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Preliminary Investigation for Storage Schemes

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INTRODUCTION.

The construction of dams is a very vast subject and would require more than one volume for thorough discussion : neither the time nor the paper is available to do justice to this subject. Certain aspects only can, therefore, be discussed.

This Paper deals with what preliminary investigation is needed for storage schemes and how it should be carried out.

Why storage is necessary has been discussed in Chapter I. It has been shown that all future projects in the Punjab will have to depend upon stored supplies. Water for storage and land requiring protection are both available. The Punjab must be industrialised if it is to remain in the fore-front. Cheap electricity can be generated at the dams. Only men with vision are needed to usher in another era of prosperity and contentment.

Requisites of a good dam site are dealt with in Chapter II. How Standard G. T. S. Plans and geological maps can sometimes be used for preliminary selection of dam sites, to what extent are different kinds of rocks suitable for construction of dams, what are the characteristics from a geological point of view of an ideal dam site, how the supply likely to be available for storage at any site can be calculated are all discussed at length in this chapter.

Chapter III deals with the investigation to be done in the office, while the method of carrying out detailed surveys in the field is fully discussed in Chapter IV.

The safety of a dam depends upon the foundation on which it rests. Most of the failures of dams can be directly attributed to inadequate foundation exploration, the defects in the foundation were not properly investigated by the Engineer with the result that they were not remedied. The importance of thorough foundation exploration is obvious. How this important work should be done has been discussed in Chapter V.

A bibliography is given at the end.

An attempt has been made to be as brief as possible, but clarity is nowhere sacrificed to brevity.

Acknowledgment. The writers are deeply grateful to Rai Bahadur A. N. Khosla, Chief Engineer, Irrigation Works, Punjab, and Mr. J. B. Auden of the Geological Survey of India for their most valuable suggestions and for their permission to quote from unpublished papers.

CHAPTER I.

Necessity of Storage

1. It is a well known fact that rainfall in the Punjab is very unevenly distributed. Not only does it vary from place to place, but there is considerable variation from year to year even at the same place. This will be clear from Table I, which gives the summer and winter rainfall in the famine areas of Hissar and Rohtak Districts.

Table I.

Year.	Summer.	Winter.	Annual.
1935-36	14·23	1·06	15·29
1936-37	7·22	3·55	10·77
1937-38	12·36	2·08	14·44
1938-39	4·19	2·96	7·15
1939-40	4·37	1·55	5·92
1940-41	10·68	3·03	13·71
1941-42	11·73	1·19	12·92
1942-43	18·33	...	...

It will be seen that the minimum rainfall is about 25 per cent of the maximum rainfall. No two years get the same amount of rain. Even this rain is not precipitated uniformly throughout the year, but most of it falls between 15th June and 15th September and a small portion between 15th December and 15th February. Thus there is little rain at the time of sowing and maturing of Kharif as well as Rabi crops and zamindars must depend upon irrigation for sowing and maturing their crops. This irrigation may be given from wells, canals, tanks, etc. Unfortunately Chahi irrigation is not only expensive, but the subsoil water in many parts in urgent need of protection is totally unfit for irrigation. Canal irrigation is undoubtedly the cheapest and the best way of supplementing rainfall.

2. Table II shows the growth of population and the area irrigated by canals in the Punjab during the period 1881-1931.

Table II.

Year.	Population.	Area irrigated by Canal.	Remarks.
1881	20,800,995	Not known	
1891	22,915,894	3,016,456	
1901	24,367,113	6,000,551	
1911	23,791,844	7,227,042	
1921	25,101,514	10,273,690	
1931	28,490,857	12,258,592	

How astonishing is the rate of increase of population : In the beginning the main spring of irrigation projects was the protection of populated areas against failure of the rains, but later on the problem was to make available the arid areas for the rapidly rising population. This problem is likely to become more acute as time passes. The easy projects are now finished. It is no longer possible to just construct a weir across a river and take off a canal as water is no longer available. Future projects will have to depend upon stored supplies alone. Lot of water is allowed to run to the sea every summer. This can be stored and used when and where necessary. Any body who has toured in the Punjab will have been struck by the fact that enormous areas of banjar but good culturable land still exist everywhere and if only water were available, they could be brought under the plough. Further Punjab must be industrialised if it is to remain in the fore-front. Power for this can be very cheaply generated at the dams. Thus water and land are both available and power can be cheaply generated. Only men with vision are required to usher in another era of prosperity and contentment.

5. Much work must, however, be done before the construction of a dam can be started. The present paper is primarily concerned with this preliminary investigation.

CHAPTER II.

Requisites of a good dam site

1. A good dam site ought to satisfy the following conditions :—

(*i*) The gorge should be narrow with sides high enough for the storage to be affected.

(*ii*) Valley above the gorge should be wide and valuable property should not be submerged.

(*iii*) Runoff should be sufficient.

(*iv*) In general, there should be no low saddles in the reservoir area.

(*v*) The site should be geologically sound.

(*vi*) The catchment should consist of firm rocks.

(*vii*) Site should be accessible.

(*viii*) Materials of construction should be available near the dam site.

(*ix*) Natural facilities should be available for diverting the river during construction and for disposal of spill water afterwards.

Each of the above items will now be considered in detail. More often than not, all the conditions are not satisfied by any site and a compromise has to be struck.

2. The gorge should be narrow with sides high enough for the storage to be affected. This is self evident. The narrower the gorge, the cheaper the dam will be. The sides must be high enough, otherwise it will not be possible to construct the dam of required height. A rough cross section of the gorge at the dam site can be plotted from $1'' = 1$ mile G. T. S. sheets. These sheets show 50 feet and sometimes only 100 feet contours with spot levels here and there. River bed at the dam site can be determined from the spot levels and the contours, which also enable the cross section to be completed. The cost of the dam varies approximately as the square of the height. Advantages of a low dam are self evident.

3. Valley above the gorge should be wide and valuable property should not be submerged. Capacity and area submerged curves. Wider the valley, the more the water that can be stored for any particular height of the dam. Usually a site immediately below the confluence of two or more main streams is the best. If capacity and area submerged curves are prepared for alternative sites, it can be easily determined which site will require the lowest dam for any particular storage.

Areas within contours at 50 or 100 feet intervals are marked out by a planimeter from $1'' = 1$ mile G. T. S. plans. Should $1'' = 1$ mile plans be not available, areas within contours at 250 feet intervals are marked out from $1/4'' = 1$ mile G. T. S. plans and the capacity between successive contours determined by the ordinary trapezoidal formula. Thus if A_1 and A_2 acres be the areas submerged at say R. L. 2,000 and 2,250 respectively, the capacity between the two contours will be $\frac{A_1 + A_2}{2} \times 250$ acre feet. Capacity at R. L. 2,250 will be the sum of capacities between successive contours from bed of the river to R. L. 2,250. Capacity and area submerged curves can then be drawn. Statement I and Plate I show respectively calculations for and graphs of capacity and area submerged curves for Larji Dam Site on the Beas.

It is obvious that valuable property should not be submerged. This is particularly important in case of low dams, where the area submerged is comparatively large as compared to the storage to be affected and cost of land bears a very high proportion to the total cost of the estimate. G. T. S. plans can usually give a tolerably good idea of what kind of property is likely to be submerged. Thus it can be seen if lot of valuable land or prosperous villages will come in the reservoir. Such tracts should, if possible, be avoided.

4. Runoff should be sufficient. This is a very important, if not the most important factor. Obviously there is no use of spending money on building a costly dam if sufficient water is not available to fill up the reservoir. The most satisfactory method is, of course, to observe regular discharges of the river at the site of the proposed dam for as long a period as possible—30 years record is about the minimum on which a dam can be designed with confidence—unfortunately so much data is available for only one site in the Punjab Himalayas, *i.e.*, Bhakra on the Sutlej. Indirect methods, must, therefore, be adopted to calculate runoff for other sites. A brief description of the methods adopted in the Beas Dams Division for some of the proposed dams is given below.

Chandni on the Giri. The Giri is a tributary of the Jumna and meets it at a distance of 16 miles above Tajewala. Chandni is situated 20 miles above its confluence. Actual discharges at Chandni were available only since 1st July, 1942, but rainfall record in the Jumna Basin above Tajewala was available for a much longer period. Discharge of the Jumna above Tajewala was also known for a pretty long period. Two indirect methods were adopted.

In the first or what may be called the proportional method it was assumed that the runoff at the dam site in any particular period would be given by the formula :—

$$\text{Runoff at Chandni during any period} = \frac{R_1}{R_2} \times \frac{A_1}{A_2} \times Q.$$

Where R_1 = Average rainfall in the Giri River above Chandni during any period.

R_2 = Average rainfall in the Jumna catchment area (including Giri Basin) above Tajewala during the same period.

A_1 = Catchment area of the Giri above Chandni.

A_2 = Catchment area above Tajewala (including Giri Basin).

Q = Runoff above Tajewala during the period.

In the second method, use has been made of rainfall and runoff curve of Messrs Strange and Inglis given on Plate I of Rai Bahadur A. N. Khosla's Paper on Rainfall and Runoff presented to the July 1942 meeting of the Research Committee of Central Board of Irrigation. The curves are reproduced on Plate II.

The results given by the proportional method and Strange's curve agreed very well with the actual discharges. Inglis' curves gave about 25 per cent higher results.

Larji on the Beas. Discharges above this are available since June 1941, but rainfall data is available for a very long period. Data for 3 rain gauge stations was available for the period prior to 1933, for 6 stations in 1933 and 1934, and for 8 stations after 1934. Ratio of actual to calculated discharge was computed for June, July, August and September 1941 and 1942, using 3 stations, 6 stations and 8 stations. These ratios were :—

			Using 3 Stations.	Using 6 Stations.	Using 8 Stations.
Average of	June	1941-42	1·9	1·5	1·52
,,	July	,,	1·4	1·2	1·21
,,	August	,,	1·4	1·2	1·17
,,	September	,,	1·9	1·5	1·66

These ratios were adopted to calculate the anticipated runoff for different periods. Thus to find out the anticipated runoff for June 1929, calculated runoff (assuming 100 per cent runoff) was multiplied by 1·9 and so on.

The ratio $\frac{\text{Actual runoff}}{\text{calculated runoff}}$ is more than unity on account of

(*i*) Inflow from the glaciers at the head of the catchment.

(*ii*) Melting of snow that fell prior to the period under consideration.

(*iii*) Water from springs fed from previous rainfall.

(*iv*) Rainfall stations not located at representative points.

Rohtas on the Khan. In this case no rain gauge station is situated in the catchment of the Khan. Nearest rain-gauge stations are Gujar Khan and Jhelum. Average of the rainfall at these two stations and Strange's curve for rainfall runoff have been used for calculating runoff. Actual discharge observations have just (June 1943) been started.

Rai Bahadur A. N. Khosla's formula. Rai Bahadur Khosla has suggested the formulas given below[1] :—

$$R = P - \frac{T}{2} + C \qquad \ldots \qquad \ldots \qquad \ldots \quad (1)$$

$$R_m = P_m - \frac{T_m}{24} + C_m \qquad \ldots \qquad \ldots \qquad \ldots \quad (2)$$

Where

R = Annual runoff in inches.

P = Annual precipitation in inches.

T = Mean annual temperature in degrees Fah.

C = A constant which allows for catchment characteristics, humidity, glacier contribution, etc., but not for evaporation and transpiration which are covered by the temperature factor $T/2$.

R_m = Monthly runoff in inches.

P_m = Monthly rainfall, inches.

T_m = Mean monthly temperature in degrees Fah.

C_m = A constant for the particular month of the year to which the formula is applied.

Formula (1) fits fairly well the data for 18 American and one British catchments when discharges for long periods say 10 years are considered. Sufficient Indian data does not exist at present to properly test this formula. Data for the Sutlej at Bhakra has, however, been analysed and it appears that C is not a constant for the snow fed Himalayan Rivers, but it is a function of temperature, snowfall prior to and at the end of period under consideration, contribution from glaciers, etc. The formula gives a fairly accurate result for the annual runoff of the river Giri at

(1) Paper on Rainfall and Runoff by Rai Bahadur A. N. Khosla, I. S. E., presented to July 1942 meeting of the Research Officers of C. B. I.

Chandni, but there are wide variations in monthly discharges probably due to time lag, *i. e.*, rainfall at the end of the month gives runoff in succeeding month.

5. Water available for storage. Whole of the annual runoff at any point is not usually available for storage. A net work of canals already exists in the Punjab. These canals have a prior claim on the river water and their requirements must be met before any water can be used for storage. One must, therefore, know how much water is required by each canal at various times of the year and what are the gains and losses in the river between the proposed dam and the offtake of the canal. A knowledge of time lag is also necessary before the supply available for storage can be calculated. This is a very complicated and lengthy process and requires a separate paper for adequate treatment. The subject is not, therefore, dealt with here, but only a reference is given to it so that it is not overlooked.

6. Live and dead storage capacity. Knowing the amount of water required for irrigation and the amount of water likely to be available for storage in different years it is easy enough to determine how much water should be stored. This will be the live storage, *i.e.*, storage which can be utilised from year to year. In addition to this allowance must be made for what is known as dead storage. This is the capacity of the reservoir, which is not available for irrigation. Some silt is deposited in all reservoirs every year and their capacity is reduced. Dead storage capacity is made equal to anticipated silting in 40—100 years. This means that the effective or live storage capacity is not impaired for this period. Dead storage capacity also enables hydro-electric power to be generated even when the reservoir is about to be depleted at the end of the season.

7. Reservoir losses. Some of the water stored must be lost by evaporation and transpiration. Some water must also be absorbed into the soil, but unless this water can escape into another catchment and this is an extremely rare phenomena—it must appear in the stream further downstream. It is available for irrigation and is not, therefore, lost. Evaporation and transpiration need, therefore, be only considered.

