 1940	94·559	+·017	·374	+·028	·0186	·0466	124	
22nd March, 1940	94·595	—·036	·309	+·025	·0203	·0453	·146	
23rd March, 1940	94·606	—·011	·282	—·021	·0203	·0413	·147	
					Average ...	·0464	·1384	

1. K_1 from S_2 (between P. T. Base+bottom of crust)
   average $S_2 = \cdot0572$
   average daily inflow $= \cdot0464$
   V=K.S. q=AVP.
   $$K_1 = \frac{q}{ASP} = \frac{\cdot0464 \times 1}{1 \times \cdot0572 \times \cdot4 \times 86400} = 2\cdot3 \times 10^{-5}$$
2. K_2 from S_2(between pit bed+P. T. base)
   average $S_2 = \cdot0801$
   $$K_2 = \frac{\cdot0464 \times 1}{1 \times \cdot0801 \times \cdot4 \times 86400} = 1-1\ 6 \times 10^{-5}$$
3. Average K between bed of pit and bottom of crust
   $K = 2 \times 10^{-5}$

TABLE IV.

Effect of evaporation with successive lowering of the pit Sheikhupura Site.

Date.	Average daily B. S. P.	Change in B. S. P.	Resultant daily P. D. P. T. base to bottom crust.	Resultant P D. between B S P. level to bottom of crust.	Calculated P D. for water table movement.	Pressure difference due to evaporation.	P. D. evaporation average of the period.	Calculated evaporation in pit.	Average observed evaporation Pitches Evaprometer.	Ratio col. 9/col. 10	Soil evaporation / pan evaporation. %
1	2	3	4	5	6	7	8	9	10	11	12
No. pit N. S. level =99·94 bottom of soil crust 14 0′ below :—											
24th February, 1940...	94·920	0 053	0·1804	0·203	0·245	—0 042					
25th February, 1940 ..	94·924	0·004	0 0902	0·1015	0·083	—0 083					
26th February, 1940...	94·941	—0·017	0·1324	—0 149	—0·078	—0 071					
27th February, 1940...	94·963	0 022	0·1480	—0·167	—0·101	—0·065	—0·065	0·00562	0 0225		24%
Depth of pit 1·0′ below N. S 10′ dia pit.											
2nd March, 1940 ...	94·853	—0·024	0 153	—0 170	—0 111	—0·059					
3rd March, 1940 ...	94·853	—0 002	0 117	—0·130	—0·009	—0 121					
4th March, 1940 ...	94 877	—0 002	0 168	—0·187	—0 001	—0 086	—0·088	0·00763	0 0203		37%
Depth of pit 2·0′ below N. S. 10′ dia pit.											
7th March, 1940 ...	94·783	0·028	—0 081	—0·090	0 120	—0 219					
8th March, 1940 ...	94·745	0 039	—0 071	0 098	0·175	—0 099					
9th March, 1940 ...	94 644	0 101	0 223	0·243	0·467	—0 224	—0·179	0 0135	0·0219		70%
Depth of pit 3 0′ below N. S. 10′ dia. pit.											
10th March, 1940 ...	94·703	—0 059	0·0119	0 013	—0 273	—0·260					
11th March, 1940 ...	94 654	—0·049	0 0115	—0 025	0·226	—0 201					
12th March, 1940 ...	94 873	—0 019	0·157	—0·171	—8 283	—0 259	—0·240	0·0209	0·0208		100·7%
Depth of pit 4 0′ below N. S. 10′ dia pit.											
13th March, 1940 ...	94·635	0 038	—0·079	—0·086	0 175	—0 261					
14th March, 1940 ...	94 643	—0 008	0 198	—0·215	—0·037	—0·178	—0·219	0 0189	0·0157		120·4%
Depth of pit 5·0′ below N. S. 10·0′ dia pit.											
15th March, 1940 ...	94·629	0 014	—0·265	—0 288	0·064	—0·352					
16th March, 1940 ...	94·650	—0 021	—0 364	—0·097	—0 290	—0·299	0·325	0·281	0·201		140·7%

NOTE 1. Conversion factor column (3 to 6)=4·628.

$$F_1 = \frac{b}{x} = \frac{8}{86400+\cdot 00002} = 4\,628$$

NOTE 2. Conversion factor column (8 to 9)=0·0864.

$$F_2 = \frac{b}{4\,628} \times 2/5 = 0\cdot 0864.$$

PART VI
GENERAL SECTION
CHAPTER IV
Hydraulic Jump and Water Profiles

The subject of the formation of hydraulic jump and the determination of water profiles, taking friction into account, was dealt with by the author in his publications 9 and 10 in October and November, 1936 in the Indian Engineering Congress, Calcutta. Students should refer to the original publications for the derivation of the formulae. It is intended here only to indicate and illustrate their use for solving actual problems.

2. Notations.

Let v=velocity of flowing water, considred as moving in the direction from left to right.

Q=total discharge of the channel section. This is considred constant for the whole section.

q=volume of water flowing per foot width of the channel. In all the analytical discussions, which follows, it is understood to be a definite constant quantity.

l=length of channel

y=depth of water flowing in the channel at any point.

y_c=depth of critical flow, *i.e.*, $g\,y_c^3=q^2$.

$y_n=\sqrt[3]{\frac{q^2}{C^2S_n}}$=depth of neutral flow. This will be more fully explained later. This is the depth of flow for a given quantity of water at which the slope of the bottom is equal to the slope required to overcome friction. Water at this depth can flow uniformly for any length with its surface a straight line parallel to the bottom of the channel. (It is sometimes called as Normal flow). The expression for it, follows from Chezy's formula.

h=elevation of water surface above any convenient datum.

k=velocity head corresponding to v, or $2gk=v^2$.

R=hydraulic radius, and in the present discussion R=y

S=slope of bottom. This will be arbitrarily considered positive for a downward slope towards the right.

S_n=slope just sufficient to maintain v and y constant or the slope required to overcome friction. This may be called the neutral slope.

$C=\sqrt{\frac{2g}{f}}$. Friction co-efficient in the Chezy's formula $v^2=C^2\,RS$.

d_l=horizontal distance between two consecutive cross sections whose relations are to be studied.

d_y=change in depth between conscutive cross section.

This will arbitrarily be taken as positive when the depth at the second or right hand cross section is less than at the other.

d_h=change in elevation of water surface between two consecutive cross sections. considered as positive, when surface is falling.

d_b=change of elevation of bottom between two consecutive cross sections, considered as positive when slope is downward.

d_k=change of velocity head between two consecutive cross sections. considered as positive when velocity increases.

d_f=head required to overcome friction in the distance d_l considered always positive, flow towards right.

3. General Equation of Flow in an Open Channel.

In the general equation of flow of water in an open channel, let AB in Fig. 1, be the water profile of a stream of depth y at A. In a length d_l, water surface drops to B by the amount d_h. The drop in water surface is due to the drop in depth and the drop in the bed.

$\therefore$ Geometrically $d_h = d_b + d_y$ (Fig. 1.)

Also, according to Bernoulli's theorem, the drop in water surface is used in friction or increasing velocity.

$$\therefore\ d_h = d_f + d_k$$

$$\therefore\ d_b + d_y = d_f + d_k \qquad (1)$$

This is the general differential equation of flow of water in an open channel.

The physical interpretation of this is that the total loss of (potential and pressure) energy for unit length of the channel due to fall in the level of the bed and in the depth of water is equal to the increase of kinetic energy, together with loss in friction per unit length of the channel.

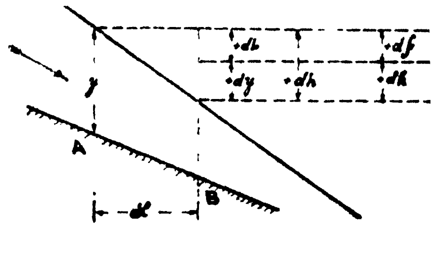


Fig. 1.

4. Critical Conditions of Flow.

When water flows, according to Bernoulli's theorem, there is a particular condition of the flow, which is of special interest and is known as the critical condition of flow. This condition of flow is more simply called critical flow. The depth and velocity are also called critical in this condition of flow. Mathematically it is defined to be the condition of the flow when the change of velocity head is just sufficient to produce the change in elevation of the water surface. Using the notations as given above, critical flow is at the place where d_k and d_y are equal.

d_k = change in velocity head

$$= d\,(v^2/2g)$$

$$= \frac{v}{g}dv \qquad (A)$$

But in a rectangular channel, discharge per unit width $q = yv$

$$\therefore\ v = \frac{q}{y}$$

$$dv = \frac{q}{y^2} \times d_y \qquad (B)$$

Substituting the values of dv and v in (A)

$$d_k = \frac{v}{g} \times \frac{q}{y^2} \times d_y = \frac{q}{y} \times \frac{1}{g} \times \frac{q}{y^2} \times d_y = \frac{q^2}{gy^3}\,d_y$$

In the critical condition of flow $d_k = d_y$

$$\therefore\ y^3 = \frac{q^2}{g}$$

The critical depth.

$$\therefore\ y_c = \sqrt[3]{\frac{q^2}{g}} = \left(\frac{q^2}{g}\right)^{\frac{1}{3}} \qquad (2)$$

Put $q = v_c \,.\, y_c$

$$y_c^3 = \frac{(v_c)^2 \,.\, (y_c)^2}{g},\ \therefore\ v_c^2 = g \times y_c \text{ or } v_c = \sqrt{g \,.\, y_c} \qquad (3)$$

$$\text{or } \frac{v_c}{\sqrt{g \,.\, y_c}} = 1 \text{ (Froude number)} \qquad (3A)$$

velocity head in the condition of critical flow

$$k = \frac{v_c^2}{2g} = \frac{y_c}{2} \qquad (4)$$

Properties of critical condition of flow may be stated as :

(1) There is neither accumulation nor reduction of pressure on account of the peculiar balance of velocity head and depth.

(2) A tangent to the curve of the pressure at this place is parallel to the bed profile. It is therefore, called the section of the parallel flow.

(3) In the critical condition of flow, the velocity head is one-half of the depth.

(4) The depth of water at the control section on a weir crest is critical It is proved later that discharge for the control section is the maximum.

(5) In the critical conditions of flow, the Specific Energy (Energy of flow) is the minimum and is equal to $\frac{3}{2} y_c$.

5. Standing Wave.

Standing wave it defined to be a wave which persists to form at the same place. The hydraulic condition necessary in an open channel for the formation of the standing waves is that the flow should be between the critical and the neutral depth and that the neutral depth is positive and greater than the critical. Flow takes place according to Newton's first Law of Motion, *i.e.*, Bernouli's theorem (Fig. 2.)

$$v_1=\sqrt{g(D_1+h)} \text{ or } v_2=\sqrt{g}\,D_1 \qquad (1)$$

This means that the velocity at the crest of the standing wave is the critical velocity of the depth at the trough and that the velocity at trough is *vice versa*. There is no impact and no energy loss in the formation of standing waves. In this case velocity at the trough of wave is no doubt greater than that at crest, but there is no concussion or impact of water moving with high velocity with that moving slowly ahead of it. It is simply a redistribution of pressures. Pressure on the plane, forming the rear of the wave, is increased and under this pressure the plane moves forward heaving up water ahead of it. When the column of water has been raised to its greatest height at the crest of the wave, it presses equally on both front and rear planes of the wave, accelerating the velocity ahead and retarding that behind. Such waves continue forming till the flow is with the neutral depth.

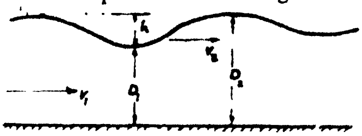


Fig. 2

Standing waves are formed downstream of irrigation falls when there is insufficient depth available downstream to form a hydraulic jump. Waves are often accompanied by splash of water and are mistaken for a hydraulic jump. Recovery of depth in a contracted flume can aptly be called a standing wave.

6. Hydraulic Jump.

Hydraulic jump is defined to be a hydraulic phenomenon when there is a distinct jump of water accompanied by an impact between the rapidly moving stream upstream and the more slowly moving column of water downstream. The hydraulic conditions necessary for the formation of the jump are that the flow upstream of the jump should be with depth below critical condition of flow and that the flow downstream of the jump should be with depth above the critical condition of flow. The neutral depth in this case should be positive. Under these conditions, there is but one hydraulic jump that forms at the same place for the specified discharge and slope of the bed. This phenomenon is quite distinct from the formation of the standing waves in hydraulic works as enunciated before.

(b) Jump on a level floor.

Theory is based on the following assumptions :—

(*i*) Jump takes place instantaneously at a section (*ii*) Stream-lines flow before and after the jump. (*iii*) Friction is neglected in the region of the jump. (*iv*) Parallel sides restraint in the region of the jump. (*v*) Horizontal bed.

Hydraulic jump takes place according to Newton's second Law of Motion, *viz.*, change of pressure is equal to change of momentum which means that pressure plus momentum is constant, Fig. 3.

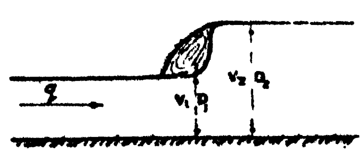


Fig. 3

$$D_1 D_2 (D_1+D_2)=\frac{2}{g} q^2=2D_c^3 \tag{2}$$

$$D_2=-\frac{D_1}{2}+\sqrt{\frac{D_1^2}{4}+\frac{2}{g}v_1^2 D_1}=-\frac{D_1}{2}\sqrt{\frac{D_1^2}{4}+\frac{2}{g}\times\frac{q^2}{D_1}} \tag{3}$$

Solution of this equation is got from Plate X, Vol. III.

(c) Energy loss in a jump on level floor.
$H_L=(D_2-D_1)^3/(4D_1 D_2)$ (4)

(d) Height of the jump.
$J=D_2-D_1$ (5)

(e) Position and lenght of jump.

Ignoring the form or splash on the hypercritical jet, which is usually equal to J in length the point when the first boil rises up should be taken as the position of the jump. Length of the jump indicating concussion downstream of the point is about 3 J on a level floor and 2 J on a glacis.

(f) Dr. Rehbock of Karlsruhe (Germany) has worked out the solution considering friction in the region of the jump on a level floor.

The resulting equation is cubic.

$$D_1^3-D_1^2H+(q^2/\delta+g)=v \tag{6}$$

Where D_1 is the depth upstream; q the dicsharge; H the Loss; δ the friction coefficient$=\cdot 9$ to $\cdot 94$.

(g) T. Blench I. S. E., Punjab Irrigation, in his article in Indian Engineering, Calcutta, 1934. Vol VXCIII & VXCV on "Mathematics of Canal Fall Cisterns" considers that the hydraulic jump may be with trough empty or with full trough. He derives the equation for full trough with D_L as the load on the hypercritical jet as in Fig. 4.

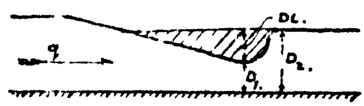


Fig. 4

$$D_1D_2\left[D_2+D_1-\frac{2D_1D_2}{D_2-D_1}\right]=\frac{2q^2}{g} \tag{7}$$

$$\text{and } H_L=D_L\times D_1-D_2+\frac{q^2}{2g}\left(\frac{1}{D_1^2}-\frac{1}{4D_2^2}\right) \tag{8}$$

He takes trough full when $D_L=J=D_2-D_1$

$$D_1D_2(D_2-D_1)=\frac{2q^2}{g}$$

$$\therefore D_2=\frac{D_1}{2}\pm\sqrt{\frac{D_1^2}{4}-\frac{2}{g}\times\frac{q^2}{D_1}} \tag{9}$$

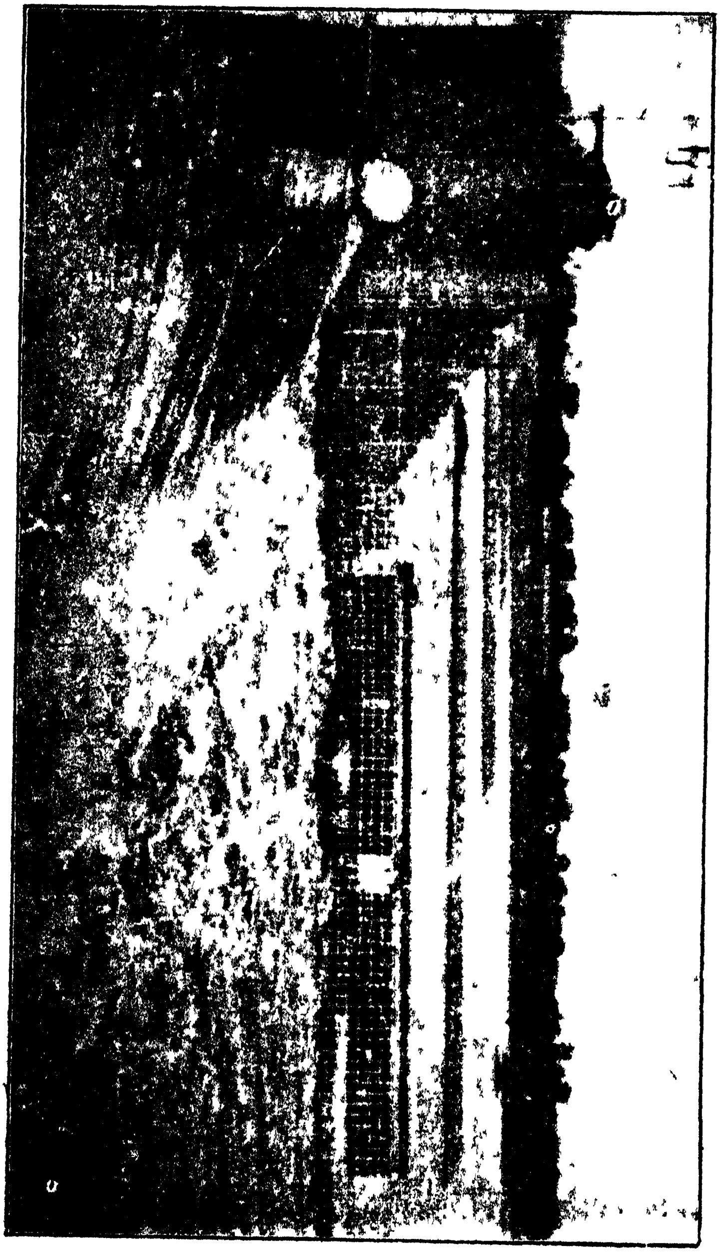
$$\text{and } H_L=\frac{(D_2+D_1)(D_2-D_1)^2}{4D_1 D_2} \tag{10}$$

The author considers that the hypercritical jet upstream of a jump simply carries away the foam or splash without changing pressures on bed. The water splash upstream of a jump is neither a part of the downstream flow nor that of the upstream, because a log of wood remains floating there for any length of time. It is only when the upstream flow is nearly at the critical conditions that the splash becomes a part and parcel of the upstream water, but in that case it shall be more near a standing wave, as explained before, than a hydraulic jump. Wave with a loaded trough may be considered an intermediate stage between a standing wave and a hydraulic jump which is not of much importance in actual practice.

Note :—In the Punjab Irrigation practice no distinction is made between hydraulic jump and standing wave. The term standing wave is applied to both.

(h) Hydraulic jump on a glacis.

The gravity component of the mass of water upstream of a jump affects the pressure plus momentum equation. Author's article, page 131, Indian Engineering, Calcutta, October, 1936, (Fig. 5)



Hydraulic jump after Reconditioning Marala Weir. Fig. 6

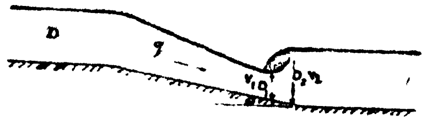


Fig. 5.

$$D_1\,D_2\,(D_1+D_2) = \frac{2q^2}{mg} = \frac{2}{m}D_c^3 \qquad (11)$$

where $m = 1 - 2\sin\phi$, and ϕ = angle of slope with horizontal

$$D_2 = -\frac{D_1}{2} + \sqrt{\frac{D_1^2}{4} + \frac{2}{mg} \times \frac{q^2}{D_1}} \text{ and } H_L = m\frac{(D_2-D_1)^2}{4D_1D_2} \qquad (12\ \&\ 13)$$

The solutions of these equations are given in Plate VIII, Volume III, for different slopes. For given D_1 and q, the corresponding depths D_2 are required on a glacis more than those on a level floor but energy loss is relatively less. There is thus relatively better recovery in a jump on a slope. Fig. 6 shows the hydraulic jump on the downstream side in the case of Marala weir in floods.

7. Position of Hydraulic Jump on a Glacis.

There are two methods in vogue—one due to Mr. E. S. Crump and the other due to Mr. A. M. R. Montagu. Both of them neglect frictional losses in flow down the glacis and are not therefore applicable to find the position of jump on a level floor. Both of them use the equation No. (2) meant for a jump on a level floor. They both apply to the discharge intensity not changing.

(*a*) Crump puts the pressure plus momentum equation in the dimensionless form.

$$xy\,(x+y) = \frac{2q^2}{g} = 2C^3;\ \textit{viz.};\ \frac{x}{C}\,\frac{y}{C}\,\frac{(x+y)}{C} = 2 \qquad (14)$$

where x is the upstream depth ; y the downstream depth ; K the total energy at weir crest ; F = Fall up to the point where jump is formed ; C = the Critical depth $= \sqrt[3]{\frac{q^2}{g}}$; L = Loss of head in jump. Fig. 7.

$$L = \frac{(y-x)^3}{4xy} \qquad (15)\ A$$

$$\frac{L}{C} = \frac{\left(\frac{y}{C} - \frac{x}{C}\right)^3}{4\,\frac{x}{C} \times \frac{y}{C}} \qquad (15)$$

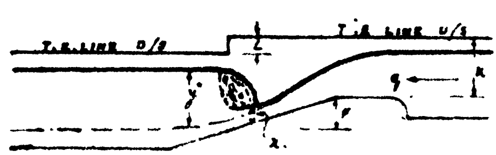


Fig. 7

Solution of these equations is given in Plates No. VI and VII, Vol III, to determine (K+F), x, y. (K+F) determines the position of the highest floor level which will be required downstream of a glacis.

(*b*) **Montagu's method.**

$$H_L = E + F - Ef_2 = (D_2 - D_1)^3/4D_1D_2$$

$$Ef_2 = E + F - H_L = Ef_1 - H_L \qquad (17)$$

Pressure plus momentum equation becomes.

$$w\left(\tfrac{1}{2}D_1^2 + \frac{q^2}{D_1 g}\right) = \left(\tfrac{1}{2}D_2^2 + \frac{q^2}{D_2 g}\right)w \qquad (18)$$

He has shown curves connecting H_L and Ef_2 for given discharge per foot run in Plate VI Central Board of Irrigation Publication No. 7 Montagu's curves of energy of flow are given in Plate (IXA & IXB) for discharges from 1 to 50 cusecs per foot width. Ef_2 subtracted from the downstream energy line fixes the position of the jump. The depths upstream and downstream of the jump are got from the plot of equation 18 in his pressure plus momentum Diagrams No. V to VII, Central Board of Irrigation Publication No. 4, With Ef_2 known, the right hand expression of equation $w[\frac{1}{2}D_2^2+(q^2/D_2 g)]$ can be worked out and for the value of pressure plus momentum, the depth upstream can be read from the above mentioned diagram.

The author considers that we can do without use of the pressure plus momentum diagrams and the depth D_2 can be read for the sub-critical flow from Montagu's Specific Energy (Energy of flow). Diagram Plate IXA & IXB for the already known value Ef_2. The depth D_1 upstream of the jump for the hypercritical jet can also be read from the same diagram using Ef_1, which is Ef_2 plus H_L. Similar remarks equally apply to author's Diagram Plate VI Fig. 1 (level floor).

8. Back Water Curves.

The subject of determination of the back water curves was first dealt with by Bresse in French in 1860. Text books on hydraulics give Bresse's tables only for positive slopes. The clearest exposition of the subject is given by Sherman M. Woodward in his publication. But the subject has not progressed further than where Bresse left it. The determination of water profiles with negative slopes, curved slopes and side expansion and contraction is not fully determined. The author in his article in the Indian Engineering, Calcutta, December, 1936. worked out the mathematics of a few more cases which we usually need for complete determination of water profiles over irrigation structures. The equations derived therein are given here and the student is advised to refer to the original publication for their proof.

Case I. Uniform Negative Slope (depth more than critical and constant width).

Differential equation (Fig. 8)

$$-d_b+d_y=d_k+d_f \qquad (19)$$

Solution.

$$L=\frac{y_n}{S}Z+y_n\left(\frac{1}{S}+\frac{C^2}{g}\right) \qquad (20)$$

where $Z=y/y_n$

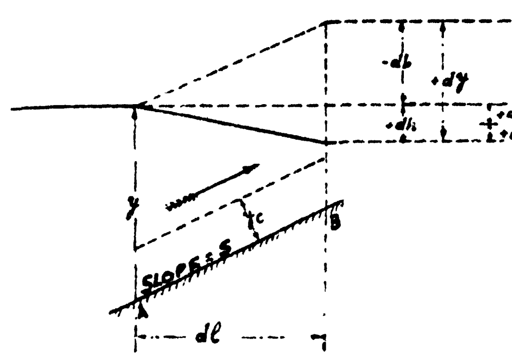


Fig. 8

$$F(Z)=\text{Backwater function}=\left[\frac{1}{6}\log\frac{Z^2-Z+1}{(Z+1)^2}-\frac{1}{\sqrt{3}}\tan^{-1}\frac{2Z-1}{\sqrt{3}}\right]$$

Case II. Variable slope with uniform constant width.

(Depth more than critical and circular approach). Fig. 9. Differential equation.

$$-d_b+d_y+d=d_k+d_f \qquad (21)$$

d_f is the increment in the centrifugal force due to mass of water moving round a circular curve.

$$\therefore \frac{S}{y}L=F(Z) \qquad (22)$$

where L=length, S=slope, y_n=Neutral depth, F(Z)=backwater function.

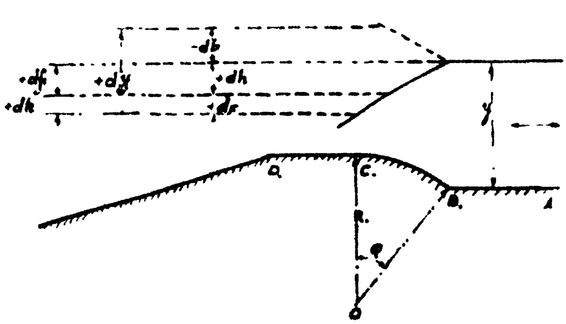


Fig. 9

$$F(Z) = Z + \frac{k^{1/3}}{540}\log(Z^3+1) - \frac{60k + k^{2/3} + 60}{180}\log(Z+1)$$

$$+ \frac{k^{2/3} + 60 + 60k}{300}\log(Z^2 - Z + 1) + \frac{k^{2/3} - 60k - 60}{60} \times \frac{1}{\sqrt{3}}\tan^{-1}\frac{2Z-1}{\sqrt{3}}; \text{ but } k = \frac{C^2S}{g}$$

Case III. Level floor, uniform width, Fig. 10., bed slope is zero, constant width.
Netural depth infinity, flow below critical,
Differential equation ; $d_y = d_k + d_f$ (23)
and solution.

$$\therefore\ L = \frac{C^2}{g}\left(y - \frac{y_4}{4y_c^3}\right) \qquad (24)$$

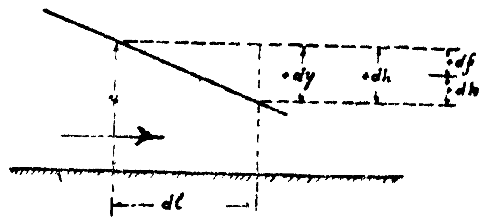


Fig. 10

Case IV. Positive slope with constant width.
Flow below critical and above neutral depth ; Fig 1.
Differential equation ; $d_y + d_h = -d_k + d_f$ (25)

Solution.

$$L = \frac{y_n}{S}\cdot Z - y_n\left(\frac{1}{S} - \frac{C^2}{g}\right) F(Z) \qquad (26)$$

where $F(Z) = \left[\frac{1}{6}\log\frac{Z^2 - Z + 1}{(Z-1)^2} + \frac{1}{\sqrt{3}}\tan^{-1}\frac{2Z-1}{\sqrt{3}}\right]$; and $Z = \frac{y}{y_n}$

Case V. Positive slope flow above critical depth, Fig. 11.
Differential equation ; $d_b - d_y = d_k + d_f$ (27)

$$\therefore\ -L = \frac{y_n}{S}Z - y_n\left(\frac{1}{S} - \frac{C^2}{g}\right). F(Z). \qquad (28)$$

Where $F(Z) = \frac{1}{6}\log\frac{Z^2 - Z + 1}{Z^2 - 1}$

$$+ \frac{1}{\sqrt{3}}\tan^{-1}\frac{2Z-1}{\sqrt{3}}$$

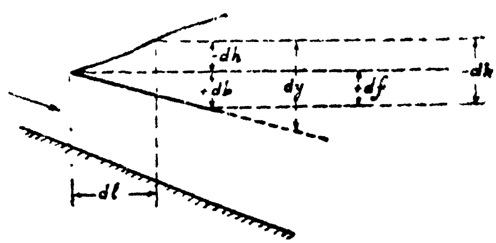


Fig. 11.

Case VI. Positive slope and uniform expansion Fig. 12.

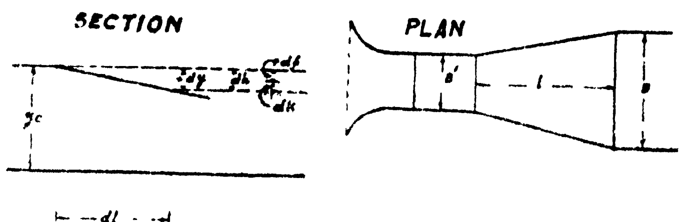


Fig. 12.

Solution. $y = B\left(\frac{1}{rC^2} + \frac{m}{g} + \frac{S}{r}k\right)$

where $k = \frac{B^2y^2}{Q^2}$ and $m = \frac{y_1 - el}{B_1 + el}$ and e = side expansion

Solution of expansion with curvature is not yet found out.

The following table gives values of F (Z) for different values of Z in case of dam, where y (actual depth) is always greater than y_n (neutral depth).

