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UNITED STATES

DEPARTMENT OF THE INTERIOR

BUREAU OF RECLAMATION

MEMORANDUM TO CHIEF DESIGNING ENGINEER

SUBJECT: SECURITY OF MASONRY DAMS ON EARTH FOUNDATIONS

by E. W. LANE, RESEARCH ENGINEER

TECHNICAL MEMORANDUM NO. 280

Denver, Colorado

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Revision

Since this report was compiled, much additional data has been received and analyzed. It indicates that a weight of one-third should be given to horizontal creep rather than one-half as suggested in this report.

With the weight of one-third the following values of weighted creep to head should be used:

Weight of Horizontal Creep 1/3

Very fine sand or silt	8.5
Fine sand	7.0
Medium sand	6.0
Coarse sand	5.0
Fine gravel	4.0
Medium gravel	3.5
Coarse gravel including cobbles	3.0
Boulders with some cobbles and gravel	2.5
Soft clay	3.0
Medium clay	2.0
Hard clay	1.8
Very hard clay or hardpan	1.6

SECURITY OF MASONRY DAMS ON EARTH FOUNDATIONS

by
E. W. Lane, Research Engineer

Synopsis

This paper gives the results of an investigation of more than one hundred masonry dams with various kinds of earth foundation, to determine the length of the percolation path necessary to prevent failure from underseepage or piping. Based on this study, a new method of analysis of such structures has been developed, which usually permits the use of smaller seepage distances than are ordinarily considered to be safe.

When a masonry dam is founded on earth, some of the water from the reservoir percolates beneath it and appears on the downstream side. If the velocity of the flow where it emerges is sufficient, particles of the foundation material will be carried away by the water, and thus decrease the resistance to the percolation. The result is an increased velocity and greater erosion, ultimately causing the formation of a channel or "pipe" beneath the dam, which may rapidly enlarge and cause the failure of the structure. This process is known as piping. To prevent piping it is necessary to so design the dam that the velocity of the seepage water as it emerges on the downstream side is insufficient to remove the foundation material. This is accomplished by making the route along which the water may percolate of such a length that the velocity of flow is reduced to a safe value. The investigation described herein was undertaken to determine from data on actual dams the length of the percolation path necessary to insure safety for dams founded on various kinds of material.

Theoretical Considerations

The law of flow of water through earth was determined many years ago in this country by Allen Hazen, M. Am. Soc. C.E. and in India by Clibborn and Boresford. Their experiments showed that (neglecting temperature effects) the discharge through a column of soil of a given composition varied directly with the head, directly with the cross sectional area and inversely with the length. This relation may be expressed

$$Q = K\frac{HA}{L} \qquad (1)$$

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$$Q = K \frac{HA}{L} \qquad (1)$$

where Q is the discharge in second feet, H is the head in feet, A is the cross sectional area in square feet, L is the length in feet and K is a constant which depends on the character of the material.

Substituting for Q the value AV, there may be obtained from equation (1) the expression

$$L = K \frac{H}{V} \qquad (2)$$

For a given class of material there is a definite maximum velocity, V_m, at which the water can emerge below the dam without carrying away the foundation material and causing the failure of the structure. Combining this value of V_m with K, which also depends on the material, to form a new constant $C = \frac{K}{V_m}$ the expression

$$L_m = CH$$

where L_m is the minimum safe length of travel path and C is a constant depending on the foundation material.

The foregoing theory seems to be generally accepted. In applying it, two difficulties arise: (1) How shall the length of the travel path L_m be measured? and (2) What value of C can safely be used for various classes of material? Before going further into a discussion of these points it may be well to give the history of the development of this phase of dam design.

Historical

The first rational basis for the design of masonry dams on earth foundations seems to have been developed in India, as a result of the investigations of Col. Clibborn and Mr. Beresford. Col. Clibborn was at one time Executive Engineer in charge of the Rolhikhand Canal Division of the United Provinces, India, where there were a number of dams founded on light sand which had often given trouble. After he became Principal of Thomason College, together with Mr. Beresford, he carried out a classical set of experiments on the law of flow through sand. From these experiments Beresford concluded that the Narora weir on the Ganges River was unsafe, because of excessive upward pressure on the apron, and made a report to that effect. At the time no special trouble had been experiences with the dam, but as a result of this report pressure pipes were placed in the apron to indicate the upward pressure beneath it. The pressure indicated in these pipes confirmed Beresford's contentions and by coincidence, the next day after the readings were taken (March 30, 1898) the apron at another part of the weir was blown up, resulting in a breach of the weir. The failure of

an important structure following so promptly after the declaration of its instability profoundly impressed the engineers of the United Provinces; and the hydraulic gradient theory of design became generally accepted there about 1898.

The first edition of Bligh's Practical Design of Irrigation Works appeared in 1907 in which the theory was advanced that the stability of a weir on a porous foundation depended upon the weight of the structure and not on the ratio of the percolation distance to the head. In his second edition, published in 1910, Bligh admits the fallacy of his original contentions and explains his well known theory that the safety of masonry dams on earth foundations depended on the length of the percolation path, which was along the line of contact of the structure and its foundation.

This conclusion was also reached independently by Mr. W. M. Griffith about the same time*. The author believes that this has not

*The Stability of Weir Foundations on Sand and Soil Subject to Hydrostatic Pressure - Proc. Inst. C.E. Vol. 197, Pt. III p. 221, 1913-14.

received the recognition in this country which its value justifies. Before this paper was published Bligh's second edition appeared, in which he also proposed that idea. The same conclusion seems to have been reached about that time by Mr. W. W. Tefft, M. Am. Soc. C.E. but did not find its way into print. The widespread use of Bligh's book, together with the publication of other articles and books* by him,

*Dams and Weirs, Bligh.
Dams, Barrages and Weirs on Porous Foundations - Bligh, Eng. News. Vol. 64, 12/29/10 p. 708, Vol. 65 1/12/11 p. 52.
Weirs on Porous Foundations and with Pervious Floors - Eng. News. Vol. 65, 4/13/11 p. 444.
Lessons from the Failure of a Weir and Sluices on Porous Foundations - Eng. News. Vol. 69, 2/6/13 p. 266.
Irrigation Headworks Repair and Dam Failure - Eng. News. Vol. 75, 6/8/16 p. 1070.

has led engineers generally to give the credit for the idea to Bligh.

Little appeared in literature for many years after Bligh's publications to aid the engineer in the practical design of masonry dams on earth foundations, but a number of measurements of upward pressure on actual and model dams have been published from time to time. Recently, methods have been devised for determining the upward pressure and path of percolation for various conditions. One of these methods is that used by Terzaghi*. It seems to have been first used for dam foundations by

*Technical Publication #215, Am. Inst. Mining & Metallurgical Engr. p.31

Forchheimer* but Mr. Terzaghi's article appears to be the first explanation of this use in English.

*Hydraulic - P. Forchheimer, 1924 p. 448

By means of a flow net,* the direction of the currents of the water and the pressure throughout the material beneath the dam can be computed for various assumed conditions. Another interesting method of obtaining the same information has been devised by Prof. N. N. Pavlovsky. He makes use of the similarity of the laws of flow of the electric current through a conductor and the flow of water through soil. As nearly as can be determined from the information available, his method is as follows: A cross section of the porous foundation material in the vicinity of a dam is represented by a thin plate of metal. Impervious material in the foundation soil, such as sheet piling, cutoff walls of the dam or impervious soil layers, are introduced by cutting out the metal at that part of the cross section which would represent their position. The upper edge of the sheet corresponding to the portion of the foundation in contact with the headwater, is connected to one pole of an electric battery and that representing the contact with tailwater with the other pole. By observing the potential at various points in the cross section and along the edge corresponding to the base of the dam, and drawing lines of equal potention, the pressure at any point within the material or beneath the dam may be determined. By drawing lines representing direction of flow in the same manner as in the flow net, the direction and relative velocities may be determined. Layers of different permeability may be represented by changing the thickness of various portions of the plate or by using metals of different frictional resistance. With either Terzaghi's or Pavlovsky's methods it is possible to obtain a rigid solution for the given assumptions, and they are very useful in obtaining a picture of the flow of the water beneath the dam. As will be discussed later however, the results obtained, if the ordinary assumptions are used, must be applied to actual cases with caution.