On the Boulder Dam in U. S. A. where maximum temperature runs to 128° F in the shade, mean annual temperature and rainfall are about 72° and 10 inches respectively, reservoir losses are assumed as 4 feet per annum over the mean exposed area. At the Shasta Dam, where the mean annual temperature is 62°, the annual loss is estimated at 3·5 feet. For Northern India, 4 feet per annum would be safe for most localities.

The mean exposed area is the area at a level at which the reservoir capacity equals the mean storage during the year, that is, it is the mean of the area at the dead storage level and the area at the full tank level in any year.

Water available for irrigation will be live storage minus reservoir losses.

8. In general there should be no low saddle in the reservoir area. The sum of live and dead storage gives the total storage and the pond level required to affect this storage can be seen from the capacity curve. G. T. S. plans are now studied to ensure that the sides of the gorge are higher than that and in the reservoir area, there is no saddle lower than the required pond level. Later on, this is confirmed by actual surveys at site. Low saddles are sometimes useful as they provide cheap spillways. It is for this reason that the words " in general " have been used above.

9. The site should be geologically sound.

Characteristics of an ideal dam site. According to Mr. J. B. Auden, M.A., Geologist, Geological Survey of India, an ideal dam site should possess, among others the following characteristics[1].

" (*a*) The rocks should be massive, homogeneous and impermeable, both laterally and vertically, throughout the region of the dam, and should possess the minimum of divisional planes, such as bedding, joints, faults, crush zones. Crushing strengths should be high.

(*b*) Following from (1), there should be no intercalations of weak strata, particularly at gentle angles.

(*c*) Chemical solution should be negligible.

(*d*) Seismic thrusts and faults should be absent."

To these may be added ;

(*e*) Bearing and shearing strength of the rock should be high,

(*f*) Rock should be proof against weathering.

(*g*) Axis of the dam should be along the strike of rocks.

(*h*) Strata should dip suitably.

10. Suitability of rocks commonly met with for construction of dams[2].

Crystalline schists (gneiss, mica schists, etc.). Generally gneiss is an excellent rock, but it becomes doubtful if it contains too much mica or weathered felspar or where it changes into mica schist.

Granite comprising all granulous rocks of igneous origin are as strong as, or stronger than concrete and their bearing strength is adequate except in portions weakened by weathering or by structural features such as faults or shear zones. The rock often consists of a net work of fractures which subdivide the mass into big parallelopipedons. Leakage may occur through the joints if the foundation is not properly grouted. The rock is insoluble and open solution channels or cavities do not occur.

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(1) Preliminary Geological Investigation of the sites of two dams on the Giri river, the Larji Dam on the Beas and the Rohtang Tunnel by J. B. Auden dated Simla the 29th October, 1942.

(2) Condensed from Engineering Geology of Dam sites by Warren J. Mead and Geotechnical Studies of Foundation Materials by Mauric Lugeon.

Volcanic rocks are generally not suitable as they are almost invariably crossed by fissures of shrinkage, unless the conditions existing at the site are completely understood and defects in the foundations disclosed by thorough foundation exploration are remedied. This has been done in the case of Boulder and Owyhee dams in U. S. A. and some dams in Western Ghats India.

Sandstone varies in composition and appearance from light coloured quartz sandstone to the darker coloured varieties containing varying amounts of felspar and ferromagnesium minerals, and is formed by the natural consolidation and cementation of sand deposits by infiltration of cementing materials, principally carbonates and silica. The strength varies. Thus while well cemented rock like quartzite is much stronger than cement, poorly cemented sand rock is weak and friable. Even poorly cemented rock has, however, the great advantage of not being susceptible to plastic deformation.

Shearing strength of cubes of sandstone varies with the nature of the cementation and jointing. Sandstone has frequently inter-stratified beds of clay or shale and sliding may occur if the layers dip at gentle angles and lie at shallow depth within the foundations.

Leakage can occur through open joints, fractures, fissures, faults and the inter-granular pores of the rock itself. Cement grout can stop all leakage except through pores. Crushed material can be taken out from faults and crushed zones and replaced by sound concrete. Very permeable sandstone may have to be rejected.

Limestone. Most limestones are as strong as or stronger than concrete, but they are soluble rocks liable to be full of solution channels and caverns. Rock appearing sound at the surface may have inadequate supporting strength due to the existence of concealed solution openings. It is of the greatest importance, therefore, that exploration be carried out in sufficient detail to disclose the existence of any such opening in the foundation or abutments, which might affect the stability of the dam. If a dam is built after thorough foundation, exploration and efficient grouting is done to seal all channels, seams, etc., there is no fear that dangerous solution channels will develop during the life-time of the dam. The rate of solution of limestone, though rapid as a geologic process, is too slow in terms of years to constitute a threat. The concrete of the dam is more vulnerable to solution and alteration than the limestone. There is a possibility of sliding occurring in case of thin-bedded limestone with well-developed bedding planes dipping down-stream at low angles or along inter-stratified shale or clay. Thorough exploration with large diameter core boring drills is needed.

Shale means fine-grained sedimentary rocks which were originally deposited in the form of clay or mud and have undergone various degrees of solidification and lithification by processes of compaction, crystallization and cementation. When originally deposited, clay or mud has a very high content of water. Under the weight of sedimentary material laid upon it, this water is squeezed out and clay is consolidated by compaction.

Factors determining compaction are :—

(*i*) Physical nature of the clay or mud, particularly grain size. More silty clays and clays with inter-stratified beds of pervious sands are unwatered more readily than fat plastic clays.

(*ii*) Thickness of deposit.

(*iii*) Amount of load of super-incumbent sediments.

(*iv*) Length of time of application of load.

(*v*) The amount of deformation of the mass by diastrophic movements.

When water has been squeezed out dry particles are brought into actual contact with one another and a sort of setting or cementing process takes place. This cementation is aided by the presence of calcium carbonate, liberation of silica, and high temperatures, which accompany deep burial and finally a hard dense rock approaching the properties of slate is produced.

Shales may, therefore, be divided into (*i*) compaction shales with little or no inter-granular cementation and (*ii*) shales which have undergone maximum compaction and in addition a cementation or setting process.

Shale of the first class, *i.e.*, compaction shales disintegrate under the action of water subsequent to partial or complete drying. Rapidity and completeness of disintegration largerly depends upon the completeness of previous drying. Well cemented shales do not, however disintegrate when placed in water after previous drying. Compaction shales may also undergo deformation due to the load of super-incumbent beds, when lateral support is removed by erosion. Shale is, therefore, a dangerous material for the foundations of important dams and thorough slaking and other tests are necessary. It is best to avoid shale.

Slate is the cleavable metamorphic equivalent of shale. Its formation is a complex process and involves the generation of new minerals and their re-orientation under pressure. Quality varies considerably. There are very good slates, which can support quite high dams. On the other hand poor slates are not suitable for even moderately high dams. Good slates are as strong as or stronger than concrete and their crushing strength is adequate except in parts weakened by weathering or by structural features such as faults or shear zones. In slates inclination of cleavage planes may be even more important than that of the bedding planes. This should be upstream for stability. At Upper Timra dam site on the Tons river the dip of the strata is obliquely down the face of the left bank at a lower angle than the hill slope with the result that considerable slips have occurred on the left bank of the river while the right bank is quite stable. The site had to be rejected on geological advice as firstly slips were likely to occur in future above and below the dam and secondly it was considered that removal of fallen debris on left bank would make the site so wide that it would be impossible to construct an economical dam at this place.

Cemented and uncemented boulders. These are generally unsuitable for masonry dams.

Alluvial deposit as foundation for earth dams. River deposited sand has a high percentage of voids 45 to 49 per cent. This open packed sand can be compressed, but the amount of compaction is not proportional to the initial percentage of voids. A low dam of great width, if built of impervious material would by its weight compress the sand so much that the loss by leakage is far less than might be anticipated. Silt too has a high percentage of voids 40—55 per cent, is susceptible to compaction, and increases in bearing strength and stability with compaction.

Compaction of sand and silt depends upon the escape of contained water. If load is applied slowly and adequate opportunity is given for escapage of water consolidation occurs. If load is applied too rapidly to permit escapage of water, or this escapage is prevented by less pervious materials, the uncompacted sand and silt tends to yield plastically and flows out from under the superimposed load.

Clay is inherently plastic and has the properties of a viscous fluid. It appears to have a certain inherent capacity to resist deformation but when loaded beyond this limit, rapid movement may take place. Clay is also compacted but it always remains plastic and does not reach the condition of close packing of the particles to become non-plastic like sand and silt. It must, therefore, be considered to be a viscous fluid having little bearing strength unless adequately restrained on all sides and incapable of safely and permanently supporting more than very moderate loads. Safe loads can be determined by investigation of the distribution of shearing stresses and measurements of shearing strength, elastic properties and degree of compaction of the material.

Frequent reference has been made above to the necessity of thorough exploration of foundations. A detailed description of this follows in Chapter V.

11. **Strike and Dip**[1]. Strike of bed, fault or joint is the bearing of the line which a horizontal plane makes with the plane of the bed, fault or joint. The slope or dip of a bed or fault plane is the maximum angle at a given locality, which the bedding plane makes with a horizontal plane. The direction of the dip is always at right angles to the strike at the same place.

In the case of bedded rocks, the axis of the dam should be parallel to the strike of the bedding and the foundations be so laid as to have an apron of an impervious bed under the edge of the dam. This is to reduce leakage. If a dam must be built (though this is not recommended) across the strike of the rocks, it should be located on an alignment, which

1. See Engineering Geology by Dr. C. S. Fox, for further details.

has the fewest disadvantages and no serious defects. In such cases the degree of dip, number of possible percolation channels which cross the dam, texture and condition of various joints become very important factors in making a choice. When the dips are gentle and the strike is only slightly oblique to the axis of the dam, there may be only a few bedding planes through which leakage can occur. On the other hand, if these beds are thin and sloping at steep angles, with a strike transverse to the dam, there will be numerous planes of percolation. Should the axis of the dam, be parallel to the strike, strata should dip at steep angles. This reduces chances of the dam sliding on its foundations and also minimises leakage. The nature of the joints must also be considered as bedding plane dips may be less important than joint dips.

Typical cross sections of some mountain valleys are given below :—

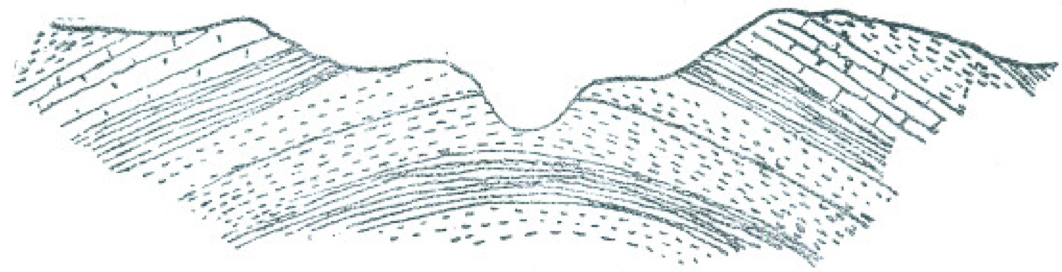

FIG 1.

Figure 1. Bed and bank of the stream consist of marls which are overlaid by porous sand strata. A dam built in this valley will be water-tight up to top of marl : above that level leakage takes place through the sandstone round the abutments of the dam. Even if porous sandstone did not exist, leakage will take place through the bedding planes as the dam is in a strike valley. Site is quite unsuitable for an important dam.

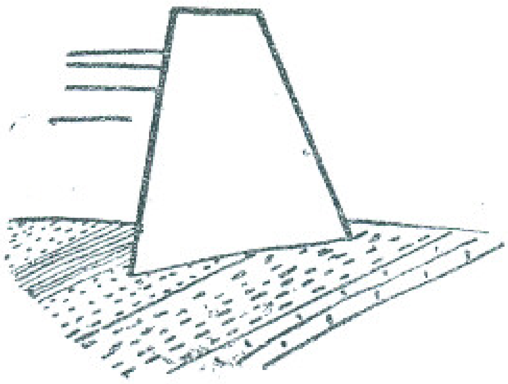

FIG: 2

Figure 2. Axis of the dam is along the strike and an impervious bed serves as an apron on the upstream side of the dam. Dips are upstream. This is an ideal site for a dam.

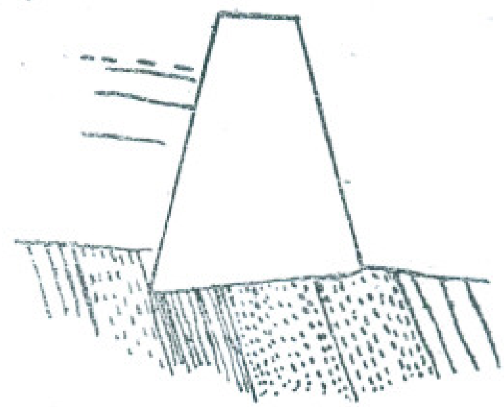

FIG: 3

Figure 3. Axis of dam is parallel to the strike. An impervious bed is on the inner side of the dam and strata dip steeply downstream. Structural features of this site are good.

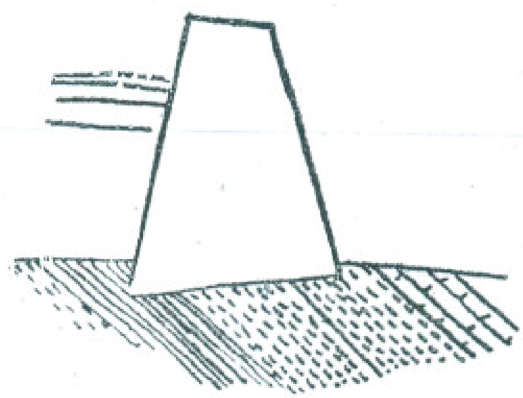

FIG. 4

Figure 4. Axis of dam is along the strike with low downstream dips. A bed of shale is on the inner side of the dam, while a band of sandstone supports the greater part of the weight of the dam. The site is inferior to the two previous cases.