TABLE 1.

Z	F (Z)	Z	F (Z)	Z	F (Z)	Z	F (Z)
1·000		1·020	2·098	1·10	1·587	2·20	0·012
1·001	3·090	1·025	2·025	1·15	1·468	2·50	0·989
1·002	2·860	1·030	1·966	1·20	1·387	3·00	0·963
1·003	2·725	1·036	1·908	1·30	1·280	4·00	0·939
1·004	2·629	1·044	1·843	1·40	1·211	5·00	0·927
1·005	2·555	1·050	1·803	1·50	1·162	7·00	0·915
1·007	2·445	1·056	1·763	1·60	1·125	10·00	0·911
1·010	2·326	1·060	1·745	1·70	1·096	15·00	0·909
1·012	2·266	1·070	1·697	1·80	1·073	20·00	0·908
1·015	2·192	1·080	1·656	2·00	1·039	50·00	0·907

In the case of a fall downstream y/y_n is always less than unity. The following table gives values of F (Z) for this case.

TABLE 2.

Z	F (Z)	Z	F (Z)	Z	F (Z)	Z	F (Z)
1·000		0·985	2·183	0·850	1·367	0·400	0·709
0·999	3·090	0·980	2·085	0·800	1·253	0·350	0·656
0·998	2·859	0·975	2·009	0·750	1·159	0·300	0·605
0·997	2.723	0·970	1·946	0·700	1·078	0·250	0·553
0·996	2·628	0·960	1·847	0·650	1·006	0·200	0·503
0·995	2·552	0·950	1·769	0·600	0·939	0·150	0·453
0·994	2·491	0·940	1·705	0·550	0·877	0·100	0·402
0·992	2·395	0·920	1·602	0·500	0·819	0·050	0·352
0·990	2·319	0·900	1·522	0·450	0·763	0·000	0·302

TABLE 3.

L/C.	(K + F)/C	x/C.	y/C.
0·010	1·548	0·841	1·178
0·107	1·773	0·675	1·417
0·206	1·949	0·611	1·529
0·332	2·148	0·561	1·628
0·520	2·424	0·511	1·738
0·603	2·541	0·494	1·780
0·783	2·785	0·404	1·857
0·929	2·976	0·444	1·911
1·030	3·107	0·432	1·945
1·525	3·729	0·387	2·089
2·014	4·319	0·355	2·202
2·540	4·940	0·329	2·305
3·101	5·588	0·308	2·400
3·999	6·697	0·281	2·530
5·204	7·950	0·255	2·677
6·053	8·834	0·241	2·766
6·935	9·847	0·228	2·850
8·316	11·340	0·212	2·968
10·254	13·418	0·195	3·112
12·294	15·584	0·180	3·244
14·424	17·833	0·168	3·364

9. Determination of Water Profiles, Using Backwater Curves.

(*a*) Weir is splitup in parts as shown in Fig. 13.

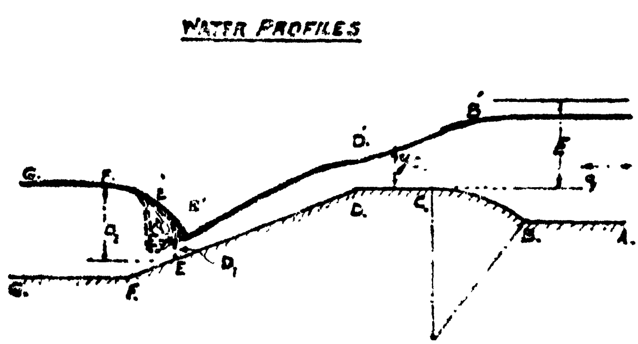


Fig. 13

1. AB flow with neutral depth
2. BC Case II
3. CD Case III
4. DE Case IV
5. EF Case V
6. FG Flow with neutral depth.

E is the position of the hydraulic jump.

Using the respective flow equations with the known depth in the beginning a few points are calculated giving the length (distance from the beginning) and the depth. They are plotted and the depth at the end of each type of flow is read. This depth is used for the next flow as the beginning point. Thus we can get A, B, D, and E. Similarly water profile can be worked for F, F, G. starting from the G side. The position of the jump is got by trial and error ; read D_1 and work out D_1 from Plate X, Vol. III. If actual depth E″E down-stream is less than this D_2 try another point downstream of it. In practice it is not difficult to get the solution puickly because the change in D_1 is very small.

(b) The objections are usually raised against this method, that there is change of equation from one part to another and that the bed slope changes abruptly None of these objections is of much practical consequence The author worked out water profiles to determine the position of the jump using this method in the case of the Rasul weir, undersluices, ten meter flumes in the Northern Branch and Sulki Branch and those in the Mithalak distributary of the Lower Jhelum Canal for different supplies. They were not found to agree with the actual, using the absolute value of coefficient of friction ·005 for Brick masonry. The backwater equations are based on Chezy's empirical equation of flow $v=C\sqrt{RS}$. The mathematical proof of this equation is evailable in all standard text books on hydraulics, equating the work done against frictional resistances equal to the energy available from drop in slope. The frictional resistance is supposed to be proportional to v^n, where v is the average velocity and n is taken as 2. The actual value of n varies from 1·83 to 2·16 (Page 134 Treatise on Hydraulics by W. C Unwin) and with high velocities it is even up to 2·3. The author selected higher values of 'f' according to the prevailing velocities in each flow instead of varying n and then found that calculated water profiles and the position of the jump agreed with the actual. A graph of the variations of 'f' for different velocities is given in Plate X which should be used to work out the value of $C=\sqrt{2g/f}$ in Chezy's equation when using backwater equations.

(c) Backwater functions.

The values of backwater functions F (Z) for all cases including Bresse's values for positive slopes are given in Plate VIII, Vol. III.

10. Merits and Demerits of Various Methods.

The friction loss is considerable on the glacis. Easily one quarter of the total drop will be lost in friction and the remaining shall be lost in the jump The determination of water profiles by taking friction into account, is by far the best method and also the accurate one. It no doubt, involves much labour which is worth while for the correct appreciation of the problem.

11. Examples.

1. Calculate the rise of level at a point half mile upstream produced by a dam arranged so as to raise the water at crest by 8·0 feet. The original depth of the stream is 2·0 feet (supposed uniform) slope in bed 1 in 500 and f = 0·006.

(a) $v=C\sqrt{R.S}$. where $C=\sqrt{\dfrac{2g}{f}}=\sqrt{\dfrac{64\cdot4}{0\cdot06}}=107\cdot5$

R = 2·0 feet nearly $\qquad v=107\cdot5\times\sqrt{2\times1/500}$

$=107\cdot5\times\sqrt{1/250}=6\cdot8$ feet per second.

Discharge per foot run = v. A = 6·8 × 2 = 13·6 cusecs.

$y_n=2\cdot0$ feet $\qquad Z=y/y_n$, Let $y=10$

$\therefore \dfrac{y_1}{y_n} = \dfrac{10}{2} = 5\cdot0$

$L_1 = \dfrac{y''}{S} Z - y_n \left(\dfrac{1}{S} - \dfrac{C^2}{g}\right) F(Z)$

From Plate X, Vol. III, F (Z_1) from curve for Z = 5 is 0 927

$L_1 = \dfrac{2}{1/500} \times 5 - 10\left(\dfrac{1}{1/500} - \dfrac{C^2}{g}\right) 0\cdot927$

$= 5000 - 10(500 - 333\ 3)\ 0\ 927$

$= 5000 - 166\cdot7 \times 0\cdot927 - 5000 \quad 1500 = 3500$ feet

(b) Let $y_2 = 6$, $\dfrac{y_2}{y_n} = 6/2 = 3\ 0$, F (Z_2) 0 95

$L_2 = 1000 \times 3 - 6 \times 166\cdot7 \times 0\cdot95 - 3000 - 953 = 2047$

(c) Let $y_3 = 4\cdot0$, $\dfrac{y_3}{y_n} = 2\cdot0$, F $(Z_3) = 1\ 03$

$L_3 = 1000 \times 2 - 4 \times 166\cdot7 \times 1\cdot03 = 1314$ feet

$L_1 - L_3 = 3450 - 1314 = 2136$

(d) $y_4 - 3\cdot0$ $\dfrac{y_4}{y''} = \dfrac{3}{2} = 1\cdot5$, $F(Z_4) = 1\cdot20$

$L_4 = 1000 \times 1\cdot5 - 3 \times 166\cdot7 \times 1\ 2 = 1500 - 600 = 900$

$L_1 - L_4 = 3500 - 900 = 2600$ which is nearly half mile

$\therefore$ the depth at half mile from the dam will be 3·0 feet

2. At a weir, the afflux level is 18 ft. above the normal bed upstream. Depth below this weir 16 feet. Slope 1 in 1000. Find out at what distance upstream of weir, the depth will be 17·0 feet.

when C^2/g 153 in Chezy's formula and $l = L_2 - L_1$

$$l \quad \frac{y_2 - y_1}{S} + y_n\left(\frac{1}{S} - \frac{C^2}{g}\right)\left[F(Z_1) - F(Z_2)\right]$$

$y_1 = 17\cdot0$; $y_2 = 18$ feet and $y_n = 16\cdot0$ feet

$y_1/y_n = 17/16 = 1\cdot062$ and F $(Z_1) = 1\cdot755$ from Plate X, Vol. III.

$y_2/y_n = 18/16 = 1\cdot25$ and F $(Z) = 1\cdot527$

$\therefore l = \dfrac{18-17}{1/10{,}00} + 16(1{,}000 - 153)(1\cdot755 - 1\cdot527)$

$= 10{,}000 + 16 \times 9847 \times 0\cdot228$

$= 45{,}700$ ft.

3. In a canal 10 feet deep with slope 1 in 5000 and $C^2/g = 200$ in Chezy's formula, it is required to find the distance between two points where depth is 8 feet and 7 feet. Flow is with increasing depth.

$y_n = 10$ feet, $y_1 = 8$ feet, $y_2 = 7\cdot0$ feet. Find F (Z_2) and F (Z_1) from curves Plate X and calculate.

$$l = \frac{y_2 - y_1}{S} + y_n\left(\frac{1}{S} - \frac{C^2}{g}\right)\left[F(Z_1) - F(Z_2)\right]$$

Ans 3400 feet.

4. A canal 50 ft. bed width, 1 to 1 side slopes, and 6 ft. deep carries a discharge of 600 cusecs. The slope is 1/10,000. The condition of channel is such that Manning's M = 0 025 A regulator in the canal heads up water by 2·0 ft. How far above the regulator water in the canal will be raised by the afflux.

The depth upstream of regulator = 8·0 feet water way of the canal = (50 + 8)8 = 464 sft.

Velocity of water = 600/464 = 1·3 per second Hydraulic mean depth

$\therefore \dfrac{A}{P} = \dfrac{464}{50 + 1\cdot41 \times 8 \times 2} = 6\cdot4$ feet

$$v=(1{\cdot}4858/N)\ R^{\frac{2}{3}}\ S^{\frac{1}{2}}$$

$$1{\cdot}3=\frac{1{\cdot}4858}{{\cdot}025}\ 6{\cdot}4^{\frac{2}{3}}\times S^{\frac{1}{2}}\ \therefore\ S=\frac{1}{25{,}000}$$

$$\text{Average slope}=\frac{{\cdot}00010={\cdot}00004}{2}={\cdot}00007$$

The difference between the average slopes and the actual.

$\cdot0001-\cdot00007=\cdot00003$

The length of backwater curve.

$l=2/{\cdot}00003=66667$ feet

5. The original depth of a wide stream is 3·0 ft. and slope of its bed 1 in 1000, the value of C=70. A weir of 10 feet height is erected across the stream. Determine the rise in water level immediately behind the weir and at points ½ mile and one mile upstream.
Ans. Rise U. S. weir=9·46 feet ; at ½ mile=6·87 feet ; at one mile=5·34 feet ;

6. A fall of 5·0 feet occurs in a channel 30 ft. wide with 300 cusecs discharge and depth=5·0 feet. The fall is of full width and the glacis slope is 1 in 5. Find out where the Hydraulic Jump will occur.

(A) The discharge per foot run=10 cusecs. (using Crump's method)

$K=\left(\frac{q}{3{\cdot}1}\right)^{\frac{2}{3}}=2{\cdot}2$ ft. critical depth$=\frac{2{\cdot}2}{3}\times2=1{\cdot}46$

Using Crump's method and his diagram Plate VII, Vol. III, for $\frac{L}{C}=\frac{5}{1{\cdot}46}=3{\cdot}42$ Read.

$x/C={\cdot}8$ $\therefore\ x=1{\cdot}46\times{\cdot}3={\cdot}44$ ft.
$y/C=2{\cdot}44$ $\therefore\ y=2{\cdot}44\times1{\cdot}46=3{\cdot}55$ feet.
$(K+F)/C=6{\cdot}0$ $\therefore\ K+F=6\times1{\cdot}46=8{\cdot}76$ feet.
$\therefore\ F=8{\cdot}76-2{\cdot}2=6{\cdot}56$ feet.

Distance from the beginning of the glacis$=6{\cdot}6\times5=32{\cdot}80$ ft.

(B) Following Montagu's method.

From Plate IX, Vol III, Ef_2 for q=10 and $H_L=5{\cdot}0$ is 3·67 ft. The jump will occur where downstream $Ef_2=3{\cdot}67$, at a point 3·67, below the total energy line. Depth with 3·67 energy of flow and 10 cusecs discharge, the depth=3·55 ft. From Plate X, which is the same value as by Crump's method for depth downstream of jump. This will be at a distance of 32·8 feet from the beginning of the glacis.

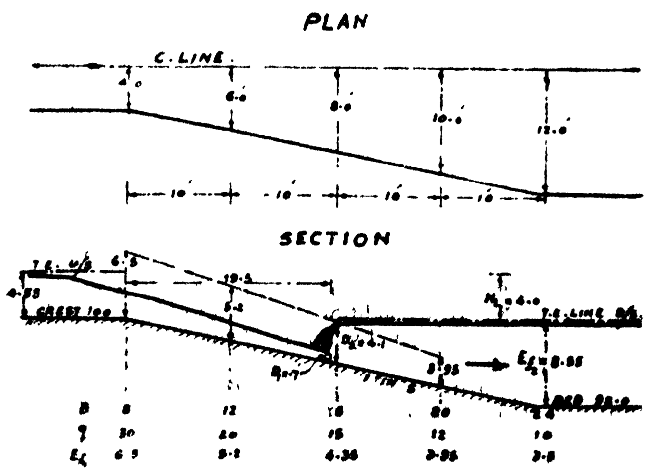


Fig. 14.

7. A 5·0 feet fall occurs in a channel of 250 cusecs with bed width 25′ and 4 feet depth the fall is contracted to a discharge intensity of 25·5 cusecs per foot run. Find the depth of the cistern below the downstream bed.

The normal value of Ef in channel 10 ft. discharge per foot run and depth 4 0 feet is 4·1 ft. from Plate IX-A, Vol III.

Again with q=25·5 cusecs per foot run and H_L= 5·0 from Plate VII, Ef_2=6·1 feet for trough empty ;

∴ depth in the cistern= ·1−4·1=2·0 feet or the floor downstream shall have to be depressed by 2·6 ft.

8. A fall of 4·0 ft. occurs in a channel of 240 cusecs and 25 feet bed width The glacis slope 1 in 5 and expansion from 8·00 feet at the crest to the bed downstream 1 in 5 on both sides Find the position of the hydraulic jump.

Divide the expansion in four equal parts, work out discharge intensity at each section and then find out Ef_2 from Plate VII for fall of 4 0 feet for the various discharge intensities. Plot the value of Ef_2 thus derived on the glacis and where they cut Ef_2 line downstream, the intersection gives the position of the jump as shown in Fig. 14 ; Distance from end of crest=19·5′.

B	8	12	16	20	24
q	30	2·0	15	12	10
Ef_2	6·5	5·2	4·35	3·95	3·5

9. (*a*) State the critical condition of flow and prove that the specific energy in these conditions of flow is minimum ? What are other conditions in this type of flow ?

(*b*) The discharge per foot run on the downstream glacis of weir is 48 cusecs. The glacis slope is 1 in 5. The depth upstream of the jump in the glacis is 1·8 feet. Find the height of the jump and the energy loss in it. *P.U. 1943*

10. Water is headed up against a sluice gate on a level floor up to a depth of 12 ft. upstream. The height of the open is 2·0 ft. What is the depth required in the down stream channel so that a hydraulic jump is formed ? Find also the uplift pressure caused by the jump in the downstream floor.

11. A sluice spans a channel of rectangular section 60 ft. wide with an opening 2·4 ft. deep and discharge 1720 cusecs. If a hydyaulic jump is formed on the downstream side of the sluice determine the height of crest of the jump above the upper edge of the sluice.

12. Explain the phenomena known as the standing wave and find the formula for the height of the wave in a rectangular section in terms of the upstream depth and the discharge. *F.P.S.C. 1940*

13. Explain the phenomena of the standing wave and state the conditions necessary for its formation.

Draw a sketch of a standing wave flume bringing out its essential parts, explain the function of each. Deduce an expression for determining the discharge of a channel on which a standing wave flume is formed. State the assumption made. *F.P.S.C. 1937*

14. What is meant by afflux in a river and under what conditions is it caused ? What steps would you take to minimise it in any given case and why is that necessary ? Deduce a formula for determining afflux caused by a bridge across a river. *F.P.S.C. 1939*

PART VI

GENERAL SECTION

CHAPTER V

Hydraulics Applied to Irrigation

In this chapter it is presumed that the reader is familiar with the principles of hydraulics and the definitions of terms usually employed in literature on the subject. Hence no attempt is made to prove the well known formulae for which the reader can refer to any of the text books. Only important formulae are given and the student should possess any of the standard text books mentioned below to slove the problems given at the end of this chapter (*a*) Lewitt (*b*) Gibson (*c*) Lea.

1. Properties of Fluids Compared with Solids.

The flow of water in a channel differs from the movement of solid on an inclined plane in the following respects

(*i*) Roughness of surface and contact affects both the solids and liquids. Disturbed liquid is more rough.

(*ii*) A sloid does not roll down before the inclination exceeds "repose." Water spreads even on a level surface.

(*iii*) Solids moving down and an inclined plane accelerate, while water maintains a steady velocity at any inclination.

(*iv*) This shows that the frictional resistance to the motion of water increases with velocity, whereas in the case of solids friction is independent of velocity.

(*v*) Friction between solids increases with pressure, while loss of head incurred by water is independent of pressure.

(*vi*) In solids friction is independent of area. In liquids it directly depends on the same.

1 cft.	water	= 62·2786 lbs.
1 gallon	,,	=10 lbs.
1 litre	,,	=2·197 lbs.=0·0353 cft.
1 gallon	,,	=4·5459631 litre=0·16057 cft.

2. Hydrostatics.

(*i*) Normal pressure on a submerged plane surface.

Total normal pressure is equal to the product of the weight of the cubic unit of water the area of surface and the head on the centre of gravity.

(*ii*) Centre of pressure of an immerced plane, Fig. 1.

$$V = \frac{\text{Moment of inertia}}{\text{static moment}} = \frac{I_o}{A\bar{x}}$$

I_o = Moment of inertia about free surface $= I_a + A\bar{x}^2$

$\bar{x}$ = depth of centre of gravity below 0·0.

A = area of surface

I_a = Moment of inertia about a horizontal axis through the centre of area.

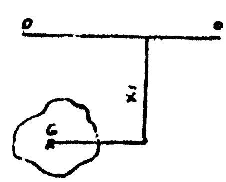


Fig. 1.

3. Time of Emptying and Filling a Canal Lock

Let H = Difference in level between headway and tailway, Fig. 2.

a = area of orifice

A = Area of the Horizontal Cross Section of the lock.

C = Coefficient of discharge

Time of emptying; $t = \dfrac{2A\sqrt{H}}{Ca\sqrt{2g}}$

4. Pressure on a Lock Gate.

Let AB be the vertical section of a plane rectangular surface.

Divide this into n horizontal strips on which the pressure of water is equal.

$$\textit{i.e.,}\ \text{pressure on any strip} = \frac{\textit{Pressure on AB}}{n}$$

Let d_1, d_2———d_r ——d_n be the respective centres of pressure of each strip. In that case the depth of the centre of pressure of the strips below the water surface is, Fig. 3.

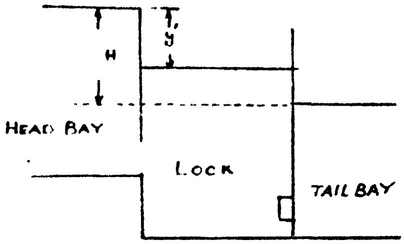


Fig. 2.

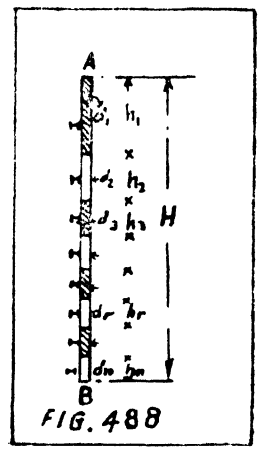


Fig. 3.

$$Ad_1 = \tfrac{2}{3}\ H\ \frac{1^{\frac{3}{2}} - (1-1)^{\frac{3}{2}}}{\sqrt{n}}$$

$$Ad_2 = \tfrac{2}{3}\ H\ \frac{2^{\frac{3}{2}} - (2-1)^{\frac{3}{2}}}{\sqrt{n}}$$

$$Ad_r = \tfrac{2}{3}\ H\ \frac{r^{\frac{3}{2}} - (r-1)^{\frac{3}{2}}}{\sqrt{n}}$$

5. Bernouillis' Theorem and Venturi Meter.

$$Z + \frac{P}{w} + \frac{v^2}{2g} = Z_1 + \frac{P_1}{w} + \frac{v_1^2}{2g}$$

H = Difference of pressure head in feet of water in the piezometer tubes, Fig. 4.

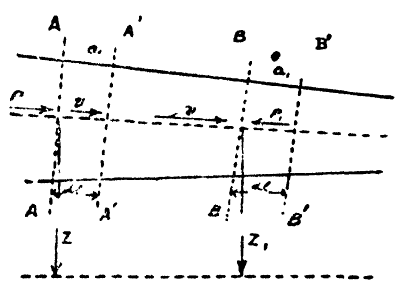


Fig. 4.

a_1 = area of enlarged end in sq. ft.

a_2 = area of the throat in sq. ft.

q = quantity of water flowing in cft./sec. (Fig. 5.)

v_1 = velocity of water at enlarged end.

v_2 = velocity of water throat end.

$$q = \frac{a_1 a_2}{\sqrt{a_1^2 - a_2^2}} \sqrt{2g\ H} = C\sqrt{H}$$

6. Flow Through Small Orifices.

(*i*) Free fall

$V = C_v \sqrt{2gH}$

$Q = C_c AV = A.C_c.C_v \sqrt{2gH} = C_d A \sqrt{2gH}$

The principal kinds of orifices met within practice with their average coefficient are given below :—Fig. 6.

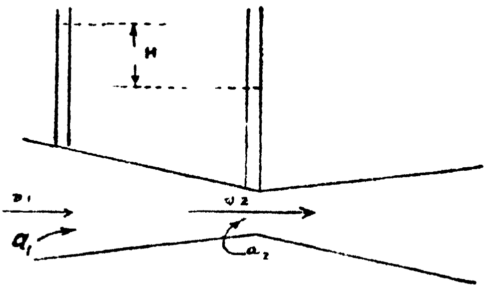


Fig. 5.

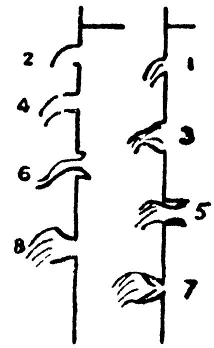


Fig. 6.

NOTE. The length of a tube must not exceed three times the diameter, otherwise the co-efficient is reduced owing to friction and the tube becomes a pipe.

For pipe outlets $C = 0{\cdot}625$. Hence $C\sqrt{2g} = 5$; $\therefore Q = 5A\sqrt{H}$

If the length of the cylindrical tube is reduced. till the jet springs clear from the upstream edge the co-efficients change to the values shown in Fig. 6. (1 to 6). There is very little change in the co efficient of discharge with variation in head.

Velocity of approach.

If the water in the reservoir has a velocity v'/sec., then the static head is increased by $v^2/2g = h_a$ and the value of H is increased to $H_1 = H + h_a$.

More than one orifice in the same wall.

If the orifices are fairly close. the discharges are increased. The following values of the co-efficients of discharge C_d show the effect of gate openings.

No. of gates open	1	2	3	4	5
C_d	0·633	0·642	0·646	0 649	0 65

In order that two orifices in the same plane may have no effect on each other, there should be no overlapping of the minimum clear margins or the minimum area of approach sections requisite for full contraction.

Margin.

The upstream surface of the wall surrounding an aperture is called the margin. It is said to be clear. when it is free from projection, leakages or anything which would interfere with the free flow of water along the wall towards the aperture. When an aperture has sharp edges, an increase in the clear margin up to a certain limit, increase the degree of contraction. When this limit is reached, the contraction is said to be complete. This limit is 2·75 times the least dimensions of the aperture.

Jet from an orifice. Fig 7.

Like any other projectile, the jet describes a parabola (neglecting air resistance). Its equation.

$$\therefore \; y = x \tan \theta - \frac{x^2 g}{2v^2} \sec^2 \theta$$

For a jet issuing horizontally the above equation is reduced to $y = x^2/4H_c$

The range of the jet on a horizontal plane H' below the orifice is $x = 2\sqrt{H_c \times H'}$

Path of water over waste weir. (Fig. 8.)

$y = D + (4/g)\ H.$

$x = (4/3)\ C\sqrt{H.y}$

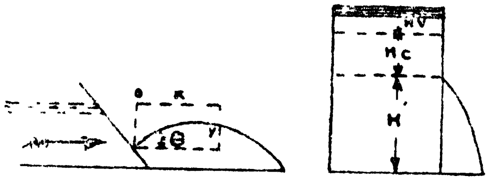


Fig. 7.

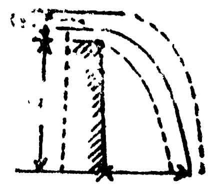


Fig. 8.

where H = height of still water above crest of weir.

x = Horizontal distance of the centre of the falling water from the downstream lip of the crest at any depth D below the level of the crest.

C = Coefficient varying from 0·63 to 0·68.

Effect of head on coefficient of discharge.

The value of the coefficient increases with head up to 4·0' beyond which it is constant.

Value of Coefficient of discharge.	C_d
Ordinary lock sluice	0·62
Small regulator opening with shallow water	0·57
Regulator opening up to 6' wide with recesses in piers	0·62
do 6' to 13' wide	0·72
do above 13'	0·82
do 6' width with straight and continuous peirs	0·72
do 6'—13' wide	0·82
do above 13' wide	0·92

Submerged orifice.

$Q = CA\sqrt{2g}\ \sqrt{H}$

H = Difference of water level between the head and tail race.

For small size of the orifice and head upto 4·0', the approximate value of C = 0·6

Short tubes. (Cylindrical)

In a cylindrical chute, the jet contracts, but it expands again, fills the tube and issues full bore. Hence the discharge through the tube is greater than that of orifice of the same bore.

For moderate Head $C_d = 0·63$

Bell mouthed tube (design)

The bell mouthed tube is made of the shape of the jet issuing. The length BE is half the diameter AB and the curves AC and BD in Fig. 9 have a radius of 1·3 times AB. This makes CD equal to 0·8 AB. The edges at AB must be rounded and not left sharp. The coefficient of discharge and velocity are in the neighbourhood of 0·96 to 0·99 ; the coefficients increasing with head.

7. Large Vertical Orifices (Free Fall).

(*i*) **Rectangle.** (Fig. 10).

Let H = height of free surface above top of *vena contracta*

b = breath of orifice

d = depth of orifice

b' = breath of *vena contracta*

d' = depth of *vena contracta*

$$Q = 2/3\ C_b\sqrt{2g} - [(H+d)^{3/2} - H^{3/2}]$$

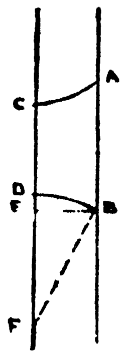


Fig. 9.

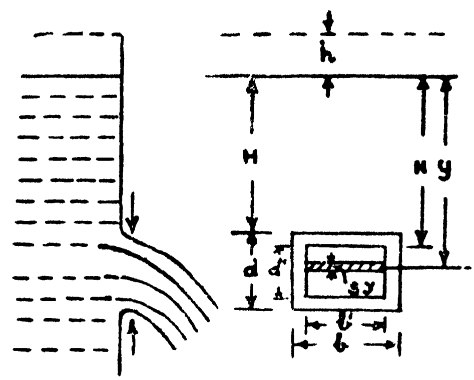


Fig. 10.

For square orifice C is nearly 0·6

For rectangular orifice the value of C depends on the ratio of length/breath and varies from 0·632 to 0·6. The following table gives the values of the coefficient for orifice 1′ wide.

Head in ft. from centre of orifice	values of d, in feet.						
	1/8	1/4	1/2	¾	1·0	1·5	2·0
1·0	0 632	0·632	0·618	0·612	0·606	0·626	
3·0	0·627	0 627	0·615	0·610	0·605	0 614	0·610
6 0	0·615	0·615	0·609	0 604	0·602	0·606	0 610
10·0	0·606	0·603	0·601	0·601	0·601	0·601	0·602
20·0	0·602	0·601	0·601	0·601	0·601	0·601	0·602

(*ii*) **Circle** (Fig. 11.)

$$Q=C\,\pi\,R^2\,\sqrt{2gH}\left(1-\frac{1}{32}\times\frac{r^2}{H^2}\right)$$

r = radius of *vena contracta*

R = Radius of orifice.

Value of C = 0·6 nearly

(*iii*) **Triangular Opening.** (Fig. 12.)