*Hydraulic Laboratory Practice - Freeman, p. 605

Three interesting and instructive papers have recently appeared, which throw much light on the problem. Two of these are papers of the Punjab Engineering Congress 1930, by A. N. Khosla: No. 138, "Hydraulic Gradients in Subsoil Water Flow in Relation to Stability of Structures Resting on Saturated Soils" and No. 142, "Stability of Weirs and Canal Works, An Application of the New Theory of Hydraulic Gradient." The third is a paper "On Percolation under Aprons of Irrigation Works" by S. Leliavsky.

Course of the Seepage Beneath a Dam

The path which the water takes in flowing beneath a dam might be determined with reasonable accuracy by either the flow net or electrical methods if all the conditions governing the flow were accurately known. This is never the case however, and for purposes of design certain assumptions have to be made. In his second edition of the Practical Design of Irrigation Works, Bligh proposes an analysis on the assumption that the water follows a path along the line of contact of the dam foundation (including the sheet piling) with the foundation material. The same method was suggested by Griffith. This contact between the dam and the foundation material is sometimes called the line of creep, and the method may be called the line of creep method. Another method which has been advocated to some extent, may be called the short path method, and is based on the assumption that the course taken by the percolating water is the shortest path through the pervious material between the headwater and the tailwater. Neither of these methods gives a true picture of the actual conditions of flow beneath dams, but are useful methods of practical design. The line of creep method has gained a wide acceptance, and most masonry dams on earth foundations have been designed according to it. It has been used in the irrigation works of the United States Bureau of Reclamation for many years with satisfactory results. The short path theory has been but little used.

Comparison of Design Methods

Although the line of creep method has been widely used in dam design, it has been subject to some criticism. Part of this is believed to be due to an improper presentation of the case in its favor. Bligh states that the water follows the line of creep and not the path of least resistance. This statement is believed to be in error, for the fact that water would take the path of least resistance seems almost axiomatic. If water flows along the line of creep instead of the shorter path directly through the foundation material it is because the resistance to travel along the line of creep is less than the shorter path. That resistance along the line of creep may be less than through the foundation material seems very reasonable, on account of the difficulty of securing as intimate a contact the more or less plain surfaces and the foundation material as between the individual particles of the foundation material.

It should be remembered that the line of creep method is intended to give a dam safe at all points. This requires that it apply to the worst condition that will happen with any reasonable care in construction. The seepage may not follow the creep line at many cross sections of the dam, but the points where there is most danger of failure

are likely to be those where contact between the dam and the foundation material is not so close, and therefore where the line of creep method best applies.

The principal weakness of the line of creep theory is that it assumes the resistance to flow along all portions of the contact between the dam and the foundation to be the same. In practice the contact between vertical and steeply sloping surfaces is more likely to be intimate than that along horizontal or slightly sloping surfaces. In a masonry dam on earth foundations there is likely to be unequal settlement which will cause less pressure at some points than at others, in places, even a slight lifting up of the masonry from its contact with the earth beneath. The earth beneath a dam may not be compact, and may settle after the dam is built, leaving void spaces beneath the floor, especially where the dam is founded on piles. This action is sometimes called "roofing."

Another cause of roofing is given by Col. J. C. Oakes*, M. Am.

*Trans. Am. Soc. C.E. Vol. 80, 1916, pp. 469 and 470

Soc. C. E. as follows: "If the bed of the foundation (of a dam) is below the water level, and pumping is required there will be a small space between the base of the masonry and the sand, caused by erosion by the flow of water from under one block of masonry while the next one is being placed. In other words the structure is ordinarily supported wholly by the piles, with almost a certainty that there will be a space between the sand and the masonry. Under such circumstances any upward pressure that developes under the dam will be uniform from the sheet piling to the toe, and will depend on the tightness of the sheeting and the ease of escape below the dam."---"The writer has watched the placing of concrete, and in no case has he found, on the works under discussion (Ohio River Dams, No. 43 and 48) that the concrete rested on the sand after sufficient time had been given it to set. The bed of the foundations of the various parts of those is about 10 feet below water level. Construction has been carried on within cofferdams, and during stages of the river from low water to 14 feet above it. Owing to the permeable nature of the material, there has always been considerable percolation, which has required pumping to keep the pit sufficiently clear of water to enable construction to proceed. The water escaping from under the concrete already placed, carries away the fine material directly under the concrete, leaving a space between it and the sand through which the transmission of pressure will be direct, and consequently, any pressure which may be developed will be uniformly exerted over the whole base of the structure." A study of the results of upward pressure measurements on actual dams discloses much evidence supporting Col. Oakes' conclusions.

On vertical or steeply sloping faces, roofing will not occur as the void spaces, if they should be formed, would be filled again by the inability of the earth to maintain the steep slope; the earth from the steep bank would move down and fill the void. For this reason the contact of the foundation with such surfaces are close and they offer more security against piping than contact beneath horizontal surfaces.

Piping failure should therefore be considered as possible from two largely independent causes: (1) direct percolation through the foundation material itself, and (2) percolation along the contact of the dam and sheet piling with the foundation material. Considerable light on the first of these causes has been obtained by experiment but the latter can only be evaluated by investigating a large number of structures. It is a weakness of the ordinary line of creep theory that it considers only the second of these causes.

Another weakness of the line of creep theory, is that it is possible to drive lines of sheet piling so close together that the short path may be so small that failure can occur by flow which will not follow the path of creep at all.

The weakness of the short path theory is that it takes no account of the greater probability of percolation along the line of contact of the structure and its foundation. In cases where the lines of sheet piling are very close together however, it may be more reliable than the creep theory. The short path principle would seem to be obviously inferior to the line of creep analysis as applied to clay or hardpan foundation, since the foundation material in this case would be so nearly impermeable that the seepage along the line of creep would be much greater. Another difficulty with the short path analysis is that it gives no method of estimating the magnitude or distribution of upward pressure beneath the dam. As some estimate of this pressure is necessary in order to determine the required thickness of the apron, additional assumptions are necessary if the dam is to be designed by the short path theory throughout.

The flow net and electro-hydraulic analogy methods are essentially the same. Both should give the same results for the same assumptions. In his article Terzaghi* does not take into consideration the greater

*Technical Publication #215, p. 31. Am. Inst. Min. & Metal. Engrs.

probability of percolation along the line of contact. Whether Prof. Pavlovsky does or not is not known. It would be possible to do it by either method, however, by assuming the relative permeability along this line as compared with that through the foundation material.

The weakness of both the flow net and electro-hydraulic analogy methods is the necessity of a detailed knowledge of subsurface conditions and the lack of data relating the results obtained as shown by the flow net to the safe limits for the various classes of material. In order to reach a reliable result, not only must the relative permeability along the contact and through the foundation material be known, but the position and relative permeability of all the layers beneath the dam must also be known, and they must not change materially in short distances along the dam, since the analysis considers flow in two dimensions only, while actually it may take place in three. In connection with the studies for this paper the Author had the opportunity to examine the boring records for a number of dams, and while he did not give them the intensive study which one would if he were to design the dam, it was apparent that it would be very difficult to form an accurate idea of the position and relative impermeability of the various layers from boring records even in a single cross section, and still less possible to consider the permeability in three dimensions. A good example of the variability of foundation materials of ten encountered is that shown by the borings for the Oradell Dam*. Some sand foundations

*Trans. Am. Soc. C.E., Vol. 89, 1926 p. 1187.

are relatively uniform, but even here the permeability would vary considerably. Moreover, even if all of these factors were known, so that the true velocity of the water emerging below the dam was susceptible of accurate determination, it would be difficult to relate this to dam safety, as safe velocities for the various classes of material are not definitely known. Some experimental work has been done along this line, but much more experimental work and comparison with actual structures is desirable.

Nevertheless, the flow net and electro-hydraulic analogy methods may prove to be useful tools in analyzing unusual conditions, and forming a mental picture of what takes place under certain conditions. It is hoped that further studies along this line will be made to clear up some of the problems of design. It is believed that there is a large field for useful experimentation in this field, especially in the determination of the upward pressure to use in design.

The results which would be obtained by the flow net or electrical analysis would roughly correspond in a homogeneous medium to that obtained by the short path method. They both are based on the consideration of flow directly through the foundation material. The short path may therefore be considered as a rough approximation of the flow net or electrical analysis.

A weakness of all of the methods, as ordinarily applied, is that the flow is considered as taking place only in a single plane. This is

not necessarily the case however, for as Griffith has pointed out,*

*Proceedings, Inst. of C.E. Vol. 197, Pt. III p. 223.

when a pipe tends to form, it provides a line of lowered pressure and water from both sides flows toward it, as well that in the plane of the incipient "pipe." Analyzing the stability of a dam by means of its cross section is a useful tool, but one is apt to be blinded to the true conditions by thinking too riddly in terms of cross section only.