12. Geological Maps.[1] An ordinary G. T. S. map with topographical features clearly shown, forms the basis of a geological map. Different kinds of rock are coloured in a distinctive way, *e. g.*, sandstone, yellow ; shales, grey ; limestone, blue ; granite, pink ; dolorite, green, etc. Position of the outcrops, dips, faults, etc., of the rocks, which occur in the area, are shown by suitable signs. If the geological details are intricate, a larger scale map is required. If the direction and degree of the dips and faults, etc. are accurately delineated, cross sections can be constructed from the map. It is possible to form an idea of the structure of the country by studying a geological map. This is good enough for a start, but later on,

1 For detailed description see Engineering Geology by Dr. C. S. Fox.

a dam site must be studied in detail by an experienced geologist. In his letter to Executive Engineer Beas Dams Division regarding possible dam sites on the Tons and the Jumna Mr. J. B. Auden, Geologist of the Geological Survey of India, says[1] "I do not think that any sound geological opinion about a dam site can be given unless the site is actually examined. In a regional geological survey covering over 10,000, square miles of the Himalayas, it is naturally impossible to study the structural geology in the detail required for a dam investigation. Moreover, the broad geological divisions include many lithological types which are common to some or all of them, so that the required data are not often likely to be obtained from a 1″ = 1 mile geological map."

The Geological Survey of India has prepared geological maps for considerable areas of the Himalayas, some of which have been published. Unfortunately these maps are not usually available in the Irrigation Branch Offices. Some idea of the strike can often be obtained from a study of the ordinary G. T. S. plans. Direction of the strike is generally the same as that of mountain ranges and spurs. Should the plan show that a mountain range or an important spur crosses a river at right angles, it can be assumed that the axis of the dam constructed at that place would be almost parallel to the strike of the rocks. These plans must however, be used with caution as it sometimes happens (*e. g.*, in case of Tons site) that the topography bears no relation to the geology. It is, therefore, essential that the results should be confirmed by actual surveys at site at the earliest opportunity. The dip of rock also requires study at site. Ordinary G. T. S. plans do not give any information regarding these.

13. The catchment should consist of firm rocks. The subject of silting of reservoirs has been exhaustively discussed by Rai Bahadur A. N. Khosla, I.S.E., and his note[2] on silting of reservoirs is well worth a careful study. In general, it may be stated that the silting of any reservoir will depend upon :—

(*i*) Type and structure of rocks in the catchment. Other conditions remaining the same, the more friable the rock, the more the reservoir will silt and *vice versa*.

(*ii*) Range of temperature. Greater the range of temperature the more the weathering of rocks and higher the silting.

(*iii*) Intensity of rainfall and magnitude of runoff. Silting is directly proportional to these.

(*iv*) Vegetable cover. The poorer the cover the greater the erosion of rocks and silting of reservoir.

(*v*) Slopes. Steep slopes are more liable to erosion than flat slopes.

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1. Letter of Mr. J. B. Auden quoted in the Report on 1939-42 Bhakra Dam Project Volume V, page 124.

2. Note on Silting of Reservoirs by Rai Bahadur A. N. Khosla, I. S. E., Superintending Engineer, Punjab Irrigation.

14. Site should be assessible. This is self evident. If the site is inaccessible, not only will inspections be difficult, but it will be very expensive to carry heavy and delicate machinery and other materials of construction to the site of work.

15. Materials of construction should be available near the dam site. The most important materials in the case of a concrete dam are cement, aggregate and sand. Should it be possible to manufacture cement near the site of work, the cost of the dam can be reduced by many lacs. The cost of the dam may become prohibitively high if good aggregate and sand are not available within a reasonable distance of the site. Plenty of good water is, of course, a necessity. If hydro-electric power can be cheaply produced, it will be a great help towards construction.

16. Natural facilities should be available for diverting river during the construction and for disposal of surplus water afterwards. This is self evident. The cheaper these arrangements can be the cheaper will be the dam. This is particularly important in the case of low dams where the cost of disposal of surplus water forms a considerable proportion of the total project cost. If it is possible to put this water in a stream of sufficient capacity the cost of the outfall channel can be cheapened and an economic dam will be possible. On the contrary, if a new long outfall channel has to be constructed or special arrangements like tunnelling have to be provided for the disposal of the surplus water, the dam would naturally become very costly.

CHAPTER III.

Investigation of Storage Schemes. 1. The usual problem that presents itself to an engineer is that he is asked to provide water for the irrigation of a particular tract of land. In this case he knows the gross area to be irrigated. Before he can start with the investigation of storage possibilities, he must know how much water is required for the irrigation of the particular plot of land. This in its turn will depend upon :—

1. Net culturable commanded area, *i. e.*, gross area minus Ghair Mumkin. Ghair Mumkin may consist of land under roads, abadies, canals, graveyards and land, which is not fit to be cultivated.
2. Area desired to be irrigated annually which depends upon annual permissible intensity.
3. Full supply factor.
4. Minimum number of days for which a canal must run.

A detailed discussion of the above factors is outside the scope of the present paper. It will be assumed that the engineer knows how much water is required for the irrigation of the tract. For the new projects hardly any water will be available for irrigation direct from river except during the monsoon months. During the remaining months water will have to come from storage. The engineer allows for the absorption between the dam site and the canal head and then calculates the water required for storage for use during the months the direct supply from the river is not available.

2. The next step would be to consult G. T. S. and the geological maps. All sites which fulfil conditions (1), (2), (5), (6) and (7) of Paragraph 2.1, will be marked on the plans. A detailed description of how the plans can be used to find this out has already been given in Chapter II. Capacity and area submerged curves will then be prepared for each of these sites and any sites, which require proportionately high dams for small storage, will be rejected.

3. The next step is to calculate runoff for each of the likely sites. This can be done in accordance with Paragraph 2.4. Any site where storage is not sufficient to meet the requirements, will be given up. Should sufficient storage be not possible at any site, the only alternative will be either to reduce intensity or to cut out area. This finishes the work in the office and field work can be started at this stage.

4. The first thing is a careful inspection of the sites. It sometimes happens that sites which appear to be very good on the plans are found not so favourable at site. Conversely, though rarely, inspection can reveal sites, which do not appear to be promising on the plans, but which are quite good. While inspecting the sites the considerations discussed in detail in Chapter II, should be carefully borne in mind. Detailed surveys can start for any sites which are found to be satisfactory. A description of the surveys follows in the next chapter.

5. A careful examination by a trained geologist is an absolute necessity. This should be done as early as possible as should the geologist reject any site as geologically unsound, money spent on detailed surveys will have been wasted.

CHAPTER IV.

Detailed surveys

1. Detailed surveys of dam sites consist of :—

1. Establishment of Bench Marks.
2. Observing cross sections of the gorge at the dam site.
3. Observing longitudinal section of the river.
4. Detailed survey of the gorge.
5. Detailed survey of the reservoir.
6. Survey of the station area.
7. Preparation of statements of property likely to be submerged.
8. Construction of burjies.
9. Observing discharges.
10. Investigation for the highest flood.
11. Investigation of road.
12. Investigation for materials of construction.
13. Taking out of stone samples and foundation exploration.
14. Analysis of silt samples.

2. Establishment of Bench Marks. It will be necessary to establish reliable bench marks at the dam site as well as in the reservoir area. For this purpose precise double levelling on "follow on" system is started from a reliable G. T. S. Bench Mark. If a reliable Bench Mark is not available within a reasonable distance of the dam site double levelling can be started from a G. T. S. triangulation point. Pamphlets called 'Triangulation in India and adjacent countries' give the details of all primary, secondary and tertiary triangulation points. The levels recorded against the triangulation points are said to be correct within a foot and these points can be used to start double levelling for the preliminary surveys. They are quite good enough for the purpose. Bench marks should be established at the dam site and one to two miles apart in the reservoir area.

3. Observing cross section of the gorge at the dam site. This can be done in the majority of cases by means of a level with stadia. Points are fixed on one bank by means of levelling and stadia readings are taken at 50 ft. vertical intervals to a staff held on the other bank. If the sides of the gorge are very uneven, the vertical interval can be reduced. In some cases the sides of the gorge are so steep and slippery that it is not possible to do any actual levelling on even one bank. In this case cross section can be observed by means of a theodolite. A base line is fixed in the bed of the river and vertical and horizontal angles can be read to the points, where slope changes.

4. Observing longitudinal section of the river. Longitudinal section of the river is prepared from the point where the pond is likely to end to about half mile below the gorge. This longitudinal section gives valuable information regarding the slope of the river.

5. **Detailed surveys of the gorge.** Usual scales adopted for this purpose are 1/2000 and 1/1000. Generally the detailed survey is done for a distance of 2,000 ft. above and 3,000 ft. below the axis of the dam. The detailed survey consists of contoured plane table survey of the gorge. The interval between successive contours depends upon local circumstances. If the sides of the gorge are steep, the vertical interval can be greater. On the contrary the contours are drawn closer if the side slope is gradual. For high dams, where the side slopes are neither very steep nor gradual, a suitable interval is 20 feet. For low dams contouring is done 10 feet apart.

The plane tabling is done with reference to G. T. S. triangulation and intersection points. Co-ordinates of these are recorded in the G. T. S. pamphlet known as "Triangulation in India and adjacent countries." A separate pamphlet is published for each degree of latitude and longitude and they can be obtained on payment from the officer-in-charge, Map Record Office, Dehra Dun. In some cases the values of co-ordinates are not recorded but latitude and longitude of the points are given. In this case co-ordinates have first to be calculated. A grid is ruled on plane table sheets and all G. T. S. triangulation and intersection points available in the locality are marked on these plans. Then these points are actually searched and a flag is erected at each point. From these points more flags conveniently situated for plane tabling are intersected and only those points, through which three rays with good intersection pass, are accepted. The position of the starting point is found with reference to the original G. T. S. and the newly intersected points. To start contouring, a peg is fixed on the required level of the contour from the nearest bench mark by means of precise double levelling. The leveller then lays down the contour on the ground. He works in conjunction with another Subordinate on the plane table. The plane tabler marks the contours on his table by means of the backward and forward way method, and the stadiá distances given to him by the leveller. The plane tabler also fills in other topographical details. There being plenty of intersection points, the plane tabler can easily check his position at any point.

In some cases the sides of the gorge are so steep that no actual levelling is possible. In this case flags are fixed at all points where the slope or topography changes and the position of these points is fixed by means of theodolite or plane table triangulation. A base line is first carefully laid in the bed of the river. A number of points are fixed on this base line and a theodolite open traverse and precise double levelling are carried out. Then all these points can be accurately laid on the plane table sheet, and their reduced levels are known. From these points rays are taken to the flags fixed on the sides of the gorge and the position of these flags shown on the plan. Triangulation can be carried out by means of a theodolite or plane table. The former is accurate but is extremely slow and is not recommended. It has been found that plane table triangulation is good enough. Reduced levels of these flags are observed by means of a clinometer or an Abney's level. After the position and reduced levels of the various flags have been plotted on the plan, contouring can be done. It is best to do this contouring at site. It has been found that contouring done in the office is not satisfactory. It is physically impossible to observe the position of all points where levels or topography change and much has to be left to the judgment of the surveyer. It is quite easy to draw accurate contours if the work is carried out at site. Due allowance can then be made for points, which are not marked on the plan but where levels or topography change.

6. **Detailed survey of the reservoir**. In the case of high dams, where plans to a scale, 1″=one mile, or to larger scales are available, it is only necessary to mark the pond level contour at site. The 50 feet contours shown on the 1″=one mile or larger plans are fairly accurate and satisfactory capacity and area submerged curves can be prepared from these contours. Should these plans be not available, contours, at not more than 100 feet intervals will have to be laid on the ground in order to prepare accurately the capacity curve of the reservoir.

The pond level and other contours are usually marked by means of levelling and plane tabling. The method is the same as has already been described for the detailed survey of the gorge. The contours are frequently checked at the bench marks established in the reservoir area.

In some cases, there is so much jungle on the sides of the stream that this method is not feasible. Many States object to destruction of jungle growth. Where it is not permissible to cut jungle, other means must be adopted. In such cases cross sections of the river valley are observed one to two miles apart and contouring is done by means of Abney's level. The contouring is started from a point with known level and is checked at every cross section. The method of using the Abney's level is given in all text books.

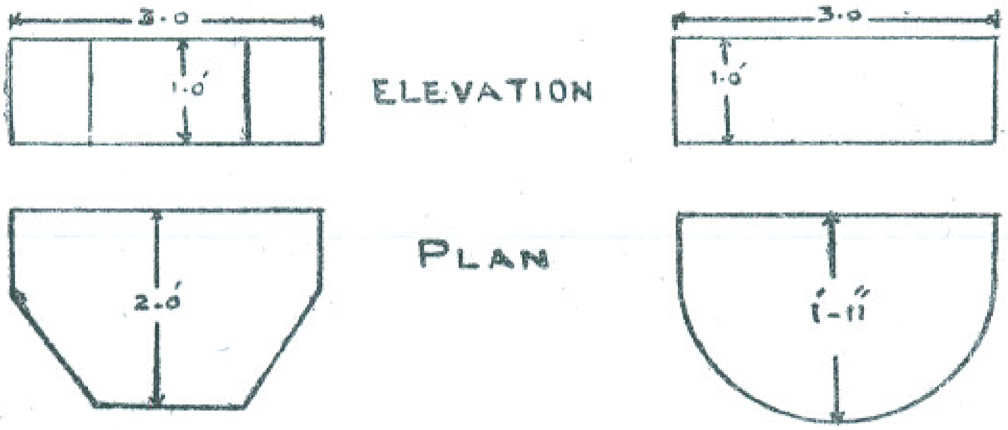
A suitable scale for this work is 4″=1 mile.