(*i*) **Base up.**

$$Q=\tfrac{2}{3}C_v l\sqrt{2g}\left(\frac{2}{5}\,\frac{H_b^{\frac{5}{2}}-H_t^{\frac{5}{2}}}{H_b-H_t}-H_t^{\frac{3}{2}}\right)$$

(*ii*) **Base down.**

$$Q=2/3\;C_v l\sqrt{2g}\left(H_b^{\frac{3}{2}}-\frac{2}{5}\,\frac{H_b^{\frac{5}{2}}-H_t^{\frac{5}{2}}}{H_b-H_t}\right)$$

Trapezoidal opening.

$$Q=\tfrac{2}{3}C_v\sqrt{2g}\left\{l_b H_b^{\frac{3}{2}}-l_t H_t^{\frac{3}{2}}+\tfrac{2}{5}(l_t-l_b)\,\frac{H_b^{\frac{5}{2}}-H_t^{\frac{5}{2}}}{H_b-H_t}\right\}$$

Sluices.

The sluice is an orifice provided with a gate of shutter. If the coefficient of discharge is only found approximately it averages from 0·61 to 0·7. If the sides and lower edges of an orifice are produced externally so as to form a shoot, the coefficient C may be greatly altered.

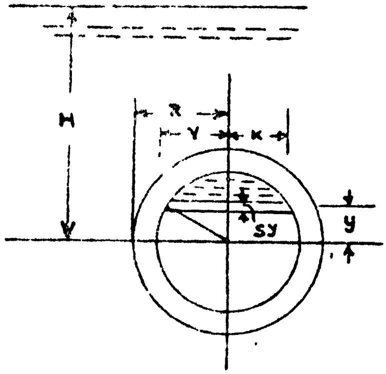


Fig. 11.

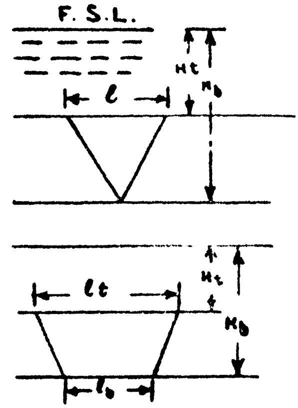


Fig. 12.

Partially submerged orifices. (Fig. 13)

Here the portion of the orifice line below the normal water level on the discharge side may be considered as a submerged orifice while the portion d_2 above this level may be looked up on as discharging freely into air.

Let B = breadth of orifice

d = depth

H = Height on the upstream-above top of the orifice.

d_1 = free depth

d_2 = drowned depth

$\therefore\ Q = B\sqrt{2g}\left[\tfrac{2}{3}C_1\{(H+d_1)^{3/2} - H^{3/2}\} + d_2C_2(H+d_1)^{1/2}\right]$. If $C_1 = C_2 = C$

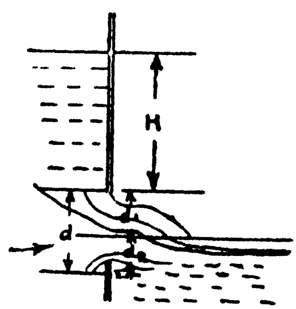


Fig. 13.

$\therefore\ Q = CB\sqrt{2g}\left[(H+d_1)^{1/2}\{\tfrac{2}{3}(H+d_1)+d_2\} - \tfrac{2}{3}H^{3/2}\right]$

8. Weirs.

When the length "l" of the weir is great relative to H end contraction does not matter. The coefficient of discharge C_d increases with H. Hence there is velocity of approach, the discharge increases faster than $H^{3/2}$.

Various kinds of weirs and their coefficients.

Weir formula. (Fig. 14.)

$$Q = \tfrac{2}{3}CB_l\sqrt{2g}H^{3/2}$$

where H includes velocity of approach $= G + h_a$

Francis formula.

$Q = 3{\cdot}53\ (B_t - 0{\cdot}1\ nH)\ H^{3/2}$, where n is the number of end contractions.

Bazin's formula.

$Q = \mu\sqrt{2g}\ B_t\ H^{3/2}$ (No. velocity of approach)

$Q = m\sqrt{2g}\ B_t\ H^{3/2}$ (with velocity of approach)

$m = \mu\ \{1 + 0{\cdot}55\ H/(G+H)\}$

$\mu = 0{\cdot}405 + 0{\cdot}00984/H$

G = height of weir crest above the bottom of the channel of approach.

B_t = width at throat on crest of the weir

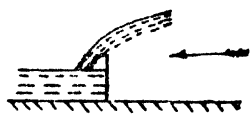
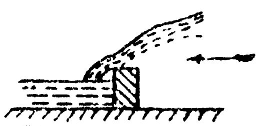
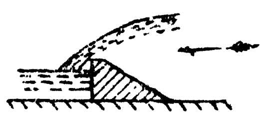
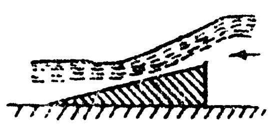
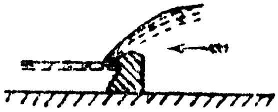
	Co-efficient C for Head = 1'0'	Manner in which the co-efficient varies as head increases
 THIN WALL	0·67	Increases slowly
 FLAT TOP VERTICAL FACE & BACK	0·5	Increases rapidly
 STEEP BACK & SLOPING BACK	0·75	Increases
 STEEP FACE & SLOPING BACK	0·61	Do
 ROUNDED O	0·85	Do

Fig. 14

The co-efficient include the allowance for velocity of approach.

Value of overall coefficients.

Description of weir	C	values of $2/3\ C\sqrt{2g}$
Broad-crested or flat-topped	0·577	3·09
Narrow-crested	0·623	3·33
Weir overfalls where w=full width of channel	0·666	3·56

Flow when air under the Nappe is excluded.

As the pressure under the sheet decreases, the discharge increase. Hence for accurate discharge observations free access of air under the sheet is essential.

Weirs with flat top and vertical face and back. (Fig. 15)

Generally the water at B holds back that upstream of discharge is less then for a weir in a thin wall under the same head. It is sort of drowned weir. B, being the tail water level.

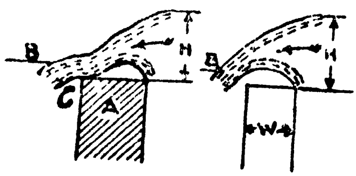


Fig. 15.

When H is about 1·6 w to 2 w, the sheet springs clear from the top and the case becomes a weir in a thin wall. But if the sheet nearly touches at C, the water gradually abstracts the air and the sheet is pressed down, touches at C and the discharge is slightly greater than for a weir in thin wall.

Miscellaneous weir. (Fig. 16)

For a fall in which their is neither a raised crest nor a lateral contraction there is no local reduction of the approaching stream due to eddies at walls and therefore no local surface fall of the kind ordinarily occurring. The surface curve is due to draw. If the slope AB is not very steep, the curve extends for a great distance.

Let v=Velocity at DE near to BC

This is both the velocity of approach and the velocity in the weir formula

If $C=0·72$; $\because$ $v=·62\sqrt{2gH}$

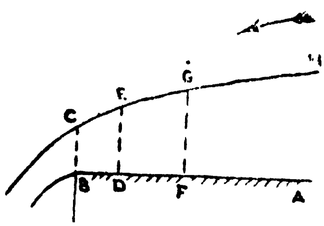


Fig. 16.

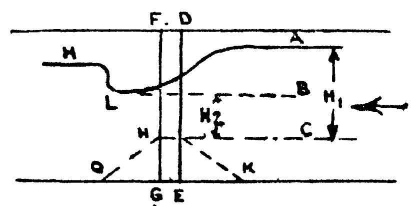


Fig. 17.

Submereged weirs. (Fig. 17)

q_1=discharge through AB $=\frac{2}{3}C_2B_l\sqrt{2gH}\times H$

q_2=discharge through BC $=C_2B_l\sqrt{2gH}.\ H_1$ where $H_2=H_1-H$

If $C_1=C_2=C$ $\therefore$ $Q=B_l\sqrt{2gH}\,(H_1-H/3)$

Simplified formula for drowned weir.

$Q=\frac{2}{3}\ C_d\ B_l\ \sqrt{2g}\ .\ H,$

Where C_d=inclusive coefficient for drowned weir and H_1=upstream Head.

Weirs with flat tops work with the above formula with a fair degree of accuracy as in their case H_2 is the critical depth for a given discharge per foot run of the weir.

Contracted channels and weirlike conditions.

If the stream is contracted at a bridge site due to piers and abutments, the degree of obstruction is measured by the loss of head and not by the reduction in surface width in that case the weir formula hold.

Weirs with sloping and stepped side walls.

Triangular weirs. (Fig. 18)

$$Q=\tfrac{4}{15}\, C\sqrt{2g}\; l\; H^{3/2}$$

Where l = top width ; since l varies with H, Q $\propto$ $H^{5/2}$

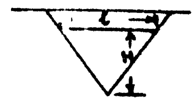


Fig 12.

Trapezoidal weir. (Fig. 19.)

$$Q=\tfrac{2}{3}\, C\sqrt{2g}\, H^{3/2}\,\{l_b+\tfrac{2}{5}\,(l_t-l_b)\}$$

where l_t = top width of water surface over weir and l_b = bottom width.

The quantity within brackets is the crest length of the equivalent ordinary weir.

The length is less than $(l_t+l_b)/2$ because the velocity of the water at the bottom of a section is greater than at the top.

If $\dfrac{AB}{BC}=\cot\alpha=r$, then $Q=\dfrac{2}{3}C\sqrt{2g}\;H^{3/2}\;(l_b+0{\cdot}8rH)$

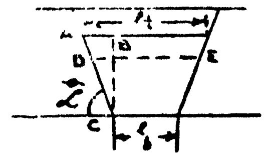


Fig. 19

9. Pipe and Syphons.

Short pipes.

When the length L is less than 100 diameters the velocity in a pipe is determined by the equation.

$$v=C_v\sqrt{2gH}$$

H is the head at the centre of the pipe or difference in water levels at the head and tail ends.

C_v ; depends on the material and diameter of the pipe, the ratio of L/D etc. For brickwork C_v varies from 3·66 to 0·48 for L/D ratio of 86 to 54. For cement or store work, it varies from 0·77 to 0·60 for an L/D, ratio, of 6 to 36.

Long pipes.

$$v=C\sqrt{R.S}.$$

Abrupt enlargement. (Fig. 20.)

Loss of head due to abrupt enlargement $=\dfrac{(v_1-v_2)^2}{2g}$

v_1 = Initial velocity and v_2 = final velocity.

Abrupt contraction.

The loss of head is caused by abrupt enlargement from EF to CD. To find the velocity at EF, divide the velocity at MN by C_c. The values of C_c are as follows :—

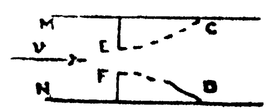


Fig. 20.

$\dfrac{\text{Area EF}}{\text{Area CD}}=0{\cdot}1$	0·2	0·3	0·4	0·5	0·6	0·7	0·8	0·9	1·0
C_c=0·624	0·632	0·643	0·659	0·681	0·712	0·755	0·813	0·842	1·0

Generally the loss of head due to contraction may be taken $0{\cdot}5\,\dfrac{v^2}{2g}$ (entry loss)

Elbow. (Fig. 21.)

Loss of head $=Z_e\,\dfrac{v^2}{2g}$

where $Z_e=0{\cdot}946\,\dfrac{\sin 2\phi}{\;}+2{\cdot}05\,\sin\dfrac{\phi}{2}$

Fig. 21.

Gradual change.

Loss of head $h = \frac{v^2}{C^2R} - \frac{v_1^2 - v_2^2}{2g}$

Discharge through syphons.

Loss of head $H = \left(L + f_1 + f_2 \frac{B}{R}\right) \frac{v^2}{2g} - \frac{u^2}{2g}$

H = Fall in ft. through syphon.
L = Length of barrel and entrance in feet and R = H.M.D.
v = Mean velocity through barrel and u = Velocity of approach.
$f_1 = 0·08$ for bell mouthed syphons $= 0·505$ for cylinderical syphons.

$f_2 = a\left(1 + \frac{b}{m}\right)$ where the values of a and b are to be taken as follows :—

Inner surface of barrel	a	b
Smooth iron pipe	... 0·00497	0·084
Encrested iron pipe	... 0·00996	0·084
Smooth cement or planed tubes	... 0·00316	0·10
Ashlar, brickwork or planks	... 0·00401	0·23
Rubble masonry or stone	... 0·00507	0·82

Lacey's formula :—

$$H = (1·08 + 0·008\ L\ R^{-1·05}) \frac{v^2}{2g} - \frac{u^2}{2g}$$

10. Open Channels and Uniform Flow : See Chapter VII, Part II.

11. Open Channel Variable Flow.

Loss of head through right angled bend $= 0·84 \frac{v^2}{2g}$

$H = \frac{v^2 \sin^2\theta}{134}$, where θ is the angle subtended by bend.

(*i*) To find the length L between two points where the depths are D_1 and D_2 (Fig. 2).
Let S' be the bed slope.
$h' = L.S'$, and $D_2 - h' + h = D_1$
or $h = D_1 - D_2 + LS'$

If $h_v = \frac{v_1^2 - v_2^2}{2g}$, then $h = \frac{v^2L}{C^2R} - \frac{v_1^2 - v_2^2}{2g}$

where v, R and C are the values of mean section between two points.

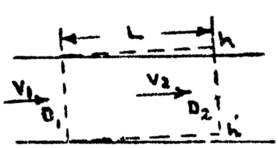


Fig. 22.

$$L = \frac{C^2R\,(D_1 - D_2 + h_v)}{v^2 - C^2RS'}$$

$$\text{or } v = C\sqrt{R} \times \sqrt{\frac{h + h_v}{L}}$$

The quantity h_v is always small compared to $D_1 - D_2$

To find surface slope S at any point.

Consider a point midway between the two sections and suppose them very near together so that the changes are very small. Let $v_1 - v_2 = v'$

Then $v_1^2 - v_2^2 = v'(v_1 + v_2)$

$$\therefore\ h = \frac{v^2L}{C^2L} - \frac{vv'}{g}$$

Let A be the cross section area and B = surface width at the midway points
a = difference in area in the length L

$$\text{Then } Q = vA = \left(v + \frac{v'}{2}\right)\left(A - \frac{a}{L}\right)$$

$$= vA + \frac{v'A}{2} - \frac{va}{2L} - \frac{v'a}{L}$$

Neglecting the last term, being very small.

$$vA = v'a \quad \text{or} \quad v = \frac{av'}{A}$$

$$\therefore \; h = \frac{v^2L}{C^2R} - \frac{v'^2a}{gA}$$

But $a = B(D_2 - D_1)$ and if d is the mean depth in the cross section
$A = Bd$

$$\therefore \; h = \frac{v^2L}{C^2R} - \frac{v^2}{g} \times \frac{D_2 - D_1}{gd} = \frac{v^2L}{C^2R} - \frac{v^2}{gd}(LS' - h)$$

$$\text{or } S = \frac{h}{L} = \frac{v^2}{C^2R} = \frac{1 - C^2(RS'/gd)}{1 - (v^2/gd)}$$

The difference in the bed slope S′ and the surface slope S.

$$S' - S = \frac{S' - (v^2/C^2R)}{1 - (v^2/gd)}$$

In order that the slope obtained by observing the water levels and the ends of a reach may agree with the local slope at the centre of the reach, the section area of the stream at the 2 ends of the reach must not differ, in ordinary cases, by more than 10%

The equations given above establish a direct connection between the depth at any or cross section and the surface slope at that section, but not the connection between the depth or slope at any section and the position of the section. To find that the profile must be worked out in that reach, by equation (*i*) for 'L' Fig. 23.

To find the length of a tangent from any point K to N where it meets the line of natural water surface.

Let D be the depth at K and D′ = Neutral depth

Let M N = x, GD = y

Then $y = xS'$ and $y + D' - D = xS$ $\qquad \therefore \; D - D' = x(S' - S)$

$$\therefore \; x = \frac{D - D'}{S' - S} = (D - D')\frac{1 - (v^2/gd)}{S' - (v^2/C^2R)}$$

Level bed.

$$L = C^2R\left(\frac{D_1 - D_2 + h_v}{v^2}\right)$$

$$\text{and } S = \frac{V^2}{C^2R} \cdot \frac{1}{1 - (V^2/gd)}$$

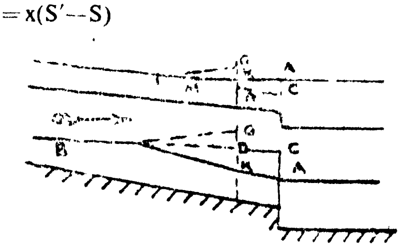


Fig. 23.

12. Strength of Pipes and Cylinders (Design of syphons)

Thin pipes : $t = \frac{p \cdot r}{f}$

r = Pipe radius (internal) in inches and p = Internal pressure in lbs./sq. inch
t = thickness of pipe in inches and f = Stress per sq. inch which the metal can bear.

Thick pipes subjected to considerable pressure.

r_1 = internal radius and r_0 = external radius

p_1 = internal pressure and r = any radius

In this case the the fibers near the centre are more heavily stressed than those further removed.

$$f = p_1 \frac{r_1^2}{r^2} \left(\frac{r^2 + r_0^2}{r_0^2 - r_1^2} \right)$$

$$f_{max.} \text{ (where } r = r_1) = p_1 \left(\frac{r_1^2 + r_0^2}{r_0^2 - r_1^2} \right) \text{ or } r_1 = r_0 \sqrt{\frac{f \pm p_1}{f - p_1}}$$

13. Units and Dimensions.

Although the essentials of doctrine of dimensions originated with Sir Isaac Newton, the first explicit formulation of the idea of dimensions, or the exponents of dimensions, of physical quantities appears to have first been made by Foureer in Theorie de la Chaleur, Ch. ii. Sec. 9, 1882.

It was not until the time of Stockes and Lord Raleigh, however, that any general use of this method of analysing the problems seems to have been made.

The method of solving certain problems by the use of dimensional analysis is rapid and simple and can be applied when the kind of physical system with which we are dealing, and the variables entering the equations, are known. It gives information in regard to the relations existing between measurable quantities associated with various phenomena. We must not overlook the fact, however, that there are limitations imposed upon its application and govern ourselves accordingly.

14. Fundamental Units.

The principal of dimensional analysis requires that all the terms of a correct and complete physical equation shall have the same dimensions *i. e.*, (measured in the same units) so that if the size of the fundamental unit is changed all the terms in the equation will be changed in the same ratio.

There are two kinds of quantities with which we must deal—dimensional and dimensionless.

A dimensional quantity is a quantity that can be expressed by one or more of the arbitrarily chosen fundamental units, which must be five in number for the treatment of our present body of physical knowledge.

These five fundamental units have been stated as ; 1 Length, 2. Time, 3. Mass, 4. Electrical permeability and 5. Temperature, for expressing respectively geometrical, kinematical, dynamical, electrical and thermal properties in some cases, however, it may be desirable to use a different choice of terms which might include such units as those of heat, force, etc.

A dimensionless quantity is a quantity whose numerical value does not change when the size of the fundamental unit is changed, so long as the relation between the derived and fundamental units is not changed. Commonly a dimensionless quantity is a ratio, as for example, a geometrical hydraulic slope, any ratio of two variable dimensions Reynolds' numbers etc., as has been stated above, reasoning by means of dimensions is very useful, simple and rapid, but there are nevertheless limitations in its application. These have been pointed out by Tolman as follows :—

(1) Equations obtained by reasoning in terms of dimensions always contain numerical constants, the values of which must be obtained by some other than dimensional sources, such as by experiment.

(2) In order to carry out dimensional reasoning, it is necessary to know or assume just what physical quantities are expected to be found connected by a physical equation. It is therefore necessary to have some knowledge of the subject being investigated, otherwise incorrect

results might be obtained by assuming that a particular phenomenon was dependent upon certain quantities that actually played no part or by ignoring some essential variable.

15. Derived Units.

It can be shown that measurements of all physical quantities with which we have to deal may be made in terms of independent units, known as fundamental units, which for our present purpose will be confind to the three common units of length, mass and time, represented by L, M and T respectively. In the case of thermal quantities we must add a fourth fundamental unit, temperature, which we shall express by θ.

Derived units are those which depend on the chosen fundamental units. Fundamental units and derived units are shown in a separate table on next page.

16. Method of Solution of Problems By Dimensional Analysis.

After we have determined the measurable physical quantities. Q_1, Q_2, Q_3 etc., in which we are interested and which we have reason to expect are of importance in describing a certain phenomena, we try to describe by an equation a relation among these physical quantities of n different kinds, in the form.

$$F(Q_1, Q_2 \ldots\ldots\ldots\ldots Q_n) = 0 \qquad (1)$$

F being the ordinary mathematical term meaning "Function of" Buckingham has given a very explicit method of procedure to follow after the physical quantities entering the problem have been determined. We shall make use of this method in solving a typical example to show the necessary steps to be followed.

He shows that if the equation

$$F(Q_1, Q_{12} \ldots\ldots\ldots\ldots Q_n) = 0$$

is a complete equation describing the relation between the n different kinds of quantities Q_1, Q_2, etc., it is reducable to the form

$$f(\pi_1, \pi_2 \ldots\ldots\ldots\ldots \pi_{n-k}) = 0 \qquad (2)$$

which Buckingham calls the π Theorem, in which each of the variables, π represent a dimensionless product of the form

$$\pi = Q_1^a, \; Q_2^b, \qquad Q_n^m \qquad (3)$$

K being the number of independent fundamental units (*i.e.* mass, length, time, etc.) needed in specifying the units of the n kinds quantities; and S is some unknown function to be found by experiment.

If there are n separate kinds of quantities, but more than one quantity of each kind, such as number of lengths, or a number of volumes, involved in the relation which is to be described by the equation, than all the quantities of any one kind may correctly be represented by specifying a single one of that kind and the ratios r′, r″................. of the others to this one. In this case equation (*i*) would be written in the form

$$F(Q_1 \cdot Q_2 \ldots\ldots\ldots\ldots Q_n, \; r', r'',) = O \qquad (4)$$

which in turn is always reducable to

$$f(\pi_1 \; \pi_2 \ldots\ldots\ldots\ldots \pi_{n-k}, \; r'. \; r'' \ldots\ldots\ldots\ldots) = O \qquad (5)$$

This π theorem as represented by equation (5) is a convenient statement for practical use, of the requirements that must be fulfilled in order to have dimensional homogeneity.

17. Laws of Hydraulic Similitude.

The fundamental condition necessary for geometrical and dynamical similarity between the two systems—model and prototype are :—

(1) That the distance between the mass elements of one system shall be proportional to the distances between corresponding mass elements of the other system.

(2) That the path of every mass elements of the one system shall be geometrically similar and proportional to the paths of the corresponding mass elements of the other system, in the same ratio as the corresponding lineal dimensions.

To the fundamental conditions of geometrical and dynamical similarity stated above we may add the following practice theorem as stated by Groat, which as he says, "throws an entirely new light on the theory of models, generally and above proves, in particular, that when hydraulic models are properly constructed and properly tested, they must produce, even in details, the performances of their prototypes."

Name of quantity.	Symbol.	Common English Unit.	Dimensional formula in terms of Mass. Length, Time.	Dimensional formula in terms of Force. Length, Time.
Length (any linear Dimen.)	L	Ft	L	L
Distance	S			
Depth	D			
Head	h			
Diameter	d			
Radius	r			
Hydraulic Radius	R			
Wetted perimeter	P			
Area	A	Ft^2	L^2	L^2
Volume	V	Ft^3	L^3	L^3
Kinematical quantities.				
Time	t	Sec.	T	T
Revolutions per time	n	R.P.S.	$\frac{1}{T}$	$\frac{1}{T}$
Angular velocity	ω	Radians/Sec.		
Angular acceleration	α	Radians/Sec.^{2}	$1/T^2$	$1/T^2$
Velocity	v	Ft/Sec.	L/T	L/T
Acceleration	a	Ft/Sec.^{2}	L/T^2	L/T^2
Acceleration due to gravity	g			
Kinematic Viscosity	$v = \frac{\mu}{\rho}$	Ft^2/Sec.	L^2/T	L^2/T
Discharge	Q	Ft^3/Sec.	L^3/T	L^3/T
Dynamical quantities.				
Mass	m	Slug	M	$[FT^2]/L$
Surface tension	σ	Lb/Ft	M/T^2	F/L
Stiffness	k			
Viscosity (Coeff. of viscosity)	μ	$\frac{Lb \times Sec}{Ft^2}$	M/LT	Ft/L^2
Pressure (unit stress)	p	Lb/Ft^2	$M/[LT^2]$	F/L^2
Modulus of Elasticity	e			
Specific weight (unit weigth)	γ	Lb/Ft^3	$M/L^2\,T^2$	F/L^3
Pressure gradient	i	$Lb/Ft^2/Ft$		
Density	ρ	$Slug/Ft^3$	M/L^3	$[FT^2]/L^4$
Impulse	I	Lb/Sec.	$\frac{ML}{T}$	FT
Momentum	M	Slug $\times$ Ft/Sec		
Force	F	Lb	ML/T^2	F
Weight	w			
Torque	T	Ft/lb	$\frac{ML^2}{T^2}$	FL
Energy	E			
Work	W			
Heat (Mech : Equivalent)	H			
Power	P	Ft $\times$ lb/Sec	ML^2/T^3	FL/T
Heat Flow (Mech : Equivalent)	H/t			

Dimensionless quantities.

Slope (Geometrical)	i	Froude's number	v^2/Lg
Angle (Geometrical)	ϕ	Thompson's number	$T \div a\rho$
Relative Elongation	0	Chezy's number	v^2/υ
Relative Detrusion	δ	Weber's number	v^2L/σ
Reynold's number	vL/υ		

When the velocities of homologous masses of floatage, water suspended materials, and detritus, flowing similarly in two geometrically similar channels, are proportional to the square roots of homologous linear dimensions, then according to kind, all the boundaries and internal force reactions of practical movement, in both streams, from distinct pairs of similar systems of forces, any two homologous of which are in fixed ratio, namely that of corresponding homologous extraneous forces; provided the conditions of flow are sufficiently removed from those of pure stream-line flow to admit the application of the ordinary theories of hydraulic friction, wind reactions and transportation of detritus.

Therefore when the theories apply, the similarity of motion must persist when once established.

18. Transference Equations.

Geometrical similarity.

(*i*) **Length.** In order that the two systems of model and full size structures shall fulfill the requirements that they be geo netrically Similar the following relation between corresponding linear dimentions must be obtained ; $L = \lambda L_m$ in which λ may be called the "scale of length."

(*ii*) **Area.** A consequence of geometrical similarity is this corresponding areas in the model must be geomentrically similar to those in the full size structure, or prototype.

The equation works out $A = \lambda^2 A_m$; where A = area in prototype and A_m = Area in the model and λ = scale of length.

(*iii*) **Volume.** $V = \lambda^3 V_m$. Where V = Volume in the prototype and V_m = volume in the model.

19. Mechanical Similarity (Weight)

In order to have dynamical similarity, it is necessary that we also have mechanical similarity.

Corresponding weights in the two systems must be mechanically similar. If W represents the weight in the prototype and W_m the corresponding weight in the model and the weight equals a volume times its specific weight, then.

$W = V \times \rho$ and $W_m = V_m \times \rho_m$

20. Kinematical Similarity.

(*i*) **Time.**

Another condition that must be fulfilled if the two systems are to be dynamically similar is that they be geometrically similar at all corresponding times while also their scale of length is a constant. This implies kinematical similarity between the two systems so that the time intervals in each must have a certain ratio which we shall denote by the Greek letter. "Tau" written τ.

$$\frac{T}{T_m} = \tau \text{ or } T = \tau \times T_m \text{ where } \tau \text{ must be constant.}$$

In the case of model experiments in hydraulics where is used in both systems and therefore the densities are equal and constant, we can easily show that $\tau = \sqrt{\lambda}$, as follows :—

$T = \sqrt{\lambda}\, T_m$

This is known as Froude's Law of Time scale for models.

(*ii*) **Velocity.**

If the two conditions of geometrical and kinematical similarity are fulfilled then the paths described by corresponding particles must be similar, and we can make use of these relations to derive the scales for corresponding velocities and accelerations. It follows that for two corresponding mass elements or particles whose paths are d_s and d_{sm} traced during the time intervals d_t and d_{tm}

$d_s = d_{sm}$ and $d_t = d_{tm}$

But velocity is a measure of the rate of travel and is equal to a distance divided by a time, in other words it is equal to the first derivative of the distance with respect to time, whence it can be derived that

$v = \sqrt{\lambda}\, v_m$

Hence, the scale of corresponding velocities of the two systems is expressed by this relationship which is also true of the components of the velocities. It may now be stated that corresponding velocities must be proportional to the square root of corresponding linear dimensions, in other words, to the square root of the "scale of length".

(*i i*) **Acceleration.**

The acceleration of a moving body may be expressed as the second derivative of distance with respect to time. It can be proved that.

$a = a_m$

21. Dynamical Similarity.

(*i*) **Force.** It has been shown that the two systems model and fullsize, may be geometrically and kinematically similar if the linear dimensions and times are taken according to independent scales, and that these two conditions are necessary for dynamical similarity, so that it can be derived that.

$F = \eta \frac{\lambda}{t^2} \times F_m$ where, $\eta = \frac{m}{m_m}$

This relation has been called. "The Newton's Law of Similarity", which may be stated as follows :—

Two material systems—model and prototype—are dynamically similar when the ratio of forces acting upon corresponding mass elements at corresponding times is constant and equal to

$$\eta \frac{\lambda}{t^2} = \omega$$

In the case of experiments with flowing water where the density and gravity are constant in both systems, so that $\frac{\lambda}{t^2} = 1$ then $\eta = \omega$

When several forces are acting at the same time to produce motion in the two systems, then the relation between each pair of corresponding forces must be constant and equal to a condition which cannot always be fulfilled, in fact the condition can be fulfilled completely only when but one kind of force acts to produce motion.

Since

(*i*) Mass

The relation for mass

$m = \lambda^3 \; m_m$

(*ii*) Discharge

$d = \lambda^{2 \cdot 5} \; Q_m$

(*iii*) Power or work done percent time

$P = \lambda^{3 \cdot 3} \; P_m$

(*iv*) Momentum

$M = \lambda^{3 \cdot 5} \; M_m$

(*v*) Impulse

$I = \lambda^{3 \cdot 5} \; I_m$

(*vi*) Work

$W = \lambda^4 \; W_m$

(*vii*) Energy

$F = \lambda^4 \; E_m$

(*viii*) Moments or Torque

$T = \lambda^4 \; T_m$

(*ix*) **Internal Friction (viscous flow).**

When the motions in the two systems -model and prototype, are due to viscous resistances and the force of gravity does not appreciably affect the resistance, then another law aplies called the Reynold's Law.