Required Length of Percolation Path

Not only have there been differences of opinion regarding the path along which the percolation should be assumed to occur, but also as regards the length of the path which is necessary to insure safety from piping failure.

As the result of his study of dams and dam failures, Bligh arrived at values of the creep-head ratio which he believed would make dams safe from piping failure. The description of the various classes of material and the values of the ratios varies somewhat in different publications by Bligh. The following table gives the various descriptions and ratios:

Safe Creep-Head Ratios Given by Bligh

Year of publication:-	Publication I 1910	II 1910	III 1913	IV 1916	V ?
River beds of light silt and mud, as the Nile	18				
Fine silt and sand as in the Nile River		18			
Very fine sand and silt			18		
River beds of light silt and sand of which 60% passes the 100-mesh sieve, as those of the Nile or Mississippi				18	
In mud and silt, such as in the Nile					18
Fine micaceous sand as in the Colorado and Hymalayan rivers	15	15			
Fine sand			15		

(Cont.)

Year of publication:-	Publication I 1910	II 1910	III 1913	IV 1916	V ?
Fine micaceous sand of which 80% passes a 75-mesh sieve, as in Hymalayan rivers and in such as the Colorado				15	
In fine; e.g., Punjab sand					15
Coarse-grained sands, as in Central and South India	12			12	
Ordinary coarse sand		12			
Coarse sand			12		
In coarse sand (this is the usual type)					12
Boulders or shingle and gravel and sand mixed	5 to 9			5 to 9	
Gravel and sand		9	9		
Boulders, gravel and sand		4 to 6			
Boulders and gravel			4 to 6		
In clay, shale or shingle					5 to 9

Publications

I The practical Design of Irrigation Works, 1910, 2nd Ed. p. 165.

II Dams, Barrages and Weirs on Porous Foundations, Engineering-News, Vol. 64, December 29, 1910, p. 708.

III Lessons from the Failure of a Weir and Sluices on Porous Foundations, Engineering-News, Vol. 69, February 6, 1913, p. 266.

IV Dams and Weirs, 1916, p. 155.

V Control of Water, Parker, 1916, p. 679.

(The original source by Bligh is not given).

The first three classifications are fairly consistent. A classification of silt by rivers, however, is not a safe procedure as that in the upper reaches is usually much coarser than in the lower. It is not known on what grounds Bligh put the Colorado River in the class with a ratio of 15, as the silt from the lower Colorado is much finer than the sizes given, in fact even finer than those given for a ratio of 18. The classifications for boulders, shingle, gravel and sand differ somewhat, in one case using a minimum ratio of 4 and in another case 5. The classification, gravel and sand, 9, and boulders, gravel and sand, 4 to 6, is the most useful, since it gives a definite value for gravel and sand without boulders, a common foundation material. It is interesting to note, however, that in his latest publication Bligh does not give a ratio lower than 5. Bligh also stated (Publication III) "that 6 is the very minimum for gravel." The classifications given by Parker are entirely different for the lower ratios and could not be located in available publications by Bligh.

Just how extensive a study of dams was made by Bligh as a basis for his data is not known, but that published is quite meagre. It includes only two dam failures,* both on fine sand foundations, and

*In a later publication Bligh discusses the failure of the Southern Alberta Land and Irrigation Co. headgate. The published data on this structure is so conflicting as to throw serious doubt on the reliability of his conclusion. See Eng. Rec. V. 63, p. 589, V 66, p. 376, Eng. News V. 69, p. 266, V. 75, p. 1070.

in one or both of these, as will be shown later, unsound conclusions were drawn. There were no dam failures on silt, coarse sand, gravel or boulders. These statements are not made to disparage Bligh's work, which has formed the basis of the design of scores of safe dams, but merely to indicate that it apparently was based on meagre data and that with much more extensive data, and better construction materials, a reduction of Bligh's values of C might be made without conflicting with well established facts.

Griffith gave in an abridged article* the following values of the ratio of creep distance to head which "had been found sufficient to ensure stability" in the United Provinces of India.

*Proceedings, Inst. of C.E. Vol. 197, Pt. III, p. 221.

Material	Limiting Safe Value of C
Fine micaceous sand	$14\frac{1}{2}$ to 16
Fine quartz sand	$12\frac{1}{2}$ to 14
Coarse quartz sand	10 to 12
Shingle	8*
Boulders	4*

*Griffith states that "the question of loss by leakage may make higher values of L/H advisable in these cases."

In an unpublished portion of this paper he suggested "a 20% reduction in the values given where reliable vertical staunching of 10 foot depth was used."

In the third edition of Bligh's Practical Design of Irrigation Works, (Preface page VI) which was revised and brought up to date by F. F. Woods, Chief Engineer of Irrigation Works, Punjab, India, Mr. Woods contends that Bligh should have used a ratio of 11 for ordinary sand instead of 15. It will thus be seen that both Griffith and Woods advocate somewhat lower values of C than Bligh. The author independently arrived at the same conclusion from a study of the data on most of the dams included in the tables of this paper.

A New Method of Analysis

As has already been pointed out, the existing methods of analysis are open to serious objections. The commonly used, method, as advocated by Bligh, does not consider the greater resistance to flow along vertical contacts as compared with horizontal ones. The short path method, and the more exact flow net and electrical methods do not consider the lesser resistance along the contact of the masonry and foundation material as compared with that directly through the foundation material. All these methods have elements of truth but all have weaknesses. A method should be devised which will combine the virtues of both without including their faults.

In the present state of our knowledge the only method of analyzing the probability of failure from flow along the creep line seems to be a study of the action of actual dams. No exact data has yet been presented to show the relative resistance to the flow along the contacts as compared with that through the foundation material. It will be very difficult to obtain this data because points where danger of flow along the contact is great will only occasionally occur, and would

probably be discovered only by very extensive observations. The flow through the foundation material can be obtained with more exactness by experimental methods, but even here the unknown conditions of the foundations and the difficulty in determining the safety of the structure even if the velocity of flow is known, make this method difficult to apply. It seems obvious therefore that while research along this line should be given every encouragement, for a long time in the future the main reliance in dam design must be on a somewhat empirical basis.

As the studies made in connection with this paper indicated faults of the existing methods of analysis, as well as the insufficiency of the factors used in the ordinary methods, the author has attempted to work out a more rational method of design, as well as to establish more accurate coefficients. In order to develop a method which could readily be used by the designing engineer in planning actual structures, in addition to the reason given above, the method developed was necessarily a somewhat empirical one.

For clarity in presenting the data on which the author's conclusions are based, it is necessary here to briefly review the basic facts of the new method developed.

From a study of all the available data it appeared that there were two distinct forms of piping, one in which the water passed along the line of contact of the structure and its foundation, as assumed in the Bligh theory, and the other in which it passed directly through the voids in the foundation material. In the former path the contact of the foundation on vertical or steeply inclined surfaces can be counted on to offer more resistance to flow than along horizontal or slightly sloping contacts. In computing the safety of a structure therefore from this type of failure, the creep distances along horizontal or slightly sloping surfaces should be given less weight than those along vertical or steeply inclined surfaces. This method of estimating the stability of a structure may be called the weighted creep method. As a result of these studies a weight of one-half is given to the horizontal or slightly inclined creep as compared with the other portion of the path.

For brevity in the remainder of this paper, creep along vertical or surfaces sloping more than 45° with the horizontal will be called vertical creep, and other surfaces as horizontal creep. It should be noted that "vertical or steeply inclined" refers to the position of the surface against which creep takes place, and not to the direction of the creep, which sometimes is not in the same direction as the inclination of the surface. The weighted creep distance is the vertical creep plus one-half the horizontal creep. The weighted creep distance is therefore practically always less than the creep distance.

As the water follows the line of least resistance, if the resistance to flow along the creep line is much greater than directly through the voids in the foundation material much more water may take the latter course and failure from piping from this cause result. The short path is a rough measure of the resistance along this path. Both the weighted creep and the short path ratios must therefore be greater than their respective critical values for the type of material on which the dam is founded.

Method of Analysis of Data on Existing Structures

In this study an intensive search was made in all available engineering literature and other sources of information. All masonry or concrete dams on earth foundations were analyzed where the data was sufficiently complete. In many cases there is considerable uncertainty. Where more than one description was found of a single structure there was often a surprising lack of agreement between them.