7. **Survey of the station area.** A fairly open tract of sufficient area is selected as near the dam site as possible and contoured and plane tabled. Sufficient area should be available for the construction of residential and non-residential buildings for the staff, workshop, etc. The area is usually selected on the upstream side of the dam, so that a good view of the reservoir is available.

8. **Preparation of statements of property likely to be submerged.** Two patwaris work with the leveller and the plane tabler when the pond level contour is being marked. These patwaris mark this contour on their shajras and they thus know what area will be submerged. They can then calculate the total area that will be submerged and area under different classes of land. Detailed statements are also prepared for residential and non-residential buildings which will be submerged.

Plinth areas are measured and type of each building, *e. g.*, pucca, katcha, cow-sheds, etc. is carefully noted. A census of the population in the reservoir is also taken. Arrangements will have to be made for shifting this population elsewhere and it is desirable to know how much population will have to be dealt with and what provision will be required for their shifting.

9. **Construction of burjies.** Cement concrete burjies are constructed at the dam site at 50 feet vertical intervals in case of high dams and 15 to 20 feet vertical intervals in the case of low dams. In addition, they are constructed in the reservoir area on all prominent spurs and near important villages. A suitable distance between the burjies is ½ to 1 mile. A type plan of the burjies constructed in the Beas Dams Division is shown below.



10. **Observing discharges.** If the discharges of the river at the place, where a dam is proposed to be constructed, are not known for a long period and this is unfortunately the case in almost every site in the Punjab, it is absolutely essential to establish a discharge site as soon as it is found that it might be feasible to construct a dam at any place. The discharge site must be selected by a responsible officer. It is not proposed to discuss the methods of discharge observations in this Paper.

They are already well known.

11. **Investigation for the highest flood.** In order to determine the maximum discharge, that has ever passed in the river, an enquiry is first made from a number of reliable old people in the locality. They are asked to show the level attained by the maximum flood in the river at various points. All these points are then levelled over to see if the levels given by different individuals agree with one another. Any doubtful points are rejected and only those points, which are corroborated by the largest number of reliable inhabitants, are accepted. It is assumed that the slopes of the river at the time of the maximum flood and at the time of the highest flood during the period under observation are the same. It is further assumed that the bed of the river at the time of the highest

flood was the same as at the present time. The value of Kutter's N is generally assumed as 0·035 in the case of a river with boulder bed and 0·025 in the case of rivers with sandy beds. In cases, where the site shows that these values are not accurate, they may be modified. These assumptions are not strictly accurate but they have to be adopted for want of a better method. The discharge is then calculated by means of Kutter's formula. This investigation is carried out at a number of places and the results correlated. It is usually possible to say what is the approximate value of the maximum discharge that has ever run in the river. When this investigation is in progress, rainfall records are also studied. Some idea of the maximum discharge that has passed, can be formed from the records and in many cases correct date of the highest flood can be found from these records.

12. Investigation of road. This follows the method described in the text-books and it is not proposed to deal with this matter in this Paper.

13. Investigation for materials of construction. The principal construction materials are cement, bajri and sand and it should be investigated where these materials can be manufactured or are available. Help of a geologist is usually necessary for carrying this out satisfactorily. Laboratory tests are also recommended.

14. Taking out of stone samples and foundation exploration. This item is discussed in Chapter V.

15. Analysis of silt samples. Silt samples are taken daily during the months of June, July, August and September at 0·6 D at three points in a cross section and analysed for coarse, medium and fine silt so as to find out the quantity of suspended silt in water. From this some idea regarding the probable rate at which the reservoir will silt can be formed. For the formation of a complete picture, a knowledge of the quantity and grade of rolling bed silt is also necessary. But unfortunately, no satisfactory method of measuring the quantity of rolling silt has so far been devised. Until this information is available, engineers can only study quantity of suspended silt and records of silting of reservoirs already in use.

During floods it is not possible in many cases to take a sample from the centre of the river. In this case samples are only taken from the sides.

It is realised that the method described above is unsatisfactory and is only a makeshift. It would be useful to carefully study the records of the existing reservoirs. Capacity surveys should be carried out at regular intervals on all such reservoirs and other relevant factors studied. In the course of time it should be possible to forecast with a fair degree of accuracy the rate of silting of any particular reservoir.

CHAPTER V.

Foundation Exploration

1. Necessity of Foundation Exploration. The safety of a dam depends upon the foundation on which it rests, and the importance of thorough foundation exploration is obvious. Most of the failures of dams can be directly attributed to inadequate foundation exploration; the defects in the foundation were not properly investigated by the engineer with the result that they were not remedied. It is now believed that a dam of some type can be designed for and built at almost any site, if the conditions existing at the site are completely understood. In America and other countries, where high dams are built, large sums of money are spent on foundation exploration, and operations sometimes extend over many years.

The term "foundation" used here is meant to denote not only the area on which the dam rests, but also the abutments, and the areas upstream and downstream, which would be subject to stresses set up by the structure.

2. Foundation exploration at a site may be divided into three parts with different objectives :—

(*a*) Preliminary investigations.

(*b*) Detailed investigations in the design stage.

(*c*) Continuing of investigations during the construction period.

Preliminary investigations are meant to determine the feasibility of a dam at a site before starting time consuming and expensive surveys. It may be that the site is situated on a seismic fault, or the rock is not strong enough to support a dam of the required height or the strata is so much intersected by joints, fissures, solution channels, etc., that it is not economically possible to make it water-tight. Such a site would be probably eliminated by preliminary investigations and the engineer would be able to concentrate on some better site. Once the engineer has made himself reasonably certain that a dam can be safely founded at a site, he can go ahead with detailed investigations for design purposes.

It is necessary to continue foundation exploration during the construction period too, due to the fact that however detailed foundation explorations might have been, there are always some defects, which are brought to light only when stripping of the rock has been carried out.

3. Preliminary investigations. Preliminary foundation exploration necessary at a site would depend upon the geological conditions present at the site. Broadly speaking, they may be divided into following operations :—

(1) Preliminary Geological examination of the site.

(2) Preliminary tests to determine the strength of the rock.

(3) Preliminary tests to determine water-tightness of the foundation.

4. Geological examination of the site. The first step after the engineer has selected a site, which appears to him the most suitable topographically and otherwise, is to arrange a joint inspection of the site with a competent geologist. The Geological Survey of India have mapped parts of the Himalayas and allied mountains and it is almost always possible for them to depute an experienced geologist, who has mapped the locality, in which the site is situated. This officer would recognise the rocks and say generally whether they are suitable or unsuitable for a dam. He would investigate the character of the strata, measure dip, strike, stratum thickness of rock, etc. He may be able to bring to light faults, crushed zones, liability of the rock to slip, etc., and say whether the faults are seismic or not. This preliminary inspection of the site by a geologist is found very helpful and should be invariably carried out before starting further investigations.

5. Preliminary tests to determine strength of rock. Tests are carried out to determine the following :—

(*a*) Crushing strength of the rock.

(*b*) Bearing capacity of the rock *in situ*.

(*c*) Shear strength of the rock in compression.

(*d*) Soundness of rock.

Crushing strength of the rock. It is determined by crushing rock samples taken from the site. As at almost all sites, the rock strata changes rapidly both horizontally and vertically, good many samples should be taken to represent all types of rock. Samples should be taken at water level and from the rock higher up the gorge too. Care should be taken to extract samples from unweathered rock only. For this it would be necessary to remove weathered rock from the surface for a few feet. The size of the samples would depend upon the capacity of the crushing machine. The bigger the samples that can be crushed, the better it is. In the Beas Dams Division, samples were crushed at the Government Test House, Alipur, and at Bhupindra Cement Works, Surajpur. The capacity of the testing machines at these places is 100 tons. Cubes about 6″ in size were taken from the site. These were reduced to the size that could be crushed by the machine, dressed true and square, faces rendered with Plaster of Paris, and then laid on the pressure plates of the testing machine. The load was always applied to the face normal to bedding planes. This is very important as most of the rocks possess much greater strength normal to the bedding planes than across it. It is generally difficult to mark bedding planes once the sample has been taken out, so they should be marked at the time of taking out by a person, who is familiar with the geology of the site. To illustrate the above, results of tests carried on rock samples from the Larji Dam site on the Beas River are given below :—

Description.		Area in square inches.	Ultimate crushing load in Tons.	Ultimate crushing strength in Tons per square inch.
Cube 3·09″	...	9·55	Not crushed at 87·95	Greater than 9·21
Cube 3·07″	...	9·42	64·73	6·87
Cube 2·58″	...	6·66″	75·45	11·33
Cube 3·03	...	9·18	56·70	6·18

Samples should also be tested wet after soaking in water for at least 21 days. In some cases, ultimate crushing strength of the rock is considerably reduced when tested wet. This is more so in the case of rocks with poor cementing materials.

The wet-dry ratio from some sandstones in Panama is about 0·65. A conglomerate may have a dry crushing strength of 500 lbs. per square inch and crumble under water. The ultimate crushing strength, determined by the above method by crushing unconfined cubes, does not represent the ultimate crushing strength of the rock *in situ* which would be much greater. It is only a point of reference. For design purposes, it is usual to adopt a factor of safety not less than ten on the average strength of the samples and not less than eight on the strength of the weakest sample.

Bearing capacity of rock in situ. The term " bearing capacity " is used to denote supporting characteristics of the rock. Tests are carried out on the rock *in situ*.

A suitable apparatus, designed for carrying out tests at the Bhakra Dam site is shown in plate III. The apparatus consists of a hydraulic jack to apply pressure, a column to transmit pressure, standards to support gauges, and about half a dozen gauges reading up to one thousandth of an inch to measure deflection. The method adopted for carrying out the tests at Bhakra Dam site is described below. The hydraulic jack used was of 100 ton capacity but it was felt that a jack of higher capacity, which was not available then, would have been more useful.

A drift 6 feet by 4 feet in section and 25 feet in length was driven at right angles to the gorge wall. The apparatus was set near the end of the drift. A plate 12″ × 12″ was placed under the hydraulic jack and the column was set on the hydraulic jack absolutely vertical. The gauges were set as shown in plate III. A plate 12″ × 12″ was placed at the top of the column and this plate was brought in contact with the rock, which was truly dressed and tested with a plumb bob and try square to see that it was perfectly horizontal and free from undulations. To ensure perfect smoothness of the surface, it was covered with a mortar layer 1/16″ thick consisting of one part soda, five parts cement and one part sand. To eliminate the compression of this mortar layer during the experiments on the rock it was compressed to the maximum possible by raising the pressure on it and keeping it constant there for a few hours before relieving. During experiments on the rock, the pressure on the plate was slowly raised to 100 tons, and then gradually relieved. The time taken for these operations was made different in different experiments and extended over many hours. Deflections were messured when raising or lowering the pressure at different intervals of pressure. It was found that pressure exerted by a 12″ × 12″ plate was not enough for the purpose, so smaller plates up to 2″ × 2″ in size were used. The rock was crushed by a 2″ × 2″ plate only at a minimum pressure of 67 tons. This gives minimum ultimate crushing strength of the rock as 16·75 tons as compared to 3 tons per square inch determined by crushing tests on unconfined blocks of the rock.

The above tests gave the bearing strength of the rock all right but it was found that the deflections measured by the gauges were erroneous as the column, plate and mortar were also compressed. The apparatus was, consequently, improved to measure the deflection of the rock directly. A gauge was set at the centre of the rock under test by means of a hole through the plate applying pressure and two gauges were set on the rock just as the edges of the plate. The deflections were plotted against pressure and some of the results are shown on plate IV. Two curves are shown in each case ; one represents the deflection when the load is being increased and the other when it is being diminished. The idea was to determine at what load did the rock cease to behave like an elastic solid *i. e.*, to find out its yield point. The object was not, however, wholly achieved on account of insufficient capacity of the hydraulic jack.

The bearing strength of the rock determined by the above tests on rock *in situ* would not necessarily represent the supporting strength of the foundation as a mass. The reason for this is that the area on which tests are carried out is very small. The rock at almost every dam site is intersected by joints, fissures, bedding planes, etc., and there may be some blocks which in spite of thorough grouting would read just themselves under heavy loads causing settlement or movement, which may not be safe.

Shearing strength of rock. It is now accepted by all that failure of a dam built by approved methods cannot take place by sliding without involving shearing of the rock or concrete first unless the bedding of the foundation rock is so horizontal that layers can slip on one another. Such a foundation would, however, be rejected as a rule.

In 1933, the late Mr. D. C. Henry, M. A. M. Soc. C. E., consulting engineer on many of the dams built by the United States Bureau of reclamation, proposed a method for calculating the factor of safety (called shear friction factor of safety) against downstream movement including allowance for shearing strength. The formula he put forward was :—

Shear friction factor of safety (Q) =
(weight-uplift) (Coefficient of Internal Friction) + (thickness of base) (Average shear strength of rock).

The coefficient of internal friction varies from .60 to .70 with different types of rocks. Mr. Henry took its value as 0·7 and recommended that shear friction factor of safety calculated by the above formula should not be less than four. Since that time, engineers of the United States Bureau of Reclamation have been considering the stability of gravity dams on the basis of the shear friction factor of safety as recommended by Mr. Henry, but the value of coefficient of internal friction is taken as .65 and the value of shear friction safety not less than five during the most severe conditions of reservoir load combined with maximum horizontal and vertical earthquake accelerations.