Relationship for kinematical viscosity

$$\lambda = \left(\frac{\Upsilon}{\Upsilon_m} \right)^{2/3}$$

The two systems, model and full size, in which only the forces of viscosity are considered as being present and acting and where the forces of gravity do not appreciably affect the resistance, are dynamically similar if corresponding velocities are directly proportional to the kinematic viescosity and inversely porportional to the linear dimensions.

$$\frac{vL}{\gamma} = \frac{v_m L_m}{\gamma_m}$$

Reynold was probably the first to make use of this relation and to determine values for the expression $\frac{vL}{\lambda}$ in connection with experiments on the flow of water in smooth pipes. For this reason it is now customary to express $\frac{vL}{\gamma}$ by the latter R where R is a dimensionless numerical quantity and is therefore the same whether metric or English system of units are used.

R is called "the Reynolds' number', and the law, stated above, is called "the Reynolds' Law for Corresponding Velocities", after the discoverer, Osborne Reynolds. We may, therefore, state that for the liquid motions in the two systems to be dynamically similar, the corresponding Reynold's numbers must be the same for each.

If liquids of the same viscosity, which implies that if they be of the same substance they be also at the same temperature, are used in both systems, model and prototype, then.

$$\gamma = \gamma_m$$

and consequently, a condition of similarity will be

$$vL = v_m L_m$$

22. Examination Questions.

Typical examination Questions usually set in the Federal Public Service Commission and the Punjab University Final Engineering Examinations.

1. State Bernoulli's theorem.

Deduce an expression for the discharge through a Venturimeter.

In a 12 inch Venturimeter the throat is 4 inches in diameter. Determine the difference in feet of head at the entrance and throat when discharging 500 gallons per minute. Take $C=0{\cdot}98$.

2. How would you determine the coefficient of contraction and coefficient of velocity for a sharpedged orifice ? What is the coefficient of discharge for such an orifice ? Which type of an orifice would you use in case of an outgoing pipe from a service reservoir, and why ?

A cylindrical cistern contains water 16 feet deep and is 1 sq. foot in cross section. On opening an orifice of 1 sq. inch in the bottom, the water level fell 7 feet in one minute. Find the coefficient of discharge.

3. Deduce an expression to determine the time required to empty a prismatic vessel to any depth through a rectangular notch under a variable head.

A tank with vertical sides and a water spread of one square mile is provided with a waste weir 50 feet long which discharges with a maximum depth of 3 feet of water on its crest. If no water enters the tank, find the time in which the surface of the water in the tank will be lowered by a foot and a half. Take $C=0{\cdot}577$.

4. State the laws of friction between a solid and a fluid, and deduce therefrom an expression for the head lost in overcoming the friction of a pipe.

A pipe is 6 inches in diameter and is laid for a quarter of a mile at a slope of 1 in 50,. for another quarter of a mile at a slope of 1 in 100; and for a third quarter mile it is level. The level of water is 20 feet above the inlet and 9 feet above the outlet end. Find the discharge, neglecting all losses except skin friction.

Draw the hydraulic gradient and mark the pressure in feet of water in the pipe at each quarter mile.

5. Explain the chief characteristic features of :—

(*i*) Thomson Turbine, (*ii*) Girard Turbine, and (*iii*) Pelton wheel. For what purpose is each most suitable ? Note the probable efficiency of each.

6. What is meant by 'Separation' in a reciprocating pump ? What is the limiting condition that separation' may not occur ? Draw an Indicator diagram for a single acting reciprocating pump showing the effect of acceleration in the suction and delivery pipes and also that of the frictional resistance of suction and delivery pipes. Clearly explain this diagram

7. Draw and explain the velocity triangles at the inlet and outlet of a Pelton wheel, adopting the usual notation.

A cup, similar to that in a Pelton wheel, deflects a jet of water through an angle of 120°. Determine the speed of the cup in terms of the velocity of the jet, so that the work done by the jet on the cup shall be a maximum. Also express this work as a percentage of the energy of the jet.

8. The following adjustages with a discharge opening of one inch diameter are successively applied to an orifice in the side of a vessel. The head over the centre of the orifice is 25 feet. Find the discharge in each case. Sketch the mouthpiece, showing how it is applied :—

(*i*) Internal cylindrical mouthpiece ; (*ii*) External cylindrical mouthpiece ; (*iii*) Orifice in a thin plate; (*iv*) Mouthpiece in the form of a contracted vein.

9. A lock 150 feet long and 16 feet wide is emptied by two sluices in the lower gate, each 2 feet deep, with their centres 3 feet above the floor of the lock. The levels of the water in the upper and lower reaches are 12 feet and 5 feet respectively above the floor of the lock. What must be the width of the sluices, so that the depth of the water in the chamber may be reduced from 12 feet to 6 feet in 2¼ minutes ?

10. A high level reservoir feeds two low level service reservoirs by means of a single main 5 miles long, 30 inches in a diameter and laid at a slope of 10 feet per mile. The main is then forked and one branch 2 miles long with a fall of 15 feet per mile serves one reservoir, whilst the other is served by a pipe 3 miles long with a fall of 12 feet per mile. Calculate the diameters of the branch pipes, so that each may deliver 4 million gallons of water during 24 hours. Take μ or $f=0{\cdot}06$.

11. Eighty-two horse-power is to be transmitted by hydraulic pressure at a distance of one mile. Find the diameter of the pipe and the pressure at a distance of one mile. Find the diameter of the pipe and the pressure required for an efficiency of 90%, when the velocity is 5 feet per second.

The frictional loss is given by the equation ; $h=[(\cdot 01\ v^2)/2g]\times 41/d$.

12. A rectangular flume 4 feet wide and 2 feet deep is roughly constructed of unpland timber and is required to deliver 80 cubic feet of water per second. Determine the necessary gradient. If it supplies water to a power house 5 miles away from the reservoir, determine the percentage loss in transmission, if the difference of level between the supply reservoir and the tailrace is 600 feet. Take $C=125$.

13. Describe by means of suitable sketches one type of each of the following. (*i*) A reaction turbine. (*ii*) An impulse turbine Compare the two in regard to their special uses and their efficiency.

14. What is meant by 'separation' in a reciprocating pump ? The plunger of such a pump moves with a simple harmonic motion The diameter is 12 inches and the stroke is 2 feet. The suction pipe is 9 inches in diameter and 80 feet long and the suction lift is 14 feet.

Calculate the maximum speed at which the pump can operate without "separation" occurring at the beginning of the stroke. Take effective height of the barometer as 28 feet of water.

15. Prove that the efficiency of a Pelton wheel is a maximum when the velocity of the cups is equal to half the velocity of the jet, if the frictional and other losses are negelected. Draw a velocity diagrams for the inlet and outlet of water in the case of a Pelton wheel.

A Pelton wheel has a mean bucket speed of 40 feet per second and is supplied with water at the rate of 150 gallons per second under a head of 100 feet. If the bucket deflects the jet through an angle of 160°, find the horse-power and the efficiency of the wheel.

16. How does a triangular notch compare with a rectangular notch for measurement purposes ?

Deduce an expression for the discharge over a triangular notch. What does this become if the angle of the notch is 90° ? Calculate the discharge if the head is 15 inches and the coefficient of discharge is 0·61.

17. Deduce an expression to determine the time required to discharge water from one prismatic vessel into another.

Two docks with vertical walls have superficial areas of 10 acres and 6 acres, and communicate with each other by gates containing two sluices each four feet square with their sills at bed level When the water in the large dock is 29 feet and in the smaller a depth of 4 feet, the shutters are opened. After what interval will the water attain the same level in both the docks and what will then be the depth; 1 acre = 4,840 sq. yards.

18. Deduce the Chezy formula for uniform flow in channels.

An irrigation channel has a gradient of 1 in 2,000, a bottom width of 16 feet, and side slopes of 1 horizontal to 2 vertical. If the depth of water is 4 feet and the values of C in the Chezy formula is 90, what is the mean velocity and the discharge in cusecs ?

19. Prove that the hydraulic mean depth (H M.D.) is half the depth in the case of earthen channels of a trapezoidal section with maximum discharging capacity.

A maximum discharging channel of best section has a depth of 8 feet and a fall of 2 ft. per mile. Calculate the discharge and the bed width. Draw a section of channel with side slopes 1 to 1 and dimensions the same. Take $C=74$.

20. Prove that in the case of transmission of power by means of water under pressure, the maximum power is transmitted by a pipe when the frictional loss of head in the pipe is one-third of the total head supplied.

The pressure at the inlet of a pipe is 1,000 lbs. per sq. feet and the pressure drop is 10 lbs. per mile. The pipe line is 10 miles. If 100 h.p. is to be transmitted over this line, find the diameter of the pipe and the efficiency of transmission. Take μ or $f=0{\cdot}006$

21. In a water power scheme the total head is 503 feet and 1,750,000 gallons of water are available per hour for utilization in an impulse turbine of the Pelton type. The proposed pipe line is 2 miles long.

Determine the diameter of the pipe required in order that the efficiency of transmission should be 80%. Also calculate the horse power. Neglect the losses at inlet to the pipe and at the nozzle. Take μ or f = 0·0075.

22. Explaine the terms (*i*) free vortex, and (*ii*) forced vortex.

Prove that the centrifugal heads-impressed on a revolving fluid is the difference between the tangential velocity heads.

A cylinderical arm full of water is rotated in a horizontal plane at 10 r.p.m. about one end. The arm is 2 feet long and its diameter is 2 inches. Find the centrifugal head impressed on the water and the total pressure on the outer end of the arm.

23. Define the terms "Unit Quantity", "Unit Speed" and "Unit Power" as applied to a hydraulic turbine. Describe the method of preparation of and the use of characteristic diagram for a turbine, the co-ordinates being "Unit Speed" and "Unit power".

24. Explain the underlying painciple, and describe, with clear diagramatic sketches the working of :—

(*i*) Hydraulic ram ; (*ii*) Reciprocating force pump ; (*iii*) Venturimeter.

25. Define Metacentric height.

A right cylinder of specific gravity S floats in water with its axis vertical. What is the relation between its length l and radius r for equilibrium to be stable ?

26. Two docks of areas a and b respectively are connected togather by a bell-mouthed sluice of area c situated below the lower water level. The difference in the water level in the two docks is H. Calculate the time in which level in the docks will be the same after the sulice is opened.

27. Describe the venturimeter, and drive its discharge formula. What are suitable ratios of throat area to pipe area and of length of approach and divergence ?

What precautions are necessary to ensure a good coefficient, and what may this be expected to be ?

28. Explain the phenomenon of separation in a reciprocating pump.

Drive a formula for determining when separation will occur with the following data :—

Suction head=h feet ; Length of suction pipe=l foot ; Area of suction pipe=a sq. ft. Piston area=A sq. ft ; Piston movement ; simple harmonic corresponding with an angular velocity ω radians per second ; and a crank length r feet.

29. Describe and explain the working of any two of the following :—

(*a*) Hydraulic ram ; *b*) Air lift pump ; (*c*) Johnson valve.

30. A pipe of cross section area a is suddenly enlarged to a cross-sectional area A. If water is flowing with a mean velocity v towards the larger section, deduce a formula for the loss of head at the joint.

31. What are the characteristics of turbulent flow, and under what conditions does it occur ? Compare laminar and turbulent flow in respect of (*a*) velocity distribution and (*b*) boundary resistance. What do you understand by (*i*) effectively smooth (*ii*) effectively rough boundaries ?

32. A wrought iron pipe, 1 foot diameter. 30 feet long, connects two tanks, the difference of level in which is 10 feet. The upstream end of the pipe protrudes into the tank, and the downstream end discharges below water level. There is a right-angled bend in the pipe and also a gate valve.

What will be the discharge of the pipe with the valve half open ?

33. State Bernoulli's theorem. How would you calculate the velocity head in a channel, knowing the velocity distribution ?

What is meant by 'critical' flow in an open channel ? In a rectangular flume of width b, what is the discharge when the critical depth is d ?

34. Describe the Pelton wheel, and state the conditions suitable for its use. What efficiency may be expected from this turbine, and how does it vary with load ? Why is it essential to maintain a constant speed in operating a Pelton wheel ? Describe different methods of securing this in practice.

35. Describe the principal forms of reaction turbines which have been used in the past. What are the advantages of the inward flow turbine which have led to its superceding the older types ?

36. A centrifugal pump, 1 ft. diameter at inlet and 2 feet diameter at outlet, 6' inches broad' at inlet and 4 inches at outlet, discharges 5 cusecs at 500 revolutions per minute.

Assuming all frictional losses to be 5 feet, determine the head pumped against. The vanes occupy 33 percent of the inlet periphery and their thickness of exit may be neglected. The outer vane ang'e is 30°.

37 (a) It is desired to cary off 150 cusecs of storm water in a circular concrete sewer of 7·0 ft diameter when flowing with depth equal to 0·8 D. Find the grade on which sewer should be laid. C in the Chezy formula is 131.

(b) If the discharge perunit of time over a flood spillway of the overfall type is 4,000 cusecs, what will be the discharge over a model of that spilway built to scals of 1 to 20 ?

38. (a) State the critical conditions of flow and prove that the specific energy in these conditions is minimum. What are other conditions in this type of flow ?

(b) The discharge perfoot run over the downstream glacis of the weir is 48 cusecs. The glacis slope is 1 in 5. The depth upstream of the jump on the glacis is 1·8 ft. Find the height of the jump and the energy loss in it.

39. Define "meta centre" and centre of buoyancy".

A cylindrical buoy is 6 ft. in diameter and 8 ft. high and weighs 1·8 tons. Show that it will not float with its axis vertical in sea water. If one end of a vertical chain is fastened to the centre of the base, find the pull on the chain in order that the buoy may just float with its axis vertical.

40. Define the terms "coefficient of viscosity" and "Kinematical viscosity".

A shaft 5 in diameter runs in a bearing of length 10 ins., the two surfaces being separated by a film of 0·001 in. thick If the coefficient of viscosity of the oil is 1·57 C.G.S. units, find the torque necessary to rotate the shaft at 50 r p m. against viscous resistance of the oil.

41. A venturimeter having an inlet diameter of 3 inches and a throat diameter of one inch is used to measure the rate of flow of air through a pipe. Mercury U-gauges register pressure at the inlet and throat equivalent to 250 and 150 m.m. of mercury respectively. Determine the volume of air flowing through the pipe in cusecs. Assume that flow takes place between the inlet and the throat under a diabatic conditions ($\gamma=1{\cdot}4$), and that the density of air at the inlet is 0 1 lb. per cubic foot. The barometric pressure is 760 m m.

42. In order to provide a brake for stopping a large Pelton wheel, an auxiliary jet is provided which impings on the back of the bukets. If the jet diameter is 2 inches ; supply head at nozzle, 1600 ft. ; and if the pressure on the buckets is equal to that on a flat radial vane calculate the time to bring the turbine to rest. Initial speed of the buckets is 150 feet per second and the kinetic energy stored in the revolving parts is 10 million foot-pounds.

43. What is a "Turgo impulse Turbine" ? Explain how it deffers from other types of turbines. Give sketches and illustrations in support of your statments.

44. An inward flow reaction turbine has a runner of 18 inches external diameter and 12 inches internal diameter with a width at inlet of $1\frac{1}{2}$ inches. The vane angles at inlet and outlet are 90° and 15° respectively. The velocity of flow is constant and 8% of the passage is blocked by the vane-thickness. If the supply head is 140 feet, hydraulic efficiency 82% and the mechanical efficiency 85%, determine the speed, output horse power and water cousumption when the speed is adjusted to give radial flow at exit from the runner.

45. A centrifugal pump is required to lift 1500 gallons of water per minute against a head of 25 ft. The water is to enter both sides of the runner and to have a constant velocity of flow of 8 feet per second. The vane thicknesses occupy 10% of the peripheril area at inner and outer diameters which are in the ratio of 1 : 3. The vanes are to be inclind back at 45° to the tangent at exit. Calculate the internal and external diameters and the width of the runner. Volumetric efficiency=96% and manometric efficiency 66%.

46. A monoplane has wings of aerofoil section with ang'e of incidence of 6° and area of each 60 sq. ft. Calcu'ate the power required by it when travelling at 230 m p.h. Assume the propeller efficiency to be 72 % and the air resistance of all parts other than the wings to be 35% of the total wind resistance. What is the lift at this speed. K_L for 6°=0·45 and K_D=0·03 ; for air $\frac{\gamma}{S}=\frac{\cdot 05}{g}$ ft.

47. (a) State and prove the conditions which control the question of stability or instability of a floating vessel.

(b) A vessel has 12,000 tons displacement. By filling alternately with 20 tons of water the boats suspended from each side, the vessal was caused to heel so that the bob of a pendulum 20 ft. long moved through a total distance of 6 inches. The distance between the boats was 50 ft. What is the metacentric height of the vessel ?

48. A water turbine develops 500 h. p. under a static head of 430 ft. The turbine has an efficiency of 82%. If the length of the pipe carrying water to the turbine is 2,330 ft. what will be the minimum diameter of the pipe ? Pipe coefficient f=0 006. Mean velocity of water in the pipe in feet per second=14·5. Explain why in practice the pipe would hardly ever be made of this minimum diameter

49. Sketch and describe a Pitot tube suitable for measuring the velocity of flow in a pressure pipe line. Explain how you would use this to determine the mean velocity at a given cross-section

50 Water is led to a turbine through a penstock 6 ft. in diameter. The penstock has a right-angled bend of 8 ft. mean radius between the vertical and horizontal sections preceding the turbine. The head at the point where the vertical section joins the bend is 150 ft. and the water velocity in the pipe is 10 ft. per second The loss of head in the bend can be taken as 0·5 ft Find the force produced in the bend by the water passing through it.

51. A power station using Pelton wheel is to develop 15,000 b.h.p. under a head of 800 ft. The speed of the wheel has to be 500 r. p m. If the mechanical efficiency of the wheel is 90% and a jet diameter must not exceed 1/9th wheel diameter,calculate the number of jets and the number of Pelton wheels required for the station. Take coefficient of velocity for jet water=0·98, and ratio of wheel velocity and theortical jet velocity=0 48.

52. Draw a diagrammatic sketch through a hydro-electric turbine power unit from headrace to tailrace, showing the component parts of the unit. Explain briefly the function of each component part.

53. A three-throw pump is required to lift 1,500 gallons of water per minute to a height of 400 ft. The stroke of the pump is 2 ft. and the diameter of the rams 12 inches The mechanical efficiency of the pump is 85% and there is a water slip of 4%. The loss of head by friction in the suction pipe is 5 ft. per sec. At what speed should the pump run, and what power would be required to drive it ?

54. (*a*) A plate is immersed in fluid stream moving with velocity v feet per second relative to the plate. The plate is inclined at an angle 0 to the direction of flow of the stream. Show that the force on the plate normal to the direction of the stream $K_L A \rho v^2$ where A=plate area and ρ=density of the fluid.

(*b*) Use the formula deduced above to find the force exerted on a 9. sq. ft. plate immersed in a stream of water and inclined to the direction of flow of the stream at an angle of 30°. The plate is stationary, and the stream is moving at a velocity of 40 ft. per sec. Take K_L=0·25.

55. An equation for the viscous retistance of a fluid in a round pipe may be obtained by equating the force on the fluid due to the drop in pressure to the viscous resistance. Working in this way prove that the complete form of the equation for the flow of fluids in pipes is

$$\frac{R}{\rho v^2}=\left(\frac{dv}{r}\right)^n$$

where R=viscous resistance per unit area of wetted surface, v= mean velocity of flow in pipe, ρ is density of the fluid and r is kinematic viscosity.

56 A centrifugal pump has a total lift of 50 ft. from well to delivery tank. The wheel is 5 ft. above the well water surface. The velocity of delivery from the uptake is 5 ft. per second ; radial velocity of flow through the wheel is 10 ft. per second ; the tangent to the vane at exit from the wheel makes an angle of 120 deg. with the direction of motion ; the water enters the wheel radially. Find (*a*) velocity of the wheel at exit, (*b*) the pressure head at exit from the wheel, (*c*) the velocity head at exit from the wheel and (*d*) the desirable direction for the fixed guide vanes. Neglect friction and other losses.

57. A centrifugal pump delivers 1500 gallons of water per minute against a head of 40 feet. The diameter of the impellers is 18 inches and the pump runs at a speed of 56 r. p. m. The water enters the wheel radially and the impeller wheel at inlet is 10 inches diameter. The vane angle at exit is 30° and impeller has a constant breadth of 2 inches. What is the pump's manometric efficiencey ?

58. It has been suggested that water-turbine blading should be made of suitable aerofoil section similar to that used for aeroplane wings, instead of the concave circular sections

generally in use. What conditions govern the use of these aerofoil sections as blades in water-turbine ? Critically discuss the suggestion.

59. (*a*) Explain the meaning of "Unit power", "Unit speed" and "Characteristic curve" in connection wi h a reaction turbine. Describe the construction of characteristic curves, illustrating your answer by graphs. Explain the usefulness of the curves in predicting the results obtainable when the turbine is operating under conditions other than those for which it was designed.

(*b*) A turbine designed to run at 200 r. p. m. under a head of 18 ft. developed 130 h. p. at full gate opening. At what speed should it run under a head of 24 feet to give the same efficiency and what could then be the horse-power ?

60. What advantages are obtained by fitting a draft tube to a Francis Turbine ? Sketch in outline. three or four types of draft tube as commonly used and detail reasons which may lead to their adoption in a friction turbine

Water enters a draft tube with a velocity of flow of 24 feet per second. The regain of pressure head in the draft tube is 6·26 ft. What is the efficiency of the draft tube ?

61. Deduce an expression for the pressure head due to the acceleration of water in the suction pipe of a reciprocating pump.

The suction pipe of a single acting reciprocating pump is 4" diameter and 20 ft. long and the suction level of the water is ft. below the pump. The pump cylinder is 8" diameter and the stroke of the plunger is 12". The pump has an air vessel fitted at the same level as the cylinder and at a distance of 4 ft. along the delivery of pipe. If the plunger moves with a Simple Hermononic Motion and the pump speed is 25 r. p. m. find the pressure in the pump cylinder at the beginning of the stroke, $f=0{\cdot}1$

62. What do you understand by the "buoyancy of a liquid" and the "centre of buoyancy" of a floating body ?

A buoy marking a submerged rock at sea is an iron cylinder 6 ft. in diameter and 8 ft. long and weighs 1·8 tons. Show that this buoy will not float with its axis unless a force is applied to an anchor fixed to the base of the buoy.

63. A "Barkers mill" which does work by the reaction of a jet is running at a speed 220 r. p. m. The nozzles are placed at a radius of 18". The mill is supplied with 2 cusecs of water under a head of 8 ft.- Find the horse power and efficiency of this machince

(*b*) A jet of water under a head of 8 ft. impinges normally on a series of flat moving vanes arranged round the circumfernce of a wheel of mean diameter 3 ft., 2 cusecs of water strike the vanes every second and the wheel rotats at 70 r. p m. Find the horse power, that the wheel can deliver and its effieiency.

64. Consider a Girad turbine which is of the axial flow impulse type. The leading dimensions are ; Mean dia. of the vane circle $=6$ ft Breadth of runner of inlet $=8$ inches ; Guide vane angle $=30°$; Runner vane inlet angle $=60°$; Outlet angle $=30°$. If this turbine is put to work under a head of 500 ft. and allowing for continuous admission of water all round the runner, calculate the h. p. developed by the wheel when 15% of the area available for admisson of water is blocked by the thickness of the runner vanes.

65. An outword flow reaction turbine is 3 ft diameter at inlet and 4 ft. diameter at outlet. The turbine rotates at 255 r. p. m. and the moving vanes have inlet and outlet angles of 90° and 30° respectively. The radial velocity of the water at inlet is 18 ft./sec. and the discharge from the turbine is radial. Calculate the guide vanes, the total head under which the turbine is working and the hydraulic efficiency. Neglect friction losses.

66 The difference in surface levels between two reservoirs is 90 ft. The total length connecting the reservoir is 1 mile of pipe line. From the top reservoir an 8" pipe is taken for 880 yards, and the diameter is suddenly reduced to 6 inches for the remaninder of the distance. Determine the discharge to the lower reservoir in gallons per minute and show the magitude of the losses in head that take place in the system, including the loss at the mouth of the 8" pipe which is sharp edged. $f=\cdot01$.

67. Define "centre of pressure" and "meta-centre".

A battleship weighs 13,000 tons. On filling the ship's boats on one side with water (this weighing 60 tons and its mean distance from the centre of the both being 30 ft.) the angle of displacement, of plumb line was found to be 2°-16' ($\tan\theta = \cdot0396$). Determine the meta-centric-height for rolling displacements.

68. Define 'Hydraulic Jump'.

The discharge intensity along a weir is 40 cusecs per foot run. The jump takes place on a sloping floor, 1 in 5 The depth of the jet upstream of the jump is 2·5 ft. Find the depth of water downstream of the jump and the loss of energy in the jump.

69. Define "Reynold's number" and "critical velocity".

The density of a fluid A is 0 8 and its coefficient of viscosity is 0·01 in C.G.S. units. The density of a second fluid B is 0·6 and its coefficient of viscosity is 0·(05. Which fluid will have lower critical velocity under given conditions of flow ? What will be the ratio of the critical velocities ?

70. A tank with vertical sides and hriorizontal cross-sectional area of 20 sq ft. is provided with a notch cut at the top of one of the sides. Water owing into the tank at a constant rate was discharged over the notch, the head over the bottom of which was 6 inches. The supply of water was suddenly stopped and it was observed that the head over the notch. started to fall at the rate of 0 14 inch per second. When the head had fallen to 3 inches it was found that it was falling at the rate of 0·05″ per second Estimate the rate of inflow into the tank when there is a Steady head of five inches over the notch.

71. Describe the working of any one type of current meter you are familiar with and show how it is used for river gauging to determine velocities. Give in brief the precautions necessary for its proper upkeep.

72. A channel of 50 ft. bedwidth, 1 to 1 side slope and 6 ft. deep carries a discharge of 500 cubic feet per second. The slope is $\frac{1}{10,000}$. The condition of the channel is suce that N in Manning's formula is 0·25. A weir in the channel of the full bed width heads up the water by two feet. How far upstream of the weir will the level of the water in the channel be affected by the afflux and what will be the height above bed of the broad crested weir working in the free fall conditions ?

73. Explain briefly the reasons for placing "air-vessels" on the suction and delivery pipe of a reciprocating pump.

A single acting reciprocating pump has a plunger diameter of 12 inches and a stroke of 18 inches. The delivery pipe is 4 inches diameter and 160 feet long. Calculate the horse power saved in overcoming friction in the delivery pipe by the provision of a large air vessel. The speed of the pump is 50 r.p.m. Assume the motion of the plunger to be simple harmonic and f = 0·01.

74 A Jet propelled vessel has a wetted surface area of 900 sq. ft. and coefficient of resistance of 0·004. The area of the nozzle openings is 4 sq. ft. The speed of the vessel is 15 miles per hour. Determine (*a*) the maximum theoretical efficiency of the Jet as a propeller, (*b*) the Jet velocity and (*c*) the horse power of the engine required for the pump. Assume pump efficiency to be 52% and the mechanical efficiency of the engines to be 82%.

75. (*a*) Discuss briefly various advantages which a centrifugal pump possesses over a reciprocating pump.

(*b*) A centrifugal pump has an impeller 8″ outer diameter. and when running at 1500 r.p.m discharge 800 gallons of water per minute against a head of 56 feet. At that discharge the water enters the impeller without shock. The inner diameter is 4 inches, the vanes are set back at outlet at an angle of 3·°. The area of flow which is constant from inlet to outlet of the impeller is 0·4 sq. ft. Determine : (*i*) the manometric efficiency of the pump ; (*ii*) loss of the head at inlet to the impeller when the discharge is reduced by 50% the speed of rotation being unchanged.

76 An inward-flow-reaction-turbine having an overall efficiency of 85% is required to develop 2500 H P. The total head H, available at the entrance to the turbine is 62 feet, the peripheril velocity of the wheel is to be $0\ 94\sqrt{2gH}$ and the radial velocity of flow at inlet is to 0·40 be$\sqrt{2gH}$. The wheel is to make 300 r p.m., and the hydraulic losses are not to exceed 29% of the available energy. Discharge takes place radially. Determine (*a*) the angles of the guide and wheel vanes : (*b*) the diameter and width of the wheel at inlet and (*c*) the specific speed of the turbine.

77. Using 'Boundary Layer Theory' calculate the Reynold's number and the drag of the hull of large air-ship when travelling at a speed of 80 miles yer hour. The length is 812·7 ft.

the diameter 135·4 ft. giving an area on chord of 91,000 sq. ft. Find also the horse power required to overcome the hull-drag at this speed. Take $K_D = \cdot 0275\ (1/B)^{1/7}$

OR

A Kaplan water turbine bladings are aerofoil section similar to that use for aeroplane wings instead of the concave circular section generally in use. Critically discuss the advantages. An axial flow impulse turbine has a mean blade ring diameter of 5½ ft. and a speed of 150 r.p.m. The runner blades are of aerofoil section and are set with an angle of incidence of 8° to the axis of the runner. Assuming the relative velocity of the water impinging on the blade to be 90 ft., per sec and that its direction is axial, find the horse power developed. There are 25 blades each having a chord of 3·5 inches and a length of 4 inches. For the blades take $K_L = \cdot 51$ and $K_D = \cdot 038$.

78. What are the essential features of water turbine governor ? Give a diagrammatical description of the principle of operation of a modern oil pressure governor.

79. (*a*) What do you understand by 'Buoyancy' ? State the principle of floating bodies.

(*b*) A cylinder has diameter of 12″ and a relative desnsity of 0·8. What is the maximum permissible length so that it may float with its axis vertical.

80. (*a*) What do you understand by specific energy of a channel's cross section and Froud Number ?

(*b*) The depth of an ordinary earth channel is 4·0 ft. Side slope 1 to 1, slope 1 in 6,000 and the discharge 7,000 cubic feet per minute. Find bottom width of the channel.

81. (*a*) Describe Pitot tube and Hook gauge.