There is frequently considerable room for the exercise of judgment as to just where the travel paths should be assumed to end. The travel path of the water was ordinarily assumed to end at its junction with the riprap downstream from the dam or at a reverse filter. If wooden cribs or loose or articulated concrete blocks were used, it was assumed to end at the upstream edge of these. Although it is true that riprap, cribbing or blocks may assist in preventing piping or a blowout it is believed that less uncertainty is introduced by considering that they do not add to the travel path than to assume that they did.

In checking over the data given by Bligh with that available from other sources, some discrepancies were found. Where the data on some dams indicated that a certain creep-head ratio had been used in design, it was usually not possible to check this exactly from the dimensions of the structure. In order that all the data might be on the same basis, the same method was used throughout, although this might not give the same ratio which the designer of the dam believed he was using. In all cases the author's best judgment was used in determining the most probable values for use in this paper. He would appreciate it if readers of this paper would call his attention to any cases in which his judgment may have been in error.

At several points in the analysis of some of the dams it was necessary to make assumptions. One of these was in the case of rows of sheet piling close together. The analysis in this case was made according to that outlined on page .

In computing the weighted creep distance it was necessary to determine the division point between steeply sloping and slightly sloping

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contacts. This division should be at the steepest slope at which a bank of earth under water would be unquestionably unstable, ie., at which the bank undoubtedly would slip. This was assumed to be on a 1:1 slope. Any contact with a slope of 1:1 or steeper was considered to be a "vertical" contact and at a flatter slope a horizontal contact. In a few cases a puddle or earth fill against a dam rested on top of a portion of the dam masonry. Although this was a horizontal contact, it was assumed to be a "vertical" contact, as the earth would press as closely on the masonry under these conditions as on a vertical surface.

In several cases dams have been built with a filling of dry or broken stone beneath them. This would offer much less resistance to seepage than the contact between solid masonry and foundation material. Where the floor above this stone filling has not been vented, to allow the water to escape, the resistance of the creep line along this section has been assumed to be half as great as for solid masonry. This is a conservative estimate from the standpoint of this study, but would not be if used in design. The use of such filling seems to be confined to old dams and is probably a bad practice. In one case it is believed to have contributed to the failure of a structure.

Considerable uncertainty has been introduced in the determination of the creep distance by the presence of weep holes or vents. In determining the distances used in this report, weep holes have not been considered as reducing the creep distance, but a separate analysis of them has been made.

There was sometimes uncertainty as to the head to use in estimating the ratios. In ordinary overfall dams it was measured from the crest of the dam to tailwater elevation, or, if the latter was not given, to the streambed below the dam. If crest gates or flashboards were used, it was measured from their top. The ordinary operating head was used, altho, in some cases, especially movable navigation dams, the structures were no doubt occasionally subjected to greater heads. For many structures, especially irrigation headgates, it was not possible to determine the ordinary head with sufficient accuracy and therefore they could not be analyzed.

Several cases have been found where dams were built on a layer of porous material, such as gravel or sand, which was underlaid by an impervious layer of clay or hardpan, into which the cutoff walls or sheet piling extended. These are not subject to analysis in the ordinary way and therefore the data on them is given without analysis.

Classification of Foundation Materials

There is a great need of a more accurate and scientific classification of foundation materials than the common terms, gravel, coarse sand, etc. Several classifications on the basis of grain size have been made, but none are entirely satisfactory. One developed by the United States Bureau of Soils* has been extensively used in the classification of soils and of materials of earth dams. For

*Grouping of Soils on the Basis of Mechanical Analysis, R. O. E. Davis and H. H. Bennett, U. S. Department of Agriculture, Circular No. 419

dam foundation materials however it is not entirely satisfactory as its classifications, clay, fine sand, coarse sand, etc., are much finer material than the engineer has in mind when using these terms. Mechanical analyses of foundation materials could be obtained for only a few dams. Those available are plotted on Figure 1, giving the grain sizes in inches and millimeters, the Bureau of Soils classification, and the size of standard sieves.

Results of Analysis of Existing Structures

The results of the analyses of the weighted creep relations for all structures for which sufficient data could be obtained are given in Tables 1, 3, 5 and 7. The first column gives each dam a number which is useful in identifying it on the figures. The headings of the second to fifth columns need no explanation. The next two columns give the vertical and horizontal creep distances. The eighth column gives the weighted creep distance, horizontal creep having a weight of one-half. The next column gives the weighted creep-head ratio. The tenth column gives an indication of the probable reliability of the computed values of travel path and ratios. In some cases the data was somewhat in doubt. The eleventh column, "Results," indicates what is known about the success of the structure from the standpoint of failure from piping. Failure from other causes was not considered. In the case of structures for which no definite data is given in this column, for the larger ones when located in the United States, it is very probable they are still giving reasonably satisfactory results or record of the failure would have been published. Smaller structures in the United States and those in foreign countries are probably still in use, although there is a slight chance that they may have failed. The next column gives a brief description of the material on which the structure was founded. The terms are not always as explicit as might be desired, due to incomplete data in the articles describing them. The last column furnishes pertinent data not included in the other columns.

Although the results of this study indicate the desirability of using the weighted creep in considering dams on porous foundations, for those who prefer to use the plain creep or short path relations, the data on these features is given on Tables 2, 4, 6 and 8. On these tables are also given the sources from which the data on these structures was obtained.

These results of the weighted creep analysis are shown graphically on Figures 2 to 5 where the weighted creep length is plotted against head on logarithmic paper and lines are drawn showing the various weighted creep-head ratios. The failures are indicated by solid dots and the dams which, so far as known, did not fail by piping, are indicated by open circles. Different sections of the same dam, or conditions of the same dam before failure and after repair are shown by dots joined by a line.

The majority of the failures are dams of very poor design, but they serve to show approximately the limits of good design. Well designed dams rarely fail, but more lessons can be learned from failures than successes. Some well-known failures are not included because the data is insufficient or too conflicting to permit reliable conclusions. These include the Hauser Lake Dam, Missouri River, the Grand Barrage on the Nile and the Southern Alberta Land and Irrigation Company headgate.

Several cases were found where the data is so complex that it is difficult to analyze. This was the case in the Alcona, the dam in the Scioto River at Columbus, Ohio, and that on the Guadalupe River in Texas.

Cross sections of the dams analyzed, for which no description appears in easily accessible literature, are given in Plates I to VI.

Dams on Clay and Hardpan

The results of the analyses of dams on clay and hardpan are given in Tables 1 and 2 and are shown graphically on Figure 2. In order to demonstrate more clearly the basis for the ratios recommended, a few of the dams constructed on this material will be briefly described. These are either dams which have failed or those with low ratios of which no failure is recorded.

Woodward Dam at Hill, New Hampshire.

This dam was founded on material so hard that if an iron bar was driven into it 12 to 18 inches, it was sometimes necessary to use a derrick to pull it out. The adjacent banks were of a hard cemented sand and clayey material with occasional

layers of pure sand. The foundations were said to have been excavated 10 to 12 feet below the natural surface of the earth. The line of creep and the short path were both about 32 feet, which, with the 30 foot head, give ratios of length of path to head of 1.1. A large proportion of the creep on this dam was vertical. The weighted creep-head ratio was 0.93.

This dam failed after five years of service with no sign of failure. Where dams fail after a considerable period of satisfactory service with heads that have been carried before satisfactorily, the probable cause of failure is piping along the line of contact of the dam with its foundation. When failure takes place suddenly, at a head which has not been held before, or with a path which is suddenly shortened by an apron blowup, the cause may be percolation directly thru the foundation material. As this dam failed after a long service apparently at no unusual head, the failure was probably due to percolation along the contact of the masonry with the earth. The layers of pure sand may have been responsible for the failure.

Dolgarrog Dam, Lake Eigiau, North Wales.

The Dolgarrog dam was founded on a glacial deposit of hard blue clay containing boulders, overlaid with a layer of peat. Part of the foundation was not carried as deep as planned. This section failed by a blowout under the foundation after the dam had been in service fourteen years. The reservoir had previously been drawn down below the dam foundation due to prolonged dry weather and the drying out of the clay may have been the real cause of failure. The portion of the dam built according to plan did not fail. The creep distance for the latter portion was 39 feet and for the part which failed 33 feet which for the 10 foot head gives creep head ratios of 3.9 and 3.3 respectively. The corresponding weighted creep ratios are 2.0 and 1.0 respectively. As the dam held fourteen years with the lower ratio this should be nearly, if not entirely safe.