The principal uncertainty involved in evaluating the shear friction factor of safety is the shearing strength of the concrete and the rock. The proper value of shear strength to use in the formula is the shear strength of the rock if the rock is weaker than the concrete or of the concrete if the concrete is weaker than the rock.

Mr. Henry determined the shear strength of rock and concrete by crushing unconfined cylinders with length at least twice the diameter. The angle of break, diameter of the cylinder and the load applied are carefully measured. The shearing strength is given by the formula

$$S=\frac{P^{1}}{2\ \mathrm{tans}(\varrho)}$$

where

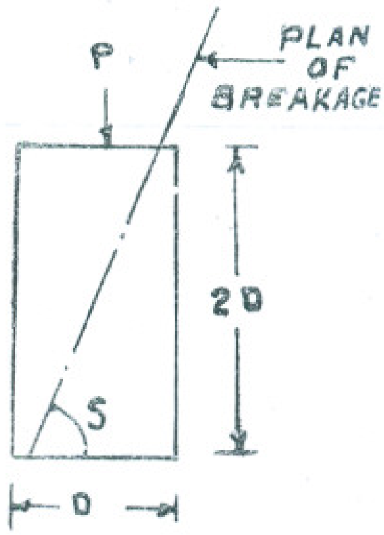
S = The ultimate shearing strength

$P = \text{Load per square inch} = \frac{P}{A}$

= Angle of break with horizontal plane

P = Crushing load.

A = Area of the face of the cylinder.



The maximum shearing force along the base or any horizontal plane in the dam if loaded to failure would be less than twice the average shear force, so safe value for average shear strength is half of the ultimate shearing resistance as determined by the above method.

In the Beas Dams Division, shearing strength of the rock has been determined by crushing cylinders with length twice the diameter. In these tests also, it is necessary to apply load to the face normal to bedding planes as shearing strength of the rock is much less along the bedding planes than across it. The results of tests carried on Bhakra rock are given below as an example :—

Crushing load tons.	Angle of Break.	Ultimate shearing strength per square inch.	Average shearing strength.
4·58	74·4°	0·64 ton = 1,434 lbs.	717 lbs.
4·61	70·4°	0·82 ,, = 1,857 lbs.	919 lbs.
3·16	71·4°	0·53 ,, = 11,87 lbs.	594 lbs.
4·02	70·4°	0·72 ,, = 1,613 lbs.	807 lbs.
3·16	70·4°	0·56 ,, = 1,254 lbs.	594 lbs.
2·83	72·0°	0·46 ,, = 1,030 lbs.	560 lbs.

1. The proof of the formula is given in most of the text-books on "strength of materials" "see Strength of Materials," by Swain page 118.

The above tests carried on unconfined cylinders give only approximate value of shearing strength. The true shearing resistance can be determined from properly conducted triaxial compression tests on cylinders with length at least twice the diameter[1].

Soundness of rock. Preliminary tests may consist of mineral classification, chemical analysis, determination of specific gravity and weathering resistance of the rock.

Minerals classification would be carried out by a competent petrologist and chemical analysis by a chemist to find out if the rock contains any deleterious minerals, which would be soluble in water or react harmfully with water or cement or would cause disintegration of the rock under new conditions.

Specific gravity can be determined in the usual manner. High specific gravity would show that the rock is compact, or made up of minerals with high specific gravity. The minerals may themselves, however, be only loosely coherent.

Weathering resistance of the rock is determined by subjecting rock samples to alternate cycles of soaking in sodium sulphate solution, and drying in warm air, and to alternate cycles of freezing and thawing when immersed in water.

6. Water-tightness of the foundation. This would depend upon the permeability and solubility of the rock at the dam site and other geologic conditions such as, dip, presence of fissures, joints, solution channels, faults, etc. With very few exceptions, no rock is free from above defects, but every foundation, if defects are completely known, can be made reasonably water-tight by grouting. It is only an economic problem.

Permeability of the rock at a dam site would not be generally serious. There would be loss of water which may be uneconomic in a small reservoir. On the other hand, if the rocks of the reservoir area are pervious there may be serious loss through sites other than the dam and it may not be possible to fill or retain water in the reservoir for the required period. Solid granites, gneiss, micaceous schist, clay slates, marl, and clay are considered more or less impermeable. Clay slate limestone, marl limestone, and clay sandstones are considered semi permeable. Slightly clay limestones, slightly marl sandstone, uncemented sandstone, sand containing very little clays, dolomitic limestones, are considered permeable.

Solubility of the rock at the dam site. Limestone, marble dolomite and gypsum are considered geologically soluble rocks. The solubility of these rocks excepting that of gypsum does not constitute a serious threat during the life time of a reservoir. However, in the case of these soluble rocks, very detailed foundation exploration is necessary before the scheme is definitely taken up.

1. For details of this method see "The shearing resistance of Soils" by L. Jurgenson, Journal, Boston Soc. of Civ. Engineer, July 1934.

A soluble rock is liable to be cavernous and intersected by solution channels, which have to be located by drilling holes, electrical prospecting, etc. These methods have not been carried out so far in the Beas Dams Division but as carried out in other countries are described, briefly below.

7. Exploratory Drilling. Various types of core drills are in use—the diamond drills, the shot drills and some of the newer devices with cutting points of special alloy steel for use, mainly, in the sedimentary rocks. The diamond drill is best for use in harder rocks and is probably a little faster than the shot drill. The more usual practice is to drill vertical holes, but inclined holes are entirely practicable and are more useful in bad foundations requiring extra precaution, or where vertical or nearly vertical seams are met as more area is covered.

The required number and depth of drill holes well depend mainly upon the local rock structure and upon the magnitude and importance of the proposed dam. They should be adequate to reveal, definitely, the character of the foundation rock. The core should be labelled and indexed methodically for both immediate and subsequent examination. All non-recovery sections of the core should have dummy fillers so marked as to indicate probable reason for the non-recovery.

In general, crystalline rocks such as granite and thoroughly indurated sedimentary rocks such as quartzite yield the best cores. The softer sedimentary rocks, such as loosely bound sandstone and shales, yield poor cores. A poor core recovery may mean a poor rock throughout; it may mean the piercing of crushed zones and seams in otherwise good rock, and it may mean too small a drill hole, or poor equipment or poor technique. Core less than about 1″ in diameter should generally be avoided because of their greater breakage tendency and their greater susceptibility to failure under inexpert drill operation. Diamond drill cores of a diameter from $1\frac{1}{4}''$ to 2″ are the most desirable, and shot drill cores, for best results should generally be from 3″ to 5″ in diameter.

In drilling operations, ground water table should always be located, if possible, its elevation carefully noted, and its fluctuation during progress of the work recorded and studied. Similarly, any water flows or seepage encountered in drilling should be noted as to position and volume, for such data serve as partial indices of rock tightness. The major facts to be observed, in the process of core drilling, may be listed as follows :—(*a*) Depth of overburden, (*b*) depth to ground water table, (*c*) depth at which drilling water escapes, (*d*) character and classification of rock, (*e*) condition and quality of rock, (*f*) evidence of processes affecting the rock, such as crushing, weathering and other forms of decay, and solution effects, (*g*) non-recovery of core and the reasons therefore, (*h*) porosity and permeability of rock and seepage conditions in the rock structure as a whole, (*i*) depths and rates of leakage into hole from ground water (*j*) rate of leakage out of hole under drilling pressure or higher testing pressure.

Large size shot drills :—Large holes about 36 inches in diameter are now drilled almost at every dam site. The drill leaves a clear hole with the rock walls undisturbed. The geologist and the engineer can descend into these holes and study rock *in situ.*

8. **Electrical prospecting.** In the last few years, electrical prospecting by resistivity measurements has been applied to the study of most of the dam sites in U. S. A., Canada and North Africa. Rocks conduct electrical current by means of their imbibed water. The more water they contain and the richer this water is in dissovled salts, the more conductive will be the rocks. Fresh rocks such as granite, gneiss, marble and in general all compact metamorphic rocks, in which the spaces or pores constitute only a very small fraction of the total volume have a high resistivity of from 200 to 3,000 ohms. On the contrary, rocks which contain an appreciable quantity of water, such as clays, marls, soft limestones as well as shattered zones and water bearing faults, have a low resistivity of from 10 to 30 ohms. Resistivity depends partly on porosity and quantity of included water and partly on the nature of the salts dissolved in the water. By this method it is thus possible to study thickness of overburden, permeability of rock, the contact of two rocks, passage of faults, etc.

The method used is very simple and is described in detail in an article 'Application of Electrical Prospecting to the study of dam sites' by C. M. Schlumberger, page 67, second Congress on large dams Washington, D. C., 1936. The equipment required is very light and one engineer with two or three unskilled helpers can carry out the studies.

Statement No. 1.

CAPACITY OF LARJI RESERVOIR

Height of dam.	Reduced levels.	Area submerged at each contour.	Mean area.	Capacity at each contour.	Total Capacity in ft. acres.
732	3,820	10,676·0	...	17,37,032	25,75,643
512	3,600	5,115·2	7,895·6	6,59,840	8,38,611
312	3,400	1,483·2	3,299·2	1,67,840	1,78,771
112	3,200	195·2	839·2	10,931	10,931
0	3,088	0	97·6	...	...

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2. Report of 1939—42 Bhakra Dam Project—Volumes I to V.
3. Preliminary Geological Investigation of the sites of two dams on the Giri River, the Larji Dam on the Beas and the Rohtang Tunnel by J. B. Auden dated Simla, the 29th October, 1942.
4. Engineering Geology of Dam sites by Warren J. Moad.
5. Geotechnical studies of Foundation Materials by Mauric Lugeon.
6. Engineering Geology by Dr. C. S. Fox.
7. Note on silting of Reservoirs by Rai Bahadur A. N. Khosla, I. S. E.
8. Strength of Materials by Swain.
9. Shearing resistance of soils by L. Jurgenson.
10. Proceedings of the American Society of Civil Engineers, May 1940.
11. Application of Electrical Prospecting to the study of Dam sites by C. M. Schlumberger.

DISCUSSION

Mr. Khangar in introducing the Paper remarked that when he took over charge in 1942 of Beas Dams Division, which dealt with Storage Projects, he found that the work in that division was of an entirely different nature to that of an ordinary division in the Irrigation Branch. The greatest difficulty was Geology. He had read the elements of Geology and Mineralogy when he was at College but unfortunately he had almost forgotten them. He had to study that subject *ab-initio*. The next snag was run off from a particular catchment area. There was obviously no use of constructing a costly dam and creating a huge reservoir if the latter could not be filled in a normal year. Hardly any discharge data existed but rainfall records were available for a number of years. Special methods had to be devised with a view to use this data to calculate the runoff. The Surveys in connection with Storage Schemes presented special problems and the methods given in the text-books had to be modified to suit local conditions. Exploration of foundations of the dams was another important problem.

2. Other officers who had to deal with similar problems might find similar difficulties. An average Engineer was a very busy individual and had seldom the time and occasionally not even the inclination to study books to obtain all the information required by him. It was believed, therefore, that a summary of writers' studies would be welcome.

3. As pointed out in the Paper the subject of storage schemes was bound to become more and more important as time passed. Population was increasing at a rapid rate and they must produce more food and fodder. Future Irrigation Projects would have to depend upon stored supplies alone. Lot of water was running waste to the sea every summer. Enormous areas of Banjar but good culturable land still existed everywhere in the Punjab and should water be made available they could be brought under the plough. Punjab was still an industrially backward Province and cheap electricity was required to stimulate industrial activity. Hydro Electric Energy could be produced very cheaply at the various dams. Raw materials were also available. Thus they had got every thing; only men with vision were required to convert Punjab into a land over-flowing with milk, gold, and honey.

If this Paper enabled even one member of that August body to appreciate better the problems in connection with storage schemes the writers would consider that their labour had not been in vain.

Sir William Stamp referred the other day to the preparation of contoured plans of a reservoir proposed to be constructed in Eastern U. P. by means of aerial photography. Mr. Palta the joint writer of this Paper would, if Mr. President permitted, give a few details of that method after the speaker had finished.

Before concluding, the speaker would like to correct a few misprints and supply a few omissions which had unfortunately crept in.

The denominator had been omitted in the formula given on bottom of page 25. This was horizontal force and the formula should read :—

$$\frac{1}{\text{Horizontal Force}} \times \left\{ \begin{array}{l} \text{(weight-uplift)} \times \text{Coeff of internal friction} + \\ \text{thickness of base} \times \text{average shear strength.} \end{array} \right\}$$

Formula on middle of page 26 should be :—

$$S = \frac{p}{2 \tan \delta}$$

where $p = \text{load per square inch} = \frac{P}{A}$ and

$\delta = \text{Angle of break horizontal plane.}$

Units had been omitted in table at bottom of page 26 Crushing load should read Crushing load per square inch and the heading of the last column should be "average shearing strength per square inch."

Add on page 28 the following footnote "Condenced from Proceedings of American Society of Civil Engineers May 1940." This footnote relates to para 7 on this page.

Mr. Palta remarked that in the case of a reservoir, situated in a deep valley, plane tabling from G. T. S. triangulation stations and points, as described in paras 5 and 6 of Chapter IV, becomes rather difficult. The G. T. S. Stations and points are generally situated on high peaks, and as such can not be sighted from the narrow valley where the surveyor works. This difficulty had been felt on the surveys of the proposed Kishau Dam reservoir, which is situated in a deep valley. Under these conditions aerial survey is the quickest and the most accurate method of survey. The field work required would be as below :—

(*a*) Running a line of precise double levels, from the proposed dam site to the tail end of the reservoir and fixing bench marks at suitable intervals.