(*b*) Find the downstream height of a hydraulic jump occuring on a level bed when the upstream depth is 3·0 ft. and the velocity 35 ft. per second. Determine the uplift back pressure in the jump and the energy losses in the example.

PART VI
GENERAL SECTION
CHAPTER VI
Canal Plantations

1. Introduction.

Plantations are an important feature of the great canal systems of the Punjab. After the plantations of the Forest Department the canal plantations are the second biggest source of fuel wood and timber in this province. As such they are of great economic importance. Apart from the supply of timber and fuel wood for public and departmental use, canal plantations provide materials for bushing and staking and for the protection of banks against erosion. They also provide shade to the traveller and afford protection to the surface of *kacha* canal banks. Shady roads detiriorate far less rappidly than those exposed to the sun with a consequent reduction in maintenance costs. It is difficult to give a cash value to this protective effect of plantations but it is certainly large. And above all is the improvement in the landscape brought about by lines of trees along the canals.

2. Suitability of the Area For Plantation.

(*a*) **Climate.**

The climate of the Punjab is characterised by extremes of temperature. The canal irrigated plains of the Punjab may be divied into two rainfall zones, as shown in the index map below.

(*i*) The arid zone with less than 15 inches annual rainfall and

(*ii*) The moist zone with more than 15 inches annual rain.

From Plate I, it will be seen that the greater part of the canal irrigated tract lies in the arid zone.

(*b*) **Soil.**

The following types of soil are met with in the Punjab plains :—

(*a*) Sandy loams or clay loams ; such soils are fertile and are suitable for tree growth.

(*b*) *Kallar* affected soils which are generally unfit for plantation.

(*c*) *Kankar* soil also unfit for plantation purposes.

(*d*) Purely sandy or water-logged areas which cannot bear plantation.

The soil of canal banks is generally moist and except when a canal is in digging, or where the area is affected by *kallar* or is water-logged. it is quite suitable for the growth of *shisham* trees of good quality.

On high spoil banks good *kikar* plantations can generally be grown.

The moisture content of the soil due to infiltration from the canals is, as a rule, sufficient to support without further irrigation, any tree growth once it is established. This is a very valuable consideration as the trees would only require thinning and protection against browsing once their roots are established.

(*c*) **Channels suitable for plantation.**

The most important consideration in the selection of channels for plantation is the possibility of affording protection to the trees. The future of canal plantations depends mainly upon the protection that it will be possible to afford to them against damage by illicit browsing and lopping. Young plants, if browsed even once in their first year surfer a serious set-back in growth and if browsed a second time have no chance of future development. Therefore, it is absolutely necessary to exclude grazing and rigorously to protect young plants during the first 3 year of their growth. Channels which are frequently inspected by the officers should be given preference over unimportant channels.

Ordinarily those reaches of the canal where the width of land, available for plantation, is less than 30 feet should be ignored for the purpose of plantation. In those reaches avenues can, of course, be established.

Choice of species.

The basic principle in the choice of species of trees is to grow only those trees in a locality which are best suited to the climatic and soil conditions prevailing in the locality.

Shisham is the most suitable tree for commanded areas with sandy or clayey loam soils. It can also be raised economically by hand-watering in uncommanded reaches where the average annual rainfall is about 25 inches.

Kikar may be grown on uncommanded areas having an annual rainfall of 15 inches or more. It flourishes best on new spoils and can even grow in soils containing a small percentage of *kallar*.

In sandy tract with an annual rainfall of about 8 inches the species known as mosquito should be tried. Low lying areas can be stocked with willows, eucalyptus and *arjun*. Mango and *jamun* may be grown in canal reaches near rest houses where they can be properly looked after while *tun*, *bahera*, *simal*, eucalyptus and *arjun* may be grown for shade and ornamental purposes in the rest house compounds.

3. (A) *Shisham* Plantation in Commanded Areas.

(*a*) **Method of growing *shisham*.**

The easiest and the best method of growing *shisham* trees is from stumps (root and shoot cuttings) of one or two years old nursery grown plants, provided the new shoot can be protected for a year or two.

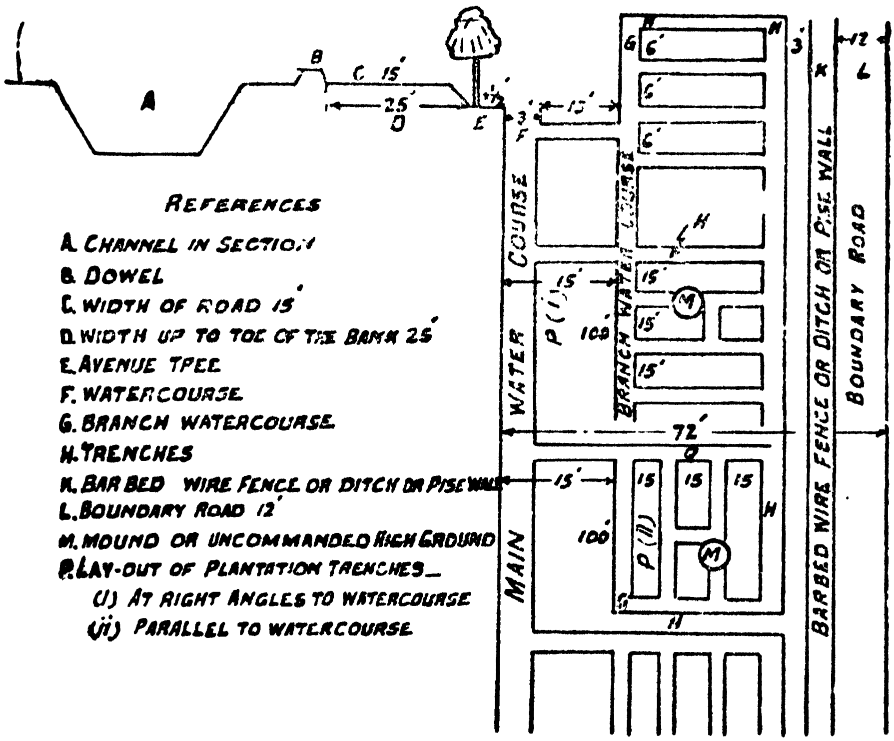


Fig. 1.

Direct sowings involve great skill, labour and time, are expensive, need more water and can only succeed in good soil. They do not flourish on canal spoil where the ground is very undulating.

Planting of entire plants is costly and results in inferior growth. It may be done only in places where protection of tender shoots from stumps is difficult.

(*b*) **Method of raising *shisham* stumps.**

For regular plantation work a canal division or a part thereof should be made self-sufficient in respect of the supply of *shisham* stumps. It is essential to have one or more local nurseries in order that stumps be available, bear at hand according to requirements and at the proper time.

(*c*) **Site for nursery.**

It is extremely difficult to find and ideal nursery site on canal spoil banks on account of their being so uneven. It should be located on the patrol bank side as far as possible. The site should be level and commanded from the canal and should consist of sandy loam soil. It should be easily accessible to inspecting officers. An old silting tank must not be selected as a nursery site as seedlings do not develop a tap root to form good stumps.

(*d*) **Lay out of nursery.**

A lay out plan of trenches for any type of plantation nursery is shown in Fig. 1.

Trenches of dimensions 12 inches top width, 8 inches bottom width and 9 inches deep should be dug 6 feet apart at right angles to the subsidiary water course which is connected with the main water course as shown in the sketch. The excavated earth should be thrown one foot away on both sides of the trenches so that the berms are left clear for sowing seed. The berms of the trenches should be made level before any sowings are carried out. The lay out should be completed before the end of March.

(*e*) **Seed.**

Shisham seed ripens in December—January and can be collected up to April, the best period of collection being the month of February. It should be collected from healthy well known trees by shaking branches on clean floor, and should be stored in a dry, well-ventilated room. Only fresh seed should be used.

60 seers of seed are required for an acre of nursery when showings are done on both sides of trenches.

(*f*) **Sowing.**

The sowing operation should be carried out from 15th March to 15th June, the earlier the better.

The soil should be loosened along both berms of trenches with rakes after a proper watering. All weeds and grass should be removed, and the seed pods should be sown breaking them into small segments, and covering them lightly with pulverised earth or preferably with silt.

Watering should be done carefully so as not to flood out the seeds which should receive moisture only throught the soil.

The soil should be kept moist till the seeds germinate. Irrigation interval depends on weather conditions. The colour of leaves is a good guide. A yellowish appearance shows either want of water or waterlogging. Too much irrigation is equally bad.

Germination commences after about 7 to 15 days depending upon the weather.

(*g*) **Weeding and thinning.**

Weeds should be removed as and when required. Young plants should not be allowed to get smothered. When plants are about 7 to 8 inches high, they should be thinned so that those that are left are about 4 inches apart. Thinning is best done after a watering when the soil is easily loosened.

(*h*) **Extraction and making of stumps.**

It takes a year for plants to yield stumps. Trenches should be flooded so that the soil becomes soft, when plants can be pulled out by hand. Shoots should be cut away leaving about a foot of the stem portion, and they should be heaped under shade. In case the soil is hard and plants too big and difficult to pull out, they may be taken out by deepening trenches and by cutting roots at about 12 inches to 14 inches below ground level by a slanting cut. Pulled out plants are then made into stumps by chopping off the stem and root without damaging the bark leaving 1 inch to 2 inches of the stem and from 9 inches to 12 inches of the root. The lateral roots should be cut off so that the remaining stumps look like truncheons The thickness at the collar (where root and shoot join) should not be more than the thickness of the thumb or less than the thickness of the middle finger of a man's hand. Stumps thicker than 1 inch and thinner than 1/2 inch in diameter should be rejected.

(*i*) **Carriage to planting site and storage.**

Stumps should be made into bundles, wrapped in leaves of grass, kept moist by sprinkling water and should be carried in gunny bags and planted without avoidable delay. They should not be kept immersed in running water nor should they be left in open to dry out but should be kept under shade with a grass cover and sprinkled with water frequently to prevent their drying out. If there has been delay in planting, cuts at both ends should be renewed by chopping off, with a cutter, an inch or less of the root and shoot portion just before planting. *Shisham* stumps can stand a long journey if not allowed to dry up. About thirty to thirty-five thousand stumps per acre may be expected from such a nursery, 100 stumps weigh from 6 to 7 seers. One coolie can easily extract and made 300 to 500 stumps per day.

(*j*) **Time for planting.**

The best time for planting *shisham* is in the spring, in the beginning of March. If irrigation water supply is available, the planting cperations should not be delayed beyond 15th of July.

(*k*) **Layout for planting.**

Trenches should be dug 15' apart and stumps put in 6 feet apart in rows. The usual size of trenches is 12 inches at the top, 8 inches at the bottom and 9 inches deep. The main watercourses are 3 feet wide at the top, 2 feet at the bottom and one foot deep, while the subsidiary watercourses are 15 inches wide at the top, 12 inches at the bottom and 8 inches at the bottom and 9 inches deep. The layout of trenches etc., is similar to that in the case of nursery.

(*l*) **Grading of trenches and watercourse.**

The whole success of a planting scheme depends upon adequate and even distribution of irrigation water supply to all the stumps planted. To achieve this, it is most essential to plant the stumps at a uniform level above bottom of the trench. An easy method of doing this is to give a trial watering and mark the maximum water level in the trenches. The berms of the trenches where stumps are to be planted should then be built by referring to this water surface level in the trenches. The depth of trenches should not be less than 8 inches. This depth will of necessity be greater in high ground.

As the canal spoil banks are very undulating, the inequalities of levels in planting sites and beds of trenches must be removed before planting operations are started, if full value is to be obtained from the plantation. This precaution should not be neglected by the overseer as the contractors are always reluctant to dig trenches according to specifications.

The grading of watercourses must be done with special care. It is useless to dig a small watercourse of more of less constant depth. as the ground is invariably uneven. Levels should be taken at 500 feet or even shorter intervals, and a proper longitudinal section prepared. Watercourses should be dug to correct levels in accordance with the approved longitudinal sections.

(*m*) **Planting in avenue line.**

Stumps should be planted along the berm of a watercourse 1½ feet from the water edge and 6 inches above the highest water level. The actual planting of stumps should be kept up after one or two waterings of the plantation area, so that the planting site becomes moist and the maximum water level in the water course is marked definitely. These precautions are necessary to avoid water-logging of the stumps.

(*n*) **Method of planting** ***shisham*****.**

Holes of depth of a little less than the length of the root portion of the stump are made by means of an iron rod pointed at one end, 2½ feet in length and 3/4 inch in diameter, the planting site having been watered and prepared the previous day. The stumps should be pushed in keeping only the shoot portion above ground. The moist earth should be pressed against the stumps to bring the earth in close contact with the whole of the root portion. The stumps should be irrigated soon after planting and the ground should be kept most until they sprout. Subsequent watering depends upon the capacity of soil to retain moisture.

(B) *Shisham* **plantations in uncommanded areas.**

(*a*) *Shisham* avenues and plantations can be successfully raised on high uncommanded areas by planting *shisham* stumps in pits and watering them by hand. The plants, if completely protected from damage by browsing, will generally need no watering after the third to fourth year of their life. Where conditions are eminently favourable, *i.e.*, high water table, good protection and adequate rainfall. Watering in the first year only may be sufficient.

(*b*) **Method of planting where annual rainfall is over 30".**

To reduce the expense of hand watering, circular pits of the shape of a shallow trough, 1½ feet diameter and 12 inches deep should be made ready for plantation by the end of February. For avenue lines these pits should be dug 10 feet apart but in plantation areas, the pits should be placed at distances of 20 feet in rows 6 feet apart.

Stumps should be planted carefully in loose earth in the centre of the pits during the month of March and should be watered immediately. Planting loose helps early development of shoots by retaining moisture and affording better aieation.

Wateiing by hand should be discontinued during thr rainy season and *beldars* should be employed to dig trenches 1 ft.×1 ft. and 3 ft. long on the upper slope to catch rain water as shown in Fig. 2.

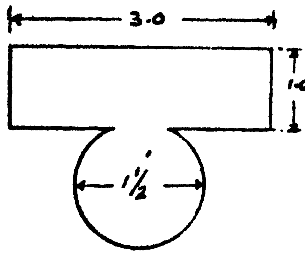


Fig. 2

One *beldar* should suffice to hand-water 300 plants, a day To begin with, the stumps may require a thorough watering every fourth day before sprouting, and every sixth to seventh day subsequently. No watering is required during rains, while three waterings per month are enough in the months of September and October. No watering is necessary in November to January, while a watering once a week is required during February and March and two waterings a month should be sufficient in April to July. Waterings may not be required later on if plants are healthy and have grown ten to fifteen feet high. In fact no rigid programme of waterings can be laid down as the ruling factor is the capacity of the particular soil to retain moisture as indicated by the condition of plants.

(*c*) **Method of planting where annual rainfall is less than 30″.**

Successful *shisham* regeneration can be achieved in the form of root suckers by severing the roots of the parent tree in the following way :—

Concentric circular trenches one foot wide and one foot deep should be dug around the stump of the felled tree at ten feet intervals, up to a maximum distance of 40 feet fiom the stump. The severing of roots induces a crop of suckers in the trenches, where the soil is good and the surface not much above the seepage level of the canal, the results are generally very satisfactory.

Where root suckers do not flourish *shisham* stumps should be planted in pits as described above.

5. *Kikar* **Plantations.**

(*a*) **Sowing.**

Where the soil is poor or the sub-soil water level is too low to support good *shisham* growth, *kikar* should be planted. Trenches four feet long, one foot wide and one foot deep should be dug and *kikar* sown on the berms and lightly coveid with earth just before the break of monsoon rains.

On unstable soil or on slopes, *kikar* seed should be sown in circular pits, about 1½ feet diameter, 6″ to 9″ deep, 6 feet apart and in rows 10 feet distant. If the young seedlings are congested, they should be thinned out, but only after they have passed through one hot season.

The best results are obtained when *kikar* seeds have passed through the goat's stomach and therefore, if possible, seeds should be collected from goat pens.

(*b*) **Protection.**

Kikar ranks next to *shisham* in economic importance. *Kikar* wood yields excellent firewood and charcoal. The timber is also of value for country cart wheels, etc., while its bark is always in demand by the tanning trade. Being prized by goat-herds for fodder and by *zimindars* for fencing it is subject to merciless lopping throughout its life and as such requires great attention towards its protection.

6. **Plantation in Compounds and Vicinity of Buildings.**

(*a*) **Species suitable for compounds and in the vicinity of buildings.**

As a variation from *shisham* and *kikar* plantations, the following trees of other species are generally grown in or near the compounds of rest houses or residential buildings.

(1) Mango (2) *Neem* (3) *Simal* (4) *Tun* (5) *Jamun* (6) *Arjun* (7) *Bahera* and (8) Eucalyptus.

(*b*) **Mango.**

Mangoes may be grown in the compounds of buildings and along reaches of canals in the vicinity of rest houses, gauge reader's huts, telegraph stations, etc. Mangoes should never be grown on berms of channels. Roots of mango trees require well drained soil and cannot stand water-logging. Not only is the fruit wasted by falling into water but the development and longevity of trees also suffer a great deal.

Mango trees should be planted at least 50 feet apart in the case of *desi* varieties, while in the case of grafted varieties a distance of 30 feet is considered sufficient. For the full development of mango trees it is essential that the get plenty of sunshine.

For planting purposes, circular pits of 4 feet diameter and about 4 feet deep should be dug at least a month or two before the commencement of the planting season, from 15th of February to end of March These pits should be filled with a mixture consisting of equal proportions of silt and rich surface soil and should be watered leberally before planting trees to allow the earth to settle fully The plants should be put in with the surrounding earth well rammed to prevent any further settlement. The top level of the earth in the pit should not be more than 6 inches below the natural surface level to avoid water-logging.

Protection against frost is most essential in the early stages of the growth of mango plants. Care should, however, be taken to see that the protective cover do not cramp the plants and that the crown is kept quite free to grow.

(*c*) *Neem.*

Seeds should be collected from trees, when thoroughly ripe, in the month of July and should be sown as soon as possible after collection. Seeds should be covered lightly with earth and watered sparingly. The soil should be kept loose to avoid caking.

Plants can be transplanted when 3 or 4 inches high or they may be kept another year in the nursery and planted with balls of earth attached to the roots. Root and shoot cuttings can also prepared and planted like *shisham.*

(*d*) *Simul.*

Seeds should be collected when ripe and should sown as soon as possible after collection in May on berms of trenches one foot wide and one foot deep. The seeds should be covered over lightly with earth and watering should be done by percolation through trenches.

The seeds take from one to three weeks to germinate. The seedlings should be kept well weeded and in case of congestion they should be thinned out.

The plants can be transplanted when two monthes old or they can be planted as root and shoot cuttings at the beginning of the following rainy season, the length of root being 8 inches and of the shoot about 3 inches.

(*e*) *Tun.*

Seeds should be collected from tree when ripe in about May and should not be picked off the ground. Seeds should be sown immediately after collection in the same manner as *shisham* seeds but in the case of *tun* seeds, great care has to be exercised in applying water in order to ensure that seeds are not washed away. Young seedlings should be screened off from hot sun in the middle of the day. Seedling should be kept 5 to 6 feet apart and soil should be loosened in the vicinity of seedlings.

Entire plants may be transplanted with balls of earth attached to the roots, or root and shoot cuttings may planted as in the case of *shisham* in the beginning of the following rainy season. Root and shoot cuttings may be planted even later, *i.e.*, during the following cold weather when the plants are leafless

(*f*) *Jamun.*

Seeds should be collected when ripe and sown immediately afterwards in the early part of the rainy season. Seeds should be sown in a nursery as in the case of *shisham* and the soil should be kept moist by percolation and not by direct watering. *Jamun* roots do not stand much exposure or injury during transplanting and therefore great care has to be exercised in the transplanting operations

(*g*) *Arjun.*

Seeds should be collected when ripe in April and May and should be sown along berms of nursery trenches immediately after collection as in the case of *shisham* trees. Seedling should be transplanted during the first rainy season with balls of earth attached to the roots.

(*h*) *Behera.*

Seeds should be collected when ripe from November to February and should be sown during March to April in the same manner as prescribed for *shisham.* Germination takes one to two months and seedlings should be transplanted during the first rains before the top root is too long.

(*i*) **Eucalyputs.**

Eucalyptus seeds should be sown in about the middle of March in bottomless pots, 6 inch size, filled with composite made of leaf mould and soil in equal quantities and very lightly covered over with ash. The pots should be kept under a light shade of trees in small nursery beds with cross bunds about 7 inches high, to hold water. If nursery beds are made in the open the plants have to be protected during the day time against sun by means of a covering made of grass or other similar material.

The soil in pots must be kept moist till germination, which generally requires about two weeks. The pots should be kept clear of weeds and only one healthy plant should be retained in each pot.

The plants will reach a height of 1 to 1½ feet by the end of July and are then fit for transplanting. The plants should be transplanted in pits previously prepared for the purpose and the whole of the contents of the pots, in which seedlings have been grown, should be carried to the new site along with the plant.

7. Maintenance of Plantations.

(*a*) **Protection.**

Protecting canal plantations against damage by illicit cutting and browsing is absolutely essential. Without proper protection it is impossible to raise any plantations worth the name. The increasing pressure of population on land renders protective measures more and more. difficult. The following methods for protecting plantations against illicit grazing, lopping and felling should be adopted wherever found necessary.

(*b*) **Against grazing.**

(*i*) Trespassing cattle should be taken to the nearest cattle pound.

(*ii*) Barbed wire fencing should be provided wherever possible. This type of fencing has the advantage that it can be used over and over again.

(*iii*) Pise walling is useful in combination with a ditch 3 feet wide and 1½ feet deep at a distance of about 1½ feet from the wall. Pise walling is useful only in those localities in which rainfall is low, otherwise it requires heavy maintenance and may prove expensive in the long run.

(*iv*) Boundary ditches with 5 feet top width and less than 3½ feet deep are only partially effective. Such ditches are useful in localities where cattle trespass is not extensive.

(*c*) **Against lopping.**

The prevention if illicit lopping is difficult without the application of statutory provisions of the Indian Forest Act. Lopping of trees for departmental purposes should in no case be done for more than half the height of the tree.

(*d*) **Against felling.**

Felling operations should be carried out systematically and care should be taken by the S.D.O. to see that no illicit felling are carried out.

(*e*) **Weeding.**

The excessive growth of grass suffocates the young shoots and provides a breeding place and shelter for the leaf defoliator. which is injurious for every type of plantation. The grass around the new shoots and plants should. therefore. be removed by roots and not merely cut, as soon as it begins to thereaten their growth, otherwise the young plant would suffer serious set dack in their growth.

(*f*) **Cleanings.**

In areas bearing a profuse crop of root suckets and coppice, it is essential to cut out the unnecessary shoots in order to promote the growth of promising stumps It is best to carry out this operation in December and January following the felling operations in the area. The shoots to be retained should be evenly spaced so as to permit sufficient growing space.

(*g*) **Pruning.**

Pruning of branches should be carried out only in the avenue line trees when they have attained a height of about 1·5 feet and are not less than two years old. Branches may also be pruned when they are interfering with traffic on the road.

Branches should be cut with a hand saw or a sharp knife quite close to the trunk without damaging the bark. The cut should first be made at the bottom portion of the branch and if the branch is a thick one, it should be first cut at about a foot from the trunk and the balance cut off next, taking care to make the cut first at the bottom portion of the branch. Pruning should not be done in the top half of the tree.

The plantation areas, the pruning of the lower branches of trees should be carried out at the time of first thinning.

Thinning should be carried out under the direction and supervision of trained forest staff.

(*h*) **Re-Juvenation of useless** ***shisham.***

Plants which have suffered a set back in their growth due to defective planting or irrigation or repeated browsing, or have become bushy or half dry, should be cut at the ground level in December or January. New shoots which are bound to be vigorous, will come up if adequate irrigation is applied The shoots must be properly protected against browsing if the new plants are to obtain normal conditions.

8. Exploitation.

(*a*) **Plantation register.**

The economic importance of canal plantations has been described in paragraphs 17·0 and 17 11. To maintain a record to this valuable property, a plantation register has been described which is kept in every sub-division of the Irrigation Branch. The instructions for the maintenance of this register are given on the fly leaf of the register and need to be carefully followed. The register is in two parts. In the first part is an account of the number of trees with their kind and in the second part an abstract is maintained which also shows the income obtained from plantations from year to year.

(*b*) **Sale of dead and fallen trees.**

To get full money out of the canal plantations the sale of dead and dying trees should be made an annual feature in every sub-division A list of such trees should be prepared by the end of Septemder and sales executed in October when the demand for fuel wood begins to rise. The disposal of dead and dying trees should not be allowed to linger from year to year, as it results in loss of timber and consequently of revenue.

(*c*) **Felling and re-stocking.**

As soon as the plantation is mature enough to be exploited, a regular programme of felling and restocking should be introduced.

The marking of trees in the reach selected for the felling and re stocking should be completed by the end of July to enable sales to be notified and carried out in August. The volume of trees should be calculated in accordance with the standard methods of the forest department. It is necessary that all fellings should be completed by the end of December to enable atrangements to be made for the construction of watercourses, trenches, etc., which should be completed by the end of March, in order to commence fresh planting operations in April.

(*d*) **Advice of canal forest officer to be obtained.**

In framing and working out the programme of annual felling and re-stocking, the expert advice of the Forest Officer and his assistants attached to the Irrigation Branch should be obtained.

The actual carrying out of the programme is, however, the function of the officers of the Irrigation Branch and every attempt should be made to work according to the seasonal time table.

9. Bibliography.

1 Institute and Maintenance of Canal Plantations, Irrigation Branch, Pb., Publication, being a note dated 6th Aug., 1908, by J. Benton, Esq., C.I.E., C.E., Irrigation Works, Punjab.

2. Report on Canal Plantations by Mr. W. E. Elewett, I.F.S., I.B., Punjab Publication.

3. How to Grow *Shisham*, by P.N. Deogun, Divisional Forest Officer.

GLOSSARY OF ENGLISH TECHNICAL TERMS

A

Absorption loss - Loss of water from Irrigation Channels or reservoir or Irrigated fields due to the pressure of the depth of water into the unsaturated soil below and lost in soil evaporation.

Accretion of levels Converse of degradation or rerogression of levels. A rise in the specific levels of the bed of the channel at any site.

Acre foot—A unit of volume used in Irrigation practice. It means the volume of water required to cover an area of one acre to a depth of one foot. It amounts to 43,560 cubic feet.
Note :—A Cusec Day is 1·98 acre feet (ordinarily taken as 2).

Acre inch—1/12 of an acre foot.

Afflux—The rise above the natural surface of water in a channel caused by placing an obstruction in the waterway.

After bay—The tail-race of a water-power plant ; a pond or reservoir at the outlet of the turbines.

Alkali soils—Soils that contain harmful concentrations of the following salts :—
Sodium Chloride, Sodium Sulphate, Sodium Carbonate or Calcium Chloride.

Alkaline—Having the properties of alkalies Soils in arid and semi-arid regions often contain an unusual quantity of Soluble mineral salts which efflorence in the form of a powder or crust, following rain or irrigation There are two main classes of alkali, namely, "Black" alkali and "White" alkali. The former generally consists mostly of sodium carbonate which is greatly injurious to crops. It is termed "Black" alkali because water containing it dissolves humus and forms black patches on the ground surface. "White" alkali usually contains a large quantity of sodium sulphate (Glauber's salt) which is less injurious to crops.

Alluvial cone—Water deposited-material at place where a mountain stream debouches on to a plain ; a debris cone.

Anaerobic—Able to live in the absence of free oxygen.

Angularity correction—The correction to be made to an observed velocity when the direction of the current is not exactly at right angles to the discharge section line.

Annual intensity - The term applied to the percentage of the culturable irrigable (commanded) area irrigated during the year to the total C.C.A on an outlet or a channel. The project intensity is the annual intensity aimed at in the project.

Annual mean monthly range—The difference in feet between the highest and lowest mean monthly gauge readings or reduced levels in any irrigation year, 1st of April to 31st of March at a particular site.

Annual mean monthly range averaged or smoothed over five years—The average of 5 successive annual mean monthly ranges (plotted in the graphs at 3rd or central year).

A.P.M. (adjustable proportionate module) - A semi-modular orifice outlet with an adjustable roof block, designed to distribute small variations in supply of the parent proportionately when set at correct level relative to the depth of the parent.

Apron—A floor or lining of stone, concrete, masonry, etc., to protect a surface from erosion, or to withstand hydrostatic pressure.

Aqueduct - An artificial channel of masonry, concrete, iron or wood, in which water flows with "free" surface, constructed across a valley, drain, canal, river, road or railway.

Arched dam—A curved dam, convex upstream, that depends on arch action for its stability. The load is transferred by the arch to the canyon walls, or other abutments.

Area assessed—The area irrigated on which water rates are levied.

Area irrigated—The area to which water has been applied

Area leased—A term used in Bengal for the area, for which cultivators have executed an agreement for taking canal water for a certain period.

Area matured—The area irrigated upon which crops have matured.

Area remitted—The area irrigated, for which water rates are remitted owing to failure to mature or other reasons.

Arid—A term applied to lands, or climates, that lack sufficient water for agriculture, without irrigation.

Authorized or designed full upply discharge—The maximum discharge for which a channel is designed. In irrigation practice the authorized full supply discharge should never be exceeded.

Automatic gate – See tilting gate.

Available supply—(*a*) *In the river.* The discharge passing at the moment.

(*b*) *In a tank* The quantity of water stored in the tank above the cill of the lowest sluices or the minimum authorized water level.

(*c*) *At the head of a canal.* The authorized share of the river discharge pertaining to a canal.

(*d*) *Other channels.* The discharge flowing.

Average supply—The average supply in a channel is the sum of the daily discharges at the head of the channel divided by the number of days when the channel is in flow. Known in Burma open "Average"

Avulsion –The breaking through by a river across the narrow neck of a horse shoe bend, or an entire change in the course of a river when it breaks through one of its bank in a deltic region.

B

Back water curve—A particular form of the surface curve of a stream of water which is concave inward. It is caused by an obstruction in the channel such as a weir or a regulator.

Baffle—A cross wall built at a distance of about five times its height from the toe of a fall or weir, to dissipate kinetic energy. (Compare with deflector).