Fergus Falls, Red River, Minnesota.

The Fergus Falls dam was founded on hardpan. It failed after eleven months service by a blowout under the power house. It appears that a spring was encountered in the power house foundation and no outlet was provided for it. This spring probably forced an outlet for itself and gradually enlarged it. The path from the headwater to the spring was too short and piping occurred from the headwater to the spring and thence along the spring outlet channel. There is no evidence to show that the spillway had insufficient length of path. The creep

distance for the spillway was 104 feet, giving a plain creep ratio of 4.5 and a weighted creep ratio of 3.8. The creep distance to the spring was 45 feet, giving a plain creep ratio of 2.0 and a weighted creep ratio of 1.4.

North Diversion Dam, Orland Project, California.

This dam is founded on rather impervious clay with considerable firmly imbedded gravel and has been in service since about 1913 with no appreciable leakage beneath the structure. It sustains a normal head of 4.7 feet with a creep distance of 5.0 feet, giving a plain creep-head ratio of 1.1. All of the creep of this dam is horizontal and the weighted creep is therefore 0.55.

Whiting Street Dam, Holyoke, Massachusetts.

This dam was founded on a very compact unyielding gravelly hardpan. It has a head of 18 feet and a creep distance of 36 feet, making a plain creep-head ratio of 2.0. The weighted creep ratio was 1.6.

Lake St. Francis Dam, St. Francis River, Quebec, Canada.

This dam is founded on stiff clay with small boulders and gravel (hardpan). The head is about 37 feet, and has been giving satisfactory service since 1917. The creep distance for the bulkhead and spillway sections is 94 and 121 feet respectively, giving plain creep-head ratios of 2.5 and 3.3. The corresponding weighted creep ratios are 1.9 and 2.4.

Kettle Creek Dam, Kettle Creek, Ontario.

The Kettle Creek dam is founded on very hard blue clay, and sustains a head of 20.5 feet. It has a creep distance of 66 feet, giving a plain creep ratio of 3.2 and weighted creep ratio of 2.0. About 27 feet of this creep is under an apron 12 inches thick. If appreciable seepage under the dam occurred, the apron would be lifted up, reducing the creep distance to 39 feet and the weighted creep ratio to 1.2.

Marysville Dam, Mashwaak River, New Brunswick.

This dam is founded on hardpan composed of firm impermeable clay containing at least 50% of large boulders. The head is 22 feet and creep 92 feet, giving a plain creep-head ratio of 4.2 and weighted creep ratio of 2.6. The creep of a gravel fill on the upstream face is neglected. The dam has stood with a

weighted creep ratio of 2.4 for a short time. The lower 30 feet of the creep distance is under an apron 12 inches thick. If much seepage should occur under the dam the apron would be blown up, reducing the weighted creep ratio to 1.9.

Dams on Gravel, Cobbles and Boulders.

The data on dams on gravel, cobbles and boulders is given in Tables 3 and 4. The results are shown graphically on Figure 3. Dams on foundations composed largely of sand but with considerable proportions of gravel are included in this group. They represent an intermediate condition between coarse sand and gravel, and to include them in this classification is on the side of conservatism.

To set forth more fully the basis for the lengths of percolation path required for various materials, the dams having most influence on the conclusions will be described at some length.

Puentes Dam, Guadalantin R., Spain.

The Puentes dam was built in 1785-1791. Most of it was on rock but in the center of the valley it was founded on piles in earth variously described as "earth", "alluvial soil and sand" and "sand and gravel at least 25 feet deep," the latter classification being assumed as being the most conservative. The dam was 151 feet wide on the base and had a masonry apron 7.3 feet thick extending 131 feet further downstream. Nearly all the creep was horizontal. The failure occurred suddenly after 11 years of use. The pressure beneath lifted the apron at the toe of the dam causing it to fail and thus greatly reducing the creep distance, resulting in a blowout. The reservoir was said to be silted 44 feet above the base of the dam and at the time of failure the head was 143 feet. It had reached 75 feet in previous years without failure.

Considering the effect of the silt, the line of creep to the end of the apron was 339 feet. With the head of 143 feet at time of failure this gave a creep-head ratio of 2.37 and a weighted creep ratio of 1.36. No signs of failure at the end of the apron appeared for this ratio, but might have, had these conditions continued. The blowout reduced the creep distance to 200 feet, the plain creep ratio to 1.40 and the weighted creep ratio to 0.86. In a previous year with 75 foot head, the respective ratios were 4.5 and 2.6 assuming the same silt conditions at the time of failure, although it is possible that they were less. This indicates that the dam was safe with a 2.6 weighted creep-head ratio, possibly safe with a 1.36 weighted ratio, and certainly unsafe with 0.86 weighted creep ratio.

Port Angeles Dam, Elwha R., Washington.

The base of the Port Angeles Dam rested upon a "glacial deposit consisting principally of gravel and small grit or coarse sand." It was 97 feet wide on the base below which a caisson had been sunk 11 feet. An earth fill was constructed on the upstream side with a height of 50 feet and base of about 100 feet. A row of sheet piling 30 to 40 feet long was driven 8 feet downstream from the toe of the dam, but not connected to it. This piling was detrimental, as it tended to concentrate the underseepage into the space between the dam and the piling. The water had been at about spillway level for nine days when signs of increased pressure were noticed below the dam and within two or three hours the entire bottom gave way.

The creep distance was about 100 feet. The head was uncertain, probably 79 feet, but may have been higher. The plain creep-head ratio was therefore not more than 2.1. The weighted creep ratio was 1.5. The shortest path was 150 feet and gave a short path ratio of 1.9.

The dam was repaired by filling the canyon with rock blasted from the side hills, reinforced with willow mats, sheet piling and a large concrete block. The short path distance is about 650 to 700 feet and the head practically 100 feet, making a short path ratio of 6.5 to 7.0. Under these conditions it has been in satisfactory service since 1913. On account of the lack of a definite contact between the fill and the original material, no creep distance can be estimated.

Pittsfield Dam, West Brook, Massachusetts.

The Pittsfield Dam was founded on an upper layer of gravel and fine material 4 to 5 feet deep, underlaid by a much finer and apparently impervious stratum of clayey material 5 to 6 feet thick, beneath which was a waterbearing gravelly mass containing much fine material with boulders from hens-egg size to possibly a cubic yard in volume. The upstream cutoff wall extended into the impervious layer, but not down through the pervious material beneath. The dam seems to have been designed on the assumption that the foundation material below the top layer was impervious. Apparently the flow thru the pervious material appeared downstream, either lifting the impervious layer or causing piping along some channel leading thru it. The failure was probably more in the nature of piping through the material rather than along the line of contact with the dam. The shortest path was 63 feet and the head 33

feet, giving a short path ratio of 1.9. The plain creep distance and ratio were 76 feet and 2.3 respectively. The weighted creep ratio was 1.6. The dam was repaired with a cutoff wall extending through the pervious layer and has been giving satisfactory service for about 20 years.

Coon Rapids Dam, Mississippi River, Minnesota.

The Coon Rapids Dam on the upper Mississippi River failed from a sudden blowout during repairs, after the dam had been giving satisfactory service (from the standpoint of piping) for over three years. The primary cause was a gap one pile wide in the sheet pile cutoff. The hole below the dam caused by scour was pumped out to build an extension of the apron, causing an increased head on the structure. The head was approximately 48 feet, and the shortest path 130 feet, giving a short path ratio of 2.7. Since the water level below the dam at this time was considerably below the bottom of the downstream piling, the water did not follow the creep line and therefore the creep-head ratio cannot be estimated.

The earth below the dam had a sloping surface and would therefore fail more easily than if the ground were level. On the other hand the gap of only one pile would considerably restrict the flow as compared with the ordinary conditions assumed in design.

Plattsburg Dam, West Brook Plattsburg, New York.

The Plattsburg Dam was founded on a glacial material of the following composition: 25% boulders, 26% fine gravel, 1% very coarse sand, 2% coarse sand, 4% medium coarse sand, 4% medium sand, 8% medium fine sand, 1% fine sand and 29% sand and silt passing a 100-mesh screen. There was less than 1% clayey material. The base of the dam was stepped up the sides of the hills. The upstream cutoff wall was said to be 20 feet deep. It is reported to have been built with wood sheeting left between the concrete and the natural earth and that the backfill was made with excavated material and construction debris. The weep holes planned for the structure were plugged. It is also reported that a trench was excavated along the upstream side of the cutoff and refilled with puddle, but this report could not be confirmed. Leakage was evident from the first, and the dam failed from a blowout, probably from seepage along the line of creep, apparently about two months after it was filled.