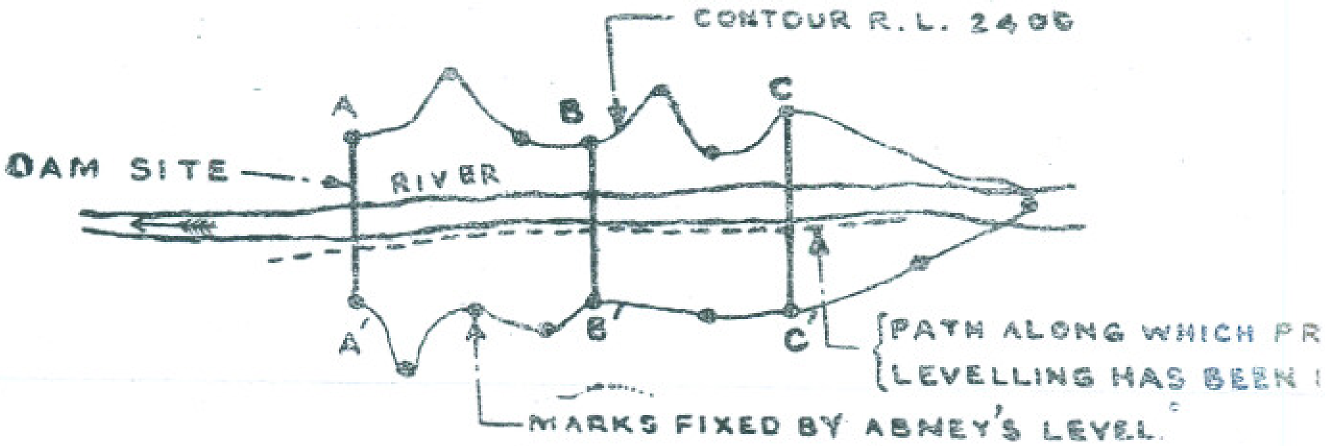
(*b*) Taking cross sections of the valley about two miles apart and fixing marks with respect to the above bench marks, at levels at which contours have to be run.

(*c*) Searching out all G. T. S. triangulation stations and points in the locality and making them prominent so that they can be picked from air.

In the sketch below, dotted lines show the path along which precise double levelling has been done. A A′, B B′, C C′, etc. are the lines along which cross section have been taken. Now suppose contour R. L. 2,400 has to be surveyed. The surveyor with Abney's level in his hand starts from point A and proceeds along the contour towards B making marks on the ground at such points as would govern contouring, *e. g.*, spurs, screes, etc. He checks his level at B, and then proceeds towards C in the same way. Marks should be made big enough so that they can be photographed from the air. Contouring on the photograph can then be easily done with reference to these marks.

The scale of the photographs can be fixed from the known distances between G. T. S. stations and points and maps can be printed.

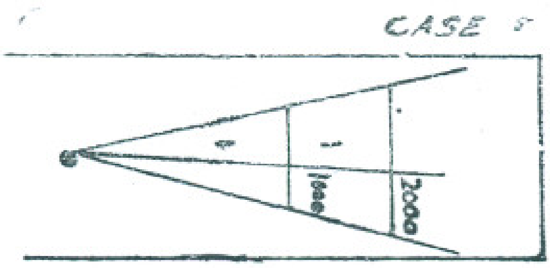
The method is quick and accurate. A surveyor with an Abney's level can cover about six miles in a day. He has neither to measure any distances not to find his position and gets many opportunities to check his levels. Of course, there is no possibility of missing any important detail.



Mr. G. S. Sidhu remarked that Trapezoidal formula given below had been suggested by the authors for calculating the capacities of the reservoir:—

$$V=\frac{(A_1+A_2)}{2}250 \text{ on page 4 of Paper No. 268}$$

where A_1 and A_2 were the submerged areas as R. L. 2,000 and 2,250 respectively.



The percentage of error that would be involved by adopting such simple equation was given below :—

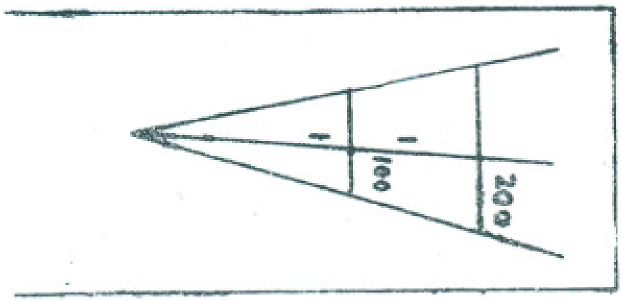
For a Conical shaped reservoir (side slope 1 in 1,000) Volume by Trapezoidal formula

$$=\frac{O+\pi\,(1000)^2}{2}+\frac{\pi\,(1000)^2+\pi\,(2000)^2}{2}=3000000\,\pi$$

Actual volume of the cone $=1/3\,\pi\,E^2H=1/3\times\pi\times(2000^2\times 2$
$=2666666{\cdot}7$

$$\therefore\ \text{Percentage error}=\frac{(3000000\pi-2666{\cdot}666{\cdot}7\pi)\times 100}{2666666{\cdot}7\,\pi}12{\cdot}5\ \%$$

CASE II



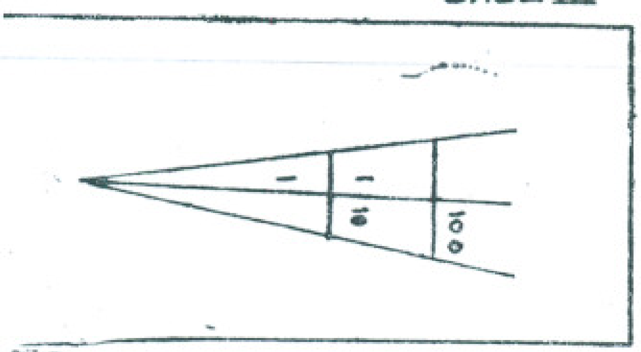
For a Conical shaped reservoir (side slope 1 in 100) Volume by Trapezoidal formula

$$=\frac{O+\pi\ (100)^2}{2}+\frac{\pi(100)^2\times\pi(200)^2}{2}=30000\ \pi$$

Actual volume $=1/3\times\pi\times(200)^2\times2=26666{\cdot}7\ \pi$

Percentage error $=\frac{(30000\pi-26666{\cdot}7\pi)\times100}{26666{\cdot}7\ \pi}=12{\cdot}5\%$

CASE III



For a Conical shaped reservoir (side slope 1 in 10)

Volume by Trapezoidal formula

$$=\frac{O+\pi(10)^2}{2}+\frac{\pi(10)^2+\pi(20)^2}{2}=300\ \pi$$

Actual Volume $=1/3\times\pi(20)^2\times2=266{\cdot}7\ \pi$

Percentage error $=\frac{(300\pi-266{\cdot}7\pi)100}{266{\cdot}7\pi}=12{\cdot}5\%$

Percentage error for all slopes lies in the neighboured of $12\frac{1}{2}\%$.

The correct formula was the Prismoidal formula

$V=\frac{h}{6}\ (A_1+A_2+4A)$ where A_1 and A_2 represented the same thing as above, and A the area at the middle height, and h the height between the two contours. The general form of Prismoidal formula resembles the Simpson's rule

$$V=\frac{d}{3}\left\{(A_1+A_{2n+1}+2\ (A_3+A_5+A_7+-\qquad)+4(A_2+A_4+A_6+-\qquad)\right\}$$

Capacities of large reservoir as given on page 29 would, therefore, be corrected as under :—

Volume at height 512 feet of the dam.

$$V=\frac{200}{3}\left\{195{\cdot}2+5115{\cdot}2+4\ \ (1483{\cdot}2)\right\}+10931$$

(assuming the capacity at 112 feet height the same as given on page 29, *i. e.*, 10931 ft. acres)

Percentage error comes to be nearly 10%.

Applying the same percentage error to the capacity at height 732 feet of the dam,

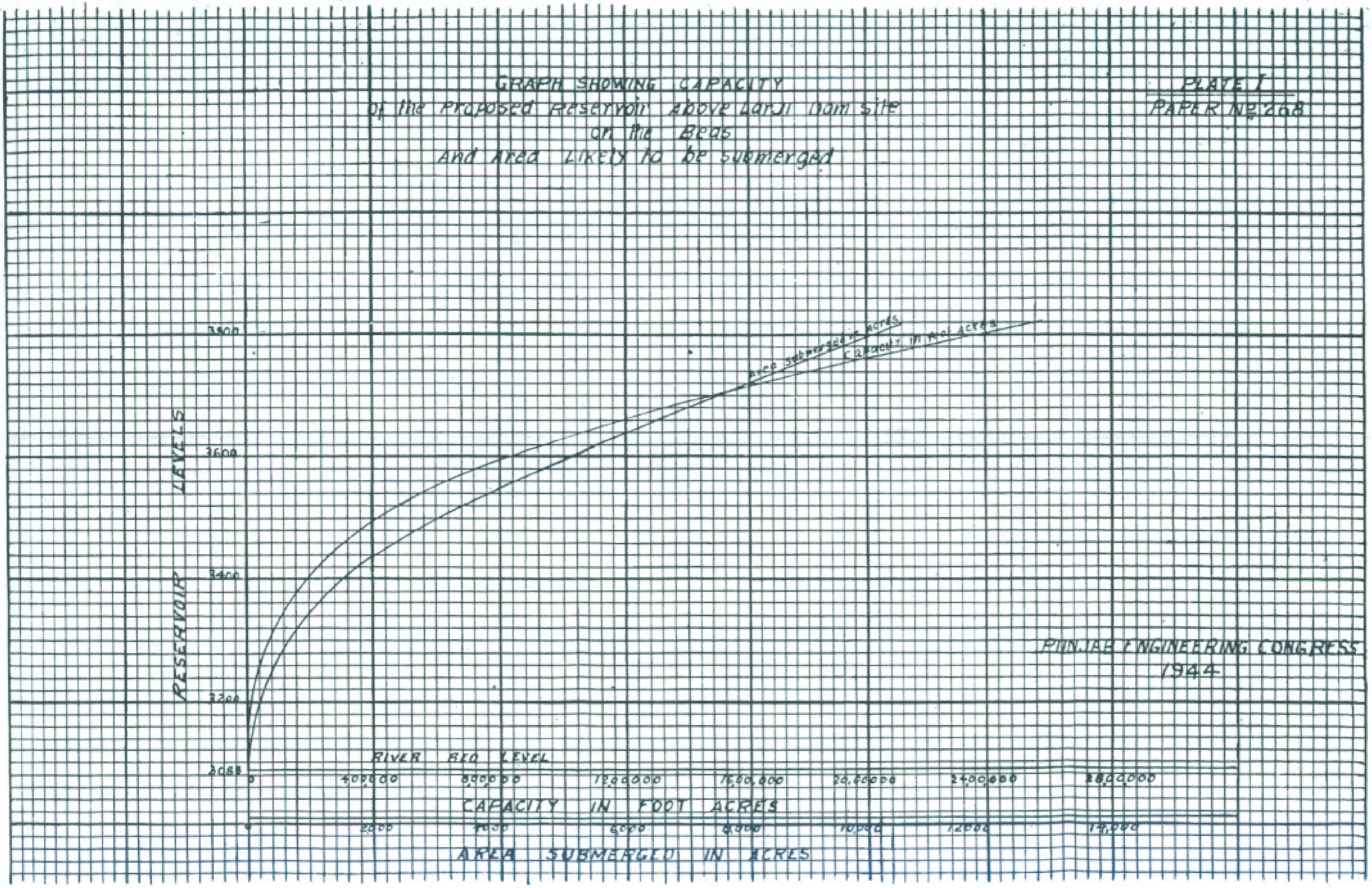
$$\text{Volume}\ \ V=\frac{2575643\times 100}{110}=2341493 \text{ ft. acres.}$$

i. e., there is an error of over 2 lakh ft. acres in the capacity of the dam at height 732 feet, if it is calculated by the Trapezoidal formula.

The Trapezoidal formula can only be used for finding out areas of surfaces but for finding out volumes of irregular solids, Prismoidal formula can only be used.

Concluding the discussion the President remarked that he was sure that the members would wish him to express their thanks to the authors of that very interesting Paper, which was of considerable importance at the present moment in view of the many storage schemes which were being investigated by engineers in the Punjab.

Replying to the discussion Mr. Khangar observed that Paper No. 268 dealt only with preliminary investigations for storage schemes and it was not worthwhile spending more time than was necessary for this purpose. The formula suggested in the Paper was good enough for the object in view. It had been found that the capacity worked out from the contours given on the G. T. S. Maps was always less than the capacity worked out from the contours laid down on the ground. This was due to the fact that there were numerous small nallas which did not appear in the contours on the plan but which increased the capacity. Thus the results worked out were on the safe side. Further the calculations made by writers showed that the difference between the results worked out by Trapezoidal and the Prismoidal formulas was only about 5 per cent. Mr. Sidhu had assumed that the catchment area was in the form of a cone. This assumption was of course incorrect and the results worked out by him were therefore inaccurate. If normal cases were examined it would be found that the difference was not more than 5 per cent and it was not worthwhile spending more time on working out results by Prismoidal formula.