Baffle tooth—A tooth-like projection on an apron, or other surface, to deflect or break the flowing water.

Balancing tank—A subsidiary reservoir for storing excess river water which is utilized during periods of short supply.

Bar—A deposit of river-borne material at the mouth of a river.

Barrage—A weir equipped with a series of sluice-gates to regulate the water surface level above it. Known as a Regulator in Madras.

Base, base days, or base period –The number of days in a crop. For example in the Punjab it numbers 183 for *Kharif* and 182 for *Rabi*. In the Bomby-Deccan, there are three base periods, *viz.*, 15th February to 14th June, 15th June to 14th October ; 15th October to 14th February. In Burma, the base periods are known as "Crop Periods" and are 183 days for main rice crop and 151 days for the winter crop.

Base exchange capacity—The important bases of soils are lime, magnesia, soda and potash. If a salt solution of one base is applied to a soil of another base, there is a rapid exchange of bases until equilibrium is established. If the soil is treated with salt solution several times, practically the whole of the bases originally present in the soil will be replaced by that of the salt solution applied. The base exchange capacity of a soil, is a measure of the extent to which such action will take place.

Base exchange reactions—The chemical actions which take place when bases are exchanged. See Base Exchange Capacity.

Basic sub-soil pressure "B.S.P."—Basic Sub-soil Pressure is defined in practice to be the manometric psessure registered by a strainertipped pipe which has been sunk along side a very deep standard pipe in the same or a directly connected stratum, till it records a reading not appreciably different from that in the standard. The level, thus recorded, is almost uninfluenced by the effect of the distortion of stream lines caused by the presence of a source or a sink. In rectilinear flow with no upward and downward movement, the B. S. P level and the phreatic surface are the same. A B.S.P. pipe responds to the water table changes uneffected by the vertical component of flow.

Basin irrigation –A method of irrigation by which land surrounded by natural or artificial banks is flooded, and when the water dries up crops are sown.

Bed load—The quantity of silt, sand, gravel, or other detritus rolled along the bed of stream, often expressed as weight or volume per time.

Bell's bunds—Guide banks for training a river at the site of a bridge or weir. Named after Mr. J.R. Bell who designed them.

Berm—A horizontal ledge or sheet along an embankment or along the water side of a canal bank separating the water from the bank. It is generally self-formed in canals by silting.

Bernoulli's theorem—A proposition advanced by Daniel Bernoulli that the energy head at any section in a flowing stream is equal to the energy head at any other downstream section plus the intervening losses.

Binni's percentage—A term used in the Central Provinces to express the estimated percentage of run-off from, to the total rainfall on a catchment area according to the table prepared by Sir Alexander Binnie.

Block—The whole area of certain specified land to which the block system is applied.
(This term is used in Bombay-Deccan).

Blocks friction - See Friction Blocks.

Block system—A system under which a supply of water is provided for carrying on irrigated cultivation under certain conditions throughout a block for a period of years.
(This term is used in Bombay-Deccan).

Bore—A wave of water having a nearly vertical front, such as a tidal wave, advancing upstream as a result of high tides in certain estuaries ; a similar wave advancing downstream as the result of "cloudburst". or the sudden release of a lage volume of water from a reservoir. The bore is analogous to the hydraulic jump in that it represents the limiting condition of the surface curve wherein it tends to become perpendicular to the bed of the stream.

Bottom contraction—The reduction in the area of overflowing water caused by the crest of a weir contracting the nape.

Branch—This term is applied to a large channel taking its supply from the main line, but whose functions are the same, *viz.*, the supply of water to distributaries. (In Madras, the term is also used for a canal taking off from a branch canal and having a head capacity of not less than 300 cusecs)

Breast wall or face wall—When applied to Irrigation Practice, a wall generally of reinforced concrete or brick work, immediately above the face of a submerged orifice.

Brix—A measure of the sucrose content of sugarcane.

Broad-crested weir—An overflow structure on which the nape is supported for an appreciable length ; a weir with a significant dimension in the direction of the stream.

Bucket wheel—The revolving portion of a current meter driven by the force of the current and whose revolutions are an indication of the velocity of that current.

Bye-wash A surplus escape for a tank flush with the ground-level, constructed where, for a sufficient width along the perimeter, of the water spread contour, the natural ground surface is at F.T.L. and the ground falls away giving a suitable surplus escape channel. It is termed 'natural' when the ground at the site is sufficiently hard to stand the surplus being passed over it without protection. Where the soil is not sufficiently hard to withstand the erosive action of the water and protective work such as drystone pitching, or masonry is required, it is termed a 'praved or masonry escape'.

C

Caisson—A chamber, usually sunk by excavating within it, for the purpose of gaining access to the bed of a stream or other body of water. If the chamber is closed on top and the water excluded by air pressure, it is called a pneumatic caisson

Canal—A channel artificially constructed or maintained for the conveyance of appreciable quantities of water or for the purposes of navigation

Canal outlet—In Madras this term is used for what is called an escape in Northern India, and what is termed a canal outlet in Northern India (*i.e.*, a pipe through the canal bank to convey water to the fields) is termed a sluice in Madras, See outlet.

Capacity—(*i*) When applied to a channel, the authorized full supply discharge.
(*ii*) When applied to a reservoir or tank the Gross Capacity is the quantity of water stored between lowest sluice level and the level of the cill of the waste weir. Sometimes known as Effective Capacity in Madras.
(*iii*) When applied to an outlet, Capacity is the designed discharge when the parent channel is running with water-surface at designed full-supply level.

Capacity curve—The graph of the volume of reservoir, tank, etc., as function of elevations.

The capacity of reservoir can only be defined by reference to some definite elevation.

Capacity factor – The ratio of the mean supply to the authorized full supply or capacity.

Note :– (*i*) If it were possible to run a canal system at full supply discharge or closed, then the capacity factor and the time factor would be the same.

(*ii*) Owing to rotational working branches and distributaries do not run for the same number of days as either the main canal or as each other.

(*iii*) The volume of discharge of a channel is given in cusec days by the sum of the daily discharges for the period in question. It is equal to :—

(*a*) Average discharge multiplied by the number of days the canal is in flow.

or (*b*) Mean discharge multiplied by the number of days in the crop.

Capillarity—The action by which liquids rise or fall in fine (capillary) or hair tubes, which is due to the attraction of the molecules of the liquid for each other or for solid with which they are in contact. Water rises higher than any other liquid in capillary tubes, *e.g.*, in soils or plant tissues, and the smaller the diameter of the tube the higher it rises. The capillary factor differs for different soils depending upon their texture. If the water contains common salt, its rate of capillary rise may increase to the extent of more than 5 percent.

Capillary fringe - Capillary fringe is defined to be the pressure transmitting zone above the phreatic surface. At any point in this zone absolute pressure is "negative" that is, below atmospheric and is equal for the static conditions to the height of the point above phreatic surface.

Capillary meniscus surface—Meniscus Surface is defined to be the upper boundary of the capillary fringe. Immediately above it there is no pressure transmitting water. Above it are air at the atmospheric pressure and Funicular or pendular water round the points of contact of the soil particles.

Capillary water – Water held above the water table in soil by capillary force.

Catchment or catchment area—Water-shed or area from which rainfall flows into a drainage line, reservoir, etc.

Caving —The erosion of a river or canal bank by the undermining section of water.

Cavitation—A condition wherein a vacuum, to any degree exists as a result of flowing water. Complete cavitation obtains when the pressure within the affected part is reduced to that of the vapour-pressure of the water.

Centrifugal pump —A water-lifting device that utilizes the centrifugal force imparted to the water by a rapidly rotating runner. It is essentially a reversed inward-flow turbine, the water being admitted to the centre of the runner and discharged at its outer periphery. It is not a displacement pump and therefore, differs materially from a rotary pump.

Centrifuge moisture equivalant – Is the moisture content of the soil when subjected to a centrifugal force of one thousand times the force of gravity for one hour.

Chemical gauging (chemical hydometry) – A process of measuring the flow of water by ascertaining the resulting degree of dilution of a chemical solution of known saturation introduced into the stream at a known rate.

Chezy formula – An empirical formula expressing the relation between the velocity of water, the hydraulic radius, and the friction slope ; thus $V=C\sqrt{RS}$ in which V=velocity ; R=hydraulic radius ; S=sine of the slope angle due to friction ; and C=a coefficient.

See also Kutter's formula ; Manning's formula.

Chute –(1) A high velocity conduit for conveying water to a lower level ; (2) An inclined drop or fall.

Cipolletti weir—A contracted measuring weir, in which the sides of the notch have a slope of 1 horizontal to 4 vertical, to compensate for end contractions ; named after Cesare Cipolletti, an Italian engineer.

Cistern –A pool of water maintained to take the impact of water overflowing a dam chute, drop or other spillway structure.

Clay—According to the American standard, this represents soil particles under ·005 mm. diameter. The standard adopted by the International Society of Soil Science however, lays down the limit as particles under ·002 mm diameter.

Clay content—Weight of clay in a soil sample.

Coefficient of discharge – A ratio connecting an actual discharge with the discharge calculated according to a specific formula.

Coefficient of roughness or rugosity – A factor which has to be applied in formulae, *e.g.*, the Kutter's, Manning's, Bazin's formulae, when calculating the discharge of a stream, which varies according to the friction and other losses engendered by the character of the confining water perimeter.

Coffer Dam -- A wall or barrier built in the water so as to form an enclosure from which the water is pumped to permit free access to the area within.

Colloids—Soil particles smaller than ·001 mm. diameter.

Colmatage—The process of causing a deposit of silt on irrigated land which generally becomes fertilized by such a deposit

Command—The height at an outlet site of the water level in a channel above the general level of the land in the area to be irrigated from that outlet.

Commutator – The portion of a current meter containing the electrical contracting device for indicating single revolution of the bucket wheel.

Conductivity –The soluble salt content of soil is often ascertained by measuring the conductivity (electric) of a solution of the soil, since conductivity varies with the salt concentration.

Continuous-flow-irrigation - A system by which each irrigator receives his allotted quantity of water at a continuous rate, without resort to rotation.

Contracted weir—A measuring notch with sides designed to produce a contraction in the area of the overflowing water.

Controlled section—A section or a reach of a conduit, where conditions exist that make the water level above it a fairly stable index of discharge. A control may be partial or complete. A complete control is independent of downstream conditions and is effective at all stages.

Control point, control flume – A fall, so designed that the water surface level above it bears a fixed relation to the discharge passing. The level is usually fixed with reference to the authorized full supply discharge.

Controlled outlet—*A modular outlet* :- The term was introduced because the name 'module' was at one time unpopular with cultivators as they associated it with reduction in supplies, (Sind).

Controlled pipe outlet —An ordinary pipe outlet with the upstream socket fitted with an annular ring, or "plug". The diameter of the hole in the plug is suitable for the discharge required. Adjustment is affected by changing the plug. (Central Provinces).

Conveyance or transmission loss – Loss of water from a channel due to evaporation and absorption.

Core wall or diaphragm wall - A wall of masonry, sheet-piling, or puddled clay built inside a dam or embankment to reduce percolation.

Creep –The movement of water under or around a structure built on permeable foundations. See also 'Piping'.

Crest – (1) The top of a dam, dike, spillway, or weir frequently restricted to the overflow portion. (2) The summit of a wave, peak of a flood.

Crib Dam—A timber barrier built in compartments, which are filled with stone or other suitable material.

Critical Flow - A condition of flow in a channel for which the mean Velocity equals one of the recognized critical velocities in steady end viscous flow.

Critical, hyper flow Flow with velocity higher than the critical velocity.

Critical, subflow – Flow with velocity lower than the critical velocity.

Critical Depth—Depth of water in a channel corresponding to one of the recognized critical velocities.

Critical velocity—(1) Reynold's critical velocity is that at which the flow changes from laminar to turbulent, and where friction ceases to be proportional to the first power of the velocity and becomes proportional to a higher power, practically the square; (2) Kennedy's critical velocity is that in open channels which will neither deposit nor pick up silt ;

(3) the critical velocity in open flumes occurs when the energy of flow is minimum. In a vertical sided flume $V_{cr} = \sqrt{gd}$

Critical velocity ratio - The ratio of critical velocity for a certain order of silt to that estimated by Kennedy's formula for a standard silt.

Crop ratio—The crop ratio of *Kharif Rabi* ratio is defined as the ratio between the anticipated areas to be irrigated for these two crops.
In madras it is the ratio between first and second crops which are generally raised between June and November, and December and April respectively.

Culturable commanded area The gross area commanded less the area of unculturable land included in the gross area

Culturable irrigable (or C.C.A. in Sind) - The gross irrigable area less the area not available for cultivation, *e.g.*, village area, roads and isolated pathes of unculturable lands.

Culturable lift area—That portion of the culturable irrigable area which can be irrigated economically by lift only.
Note :— In Sind, Culturable Irrigable area = Culturable Commanded Area + Culturable Lift Area.

Current meter – A device for determining the velocity of flowing water by ascertaining the speed at which a stream of water rotates a vane or a wheel.

Curtain wall – A cross wall built under the floor of hydraulic structure with the object of dividing the work into suitable compartments or to provide cut offs.

Cusec—The Unit of discharge use in irrigation practice and means a rate of flow of one cubic foot per second.

Cusec day – A Unit of volume used in irrigation practice and means the volume of water resulting from a discharge of one cusec for one day (24 hours). It amounts to 86,400 cubic feet of water.

Cusec month – The volume of water resulting from a discharge of one cusec for one month.

Cut-and-fill—A process of building canals by excavating part of the depth and using the excavated material for the adjacent embankments. In a balanced cut-and-fill the excavated material is precisely enough for the embankments with an allowance for settlement.

Cut off—A wall, collar, or other structure intended to reduce percolation water along other-wise, smooth surfaces, or through porous strata. Also the difference between the water levels upstream and downstream of regulator, or the difference in levels between a parent and an offtaking channel. Also a channel excavated to reduce the length of the course of a stream or river.

Cut-off trench or key trench - An excavation in the base of a dam or other structure filled with relatively impervious material to reduce percolation.

D

Dam – A structure erected to impound water, thus forming a reservoir.

Dam arched—A curved dam, convex upstream that depends on arch and cantilever action for its stability. The load is transferred by the arch to the canyon walls or other abutments.

Dam axis—The axis of an arch dam is a vertical cylindrical surface which pierces through the top of the dam at the upstream face.

Dam buttress – A masonry structure which carries the water load from an impervious deck on the upstream face through buttresses or counterforts to the foundation.

Dam debris—A barrier built across a stream or channel to collect debris such as sand, gravel, silt, drift-wood.

Dam flat deck – This type of dam is made of a flat reinforced concrete slab, which transmits water pressure to a series of parallel buttresses which rest directly on the foundation or upon a concrete slab resting on the foundation material.

Dam gravity – A dam which depends, for its stability entirely on its weight. It may be straight or slightly curved in plan.

Dam gravel fill—It is an embankment composed of gravel or shingle with the downstream part made of relatively coarse matetial and the upstream or water side part made of finer gravel and sand.

Dam hydraulic fill – A dam composed of earth, sand, gravel, etc., sluiced into place, generally the fine materials are washed towards the centre for greater imperviousness.

Dam ultiple march —This type of dam consists of series of inclind arches supported by buttresses. The load on the upstream face is transmitted by the arches to the foundation through the buttresses.

Dam overflow—A dam designed to be over topped in floods.

Dam rock filled It is a modified from of the earth dam using rock of all sizes to provide stability and an impervious membrane on the upstream side to provide water tightness.

Datum – Plane of reference for elevation.

Dead storage level – It is the water level below which a reservoir is not depleted in order that the minimum designed head for the hydro-electric generation is reduced. The capacity below this level is for silt deposit.

Debris — Any material, including floating trash, suspended sediment, or bed load, moved by a flowing stream ; *detritus*, (See also ; Sediment in Rivers and Canals.)

Debris cone — An fan-shaped deposit, of soil, sand, gravel, and boulders built up at the point where a mountain stream meets a valley, or otherwise where its velocity is reduced sufficiently to cause such deposits, See also, Alluvial Cone.

Debris dam—(Also see under Dam).
A barrier built across a stream channel to collect debris, such as sand, gravel, silt drift-wood, etc.

Deficiency – The amount by which a series of quantities fall short of a given demand ; in other words, the deficiency of natural stream flow to meet a given irrigation demands determines the storage required, the additional supply necessary, or the limitation of the irrigable area.

Deflector — A deflector is similar in design no baffle, but its purpose is markedly different. A baffle is required primarily to dissipate energy ; a deflector to deflect the high velocity stream near the bed and causes a horizontal bed roller to form, which causes the downstream bed to bank up against the down-stream face of the deflector to 1 to 5 slope

Deflocculation – This is the reverse of flocculation. When sodium carbonate is added to calcium clay the particles disintegrate forming a stable suspension and the clay becomes impervious to water.

Degradation of level – A reduction of specific bed levels, Retrogression of bed.

Delta—An expression used in irrigation practice to mean the depth of water that would result over a given area from a given discharge for a certain period of time Alternatively, the delta may be defined as the total volume of water delivered, divided by the area over which it has been spread.
Note : – A cusec day on one acre results in a delta of 2 feet. It is clear that owing to the total losses in a channel, the delta will vary with the place at which the discharge is measured which should be stated thus ; at field. outlet or head of channel.
This terms also applies to the alluvial tract formed by the deposit in the sea of the sediment carried down by rivers.

Demand – (*a*) *At the outlet* :—The cultivators' water requirements.
(*b*) *At a head of a channel* :—The sum of all the useful discharges required, plus total losses (Punjab Indent)

Demand Factor—When applied to an electric Iastallation Demand Factor of a system or any part thereof is the ratio of the maximum demand of the system or part thereof to the total connected load on the system or port thereof under consideration.

Densification - Is one of the methods used in the consolidation of stabilised earth-work and consists in the application of such a load as penetrates the unconsolidated material and compacts the layer, from the bottom upwards in order that the material shall be consolidated uniformly throughout its thickness without stratification. For densification the soil is deposited in layers of specified thickness and at a particular moisture content.

Dentated sill – A notched sill at the end of an apron to check the force of flowing water and thus reduce erosion downstream by deflection of high velocity jet.

Depression—In a semi-module, the depth below supply level of some point of a semi-module fixed by its hydraulics such that as supply level varies, a constant co-efficient multiplied by the correct power of that depth, gives the discharge.

Depression ratio –The ratio between the depression and the height of the opening of an orifice outlet.

Designed or authorized full supply discharge—See under Authorized or designed ful supply Discharge.

Detritus—See Debris.

Direction float — A standard metal float carrying a small flag used for indicating the direction of flow of a river so that the angle that direction makes with the discharge section line at an observation point may be measured.

Direction peg line –The line parallel to the discharge section line on which the direction pegs are located.

Direction pegs-- The points through which rays from the observation points pass when converging on to the pivot point.

Direct outlet—An outlet constructed in a main line or branch canal.

Discharge –The rate of flow at a stated site, *i.e*, the quantity of water passing in unit time.

Discharge section line—The line along which depths and velocities of water are measured between two points one on each bank of a channel.

Discharge or velocity rod—See "Rod-Float."

Dispersion coefficient—The percentage of the total clay which passes into suspension on being left in contact with water for 24 hours.

Distributary - A Government channel taking its supply from a main line or branch the function of which is to supply water to minors and outlets. In Madras it is a channel taking off from a main or branch canal or from another distributary having a head capacity of about 25 and below 300 cusecs in the case of a major distributary and of 25 cusecs and under in the case of a minor distributary

Distributary proportionate –See Proportionate Distributor.

Distribution system—(1) The system of distributaries and their appurtenances, conveying irrigation water from the main to the farm units ; (2) any system by which a primary water supply is distributed to consumers.

Ditch –A channel constructed by the side of and parallel to the parent channel. (This term is not in general use in India.)

Ditch channel—A channel constructed by the side of and parallel to the parent channel, known in Madras "side channel".

Diversion dam, or intake weir—A barrier built for the purpose of diverting part or whole of the water from a stream into a different course.

Diversity factor—The diversity factor of any electrical transmission system, or part of a system is the ratio of the sum of maximum power demands of the Sub-Dns. of the system, or part of a system, to the maximum demand of the whole system or part of the system under consideration measured at the point of supply.

Divide wall, or groyne - A wall or groyne separating the undersluices channel from the main river with the object of facilitating silt regulation.

Dominant discharge—The dominant discharge is defined as that discharge which determines the meander length It is the discharge which controls the meander length and breadth. It appears to be slightly in excess of bank full stage.

Double crop—The raising of two successive crops on the same field in one irrigation season (in Tanjore, June to January). Applied generally in connection with rice irrigation. The crop that is first cut is called the "First crop" and the crop harvested later in the season is called the "Second crop". The term "Second crop land" is sometimes used loosely instead of "double crop lands."

Drain—A conduit or channel, either artifical or natural, for carrying surplus ground or surface water.

Drain tile—Pipe of burnt clay concrete, etc., in short lengths, usually laid with open joints to collect and remove drainage water.

Drainage—The process of removing ground or surface water by artificial means, or that which is removed by drains.

Drainage area or drainage basin—The area from which water is drained.

Drainage channel—A natural drainage when improved by "clearance' or by excavation.

Drainage cut or seepage drain—An artificial channel, deliverately excavated on a line which is naturally a drainage, for the disposal of water

Drainage Line, or drain—A conduit or channel either natural or artificial along which drainage takes place.

Drift—The distance in feet a discharge boat travels down-stream with the current whether anchored or not during the time taken to make a velocity observation.

Drop—See Fall.

Drop down curve—See Free Over-Fall curve.

Drowning ratio—The ratio of the tail water elevation to the head water elevation, when both are higher than the crest, the overflow crest of the structure being the datum of reference. The distances upstream or downstream from the crest at which head water and tail water elevations are measured have not been standardized but should be such that the levels are not in the influence of the work.

Dry crop—A crop which is raised entirely with the help of rainfall Also any crop other than rice or sugar in Sind.

Dual purpose canal—A canal constructed for the combined purpose of navigation and irrigation.

Duration curve—A graphical representation of the number of times given quantities are equalled or exceeded during a certain period of record. For example, if in a 10-year record of daily stream flow, the percentage of time the flow was above certain values (100, 200, 300 cu. ft. per sec., etc ,) was plotted against flow the graph would constitute a duration curve for that stream and period. From it could be read the percentage of time the flow was greater or less than any given value within the range that occurred during the period cited; the duration curve is the integral of the frequency curve.

Duty—Duty is the relation between the area irrigated and quantity of water required to irrigate it when applied to a channel; it is the area irrigated during a base period divided by the mean supply utilized in cusecs.
When applied to tank irrigation it is the number of acres of irrigation per million cubic feet of impounded water.

Duty on capacity (punjab full supply duty)—The full supply factor attained by a project system or channel after it has been opened for irrigation.

Duty of water—The relation between the area of land served and the quantity of irrigation water used. When applied to a channel, it is the area irrigation during a base period divided by the mean supply utilized in cusecs.

E

Eddy loss—The energy lost (converted into heat) by swirls, eddies, and impact, as distinguished from friction loss.

Effective capacity—See capacity.

Effective size (of sand or soil particles)—The effective size is the size such that 10% by weight of the particles are smaller than it.

End contraction—The contraction in the area of overflowing water caused by the ends of a weir notch.

Energy—The capacity to perform work ; kinetic energy is that due to motion ; and potential energy is that due to position. In a stream the total energy at any section is represented by the sum of its potential and kinetic energies.

Energy gradient—The slope of the energy line with reference to any plane

Energy head—The elevation of the hydraulic gradient line at any section plus the velocity head of the water at that section.

Energy line—See total energy (or T. E.) line.
Entrance or entary head—The head required to cause flow into a conduit or other structure; it includes both entrance loss and velocity head.
Entrance or entary loss—The head lost in eddies and friction at the inlet to a conduit or structure.
Escapage—The water escaped through spill way or otherwise discarded from an irrigation system after having been diverted into it.
Escape—A channel through which surplus or excess water may be removed from a canal, reservoir or stream (see spillway).
Evapo-transpiration—Combined loss of water from soils by evaporation and plant transpiration.
Exit gradient (Khosla)—Water exerts a certain residual force along its flow through the subsoil. This force has a vertical upward component tending to lift the soil particles. In the critical conditions, the weight of the particle just balances the upward uplift component. The grade of flow under these conditions is called the exit gradient. This is sometimes called 'flotation gradient (Terazaghi, Egypt) ; Bursting gradient (Heigh, Pb.) or critical gradint (Dr. Bose, Pb.)
Exchangable Bases—Those soil bases such as calcium, mangnesium, potassium and sodium which are readily exchangable when brought into contact with solutions of other salts.
Extreme limits of river oscillation—The belt within which a river has ranged within historic time.

F

Fall—A work designed to secure the lowering of the water surface in a channel and the safe distruction of the surplus energy. Known in Madras as a "Drop."
Feeder—A channel constructed primarily to convey water from one source of supply or system to another, or within the same system.
Field channel—See Irrigator's channel.
Fish ladder—A structure provided at a dam or weir consisting of a "ladder" of small steps or falls over which water is allowed to flow, to enable fish to proceed from the downstream to upstream side.
Flank wall—The retaining wall in continuation of abutments both upstream and downstream.
Flash-board—A baulk, needle, or plank usually of timber, which is placed horizontally and held by verticle supports on the crest of a weir or dam, or in a regulator, to head up water.
Field moisture equivalent—The maximum moisture content at which a drop of water placed on a smooth surface of the soil will not be immediately absorbed, but will instead spread over the surface and give it a shiny appearance.
Flexibility (outlet)—The ratio which the rate of change of discharge of the outlet bears to the rate of change of discharge of the supply or parent channel.
Float gauging—Measurement of the discharge of water by floats to determine velocities.
Float run—The fixed distance over which surface float is timed.
Flocculation—When calcium salts are added to the suspensions of sodium clays, the colloidal particles form aggregates and are precipitated. This is known as flocculation. The clay then becomes previous to water.
Flood absorption capacity—Is the capacity of a reservior between high flood level and the normal reservior level, provided for the absorption of floods.
Flood lift—The term is used in a Central Provinces to express the difference between the High Flood Level and the Full Tank Level.
Flood storage—A term used in Central Provinces to denote the volume of water that a reservior can accommodate between Full Tank Level and High Flood Level. Known in Madras as "Flood Absorbing Capacity."
Flow-hyper-critical—Flow at velocities greater than the critical.
Flow sub-critical—Flow at velocities less than the critical.

Flow irrigation or flow area—Area which can be irrigated from the source of water, by flow under gravity alone.

Flume—An artificially constructed water way.

Forebay—A reservoir or pond at the head of a penstock or pipe line.

Foundation mattress or raft—A slab of concrete, usually reinforced, placed over a yielding foundation to distribute the superimposed load.

Free board—The vertical distance between the full supply level in a channel and the top of the containing banks, or the vertical distance between the full supply level in a reservoir and the top of the dam or *bund*.

Free flow—A condition of flow through or over a structure not affected by submergence.

Free over-fall curve—The convex surface outlines assumed by flowing water enlarging from a flume or other construction of a channel.

Free water—Water in the soil in excess of hydroscopic and capillary water; also termed "Gravity Water".

Free weir, free overfall, or free fall weir—A weir that is not submerged in which the tail-water is below the crest or the flow is in way affected by the elevation of the tail water.

Friction blocks—Obstructions placed on the downstream floor of a weir or fall to introduce resistance to the jet so that the filaments of high velocity may be diverted towards the surface and so as to maintain the standing wave on the glacis.

Friction head (or loss)—The head or energy lost as the result of the disturbances set up by the contact between a moving stream of water and its containing conduit. In laminar or stream-line flow the friction head is approximately proportional to the first power of the velocity; in turbulent flow to a higher power practically the square. For convenience, friction losses are best distinguished from losses due to bends, expansion, obstructions, impacts, etc., but there is no recognized line of demarcation between them, and all such losses are often included in the term, "friction losses".

Friction slope—The friction head or loss per unit length of conduit. For most conditions of flow the friction slope coincides with the energy gradient, but where a distinction is made between energy losses due to bends, expansions, impacts, etc., a distinction must also be made between the friction slope and the energy gradient. Friction slope is equal to the bed or surface slope only for uniform flow in uniform channel.

F.S.L.—Full supply Level.

Full meander—Consists of two loops, one flowing clockwise and the other anti-clockwise. (Applies to rivers and stream).

Full supuly discharge—The discherge required for irrigation purposes (Madras). Also see Authorized or Designed Full Supply Discharge.

Full supply factor—The area estimated to be irrigated during the base period divided by the authorized or the designed full supply discharge of the channel at the head (F.S.F. at the distributary head) or at the outlet (F.S.F. at the outlet).

Note (i)—The full supply factor is assessed for the purposes of project making, in the light of experience. *Note (ii)*—Once a project is opened for irrigation the full supply factor attained is usally known as the Duty on capacity.

Full tank level (F.T.L.)—The level of the crest of the waste weir of the reservoir or tank.

Funicular water—B.A. Keen (The physical properties of the soil, page 116). Funicular water is held round the points of contact of the soil particles by the Funicular menisci due to molecular *affinity* of soil particles for water and is continuous. The air passages in the cellular pore spaces are also continuous. The range of pressure deficiency of funicular menisci is from 12·9 T/r to 6·9 T/r.

G

Gallery—(1) A Sub-surface collectar of percolating water; (2) a passageway; as in a dam; (3) and undeground conduit or reservoir.

Gauge discharge curve—The curve resulting upon the plotting of discharges against equivalent gauges.

Gauge line—The line across a channel passing through the permanent gauge in a fixed direction.

Gauge line pillars—The masonry land marks fixing the position of the gauge line.

Gauge permanent—A contrivance the position of which is never changed and against which the height above sea level of water surface levels are recorded in feet

Gauge temporary—Any temporary gauge fixed for the purpose of recording water surface level above sea level.

Glacis—The sloping floor below and in continuation of the raised crest of a weir.

Government channel—A channel or canal which is owned, maintained and controlled exclusively by the Government.

Grade or gradient—(1) The slope of a road, channel, or natural ground, (2) the finished surface of a canal bed, road bed, top of embankment, or bottom of excavation; (3) any surface prepared for the support of a conduit paving ties, twills, etc.

Gravel—Soil particles retained on a 10 mesh sieve which has openings 2 mms. in diameter.