The length of the line of creep varied for different sections of the dam and the creep-head ratios varied from 7.1 to 3.2, the higher ratios being for the lower parts of the dam where it was stepped up the side of the valley. The corresponding weighted creep ratios were 6.1 to 2.4. The shortest path ratios varied from 5.8 to 2.6. Where the failure took place the head was probably approximately 22' and the creep 87', giving a plain creep ratio of 4.0 and a weighted creep ratio of 3.1. The shortest path at this point was probably 70', giving a shortest path ratio of 3.2.

Pinhook Dam, Maquoketa River, Iowa.

This dam and power house are founded on what is reported to be "fairly well graded sand, altho there is considerable gravel and occasional boulders as large as 1/3 cu.yd. (A sieve analysis is given on Figure 1.) The head is 25' and the creep distance to the end of the fixed apron below both the power house and spillway is 130', giving a plain creep-head ratio of 5.2 and a weighted creep-head ratio of 4.0. Below this fixed portion however, there are concrete slabs constructed to allow settlement without cracking, but without watertight joints. Since these slabs no doubt to some extent increase the resistance to piping, it would not be correct to assume that the success of this dam indicated that the 4.0 ratio was safe. The creep distances to the end of this slab apron for the power house and the spillway section are 206' and 231' respectively, which give plain creep-head ratios of 8.0 and 9.2 and weighted ratios of 5.7 to 6.3. The shortest paths, not including the block apron are 92' and 91' for the power house and spillway respectively, giving ratios of 3.2 and 3.1. Including the block apron these distances are 161' and 179' and the ratios 5.6 and 6.2 respectively.

Richmond, Indiana Dam.

This was a small dam, founded on sand and gravel, sustaining a head of 5 ft. with a creep distance of 27 ft. giving a plain creep-head ratio of 5.4 and a weighted creep-head ratio of 3.8. The dam is believed to have been washed out in a flood about seven years after construction.

Boise Intake Dam, Boise River, Idaho.

This dam is founded on very compact gravel and cobbles. It sustains a head of 33 ft. with creep distance of 96 ft.

giving a plain creep-head ratio of 2.9. Much of this creep is horizontal and the weighted creep ratio is 2.2. The shortest path is 92 ft. and short path ratio 2.8. This dam has been giving satisfactory service since about 1906.

Oswegatchie River, New York.

This dam, for part of its length, is reported as being founded on "glacial boulders" but as light sheet piling was driven, the boulders were more than likely combined with sand or gravel. The head was 23'. The cutoffs were formed by concrete walls with 2x6 inch sheet piling on each side of the wall driven down 4 to 7 ft. beyond the bottom of the concrete. Except for springs encountered during construction, there was no seepage. The piling is assumed water tight and that the resistance to travel between the two adjacent rows is equivalent to a creep twice the distance between the piles. The creep distance is 82 ft. and the head 23 ft., making the plain creep ratio 3.6. The weighted creep ratio is 2.7. The shortest path is 57 ft. giving a short path ratio of 2.5.

Granite Reef Dam, Salt River, Arizona.

The Granite Reef Dam is founded on gravel and boulders and sustains a head of 20'. The total creep distance to the porous apron is 77', giving a plain creep-head ratio of 3.8 and a weighted creep-head ratio of 3.1. The shortest path to the upstream edge of the porous apron is 58', giving a short path ratio of 2.9, and the short path to the weepholes is 47, giving a ratio of 2.3.

When first put into operation there was a seepage of two' to three sec. ft. under the 300' length not founded on rock, but the pool above the dam filled with sand and silt, which reduced or stopped the seepage. It is possible that the rock fill under the downstream apron acted somewhat as a reverse filter and increased the security of the structure. After some years in service the apron was destroyed by a flood, and the dam stood some time without it before it was repaired, showing that at that time it was not necessary for security.

Avignonet Dam, Drac River, France

This dam was built on gravel about 1903 and sustains a head of 56 ft. It has a creep distance of 266 ft., giving a plain creep-head ratio of 4.8 and a weighted ratio of 3.2. The short path is 217 ft., making the short path ratio 3.6.

Dams on Coarse Sand

The data on dams on coarse sand is given in Tables 5 and 6 and shown graphically in Figure 4. A more detailed description of the dams which have an important bearing on the safe length of path necessary for this class of material follows:-

Riverdale Dam, Apple River, Wisconsin.

This dam was founded on torpedo sand, and held a head of 24'. It was completed in 1905 and was still apparently in service in 1909. The maximum line of creep is 101', but this includes 20' for sheet piling composed of steel rails driven as close together as possible, which would probably not be watertight, and the probable creep was therefore 81', the plain creep-head ratio 3.4 and the weighted ratio 2.1. Considering the rails to have made an effective cutoff, the plain creep ratio would be 4.2 and the weighted ratio 2.9. This value will be used as it is more conservative. The shortest path is 71', and the short path ratio, 3.0. Downstream from the dam is an apron of large stones which may have acted to some extent as a filter. The foundation was full of springs during construction.

Godavery Weir, Godavery River, Madras, India.

The portion of this weir founded directly on coarse sand had a creep distance of 100 ft. and sustained a head of 10.8 ft., giving a plain creep-head ratio of 9.3 and a weighted creep ratio of 6.9. The short path was 77 ft. and short path ratio 7.1.

Dam No. 1., Big Sunflower River, Mississippi.

This dam was founded on sand, with a little clay, some gravel and some clayey sand. It sustains a head of 16.9 ft. with a creep distance of 111 ft., giving a plain creep ratio of 6.6 and an equal weighted creep ratio. The rows of sheet piling are close together and the upstream one considerably longer than the downstream one, so that the path is assumed to cut across at a 2:1 slope from the end of the downstream piling to the upstream piling, as described on page 53. The dam has given successful service since about 1917.

Prairie De Sac Dam and Power House, Wisconsin River, Wisconsin

This dam was built about 1912 and is founded on pure coarse sand. It is hollow concrete, the buttresses are

supported by piles and there is no floor under the dam. Seepage is prevented by a 50' row of sheet piling and an upstream clay blanket about 20' wide. The inside of the dam is filled with loose rock, through which any water passing under the sheet piling may rise and until the vents were closed escaped thru them in the downstream side of the dam. Water may also seep out thru the sand and under the downstream sheet pile cutoff. Since the plain creep ratio from the inside of the dam to tailwater is much greater than between the headwater and the vent, the underflow undoubtedly escaped through the vents, until they were closed. The head from headwater to vents is 29', and the total ordinary head 32'. The creep distance to the inside of the dam, where it formerly could discharge thru the vents, was 124'. With the vents closed, the total creep distance is 198'. The creep-head ratio to the inside of the dam, with the vents open was therefore 4.3 and with the vents closed the ratio for the entire structure was 6.2. The corresponding weighted creep ratios are 3.9 and 5.1.

The shortest path with the vents open is 104', giving a ratio of 3.6, and with vents closed 149', with ratio of 4.7.

The creep distance under the power house is 301', which, with a head of 32', gives a plain creep ratio of 9.4 and weighted creep ratio of 7.2. The shortest path is 216' and short path ratio 6.8.

Dams on Sand, Fine Sand and Silt

The data on dams on sand, fine sand and silt is given on Tables 7 and 8 and shown graphically on Figure 5. A brief summary is given in the following paragraphs of the percolation conditions for those dams which have an important bearing on the limitations of the length of the seepage path.

Narora Weir, Ganges River, India.

This weir was founded on fine micaceous sand, probably falling into Bligh's class "of which 80% of the grains pass a 75-mesh sieve." It sustained a head of about 13 feet. After about 20 years of service it failed and was repaired with a much longer creep distance.

This failure has occupied a large place in discussions of the design of dams on porous foundations, and therefore has been studied in considerable detail. Bligh contended that it was a case of piping, which had been going on for twenty years, and on the basis of this failure, and that of the Khanki weir,

built the structure of his widely used methods. A thorough study of the data leads to considerable doubt as to the accuracy of some of Bligh's conclusions. There are many differing opinions on the exact manner in which the failure took place. In Bligh's Practical Design of Irrigation Works, 3rd Edition, Mr. F. W. Woods records several opinions, and Griffith has still another. Woods contends at length, apparently with considerable justification, that the failure was primarily due to undercutting of the upstream portion of the weir by a lateral current and that this occurred in the river channel section where it was built on a fill.