GRAPH SHOWING CAPACITY
of the Proposed Reservoir above Larji Dam site
on the Beas
And Area Likely to be submerged
PLATE I
PAPER No 268
Area submerged in acres
Capacity in foot acres
RESERVOIR LEVELS
3800
3600
3400
3200
3088
RIVER BED LEVEL
0
400,000
800,000
1200,000
1600,000
2000,000
2400,000
2800,000
CAPACITY IN FOOT ACRES
0
2000
4000
6000
8000
10000
12000
14000
AREA SUBMERGED IN ACRES
PUNJAB ENGINEERING CONGRESS
1944

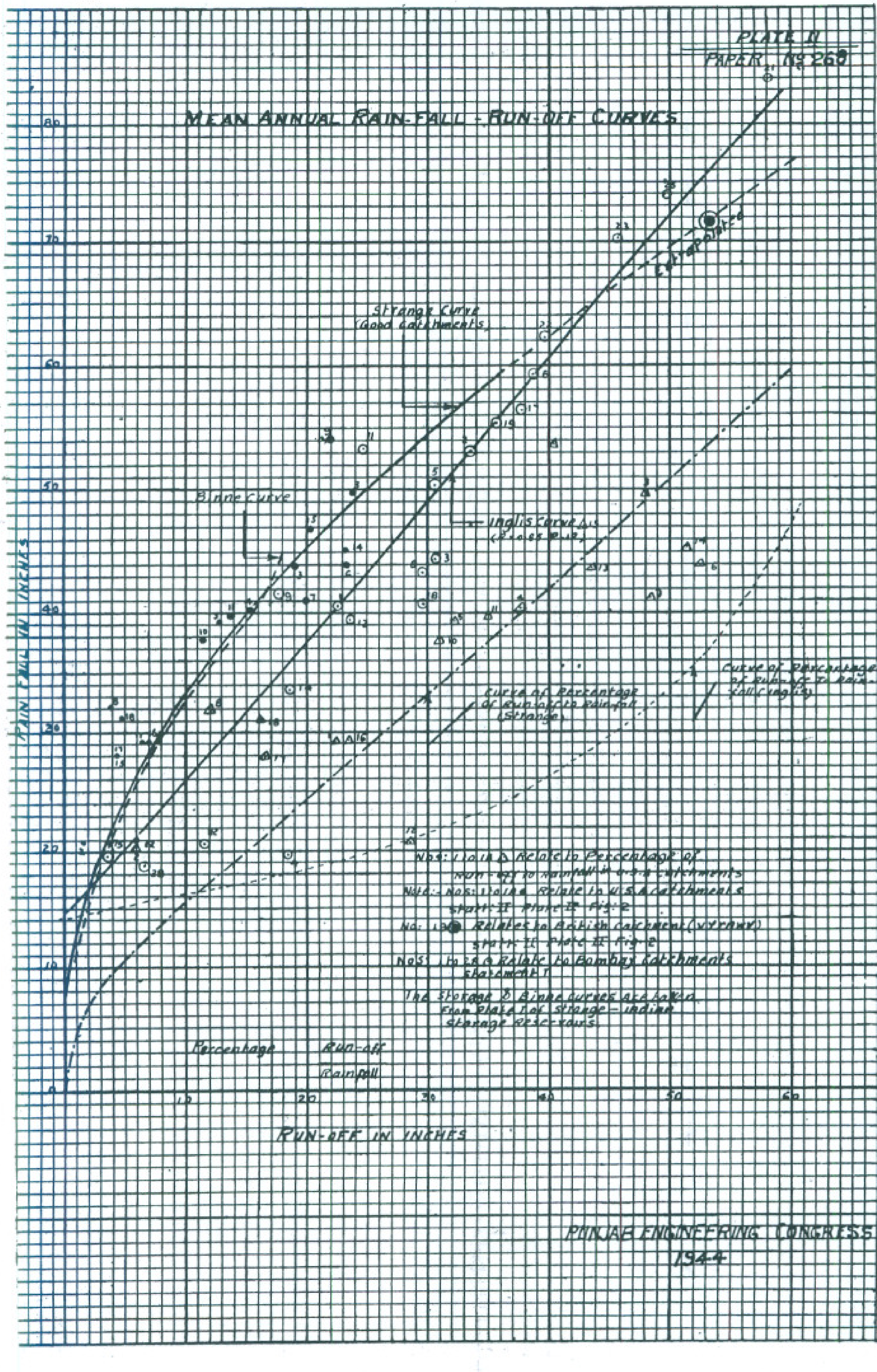
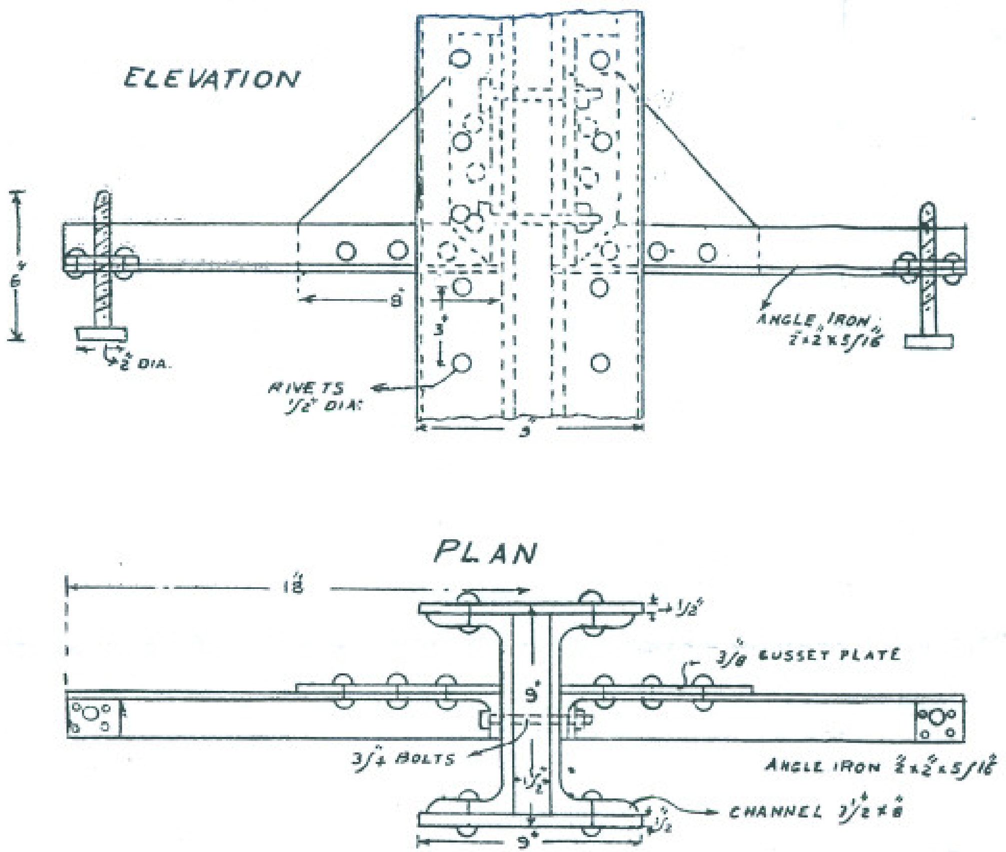


PLATE II
MEAN ANNUAL RAIN-FALL - RUN-OFF CURVES
Strange Curve (Good Catchments)
Binne Curve
Inglis Curve
Extrapolated
Curve of Percentage of Run-off to Rain-fall (Strange)
Curve of Percentage of Run-off to Rain-fall (Inglis)
Percentage Run-off / Rainfall
RAIN FALL IN INCHES
RUN-OFF IN INCHES
PUNJAB ENGINEERING CONGRESS
1944

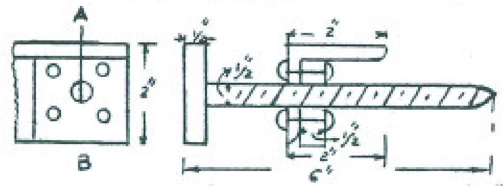
DETAIL OF BUILT UP BEAM WITH BRACKETS

SCALE 2" = 1 FT.



DETAIL OF ADJUSTING HEAD

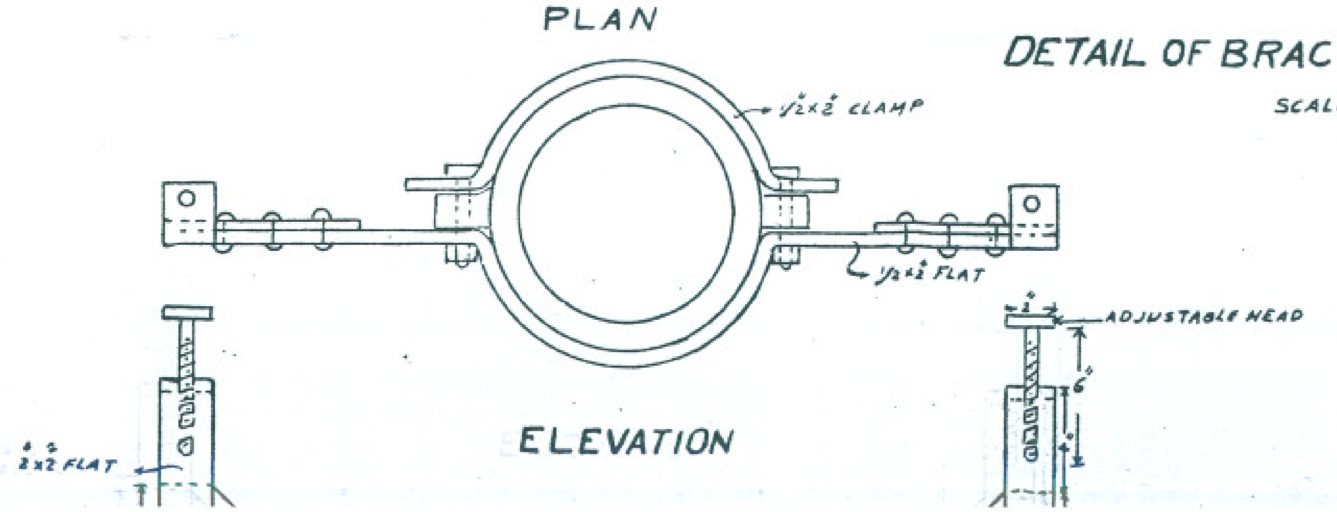
SCALE 4" = 1 FT.



SECTION THROUGH A.B.

DETAIL OF BRACKETS FOR 100 TON

SCALE 2" = 1 FT.



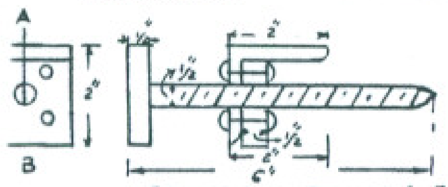
SIDE EL.

PLATE III
PAPER No. 268

APPARATUS FOR COMPRESSION TESTS ON ROCK IN-SITU

ETAIL OF ADJUSTING HEAD

SCALE 4" = 1 FT.



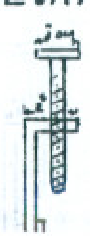
SECTION THROUGH A.B.

F BRACKETS FOR 100 TON JACK

SCALE 2" = 1 FT.

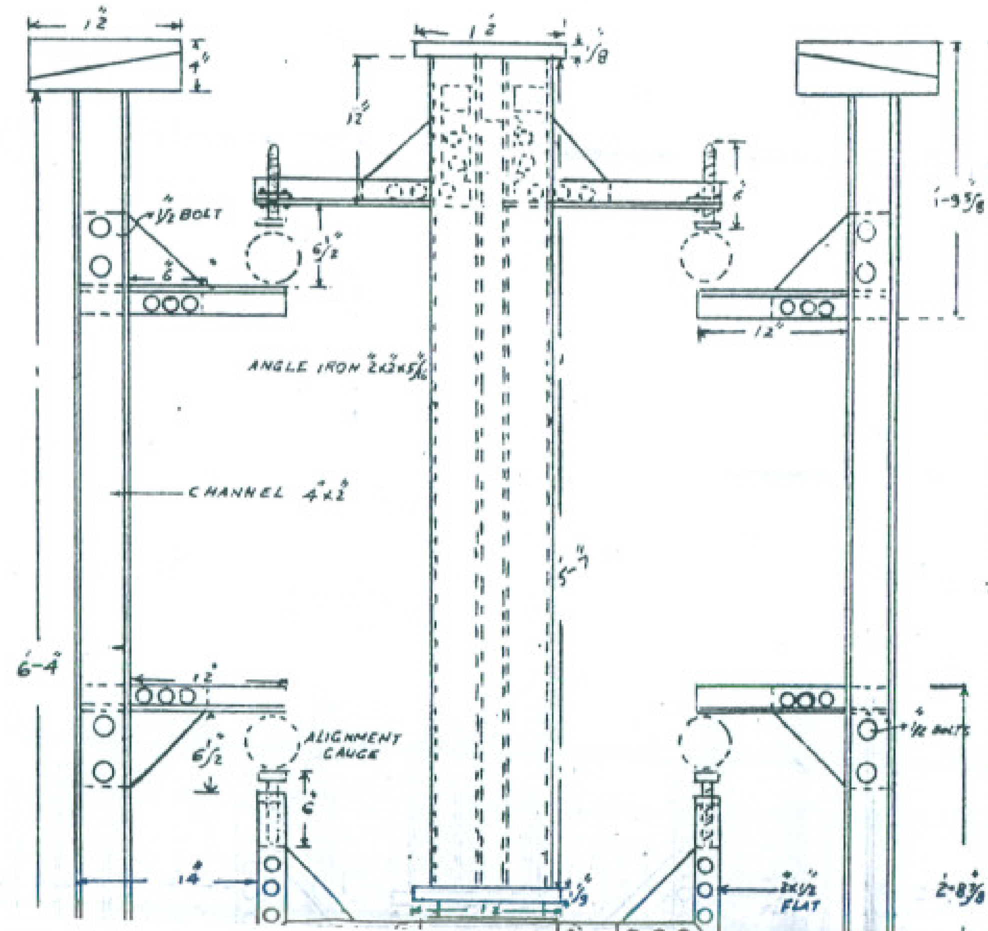
SIDE ELEVATION

TABLE HEAD



GENERAL ARRANGEMENT

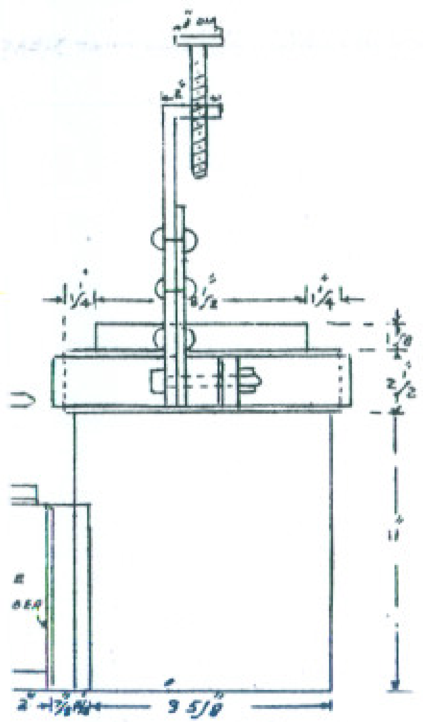
SCALE 1" = 1 FT.