Gravity dam—A dam depending solely on its weight to resist the water pressure.

Gravity water—Water that moves through soil under the influence of gravity.

Gross area—The total area within the extreme limits set for irrigation by a project system or channel.

Gross commanded area—This is total area which can be irrigated economically from a canal scheme on the supposition that unlimited water is available.

Note:—In Sind and United Provinces by economical lift also.

Gross head When applied to a dam. The gross head is the total fall or difference between the elevation of water surface in the diversion pond and that in the lower end of the tail race.

Gross irrigable area—The gross area less such large compact areas are excluded from the project by reason of their being unsuitable for irrigation either on account of the nature of the soil, or because the ground is too high to be economically irrigated by 'Lift''. In Madras it refers to the area that has been determined as capable of being irrigated under a project considering the water supply available and the designed distribution system.

Gross lift area—That portion of the gross irrigable area which can be irrigated economically by "Lift" only. Gross commanded area plus Gross Lift Area equals Gross Irrigable Area.

Ground-water, phreatic water or gravity water—Water below the water table.

Groyne—An obstruction of stone, timber or brushwood, constructed from the bank of a canal or river and projecting into it, for diverting the flow. Sometimes known as a Spur

Guide bank—A protective and training bank constructed at the site of a weir, bridge etc., to guide the river through the water-way provided in the tructure.

Gypsum—Calcium Sulphate ($CaSO_4$). Used for the replacement of Sodium by Calcium in alkali soils.

H

Head or head regulator—This term is usually applied to the control work constructed at the off-take of a channel subsidiary to a main canal.

Head race—A channel leading water to a water-wheel; a forebay (See 'Race' also).

Head wall—A wall built across a small channel and provided with a regulating arrangement to head up water on the upstream side.

Head works—The works constructed at the off take of a main canal. It includes the weir on the river, the dam at the storage site, etc.

High flood level (H.F.L.)—The highest level the water surface of a reservoir or tank. Also applied to streams and rivers. Known in Madras as Maximum Water Level.

Hollow dam—A barrier usually of reinforced concrete consisting essentially of slabs supported by transverse butteresses. The load is taken by the slabs and transferred to the foundations through the butteresses. (See also Multiple Arch Dam).

Hook-gauge—A pointed hook attached to a graduated staff or Vernier scale of measuring accurately the elevation of the surface of still water. The hook is submerged, and then raised until the point makes a pimple on the water surface.

Hydrated—Combined with water or the elements of water.

Hydraulic friction—The force exerted by the wetted perimeter or wetted border of a channel or conduit by reason of its roughness, tendings to retard the flow therein.

Hydraulic gradient - The slope of the hydraulic gardient line.

Hydraulic gradient line—In a closed conduit a line joining the levels to which water rises in pressure pipes (manometer tubes). In an open channel the hydraulic gradient line is the water surface In embankments, earth dams, and earthern structures retaining water it is the line of saturation in the earth.

Hydraulic jump - Defined to be the hydraulic phenomenon when there in a distinct jump of water accompanied by an impact between the rapidly moving stream upstream, and more slowly moving column of water downstream. The change takes place according to Newton's 2nd Law of Motion *viz.*, change of Momentum—Change of Pressure, which means pressure plus momentum is constant.

Hydraulic mean radius or hydraulic mean depth - The right cross-sectional area of a stream of water divided by the length of that part of its periphery in contact with its containing conduit ; the ratio of area to the wetted perimeter

Hydraulic ram—A device for lifting water by checking the flow of water in a large drive pipe periodically so as to produce pressure to drive water through a small pipe to a relatively great height.

Hydraulic sluicing - The process of moving materials by water ; coloquially. "Hydraulicking".

Hydrograph—A graph showing the stage, flow, velocity, or other property of water, with respect to time. The curve resulting from the plotting of discharges against each day of the year.

Hydrography—Water surveys. The science of measuring and analysing the flow of water, precipitation, evaporation, and analogous phenomena.

Hydro-isobaths -Contours of similar depth of the sub soil water table below the ground surface. A 5 ft Hydro-isobath is a line indicating where the sub-soil water table is at a depth of 5 ft. from the ground surface.

Hydrology—The science treating with the waters of the earth, their occurence' distribution, movements, etc , often restricted to underground waters in distinction to hydrography as relating to surface water.

Hydrolysis—Chemical process of decomposition of the elements of water.

Hydrometry—The determination of the specific gravity of liquids or suspensions of finely diveded solids in liquids.

Hydroscopic coefficient—The moisture in percentage of dry weight that a dry soil will absorb in saturated air at a given temperature

Hydroscopic moisture or water Immobile soil moisture that can only be driven off by heat.

I

Impact loss—The head lost as a result of the impact of particles of water; included in and scarcely distinguishable from eddy loss.

Incised river—An incised river is one which has cut its channel through the bed of the velley, as opposed to one flowing on a flood plain

Indent—See Demand.

Infiltration—The percolation flow of ground-water into a drain, gallery, or other underground conduit.

Inlet—A cross drainage work consisting of and opening in a canal bank suitabely protected, to admit upland drainage water into the canal.

Intake weir - See Deversion Dam.

Integration method—A means of determining the mean velocity of a stream by noting the total number of revolutions of a current-meter vane and the time taken, while the meter is slowly lowered from the surface to the bed and returned one or more times.

Intensity (annual)—The term applied to the percentage of the culturable irrigable area irrigated during a year or during a crop. The project intensity is the annual intensity aimed at in the project.

Intermittent canal—A canal designed to irrigate intermittently and which usually takes its supply from a storage or reservoir. It should be distinguished from an Inundation canal.

Inundation canal—This term is ordinarily applied to a canal with or without some form of head regulator, and dependent upon the surface level of the water in the river for its supplies. It follows that the Inundation Canal will only run when the supply in the river rises to a level which permits of feeding the canal In Burma such canals are known as freshet channels.

Invert—An invert arch as at the bottom of a sewer.

Ion—One af the small electrified portions into which a certain proportion of the molecules of a substance are spilt up when it is dissolved in water. Such a solution is subjected to electrolysis (Chemical decomposition by the action of an electric current) those ions which appear at the anode (positive pole) are known as anions, while others are called cations.

Irrigation—The artificial application of water to land for agricultural purposes.

Irrigation canal—A canal constructed primarily for conveying water from the source of supply to areas in which it can be used for irrigation The consumption of water implies a steady rate of flow which interferes with navigation.

Irrigation requirment—The quantity of water, exclusive of precipitation, that is required for crop production. It includes economically unavoidable wastes.

Irrigation water—The quantity of water artificially applied in processes of irrigation. It does not include precipitation

Irrigator's channel—(Commonly known as Water Course). A small channel taking its supply from a Government channel but owned and maintained by the cultivators. Known as a Field Channel in Madras.

J

Jump—See Hydraulic Jump.

K

Kankar* layer or *kankar—A form of limestone found in the form of modules on the surface or distributed through the top soil, or in layers beneath the surface. Very good hydraulic lime can be made from it.

Kennedy's theory—See Critical Velocity.

Key trench—See Cut-off Trench.

***Kharif* channel**—A channel which is designed to irrigate during the "*Kharif*"; or summer season only.

Kinetic energy—Energy due to motion. The kinetic energy of a given discharge is generally taken as proportional to the product of the weight per unit of time and the velocity head of its mean velocity. For a constant discharge kinetic energy may be represented by a line at a distance above the flowing water surface proportional to the velocity head of its mean velocity. The elevation of such a line above any datum represects the total energy (potential plus kinetic) of the given discharge above that datum. Strictly, the kinetic energy of a given discharge is the integral of the kinetic energies of its particals.

Kuter's formula—An empirical formula expressing the value of the coefficient C in the Chezy formula, in terms of tne friction slope, hydraulic radius, and a coefficient of roughness.

L

Lacey's silt factor—Lacey's Silt Factor is based an the theory that in all regime channels of the same silt grade the ratio V^2/R which characterizes turbulence is a constant. The silt factor is given by the expression $f=0{\cdot}75V^2/R$, when the silt factor is unity the grade of sandy silt closely approximates to that of the regime canals in the northern Punjab, the value of Kutter's N associated with such standard grade being appraximately 0·0:25.

Laminar flow—See Stream Line Flow.

Laminar velocity—The velocity below which in a particular conduit, laminar flow will always exist, and above ,which the flow may be either laminar or turbulent depending on circumstances.

Leaching—The washing out of salts from the upper zone of the soil by flooding. The salts dissolve in the water which is drained off either on the surface or through the subsoil.

Level crossing A work designed to pass one channel across another comprising a barrage or regulator on each. Water arriving at the crossing from either channel can then be distributed between them as desired

Lift irrigation or lift area —Area of which the level is too high to allow of irrigation by flow from the source but which can be economically irrigated by water raised by pumps or other lifting devices to the necessary level at some point in the supply system

Limits of oscillation—The width within which a river has ranged within historic times.

Lining—A protective convering over all, or over a portion of the perimeter of conduit, or reservoir, to prevent seepage losses, to withstand pressure, or to resist erosion. Channels are sometimes lined to reduce friction or otherwise improve conditions of flow.

Liquid limit – Is that moisture content expressed as a percentage of the weight of oven dried soil at which the soil will just begin to flow when tightly jarred, At this stage cohesion and internal friction are practically zero.

Long crested weir—See broad crested weir, which should preferably be called long crested weir.

Long crop—The term is used generally to denote a crop, that takes more than four months to mature. As a relative term, it denotes the longer of the two crops on a double-cropped land, the other crop being called "Short-crop".

Lost head or loss of head—The energy of a given flow that is lost (converted into heat, and, therefore, useless) as a result of friction, eddies, and impact expressed as a head, that is, as a head, that is, as the height through which the flow would have to fall to produce and equivalent amount of energy.

M

Main line or main canal—This term is applied to the principal channel of a canal system off-taking from a river or other source of supply.

Major distributary —(Sometimes known as a distribntary) A Government channel taking its supply from a main line or branch, the function of which is to supply water to minors and outlets.

Manning formula—An empirical formula for the value of the coefficient. C, in the Chezy's formula, the factors of which are the hydradlic radius and a coefficient of roughness,

$$V = \frac{1{\cdot}4858}{N} R^{\frac{2}{3}} \quad S^{\frac{1}{2}}$$

Manometer - A tube containing a liquid, the surface of which move proportional to the changes of pressures; a U-tube type of differential pressure indicator ; a pressure gauge.

Manometric pressure difference "P. D."– In working with a U-tube (with suction or otherwise) it is given by the difference between the readings of the limbs

Manometric gradient—Manometric gradient is the rate of loss of head in flow and is got in practice by dividing the observed P.D. by the vertical distance between the two tapping points.

Maximnm supply discharge—The discharge which during rains, the canal will have to carry when cross drainages enter it.

Mean depth—Cross sectional area of a stream divided by its surface width.

Mean monthly discharge - Discharges, observed or interpolated daily. and averaged over a calendar month.

Mean monthly discharge averaged or smoothed over five or nine years—Mean monthly discharges for the same month averaged over five or nine successive years and ploted in the hydrographs at the central year in each case.

Mean monthly gauge reading or levels—Gauge reading or reduced levels read daily and averaged over a calender month.

Mean supply—The sum of daily discharges at the canal head divided by the number of days in the base period. Known in Burma as "Period average". In Madras the quantities escaped are excluded from the daily discharges.

Mean velocity—(1) The velocity at a given section of a stream obtained by dividing the discharge of the stream by the cross-sectional area at that section, (2) mean velocity may also apply to a reach of a stream by the dividing the discharge by a average area of the reach.

Mean velocity position The point lying between water surface and the bed a channel at which the velocity is equal to the mean velocity (·6 D below surface).

Meander—A meander consists of two consecutive loops one flowing clock-wise, the other anti-clock-wise.

Meander belt—The distance between lines drawn tangential to the extreme points of successive fully developed meander

Meander length—The tangential distance between corresponding points at extreme limits of successive fully developed meanders.

Meander ratio—The ratio of the 'meander width' to the meander length.

Meander width—The amplitude of swing of a fully developed meander, measured from mid-stream to midstream.

Meandering river -A meandering river follows a sinuous path due to nature's physical causes not imposed by external restraint, and occurs where varying discharges and silt charges lead to curved flow and erosion of the banks.

Measuring weir – A device for measuring the flow of water. It generally consists of a rectangular, trapezoidal, triangular, or other shaped notch in a thin plate with a verticle plane through which the water flows. Ths weir head is an index of the rate of flow.

Mechanical (or physical composition of soil)—The composition of the soil which is defined by reference to the sizes of the particles forming the soil. This is usually measured by sieving or sedimentation in water.

Meter—A device for measuring quantities of water passed or the rate of flow.

Meter flume—The device for measuring discharge from the direct measurement of the depth of wates flowing over it.

Million cubic feet (M. C. Ft.)—A unit of volume of reservior capacity sometimes used as I. M. C. Ft. 22·96 Acre Feet.

Minimum modular head (M. M. H.)—The minimum working head required for the modular working of an outlet.

Minor—(Sometimes known as Minor Distributary). A small Government channel, usually taking its supply fram a major distributary, the function of which is to supply water to the outlets. In Madras, "Minors" are distributaries carrying 25 cusecs or less.

Module—A device for ensuring a constant discharge of water passing from one channel into another irrespective of the water level in each, within certain specified limits.

Modular limits—The extreme values of any factor at which a module or semi-module ceases to be acting as such

Modular range - The range of conditions between the said limits within which a module or semi-module works as designed.

Moisture equivalent—An arbitrary ratio used in the indirect determination of hygroscopic and wilting coefficient of soils. It is given in terms of 100 c/w in which c, is the weight of water that remains in the soil sample after it has been saturated and subjected to a centrifugal force equal to 1000 times that of gravity ; and w, is the soil sample when dired

Moisture gradient—The rate of change of moisture content of the soil with depth is called the moisture gradient.

Mole drain—A drain which takes the form of a burrowing beneath the ground surfuce similar to that made by moles. It is usually at a depth of 2 to 2½ ft and is made by a special mole plough which is dragged through the ground.

Monsoon and *rabi* sections of a canal - These terms are used in Bombay-Dacoan. The *Rabi* Section is the area of a canal in which assured supply is available, for maturing two season and *Rabi* crops only. The rest of the canal is the Monsoon Section in which water may be supplied when available, in the Monsoon and *Rabi* seasons without any assurance

Monsoon period—June to September.

Modular outlet—An outlet of which the discharge is independent of the levels in the channel and the water-course, within certain limitations of such levels and discharges, known as the Working limits.

Mulching—Covering the soil with straw or similar loose material to prevent evaporation.

Multiple arch dam –A barrier consisting of a series of arches supported by buttressess or piers. The load is transferred by the several arches to the foundation through the butressess.

M. W. L. – Maximum Water Level. See High Flood Level.

N

Nappe—A sheet or curtain of water over-flowing a weir, dam, etc. The nappe has an upper and a lower surface

Navigation canal—A canal whose primary object is transport by water. In a purely Navigation canal, the flow of water is reduced to a minimum.

Needle—A wooden, metal, or combined wood and metal plank or baulk used for closing and opening for the control of water. It is generally inserted vertically.

Needle regulator—A regulator in which the supply is controlled by the use of needle.

Nett head—When applied to a dam-nett head is the gross head less all losses in the conduit and tail race, (losses within the turbine casings, the turbines and the draft tubes are not included in the conduit losses, being accounted for in turbine efficiency.

Nett duty of water, depth of water or delta—The depth of water that would result over a given area from a given discharge for a certain length of time. Alternatively, the delta may be defined as the total volume of water delivered, divided by the area over which it has been spread.
Owing to losses, the delta will vary with the place at which the discharge is measured, and it should be stated thus ; delta at the field, delta at the outlet, or delta at the head of the channel. A cusec day on one acre results in a delta of 2 feet.

Non modular outlet – An outlet the discharge of which is dependent on the levels both in the canal and in the water-course.

Non-perennial area –The area served by a non-perennial channel.

Non-perennial channel—A channel which is designed to irrigate during only part of the year "*Kharif*" or summer season and at the beginning and the end of the "*Rabi*" or winter season.

Non-uniform Flow—Flow of which the velocity is undergoing a positive or negative acceleration. If the discharge is constant, it is referred to as a "steady non-uniform flow".

Normal depth (Neutral-depth)—The depth of water in a channel carrying a specified discharge. It is a hypothetical depth under conditions of steady non-uniform flow.

Notch fall - -A fall the crest of which is usually at or near the bed level, usually without a glacis. In irrigation practice, notches are designed primarily to maintain the depth-discharge relation of the canal, at all stages of discharge.

Notched weir - A weir with a notch or an opening which may be rectangular, triangular or trapezoidal in shape, for the passage of water.

O

Ogge—The overfall of spillway in the shape of a double or S curve, which is convex at the top and cancave at the bottom.

Optimum moisture content—The moisture needed for maximum densification is soil compaction, if the soil is compacted at the optimum moisture, it has maximum dry weight a unit volume of the compacted soil.

Outfall –The point at which water discharges from a pipe conduit or channel into the sea, a lake, drain, or other channel.

Outlet—The term used to designate the work which passes water from a Government channel to a water-course. In Madras the term is applied to a structure constructed to pass surplus water out of a canal, distributary or minor. (See also Controlled Pipe Outlet).
Note :—Ordinarily outlets are not built in the main line or branches. When built such outlets are termed direct outlets.

Outlet area—The unit of area in irrigation practice for final distribution. It is the area served by the individual outlet. The village area may be divided into several Outlet Areas or alternatively an Outlet Area may consist of portions of several villages. Its boundaries are generally defined by the configuration of the ground. Whereas village boundaries are not so limited. In the Punjab and the United Provinces this is known as a "*chak*"

Overfall—(1) The part of a dam or weir over which the water pours ; (2) the over-pouring water.

Overlap—Overlap occurs when new cane is planted in a block before the old can has been removed so as to exceed the sanction cane area.

P

Parabolic weir—A measuring weir whose notch is bounded on the sides by parabolas such that the flow is proportional to the head.

Pendant—A sheet metal disc bearing the observation point number and carried by the pendant wire.

Pendant wire—The wire exactly marking a section line and carrying pendants upon it to indicate the exact position of observation point.

Pendular water—B. A. Keen (The Physical Properties of the Soil, page 115). Pendular water is the water held round the points of contact of solid particles due to the molecular affinity of the soil particles for water. The air is continuous in the cellular pore spaces but water is discontinuous and is bound by the Pendular menisci. the curvature of which is a measure of the pressure deficiency. The range of pressure deficiency for pendular stage is from infinity to $4{\cdot}1 \frac{T}{r}$; where T is surface tension of water and r is radius of the soil particle.

Penstock—(1) A closed conduit for supplying water under pressure to a water-wheel or turbine (2) A sluic or flood gate for restricting or regulating the flow from a head of water formed by a "pen" as in a water mill.

Penta commutator—The special commutator which indicates every fifth revolution only of the bucket wheel and is interchangeable with the commutator for indicating single revolution of the bucket wheel.

Percolation intensity coefficient (P. I. C.)—See seepage intensity co-efficient.

Period average—See Mean Supply.

Permissible area—That proportion of area served by an outlet which is intended to be irrigated in a base period is known as the Permissible Area for the period or crop. Similarly the area proposed to be irrigated annually is termed the Annual Permissible Area.

Permissible velocity—The highest velocity at which water can flow in a channel without scouring it.

Perennial area—The area served by a perennial channel.

Perennial channel—A channel which is designed to irrigate all the year round.

Phreatic surface—Phreatic Surface is defined to be the water surface at atmospheric pressure in the sub-soil.

pF value—It is a term introduced by soil chemists. It is nothing but the logarithm of capillary potential. If a soil can support 10 ft. fringe, the pF for particular moisture content if therefore 1. In the pedular state it may reach many atmospheres so it is convenient to express this in pF instead of a negative pressure of many atmospheres.

pH value—Is related to the hydrogenion concentration of a solution, which is a measure of the intensity of acidity or alkalinity. Distilled water, which is neutral, has a pH value of 7-Values, above 7 indicate the presence of alkalies, while those below 7 indicate acids.

Pick-up-weir—A weir constructed across a river at the head-works of a canal to raise the level of water sufficiently high for it to flow into the channel. This term is generally applied to a weir across a river on which there is a storage reservior higher up.

Piezometer—An instrument for measuring pressure head, usually consisting of a small pipe tapped into the side of a closed or a open conduit and flush with the inside, connected with a pressure gauger mercury, water column, or other device for indicating pressure head.

Pile line—A long line of inter-locked piles driven into the soil to form an impermeable cut off.

Pipe outlet—See controlled Pipe Outlet.

Piping—The flow of water under or round a structure built on permeable foundations, which if not preveted or stopped will remove material from beneath the structure and cause it to fail.

Pitching—See Riprap.

Pitot tube—A device for observing the velocity head of flowing water, consisting essentially of an orifice held in point up-stream in flowing water and connected with a tube by which the rise of water in the tube above the water surface may be observed. It may be constructed with an upstream orifice and two water columns the difference of water levels being an index or the velocity head.

Pivot point—The point at a fixed distance from the discharge section line on to which rays from observation points converge.

Pivot point line—The line from zero point of the discharge section line, passing through the pivot point.

Pivot point layout—A geometrical lay out of points on one or both banks for the purpose of locating observation points in a river without direct measurement along the discharge section line.

Plant consumption—The water used by plants in the processes of growth. It includes that stored in the body of the plant and that dissipated from its leaf and body surfaces by transpiration.

Plant factor—(When applied to electric installation). Plant factor (or capacity factor) is the ratio of the average load to the rated capacity of the plant.

Plastic limit—The lowest moisture content expressed as a percentage of the weight of the oven dried soil at which the soil can be rolled into thread 1/8″ in diameter without showing signs of crumbling.

Plasticity index—The numerical difference between the liquid and the plastic limit. This shows the percentage in moisture content through which soil remains plastic.

Pocket (undersluice)—The undersluice pocket may be defined as the area adjacent to the head regulator bounded on one side by the flank and on the other by the divide wall.

Porosity—An index of the void characteristics of a soil or stratum as pertaining to percolation; degree of perviousness.

Potential energy—Energy due to position. The potential of a given volume of immobile water with reference to any datum, is proportional to the product of weight and the elevation of the centre of gravity above the datum. The potential energy per unit of time of a given discharge at any instant with reference to any datum, is proportional to the product of its weight per unit of time and the elevation of its hydrulic gradient line above that datum, at that instant.

Power primary or firm—(When applied to hydro electric insiallation) is the minimum power which can be generated under the worst working coditions.

Power secondary—(When applied to hydroeletric installation). Any power generated over and above firm power due to variation of flow and head is called secondary power.

Precipitation or rainfall—The total measurable supply of water received directly from clouds, as rain, snow, and hail; usually expressed as depth in a day, month, or year, and designated as daily, monthly, or annual precipitation.

Pressure connection—When the zone between free water in a canal and the sub-soil water is saturated, a "Pressure connection" is established, as in a pipe, and any increase or decrease in the head of water in the canal is immediately reflected in the sub-soil water table.

Pressure deficiency—Pressure deficiency is the difference of pressure inside and outside a meniscus.

Pressure head—The head on any point in a conduit represented by the height of the hydraulic gradient line above that point.

Pressure sounder—The device for determining depths of water from the cubical measurement of water trapped within it due to the different pressures created at different depths.

Priming—(1) The first filling of a canal reservoir or other structure that is, either the absolutely first, or the seasonally first.
(2) Starting the flow as in a pump or syphon.

Proportionate distributor—A bifurcation comprising two or more semimoduler weir of same profile and with crests at the same level. Such a device will distribute water reaching it in proportion to the length of the crest.

Puddle—(1) Clayey material placed to form a compact mass to reduce prcolation : (2) to place such material.

R

Rack and pinion—The machine incorporating a toothed wheel and a toothed rod to the bottom of which the swivel and current meter are attached.

Race—The channel that leads water to or from a water wheel ; the former is the "head-race" the latter the "tail-race".

Radial gate—A pivoted gate the face of which is usually a circular arc with centre of curvature at the pivot ; a Tainter gate.

Ram—See hydraulic Ram.

Rapid—(1) Flow down a steep incline, on which energy is dissipated by friction,, impact against stones and small natural standing waves caused by unevennesses. (2) A structure on which such flow takes place.

Rating—(1) The relation, usually determined experimentally, between two mutually dependent quantities, such as gauge and discharge of a stream ; currentmeter vane revolutions, and water velocity, etc., calibration ; (2) the taking of measurements or the making of observations to establish a rating ; calibrating.

Rating curve (table)—A graphic (Tabular) representation of rating ; a calibration.

Rating flume—An open conduit buit in a channel to maintain a consistent regime for the purpose of measuring the flow and developing gauge discharge relation.

Rating tank—A tank containing still water of rating currentmeters, pitot tubes, etc.

Rectangular weir—A measuring weir with a rectangular notch. Unless a suppressed weir is specified the term may be taken to mean a contracting weir.

Red cement motar—A motar prepared with red cement and sand.

Regime gauge reading or levels—The level or the gauge reading of the water surface for a given discharge during period of time when the channel is in permanent or temporary regime.

Regimen or regime—A channel flowing in incoherent silt is said to be in regimen (or regime) for the particular type of silt carried when its slope and shape have reached stability under constant flow. In general, a channel is in regimen (or regime) when it is stable *i.e.*, it does not silt or scour.

Regulation—Is the process of distribution of supplies available in a river between different canals taking of at or between channels on a canal.

Regulator—A structure through which the discharge can be varied at will ; also applied to a structure provided with means of varying the water surface level above it. See Barrage.

Reservoir—A pond, lake or basin, either natural or artificial for the storage, regulation and control of water.

Retrogression of levels—The lowering of the water surface level due entirely to the obstruction of water and consequent diminution in the flow downstream, and or the lowering of the specific level, *i e.*, of the level of the water surface for given discharge.

Reynold's number—See critical velocity.

Ridging—Making small embankments or "*bunds*" in the fields to control irrigation water.

Riprap—Broken stones placed on earth surfaces for there protection against the action of water (this is sometimes known as "pitching"); also applied to brush or pole mattresses, or brush and stone or other similar materials used for protection.

Riparing—Pertaining to the banks of a stream or body of water ; a riparian owner is one who owns the banks ; a riparian right is the right to control and use water by virtue of the ownership of the bank or banks.

Ripple – (1) Surface ripples are small undulation caused by unevennesies in the bed. (2) Sand ripples result from the movement of bed sand not being uniform (sometimes known as "riffle").

Rivetted – Pitched with stone.

Rod float—A rod or staff designed to float in a practically vertical position for the purpose of observing velocities

Roller horizontal and vertical gate – A hollow cylindrical gate with spur gears at each end meshing with an inclined rack anchored to a recess in the end pier or wall. It is raised or lowered by being rolled on the rack. It may close at greater depth than its diameter by means of a shield of apron attached to the cylinder. See also Sector Gate.

Root zone—The part of the soil invaded by the roots of plants.

Rotary pump – A displacement pump for raising a liquid by the use of rotating elements instead of by piston. It may operate at almost any speed, and does not depend on centrifugal forces to lift the water.

Rotational working or Roster – When the demand exceed the available supply, recourse it had to the system is known as Rotational Working. This system is applied to channels or to groups of outlets. Each channel or group of outlets takes a turn of full supply for certain number of days, the others being closed to admit of this. The unit period for which the channels or outlets run, or are closed is known as the Rotational Turn.

Run-off—The portion of precipitation that appears as flow in streams. The volume of water discharged by the stream draining the area or into the reservoir receiving the drainage.

Run-off coefficient – The ratio of Run-Off to precipitation.

S

Salt index – A formula for ascertaining whether water is suitable for irrigation.

Salt Index = (Total Na – 24·5)—(Total Ca – Ca in $CaCO_3$) × 4 85.

All quantities in the formula refer to parts per 100,000.

The Salt Index is negative for all good waters and positive for those unsuitable for irrigation.

Sand – See Sediment in rivers and canals.

Sand core – To prevent leakage through an embankment a core of sand is often provided from the base of the core trench to at least one foot above the high flood level. It also prevents the formation of rat holes.

Saturated soil (Effectively) – If the pore space in the soil is less than 40% say 32% by volume it is said to be effectively saturated ; then the moisture content is found to be 20% by weight.

Saturated soil (Fully)—When the soil pores are fully saturated with water. The normal pore space in the Punjab soil is 40% by volume. In a fully saturated soil, the moisture content shall, therefore, be 25% by weight or more than this. This is taken to be the standard moisture content to indicate full saturation.

Schocklitsh number—A number characteristic of silt, derived from the analysis of grain size.

Scouring sluices – See undersluices.

Sector gate – A roller type of gate in which the roller is a sector of a circle instead of a complete cylinder. See also Roller Gate.

Seconds pendulam A small weight attached to a cord held at a point 39" from the centre of gravity of the weight resulting in its swinging to and fro in exactly one second of time, over its normal swinging range irrespective of the circumferential length of the to and fro distance swung.

Sediment in river or canal—The deposit shingle, sand or clay in the bed of a river or a canal.

Segment—A specified length of the total discharge section line.

Sensitiveness—The variation (per cusec) of discharge of a semi-module for a tenth of foot variation of supply level.

Sensitivity—The ratio that the rate of change of discharge of an outlet bears to the rate of change in level of the distributary water surface referred to the normal depth of the channel.

Seepage—The percolation of water through soil ; infiltration. Seepage from canals and reservoirs accurs in the form of percolation under positive pressure ; but if the sub-soil water table is at more than a few feet below the bed flow becomes capillary under negative pressure reducing losses. Losses are generally described in terms of cusecs per million sq. feet ; but in many cases loss percent per mile gives more useful information. Seepage into a body is refferred to as influent seepage ; that away from a body, as effluent seepage.

Seepage intensity coefficient S.I.C.) – $q = \sigma H$. Where q-loss in cusecs per million square ft. H-percolation Head, difference between B. S. P. level and the water surface in the channel. σ – Percolation Intensity Coefficient @ 20°C per foot infiltration head.

Note—P.I.C. should be used preferably instead of S.I C.

Seepage head—Difference in the water level in canal or any other source and the Basic soil pressure represents the head causing seepage loss.

Semi arid – A term applied to a country or climate, neither entirely nor strictly humid. In which inferior crops can be grown without irrigation.