For the purpose of this paper it is not necessary to determine exactly how the structure failed. Certain facts seem to be well established. The greater portion of the structure which was not built on a fill stood without failure for twenty years, although some trouble with springs at the downstream edge seems to have been experienced. This portion had a creep head ratio of 13.0 and a weighted creep ratio of 8.0. The upstream puddle apron in the section where failure occurred had been washed away. In some portions of the weir, springs of considerable magnitude came up through the upstream edge of the grouted riprap. The spaces between the wells forming the downstream cutoff were said to be "indifferently closed" by piles.

Bligh apparently was not in possession of all these facts when he reached his conclusions. He assumed that the line of creep followed the line A C D E F G H I J K L M N. (Plate IV) This gives a length of approximately 169 feet, and a creep-head ratio of 13.0. The upstream puddle is known to have been washed away, and this reduced the creep ratio to 11.0. However it is very likely from the accounts of the failure that the seepage path ended in springs at the upstream edge of the grouted pitching. If this is the case, the ratio would be still further reduced to 7.3. If, in addition to this, the creep path passed through the indifferent staunching between the wells of the downstream cutoff, the ratio would reach the low value of 5.4. It is apparent therefore that Bligh's conclusion that the failure occurred with a ratio of 13.0 is incorrect, and if the dam failed from piping at all it occurred with a ratio of not more than 11.0, probably with 7.3 and perhaps as low as 5.4. The corresponding weighted creep head ratios are 7.2, 5.4 and 3.5 respectively. The data for the distances and ratios for plain creep, weighted creep and short path are as follows:

	Path	Plain Creep Dist.	Plain Creep Ratio	Weighted Creep Dist.	Weighted Creep Ratio	Short Path Path	Short Path Dist.	Short Path Ratio
Conditions Assumed by Bligh	A,C,D,E,F, G,H,I,J,K, L,M,N	169	13.0	104	8.0	A,D,E,H, I,M,N	135	10.4
Puddle Apron Missing	B,C,D,E,F, G,H,I,J,K, L,M,N	143	11.0	93.5	7.2	B,C,D,H, I,M,N	113	8.7
Exit of Percolation at J	B,C,D,E,F, G,H,I,J,P	95	7.3	70	5.4	B,C,D,H, I,J,P	80	6.2
Puddle Missing, Exit at J and Percolation Between Wells	B,C,D,E,F, G,J,P	70	5.4	45	3.5	B,C,D,E, G,J,P	67	5.2
Reconstructed Weir	- - - -	201	15.4	130	10.0	---	158	12.2

Khanki (Lower Chenab) Weir, Chenab River, Punjab, India.

This weir was founded on fine sand and failed about three years after construction. When it was repaired, the apron was extended and sheet piles were added. The data on creep distance from various sources differs considerably and the values used are therefore somewhat arbitrarily selected.

Bligh's method of design and his constants were based principally on the results with this structure and the Narora Weir. There is unfortunately considerable uncertainty regarding the cause of failure. Bligh contended that it was from piping but Woods argues very convincingly that it was due to subsidence resulting from undermining on the upstream side by a lateral current, that this occurred where the dam was built on a fill and that the weir was safe where not undermined. Buckley however states that there were bad springs below all bays of the weir before it failed, and since failure occurred only three years after construction, it is reasonable to conclude that the original design was not very secure, even if the failure did not result from piping.

The original design included very little vertical creep. It was repaired by extending the upstream apron and adding a

cutoff of wells or sheet piling, the nature of the repairs differing at different parts of the weir. Bligh gives the head as 13 feet. Buckley states that the maximum head may have been 12 feet, and the average 8 to 9 feet. In the following table is given a summary of the analysis of plain creep, short path and weighted creep for both 12 and 13-foot heads. The 12-foot head is adopted as being more conservative.

Before Failure

	Bligh Creep		Weighted Creep		Short Path	
	Dist.	Ratio	Dist.	Ratio	Dist.	Ratio
Using 13-foot head	119	9.2	67	5.2	110	8.5
Using 12-foot head	119	9.9	67	5.6	110	9.2

After Reconstruction

	Bligh Creep		Weighted Creep		Short Path	
	Dist.	Ratio	Dist.	Ratio	Dist.	Ratio
Using 13-foot head	209	16.1	128	9.8	188	14.5

Deoha Barrage, Deoha River - United Provinces, India.

This barrage was constructed on fine sand, with a creep-head ratio of 17, the entire travel being along the base of the structure, as there was no sheet piling. It was put into service in July 1925. Trouble was experienced with cracking in 1925-26. In 1927 some evidences of piping and subsidence were observed and strong springs with sand discharging downstream from the canal headgate developed during a flood but ceased as the flood receded. On account of danger of undermining at the downstream edge of the apron in 1928-29 steel sheet piling 15 feet long was driven, but difficulty was experienced in getting a continuous cutoff properly tied into the floor. Thickening of the floor on account of the increased upward pressure was found to be unnecessary. Interlocking steel sheet piling 20 feet long was also driven upstream from the headgate and no more trouble appeared from

the headgate springs. A line of interlocking steel sheet piling, 25 feet long, was started along the upstream edge of the floor.

In September 1929, when it was necessary to carry the full 12 feet head, springs suddenly appeared downstream of the sheet piling in the first bay, cracks soon developed and the floor was slightly raised. A distinct channel formed leading from the end of the upstream line of sheet piling at one abutment to the downstream row of piling, thru which the water escaped. Portions of the floor were undermined and one portion upstream of the gates was forced down by the weight of the water over it, and a section downstream was blown up.

The length of the creep path was more than 320 feet, giving a creep-head ratio of at least 26.6. No appreciable part of this path was along a vertical or inclined surface. This distance is so extreme that it is reasonable to assume that the channels along at least a portion of the path had started to develop before sheet piling was driven. As the length of the path which developed under the various conditions is unknown the structure at the time failure occurred cannot be analyzed in a quantitive way but it does serve to bring out several important points. It is evident that the original creep distance giving creep head a ratio of 17 was insufficient. The long distances that creep can develop under a horizontal foundation surface are strikingly shown, emphasizing the value of creep distances along vertical surfaces. The necessity of tight sheet piling well connected to the foundation masonry and of tying in the ends of the sheet piling cutoff at the abutments so that the water cannot find its way around the end of the cutoff are convincingly illustrated.

Kitcha Bye-Wash Weir, Rohilihand Canals, United Provinces, India.

This structure was founded upon dry sand above ground water level. It had a puddle upstream apron covered with grouted riprap and a stepped masonry drop, with 12' sheet piling at the downstream edge. There was no upstream cutoff. Normally no water was against this structure. Several times small flows passed over it, each time causing piping and failure of the floor. The first time a real test of the structure occurred, it failed to such an extent as to require abandonment.

There are differing opinions as to the cause of the failure, evidently it was a case of piping, but the path of the piping is not definite. The water seems to have passed thru cracks in the puddle, which had probably dried out, or thru the contact of the puddle and the masonry. The head was about 8.0

feet, and the creep distance which the water actually took, probably about 58 feet, making the plain creep-head ratio 7.2 and a weighted creep head ratio of 5.9. The corresponding figures for short path are 47 feet and feet. Possibly the settlement of the dry sand when brought into contact with water contributed to the failure of the structure. It is possible also that a head brought suddenly on a dry sand would give rise to greater percolation than sand continuously wet. This failure serves to bring to attention the danger of puddle aprons where they may dry out, and the possible danger of founding a structure on dry soil.

Kulli Dam, Rohilihand Canal, India.

This dam was originally constructed with an apron of insufficient weight, which was blown up, necessitating the reconstruction of the weir. The reconstructed weir stood without any signs of trouble for about ten years, when it suddenly failed, apparently from piping along the line of creep. The structure was founded on fine sand, without any cutoff walls or sheet piling. The creep ratio was between 1:14 and 1:16. The structure was rebuilt with much the same section as before, but with an upstream cutoff of interlocking steel sheet piling, and has given no trouble since. The data available is insufficient to give exact data on this structure, but its failure is of major importance as showing the necessity of vertical staunching if shorter creep ratios than those advocated by Bligh and Griffith are to be used.

Kulli Bye Wash, Rohilihand Canal, India.

This structure had several failures of the upstream apron which were patched up with reinforced concrete, but after 3 or 4 years it failed beyond repair. It was built on fine sand, without cutoff walls or sheet piling and had a plain creep ratio of between 14 and 16.