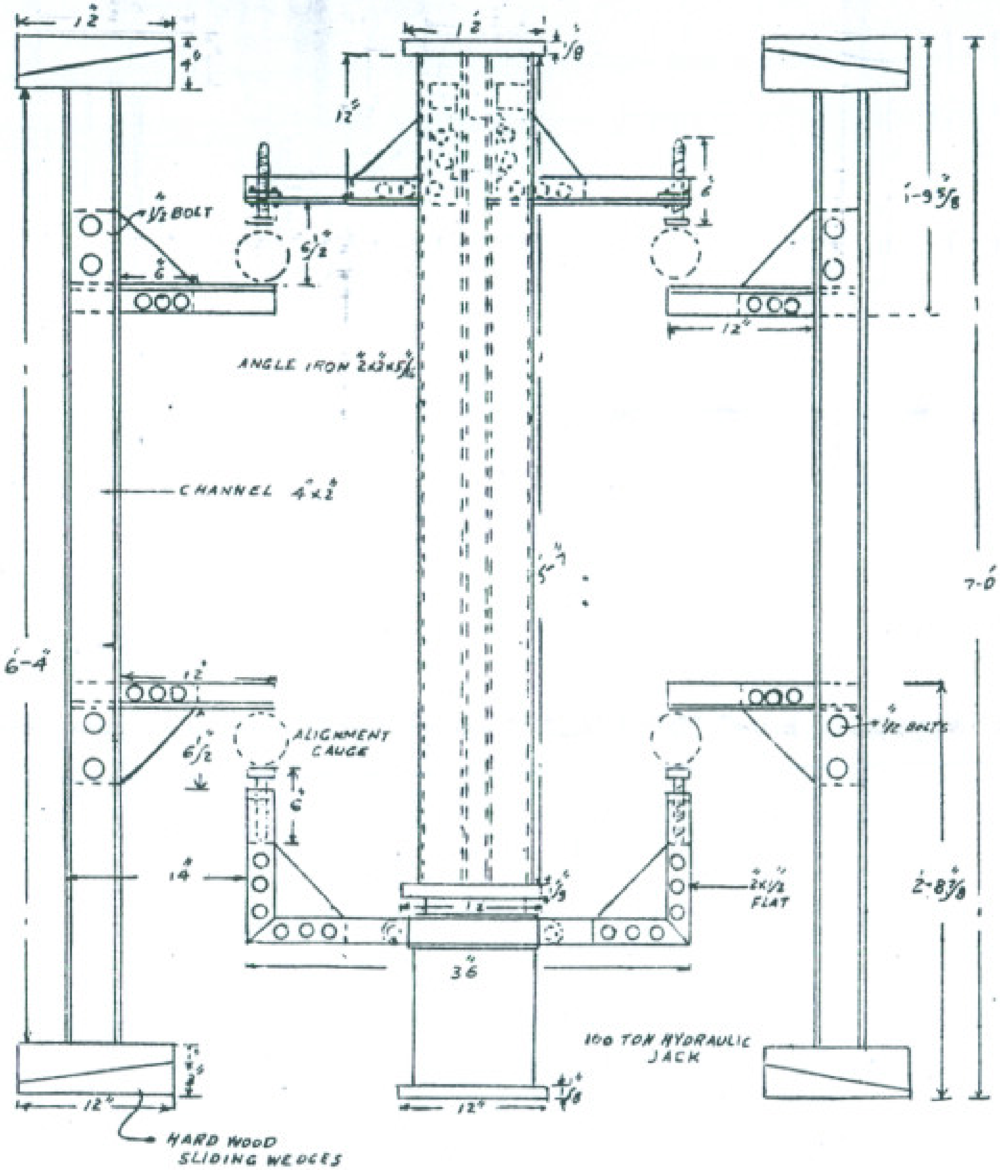


A.B.

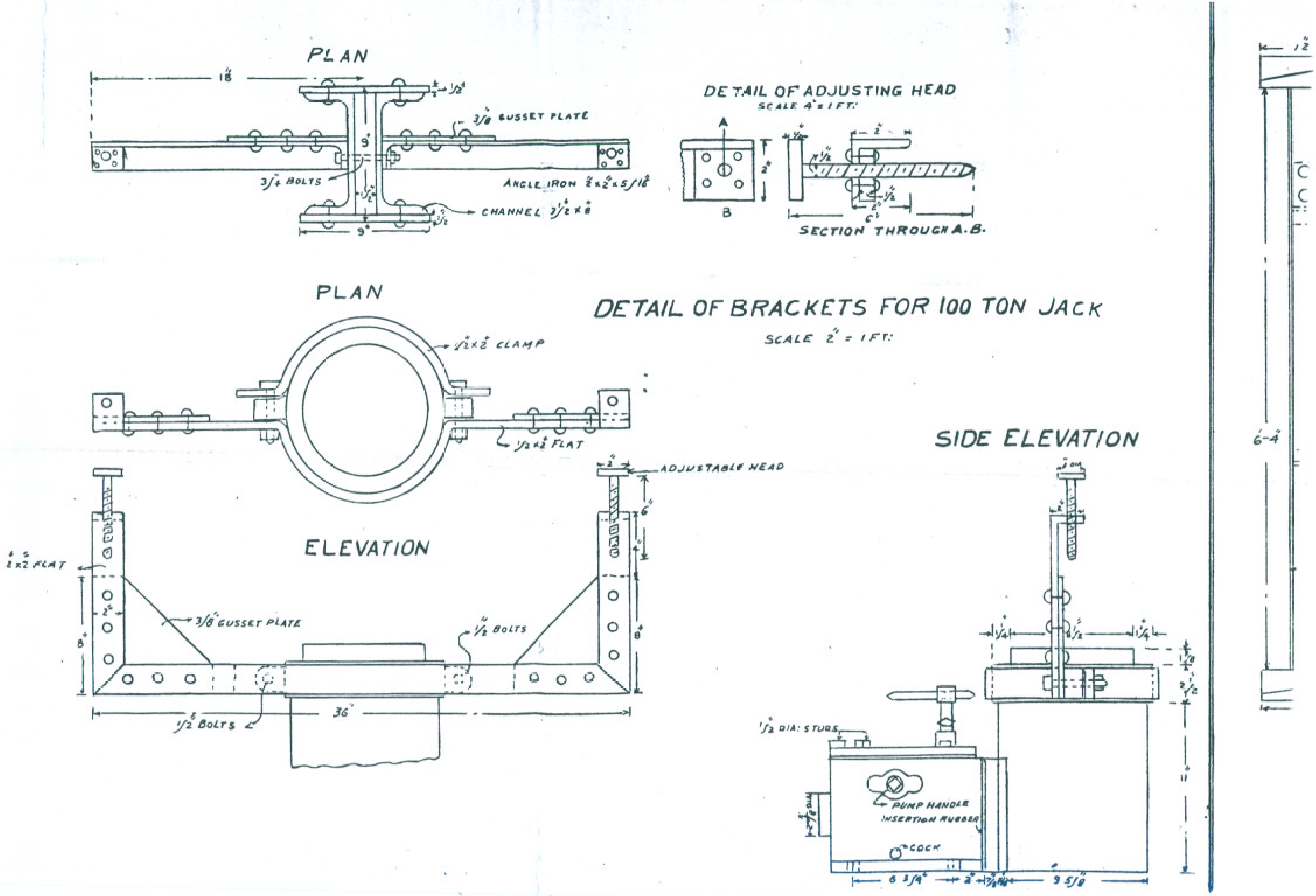
00 TON JACK

IDE ELEVATION

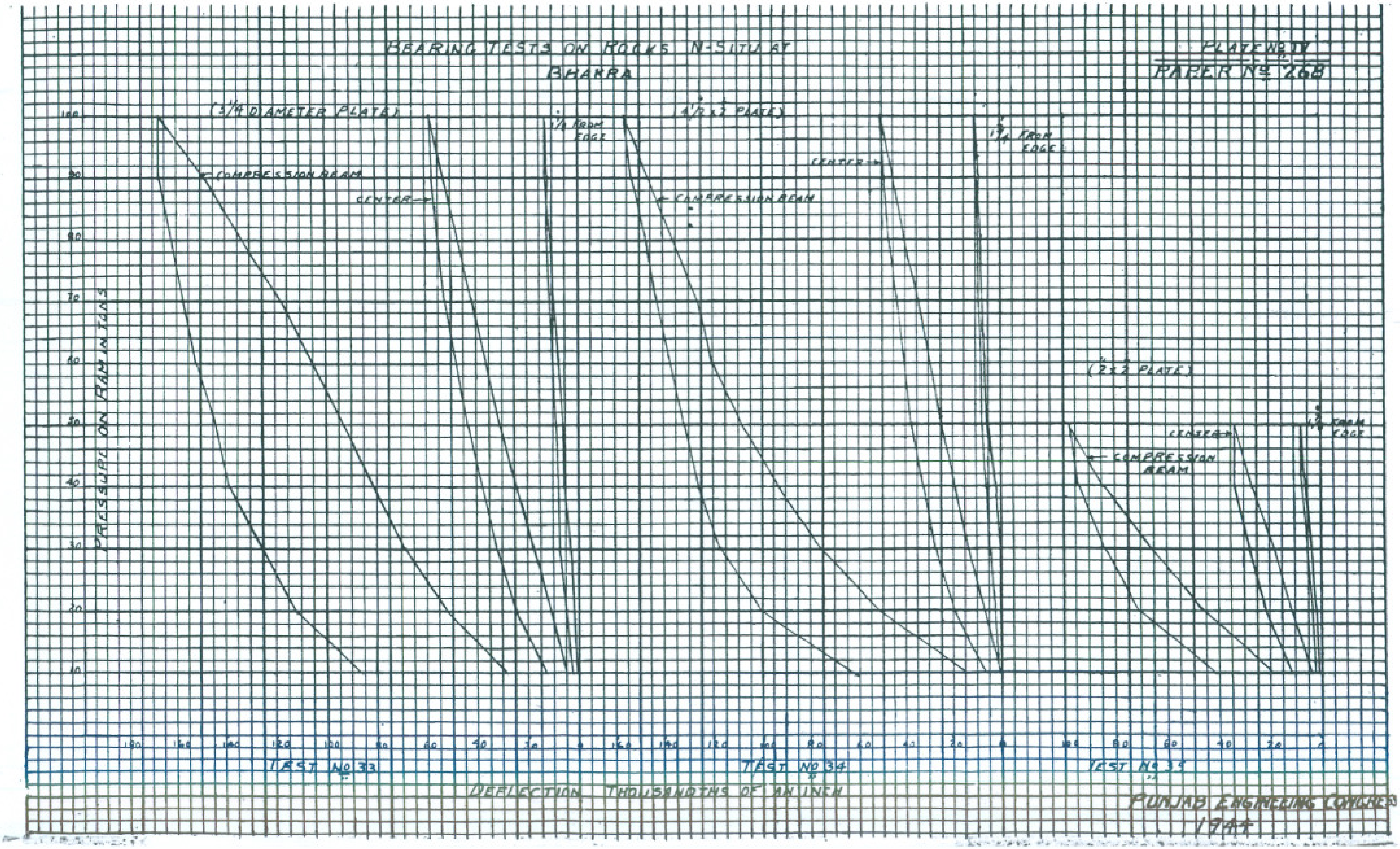




PUNJAB ENGINEERING CONGRESS
1944



PLAN
18"
1/2"
3/8" GUSSET PLATE
9"
3/4" BOLTS
ANGLE IRON 2"x2"x5/16"
CHANNEL 3 1/2" x 8"
9"
DETAIL OF ADJUSTING HEAD
SCALE 4" = 1 FT.
A
B
2"
6"
SECTION THROUGH A.B.
PLAN
1/2"x2" CLAMP
1/2"x2" FLAT
DETAIL OF BRACKETS FOR 100 TON JACK
SCALE 2" = 1 FT.
ADJUSTABLE HEAD
ELEVATION
2"x2" FLAT
3/8" GUSSET PLATE
1/2" BOLTS
8"
36"
1/2" BOLTS
SIDE ELEVATION
1 1/4"
1 1/2"
1/2" DIA. STUDS
PUMP HANDLE
INSERTION RUBBER
COCK
6 3/4"
2"
3 5/8"
6'-4"



BEARING TESTS ON ROCKS IN-SITU AT
BHAKRA
PLATE No. IV
PAPER No. 268
(3 1/4" DIAMETER PLATE)
(4 1/2" x 2" PLATE)
(2" x 2" PLATE)
COMPRESSION BEAM
CENTER
1/4" FROM EDGE
1 1/4" FROM EDGE
PRESSURE ON RAM IN TONS
TEST No. 33
TEST No. 34
TEST No. 35
DEFLECTION THOUSANDTHS OF AN INCH
PUNJAB ENGINEERING CONGRESS
1944