Semi-modular outlet—An outlet of which the discharge is independent of the level in the water-course, within the working limits.

Semi-modular outlet submerged An A. P. M. is a Submerged Semi-Modular-Outlet. The discharge varies as the depression of the roof block and not as the head measured from the distributary water surface to the centre of the orifice.

Setting—The term applied to the adjustment of the discharge of an outlet by altering the Work Head. *Note*—A module is said to be "rateable" when it can be rated or set to give a fixed discharge under a given fet of conditions.

Sharp crest weir – A measuring weir consisting of a thin metal plate fixed vertically, over which the water flows.

Shrinkage limit—The moisture content expressed as a percentage of the dry weight of the sample at which the removal of additional water produces no further change in the volume of the sample. In other words, the amount of water required to fill the pores of a soil sample which has been dried to constant weight from a wet condition.

Shutter – When applied to a weir, a plate of steel or wood construction hinged to the crest. Shutters are used to regulate the level of the river above the crest. The size is limited by that which can be raised against a modest head by manual labour.

Side channel – See Ditch channel.

Side slope—The slope of the sides of a canal, dam or embankment ; custom has sanctioned the naming of the horizontal distance first as 1·5 to 1, (or frequently 1½ to 1) meaning a horizontal distance 1·5 ft. to 1 ft. vertical.

Silt – (1) Water borne sediment. The term if generally confined to fine earth sand or mud but is sometimes broadened to include all material carried, including both suspended and bed load. (2) Deposits of water borne materials as in a reservoir on a delta or on overflowed lands.

Silt charge—This is the total silt charge per unit volume by weight in water.

Silt excluder—A silt regulator located at the head of a channel.

Silt extractor or ejector—A silt regulator located on a channel other than at the head.

Silt factor 'f'—See Lacey's Silt Factor.

Silt grade—Average diameter of the silt particles.

Silt regulator—A regulator provided with under-sluices so arranged as to secure the escapage of the heavily sand laden bottom water.

Silt vanes—Verticle vanes arranged in the bed of a channel with the object of diverting the heavily sand laden bottom water in order to control the sand entering an offtake.

Silt wall—A vertical diaphragm wall connecting the downstream wing of an offtake with a point in the waterway so located that the sand distribution of the water passing, it is not distributed by the draw of the offtake.

Single crop—Raising of only one irrigated wet crop in one season. This practice is, of necessity, predominant in the lower reaches of the irrigation system.

Siphon or inverted siphon—A tube or "sealed" channel constructed to carry water under an obstruction such as a river, canal road or railway at a level lower than that at which the open channel normally flows.

Siphon spillway—A discharging device on the siphon principle for discharging surplus water over dams.

Slope—The relation between difference of level and the length of the river either direct or along the main stream, as the case may be, between two gauge sites.

Sluice—(1) A conduit for carrying water at high velocity ; (2) an opening in a structure for passing debris ; (3) to cause water to flow at high velocity for wastage for purposes of excavation, ejecting debris, etc. In Madras presidency the term is used for an outlet.

Sodium clay—Clay in which the principal, exchangeable base is sodium. These when defloculated are impervious to water.

Sodium salts -Compound of sodium metal such as Sodium Carbonate ($NaCO_3$) Sodium Sulphate (Na_2SO_4), Sodium Chloride, (Na Cl) Common salt.

Soil—Finely divided material composed of disintegrated rock mixed with organic matter, the loose surface material in which plants grow.

Soil compressibility—In a soil, is the degree of resistance to change in volume under the pressure of heavy loads.

Soil cohesion—In a soil, is the resistance of particles against motion because of their stickness, coheison is high in clays, but may be very low in silt and is entirely lacking in sand.

Soil compaction—Compaction is one of the method used in the consolidation of stabilized earth work and consists in the application of load at the top of an unconsolidated or partially consolidated layer of a graded mixture, which contains enough granular material provided for mechanical interlocking. The load is readily transmitted vertically and causes consolidation throughout the thickness of the layer. The soil is rolled in relatively thin layers and as a rule in a moistened state, neither thickness of layers nor the moisture content is critical.

Soil density—The density of a soil is its weight per unit volume. A soil which consists of solids and pores has two densities that of the mass termed "bulk" density and that of the solids termed "absolute" density.

Soil water or suspended water —Water in the soils above the phreatic surface is called the soil water (physists) or suspended water (Hydrologists). It comprises Capillary, Funicular and Pendular Waters.

Soil evoporation—It is the evaporation of water from moist soils and the ground water reservoir below.

Sounding rod—The graduated pole with which depths of water are measured in feet at observation points,

Specific gauge reading or level—The gauge reading or level of the water surface at any particular site for a given discharge.

Stability of soil—In a soil, this may be defined as the resistance to natural flow, when loaded denoting its structural strength which depends upon the shear strength representing the combined effect of internal friction of the soil particles.

Strike—The strike of a bed, fault or joint in a rock is the bearing of the line which a horizontal plane makes with the plane of the bed, fault or joint.

Supply—Is taken to be supply utilized, that is supply entering the canal head less escapage, in modern and efficient canal irrigation, there is practically no escapage.

Supply authorized full or designed full or full supply discharge—Is the maximum discharge for which a channel or work is designed.

Specific energy—The energy of a stream referred to its bed; namely, depth plus velocity head of mean velocity.

Specific retention—The ratio of the volume (or weight), water which a soil will retain against the force of gravity (after having once been saturated) to its own volume (or weight).

Spill-way—See Waste Weir.

Spillway siphon—See Siphon Spilway.

Spur—See Groyne.

Standard deviation—A measure of the dispertion of a series of statistical values as precipitation, stream flow, etc., it is the square root of the sum of the squares of the deviations from the mean, divided by the number of values or events in the series.

Standing wave – Standing wave is defined to be a wave which persists to form at the same place. The wave is formed according to Bernoulli's theorem witout impact. The velocity at the crest of the wave is the critical velocity of the depth at the trough and the velocity at the trough is *vice versa*.

Static head – The total head without deduction for velocity head or losses ; for example the difference in the elevation of head water and tail-water of a power plant.

Static or equilibrium gradient in soils – In a column of soil which is saturated at the bottom the conditiun of static equilibrium are attained. Hydrostatic pressure drop from one point to another divided by the distance in such a case gives the equilibrium or the static gradient. In Pendular, no stage flow is possible as water is disconnected. In capillary or funicular stage flow is possible if pressure gradient is more than the equilibrium gradient, *i.e.*, $\phi = gh$ (Wilsdon 1923).

Staunching—Sealing, *e.g.*, staunching of canals by silt deposits. Sometimes used loosely for lining canals.

Steady flow – A constant flow ; that is, the same valume in equal units of time.

Still pond system of regulation—A method of reducing silt entry into a canal by alternately scouring a deep pocket in front of the regulator with the canal closed and using this pocket as a sand trap when the canal is in flow.

Stilling-well or chamber – A pipe, chamber, or compartment with closed sides and bottom except for a comparatively small inlet or inlets communicating with a main body of water. Its purpose is to damp the waves or surges while permitting the water level within the well to rise and fall with the major fluctuations of the main body.

Storage dead—Is the capacity of a reservoir below dead storage level.

Storage live—Is the capacity of the reservoir above dead storage level.

Stream-line flow – That type of flow in which each particle moves in a direction parallel to every other particle and in which the head lost is approximately proportional to the first power of the velocity. It is sometimes designated as "Laminar flow" or "Viscous flow".

Sub-critical flow – Flow at velocities less than one of the recognized critical values.

Sub-minor – Sometimes known as a Sub-Minor Distributary. A channel offtaking from one already defined, as a Minor.

Sub-meander—A small meander contained within the banks of a perennial river channel. These are caused by relatively low discharges after the flood has subsided.

Submerged orifice – An orifice which in use is drowned by having the tail-water higher than all parts of the opening.

Submerged weir – A weir which in use has the tail water level higher than the weir crest, by which the discharge is effected.

Submergence – The ratio of the tail water elevation, to the head water elevation, when both are higher than the crest, the overflow crest of the structure being the datum of reference. The distances upstream or downstream from the crest at which head-water and tail-water elevations are measured are important, but have not been standardized.

Sub-soil flow—(1) Movement of water through a pervious sub-surface stratum ; the flow of percolating water under a structure. (2) The rate of flow or discharge of sub-surface water.

Sub-soil irrigation – Watering plants by applying the water below the ground surface.

Super-critical flow (Hypercritical flow)—Flow at velocities greater than those of the recognized critical values.

Superpassage – A work which carries one channel over another without lowering the bed level of the lower channel.

Superphosphate Fertilising material largely consisting of soluble phosphates. Animal bones are largely used for the manufacture of these fertilizers.

Supply average – The average supply in channel during a certain period is the sum of the daily discharges run at the head of the channel in the period divided by the number of days when the channel is in flow.

Supply-mean – The mean supply in a channel is the sum of the daily discharges at the channel head divided by the number of days in the base period.

Supply normal—Is a term peculiar to the Sutlej Valley Canals and denotes the discharge corresponding to 55% of the authorized Full Supply Discharge.

Supply share—Is a term peculiar to the Sutlej Valley Canals and denotes the discharge allowed to any particular channel according to the term of the agreement between the three parties, namely, Punjab, Bahawalpur and Bikaner.

Supply unilized—The supply entering a channel less any supply escaped. Sound irrigation practice requires practically no escapage.

Suppressed weir—A weir the length of which is the same as the water surface width of the channel upstream of it, and its base is at the same level as the bed of the channel upstream of it.

Surface curve—The longitudinal profile assumed by the surface of a stream of water flowing in an open conduit ; the surface curve is the curve of equilibrium of all forces acting on the flowing water ; (2) the hydraulic gradient line.

Surface float—A wooden disc or other floating matter used for timing over a fixed distance in order to determine surface velocity.

Surface slope—The inclination of the water surface expressed as a charge of elevation per unit of slope length ; the sine of the angle which the water surface makes with the horizontal. The tangent of that angle is ordinarily used, no appreciable error resulting except for the steeper slopes.

Surface velocity—The rate at which the surface layer of water moves.

Suspension rod—The hand operated rod used in shallow water instead of a Rack and Pinion.

Swival—The device fixed between current meter and its means of suspension so that it may be free to swing in a horizontal plane.

Sympathetic retrogression—A retrogression of levels in one river caused by a reduction of water levels at the confluence due to changes upstream in the water level of the river joining it.

Syphon—See Siphon.

T

Tail—This term is usually applied to the work constructed at the end of a channel for the distribution of the water thereat, *e.g.*, tail cluster, tail regulator, etc.

Tail race—A channel conducting water away from a water wheel. (See also "Race").

Tail water—The water just downstream of a structure.

Tainter gate—See Radial Gate

Talus—A protection at the downstream end of a weir or fall, consisting of blocks of concrete or masonry.

Tank—A small reservoir.

Tank percentage—This term is used in Central Provinces to express the percentage ratio between the net capacity of a reservoir or tank and the normal yield from the average monsoon rainfall.

Tarungars—Wire crates containing loose bricks or stones placed downstream of a structure to protect it from scour.

Temporary gauge to permanent gauge—A temporary contrivance for measuring water surface level along the gauge line of a permanent gauge when that gauge itself is left high and dry, and fixed so that it measures as if it was the permanent gauge.

Terraces—(1) Sloping ground cut into a succession of benches and steep inclines for purposes of cultivation. Often the inclines are made quite steep and are protected by riprap, or retaining walls are substituted, thus giving greater areas for cultivation ; (2) areas bordered by low broad ridges constructed on cultivated land of such alignment, height, and spacing as to conform to the topography and to permit travel by cultivating and harvesting machinery, the object being to prevent loss of soil by erosion.

Tilting gate or automatic gate—A gate hinged at the top or bottom and counter-blanced by weights, automatically opening or closing with the change in head.

Time factor—The ratio of the number of days the channel is in flow to the base day.

Time-lag—The time elapsing between the occurrence of any alterating of discharge or level at one point on the river and its occurrence at another point.

Titration—A method of analysis in which an increasing quantity of a liquid of strength is added to a measured quantity of a substance in the presence of a third substance called the indicator until a certain action, *e.g.*, change of colour, takes place. By this method the strength of an alkali solution may be ascertained by neutralizing it with an acid solution of known strength.

Toe-wall – A shallow wall constructed below the foundation level to provide a footing for the pitching of the face of an embankment. When the sub-soil water level is high, the toe-wall takes the form of a series of shallow walls.

Tortuosity – The ratio of actual length measured along the middle of the main river channel to the axial length of the river.

Total energy line--A line joining the elevations of the energy heads of a stream. * The energy line is above the hydraulic gradient line a distance equivalent to the velocity heads at all sections along the stream.

Total losses – The sum of loss of water by absorption, percolation and evaporation. The total losses in a channel may be defined as the difference between the discharge at the head of a channel and the useful discharge, *i.e.*, the sum of the offtaking discharge. These are also called "Canal Losses" and are usually expressed in terms of cusecs per million square feet of wetted perimeter.

Toxic – Poisonous ; tending to reduce the yields of crops below the Normal

Transition – A short conduit uniting two others having different hydraulic elements; a conversion.

Transmission co-efficient (or constant) – The quantity of water that is transmitted in unit time through a column of soil of unit length and cross-section under unit difference of head at the ends. It is the rate per unit area that water will move vertically downward through a completely saturated uniform soil column.

Transpiration—The process by which plants dissipate water from their leaf and body surfaces in the phenomena of growth. The water is brought to the surface of the plant body and dissipated there.

Transpirations ratio—The ratio of the weight of water passed through a plant, to the weight of dry plant substance produced.

Trapezoidal weir—A contracted measuring weir with a trapezoidal notch. See also Cipolletti Weir.

Trash rack—A grating, usually made up of M.S. Flats, provided at the entrance of a submerged outlet to prevent entry of debris jungle, etc.

Triangular weir—A contracted measuring weir notch with sides that form an angle with its apex downward ; the crest is the apex of the angle ; a V-notch weir.

Turbulence—A state of flow wherein the water is agitated by eddies ; opposed to a condition of flow that is quiet or quiescent. It exists where the Reynold's Number exceeds 2,000.

Turbulent flow – That type of flow in which any particle may move in any direction with respect to any other particle, and in which the head lost is approximately proportional to the second power of the velocity.

Turbulent velocity—That velocity above which, in a particular conduit, turbulent flow will always exist, and below which the flow may be either turbulent or stream-line, depending on the circumstances.

U

Under-flow—See Sub-Soil Flow.

Under-sluices – Weir sluices designed to maintain the course of a river in the desired position.

Uniform flow – A constant flow or discharge, the mean velocity of which is also constant. Uniform flow is also referred to as "steady uniform flow". It is an ideal condition that can only be approximated in fact. If the velocity of the constant discharge varies the flow is defined as "steady-non-uniform".

Uplift—The upward water pressure on the base of a structure.

V

Vanes – Longitudinal walls usually built in continuation of piers to fan out the flow as desired.

Velocity head—The distance a body must fall freely under the force of gravity to acquir the

velocity it possesses. See also Kinetic Energy.
Velocity-rod correction—The correction to be applied to a velocity-rod velocity in order to convert it into mean velocity.
Velocity rod or discharge rod—See "Rod Float.'
Velocity of approach—The mean velocity immediately upstream of a weir or other structure
Velocity of retreat—The mean velocity immediately downstream of a structure.
Vana contracta—The most contracted sectional area of a stream, jet, or nape beyond the plane of the orifice, or notch, through which it issues.

Venturi flume—A flume in which the velocity in the throat is less than the Critical Velocity; the discharge being dependent on the Ventri head (difference of water level upstream and in the throat) and the cross-sectional area of water in the throat section.
Venturi meter—A measuring device consisting of a Venturi tube with entrance and throat piezometer tubes the discharge depending on the Venturi head and the area of the throat.

Venturi tube—A closed conduit which is gradually contracted to a throat causing a reduction of pressure head by which the velocity through the throat may be determined. The contraction is generally followed, but not necessarily so, by gradual enlargement to original size. Piezometer connected to the pipe above the contracting section and at the throat indicate the drop in the pressure head which is an index of flow.

Vertical velocity curve—A graph of the relation between depth and velocity along a vertical line in a stream, as determined by a set of observations.
Viscous flow—See Stream-Line Flow.
V-notch weir—See Triangular Weir.

W

Warp—Bend in a river imposed by external restraint.
Waste weir—The escape provided for the passage of surplus water from a reservoir or tank.
Water account—Is an account maintained of distribution of supplies between units of interlinked canals or different channels of one canal.
Water allowance—The outcome of all considerations of the duty of water, intensity, proposed crop ratio, water available, etc., is the fixing of the water allownce. Water Allownce may be defined as the number of cusecs of outlet capacity, authorised per 1,000 acres of culturable irrigable area. The Water Allowance therefore, not only defines the size of the outlet area but also forms the basis for the design of the distributing channels in successive stages.

Water course—The term applied to an irrigator's channel taking its supply from a Government channel, from which fields are irrigated directly. A watercourse is sometimes used for conveying water to tanks or ponds or to municipalities.

Water cushion—A pool of water maintained to take the impact of water overflowing a dam, chute, drop, or other spillway structure.

Water hammer—The phenomena of oscillations in the pressure of water in a closed conduit, resulting from checking the flow. Momentary pressure greatly in excess of the normal static pressure may be produced in this manner.
Water-logged—Land may be classified as water-logged when the water table is permanently located at ground level. The approach of this condition is indicated when the yield of crops commonly grown in the locality is reduced, by the rise of the water table below the normal that would be expected from the type of soil of that area.

Water requirement—The total quantity of water, regardless of its source, required, by crops for their normal growth under field conditions. See also Irrigation Requirements.
Water-shed—(1) The area drained by a stream or stream system; (2) the dividing line between drainage basis.
Water table—The upper surface of zone of saturation in soil or in permeable strata or beds.
Weep-holes—Openings left in the retaining walls, aprons, linings, foundations, etc., to permit drainage, to reduce pressures, etc.
Weir—An artficial barrier across a river (or canal) to raise the water above the natural level in order to supply a canal or canals taking off above it and to pass over its top the excess water.

In Madras an escape is termed a weir.

Weir controlled canal—A canal tacking its supply from a river or branch of a river in which there is some artificial obstruction which raises the level of water in the river above its natural level so that the canal may be fed.

Wet area—This term is used in the Central Provinces for an area which can be irrigated from a private source and which in consequence is assessed at higher rates for land revenue purposes.

Wet crop—A crop which depends on irrigation for its growth.

Wetted perimeter—The length of the wetted contact between a stream of water and its containing conduit, measured along a plan at right angles to the direction of flow ; that part of the periphery of the cross sectional area of a stream which is in contact with its container.

Wilting coefficient—The moisture in percentage of dry weight remaining in the soil within the root zone as plants reach a condition of permanent wilting.

Working head—When applied to an outlet, it represents the difference of level between the water in the distributary and in the water course.

GLOSSARY OF HINDUSTANI TERMS

A

Ab—Water.

Abadi– Cultivated land.

Abadi—A village or habitation.

Abiana—Water rate.

Abpashi—Irrigation.

Abzaya—Waste of Water.

Ad (**or Add**)—The water course which conveys. water from the water wheel into the fields.

Amin—Supervisor of Patrols (See Patrol).

Arzi kasht—Temporary Cultivation.

Asal - Land revenue

Asami or kashtkar—Cultivator,

Asami maurusl—Occupancy tenant

Asami ghair maurusl—Tenant-at-Will.

Asami qanuni—Statutory Tenant.

B

Bandis—Closure of a channel.

Banjar—Unculturable land.

Banjar jadid—New fallow.

Banjar qadim—Old-fallow.

Baqaya—Unalloted area of a village.

Bar—This high land of *doab* as distinct from the *Khadir*

Barani—Lands on which crops are grown on rain water only or depending on rain.

Barsat—Monsoon season,

Bela—River valley, or shoal or island in a river.

Beldar—A labourer.

Bhal—Silt or soil from the clearnce of a water course.

Bhandal—Bundle.

Bigha—An area equal to about 5/8 of an acre. (different area in different localities).

Biswa—The twentieth part of a bigha.

Biswansi—The twentieth part of biswa.

Bund—Earthen Embadkment.

C

Chah or kuan—Well.

Chahi—Land irrigated from wells.

Chak—Area irrigated from an outlet.

Chak-bandi—The distribution of an area into chaks.

Chak boundary—Is the limit set for irrigation from any particular outlet.

Charus—Water lifting arrangements used in the Punjab with a rope and a leather bucket, called *mot* in U.P.

Choa—Hill Stream.

Colaba or mori—Pipe or outlet.

Crore—Ten Million (1,00,00,000)

D

Daff—An obstruction in a channel for raising its supply level.

Dahana—Ventage of outlet.

Darbar—A convocation or assembly more or less of a ceremonial character, convend by high Government officials.

Darkhast abpash—An application for water and the form on which such application can be made.

Dharya or nadi—River.

Dhava—River side land revined by river.

Denkli—Water lifting arrangement with lever arm called *Rati* in U.P.

Desi Of the country.

Dhaincha—Fodder crop used as green manure.

Do-fasla—A field irrigated twice during a year *i.e.*, sown, with *Rabi* after a *Kharif* crop.

Do-fasli—Double cropped or two crops per session.

Doab--The country lying between two rivers.
Dobara kasht—A field sown twice during the same crop.

E

Ekfasli—Yielding one crop per season.

F

Fard—List.
Fard partal—Check list.
Fard mushtabba—A list of fields, the irrigation of which a doubtful.
Fard raftar—Tour programme.
Fasal—Crop.

G

Ghair mumkin—Unculturable land *i.e.*, area under roads, houses, etc.
Ghara bandi—Earth work repair to rain cuts in a bank.
Gharqaba—Crop flooded by water from a breach in a channel or by rain.
Ghat—Generally a passage across a channel. The term also applies to a site in the bank of a channel where cattle can drink water.
Girdawari—Inspection and measurement of crops.

H

Halqa (or circle)—A group of villages in charge of a canal Patwari.
Hissa—A part, usually understood to be one-tenth of foot.
Hissa—A share.

I

Isteghasa—Complaint against *khasra* measurement.

J

Jamabandi—Register of holdings of owners and tenants showing the land held by each and the amount payable as rent, land revenue and cesses. Record of rights in land, after every five years.
Jagir—A rent-free grant of land.
Jagiadar—A holder of *Jagir*.
Jhal—A fall of water.
Jhalar—A contrivance used for lifting water.
Jhalari—Area irrigated bay *Jhalar*.
Jham—A big *Kassi* used as dredger in well sinking.
Jhatta—Area irrigated by lift.
Jhil—A shallow lake.

K

Kacha—Made of earth or perishable material.
Kalar—Salt integrated soil in Sind. The following types of "Kalar" occur :—

(*a*) *White Kalar*. Main salts are sodium chloride, sodium sulphate, and magnesium sulphate. This is the commonest type of *kalar* in Sind.

(*b*) *Dark Kalar*. In addition to the salts in white *Kalar*, this type contains the chlorides of magnesium and calcium. Found on land affected by water-loggings and seepage.

(*c*) *Black Kalar*. Contains sodium carbonate, a salt most toxic to vegetation, in addition to the salts present in white and dark *kalar*. The large amounts of lime present in Sind soils prevent excessive formation of black *kalar*. Present in badly drained localities.

(*d*) *Brown Kalar*. Contains the nitrates of potassium and sodium in addition to the salts present in white *kalar*. Brown *kalar* is used as a top dressing on crops after the manuer or artificial fertilizers.

Kalarathi zamin—Land impregnated with salts.
Kamin—Cultivator's servants.
Kar—Hand soil crust, called *mota* in U.P.
Kasht—Cultivation.
Kashtkar—Cultivator.
Kassi—Soil digging Indian instrument.
Khadir—River valley ; low alluvial land, and as adjective means riparian.
Khal—Water-course.
Kharaba—Failed crop.
Kharanja--Brick on edge.
Kharif—Season, April to September.
Kharif crop—Autumn; summer crop harvested in Autumn.
Khasra—Field, or field book.
Khasra canal—Register in which irrigation is recorded.
Khasra shudkar—The field measurement book in which the canal patwari records the final measurements. It is combined with the initial record.
Khasra bandobast—Record of standard field areas.

Khata—A holding or estate.
Khataunni—The demand statement of canal revenue sent to the Deputy Commissioner showing recoveries to be made from each cultivator.
Khet—A field.
Khud khast—When the zamindar does not lease out land but cultivates it himself.
Khush haisiti—Increase in value of land due to introduction of irrigation.

Khushak—Dry.
Kiari or kiara—Sub-division of a field into small portions for efficient irrigation.
Killa - The 25th part of a square or a *murabba*, which is a rectangle of 25 acres measuring 1100×990 sq ft.
Kila bandi—Substitution of rectangular fields of uniform size (usually an acre) for irregular shaped fields
Kist—Land assessment (half yearly portion).
Kore—Crop which has not received a watering after being sown.
Kungi—A disease of wheat crop when the blade of wheat is charred black.
Kore-watering—First watering after the germination of a crop.
Kumad - Sugarcane.

L

Legan Assessment of land rent,
Lalkitab—Patwari's note book (diary).
Lamberdar—Headman of the village.

M

Maira—Sandy loam.
Makhlut—Mixed crops.
Mal—Land revenue.
Malba—Village common fund.
Malik—Owner.
Malkana—Owner's rate.
Malguzar—Land owner ; a person responsible of paying the land revenue
Mamla - Land revenue.
Maurusi - Hereditary tenant.
Mauza—Village.
Mazrua—Area under cultivation.
Moga—Outlet.
Murabba - Literally a square —the term means a square or a rectangle of land. On the Lower Chenab Canal, land was alloted, in squares of about 28 acre each On the new canals rectangles of 25 acres were allotted.
Mudh—The beginning of a water-course.
Mumni—Undesirable growth in wheat crop.
Muzara—Tenant.

N

Nabud - Crop which fails to germinate.
Nadi—River.
Nahar—Canal.
Nahari—Canal irrigated.
Najaiz—Unauthorised.
Najaiz abpashi—Irrigation done in an unauthorized manner.
Najaiz kasht—Unauthorized cultivation.
Nakka—A cut in a water-course to pass water to the fields.
Naka—Cut or breach in a channel.
Nali—When applied to the sowing of a crop. means sown in rows.
Nali -A small water-course.
Nikal—When a rotation of turns on a water-course is completed, the balance supply left over the length of the water-course from head to tail is termed "Nikal".
Numbershumari The recording of the area cultivated by each zamindar for the purpose of recovering from him the land revenue due to the Government.
Numbardar—See Lambardar.

O

Orae—Winter-crop.

P

Pacca—Superior or pamanent. Pacca building is usually one constructed of burnt brick walls, with a superior type of roofing
Pachotra—Renumeration paid to Lambardar usually 5% but for canal irrigation collection 3%.
Pahr—A period of three hours.
Paimaish—Measurement of Irrigation.
Panchait—Village legislative assembly.
Pand—Tail of a Water-course.
Pallar—Rice trash (Punjabi).
Pani—Water.
Pansal—Gauge.
Parcha abpashi or parcha—A slip showing the revenue recoverable from an individual.
Partal—Check of patwari's record and measurement of irrigation.
Partal kham or pucca—A check of intial irrigation and check of measurement of crops
Patta—A lease of irrigation.
Patti—A ivison of village.
Patwari—Village Government civil official. In the Punjab the petty canal official corresponding to a patrol in the United Province.
Pushta—Berm.
Purab—East.

Q

Qanugo—Civil revenue official.
Qasba—Large village.
Qulaba—Outlet.

R

Rabi—Winter crop.
Rajbaha—Distributary Channel.
Rakhar—Land with a low salt content but high pH values, *i e.*, an alkaline soil.
Raoni—First or prelminary watering of a field.
Reh—Land with high salt content usually sodium chloride or sodium sulphate.
Ret—Sand (Retli—sandy).

Rohi—Stiff soil containing a considerable amount of clay.
Roznamcha—A dairy of daily work done by a Patwari or Zilladar.
Ryat—Cultivator.

S

Sabaz khad—Green manure—a crop ploughed green to serve as manure.
Sabzi, tarkari—Vegetables.
Sailab—Flood, Inundation.
Sailaba—Flood irrigation.
Sal—A heavy wood used for canal ballas regulating planks, etc.
Salami—Slope
Sawni—Summer crop.
Ser—A seer.
Sem—Seepage
Sema—Water-logged area.
Shahukar—A money lender.
Shajra—A field map.
Shakh—Branch or distributary.
Shamlat—The undivided area in a village which is the property of the whole village.
Shisham—A good wood for door, panels, etc
Shor or shora—Impregnated with salts.
Shudkar—Intial record of irrigation in the khasra.
Seir—Leakage.
Sua—A minar irrigation channel.
Surkhi—Finely grouned bricks.

T

Taccawi (work)—A work executed at the cost of the cultivators.
Takavi—An advance given by Government to an agriculturist for buying seed, etc.
Tahsil—Sub-division of a civil district.
Tahsildar—A civil officer in charge of a tahsil.
Talab—Tank or pond.
Talaf—A crop which fails completely after germination.
Talfi—Failure of crops.
Tanaza—Objection to assessment.
Tarungar—A wire crate filled with stone used as a prorection against erosion.
Tatil—Closure of an outlet or a minor.
Tawan—A punitive water rate for an unauthorized irrigation or waste of water.
Thur—Land with high salt content usually sodium chloride or sodinm sulphate.
Tibba (sand)—Wind-drifted and dunes.
Tor abpsahi—Flow of irrigation.

U

Usar—A highly impermeable salt impregnated soil. Main salt constituent conists of sodium bicarbonate with somewhat less sodium carbonate, a small amount of sodium chloride, and variable traces of sodium sulphate.
Uttar—North.

W

Wadh-wattar—A crop grown on the moisture of a previous crop (*N.B.* Not necessarily a *Rabi* crop).
Wah—Main water channel.
War or wara or wari—A cultivator's turn for taking water.
Warabandi—Schedule of turns or time-shares of supply from a water-course.
Wara shikhni—Interference in a regular schedule of turns of supply.
Wats—Field ridges.

Z

Zaid kharif—Late summer crop.
Zaid rabi—Late winter crop.
Zaildar—Superior of a few Lamberdars.
Zamindar—Land owner.
Zilladar—Supervisor of Mirabs and Patwaris.

ॐ नमः