Corpus Christi Dam (Abutment Failure) Nueces River, Texas.

The author believes that the failure of the Corpus Christi Dam abutment was due to the existence of a short percolation path under and behind the abutment. This is the only record of a failure due to this cause which he has found, and the only warning of this possibility of such a failure seems to have been given by Griffith.* The short path was apparently down

*Proc. Institution of Civil Engineers Vol. 197, Pt. III, P.229

the upstream face of the dam directly over the edge of the abutment, under the abutment to its rear edge, up over the top of the sheet piling, down to the bottom of the abutment and thence to the drain. There were two lines of tile drain under the dam. The creep path to the end of the upstream one of these was apparently about 103 ft. and to the downstream one 154 ft., which, with a head of approximately 37 ft. would give plain creep ratios of 2.8 and 4.2 and weighted creep ratios of 1.9 and 2.6 respectively. The data available does not indicate a filter around the tile drain and fine material seems to have escaped through one of the drains, until a "pipe" was formed leading to that drain. As this enlarged the flow became greater than the drain could carry without pressure, and a pressure under the apron was built up which eventually blew up the apron. The water rushing through this opening and the "pipe" already formed, rapidly enlarged its passageway and undermined the abutment. It may be concluded from this that the weighted creep ratio responsible for failure was not over 4.2 and may have been 2.8. The material at the abutment was sand.

Nepean River, New South Wales.

The weir was founded on sand and sustained a head of 6 ft. with a creep distance of 90 ft., giving a plain creep-head ratio of 15.0 and weighted creep-head ratio of 9.8. Some springs appeared below but were too small to effect the stability of the structure. It was built about 1905 and was probably in service at least seven years.

Islam Weir, Sutlej River, Punjab, India.

This weir was founded on sand. It was in service about two years when it failed from retrogression of the tailwater level. It seems to have been satisfactory from the standpoint of piping. The percolation data is as follows.

	Head	Plain Creep		Weighted Creep		Short path	
		Dist.	Ratio	Dist.	Ratio	Dist.	Ratio
Main Sluice Section	18	249	13.8	162	9.0	195	10.8
Sand Sluice Section	18	264	14.7	169	9.4	210	11.7

Zifta Barrage, Nile River, Egypt.

This barrage was founded for part of its length on running sand and mud. It sustains a head of 13 ft. with a creep distance of 210 ft., giving a plain creep-head ratio of 16.2 and weighted creep-head ratio of 10.2. It has an inverted filter at its downstream edge. It has been in service since about 1902.

Iron Mountain Dam, Menominee River, Michigan.

The bed of the river at the damsite is covered with a layer of sand and gravel, paved with small boulders, beneath which is fine sand. The dam probably rests on this sand layer. It sustains a head of 30 ft. with a creep distance of 257 ft., giving a plain creep-head ratio of 8.6 and a weighted creep-head ratio of 6.3. The downstream apron of this dam is thin and the pressure beneath it is relieved by a drain of tile surrounded by gravel. This dam has been giving satisfactory service since about 1926.

Important Facts Shown by Experience With Existing Dams

A study of all the available records of percolation distances of existing masonry dams on earth foundations, and those which have failed from piping brings out three very important facts. These facts are (1) that several dams have failed from piping with percolation distances which judged by the ordinary standards should be safe, (2) that a number of dams have stood successfully with percolation distances much less than previously recommended (3) that the dams which failed had very little of their creep paths along vertical or steeply sloping surfaces, while those which stood with much smaller distances had a considerable proportion of such creep.

Failures have occurred with creep distances which ordinary standards would consider safe in the case of the Kulli Dam, Kulli By Wash and the Deoha Barrage. The first two of these were founded on fine sand and failed with creep-head ratios of between 14 and 16. For fine sand Bligh recommends a ratio of 15 and Griffith 14½ to 16 for fine micaceous sand and 12½ to 14 for fine quartz sand. The Deoha Barrage on fine sand was threatened with a creep-head ratio of 17 and finally failed with a piping path with length at least 26.6 times the head. None of these structures had vertical cutoffs.

The second fact is based on the data on dams given in the following table, which shows that much lower ratios than recommended by Bligh or Griffith have been successfully used. As Bligh and Griffith do not give ratios for material of the exact description of that existing at some of these

dams, the author has used his best judgment in estimating from the classifications they did give what they would have probably recommended.

Dam	Material	Creep-Head Ratio	Recommended Ratios (approximate) Bligh	Recommended Ratios (approximate) Griffith	Actual Ratio in % of Recommended Bligh	Actual Ratio in % of Recommended Griffith
Dam No. 48 Ohio River	Very fine sand	13.2	18	$14\frac{1}{2}$ to 16	73	83 to 91
Dam No. 47	Sand, very fine to medium, a little fine gravel	13.5	18	$14\frac{1}{2}$ to 16	75	84 to 93
Iron Mountain	Fine sand	8.6	15	$12\frac{1}{2}$ to 14	57	61 to 69
Riverdale	Coarse sand	4.2	12	10 to 12	35	35 to 42
Prairie du Sac	Coarse sand	4.3	12	10 to 12	36	36 to 43
		6.2	12	10 to 12	52	52 to 62
Dam No. 1, Big Sunflower River	Sand with a little clay, gravel and clayey sand	6.6	14	10	47	66
Granite Reef	Gravel and Boulders	3.2	5 to 9	6	36 to 64	53
Oswegatchie R.	Sand and Boulders	5.6	5 to 9	8	40 to 72	45

Considering the three failures before mentioned where the creep distances were substantially equal to the recommended values with the ratios in the preceding table where the ratios are much less than recommended, it is a very significant fact that the group which failed had nearly all the creep distance horizontal and the latter had substantial portions of their percolation distances along vertical or steeply sloping surfaces. In fact only one case of failure by piping which might have been along the line of contact with a properly constructed deep cutoff was found. This was in the case of the Floderborn Dam, where the creep ratios were

much less than this study indicated would be safe. The failures of the Coon Rapids and Plattsburg dams were apparently due to defective construction and the Pittsfield dam to a blowout directly through the foundation material from a porous layer beneath the impervious layer on which the dam was founded and not along the line of creep.

The logical conclusion from these facts is that in the design of masonry dams on earth foundations greater weight should be given to the creep along vertical or steeply sloping surfaces than along horizontal or slightly sloping ones. This has lead the author to the development of the weighted creep analysis of existing dams and dam failures.

Origin of the Weighted Creep Analysis

The author's attention was first directed to the greater value of vertical creep by the difficulty of explaining the stability of the Prairie du Sac Dam with its extremely low ratio of 4.3 on a coarse sand foundation. The evidence from one structure however was not sufficient to justify a general conclusion.

Frequently when one forms a correct impression and thoroughly studies the literature of the subject he finds the same idea expressed by others. There are few absolutely unique ideas in engineering. The idea apparently was first expressed by Mr. W. M. Griffity, who suggested in his abridged paper that a reduction could be made in his suggested creep-head values for reliable vertical staunching. He explains the greater effectiveness of vertical creep as follows: "First, the line of creep is subject to a greater pressure where carried to a greater depth; secondly, at the greater depth larger and heavier sand will probably be met; and thirdly, sand carried out must be lifted up the curtain wall, presumably requiring greater velocity of percolation." Mr. Griffith kindly gave the author data on the Kulli Dam and Kulli By Wash and expressed his present conviction that the creep-head ratios he gave were probably too low unless some vertical staunching was used. The author's explanation for this difference seems to have been independently reached by Mr. A. A. Musto*.

*Punjab Engineering Congress 1930, paper No. 142, p. 2121

The effect of sheet piling in reducing upward pressure and cutting down the flow through the voids in the foundation material has been discussed at length by Khosla and Leliavsky. Both seem to have reached the conclusion that such piling reduces the pressure by a greater amount per unit of contact length than horizontal creep.

Relative Weights of Vertical and Horizontal Creep

The proper relative weight to give to vertical and horizontal creep can be determined only by analysis of actual dams, although some light may be thrown on it by an analysis of observation of upward pressure on actual dams. While the results of the analyses of all available data on percolation distances of dams does show conclusively that the horizontal creep distance is not as effective in resisting piping as vertical creep distance, it does not indicate accurately just what their relative effectivenesses are. Although several dams without appreciable vertical creep have failed with distances normally considered sound, no failures were found with considerable vertical creep except at ratios much less than ordinarily considered safe and therefore the upper limit of weight must remain uncertain.

In arriving at the decision to use a weight of one half for horizontal creep the author listed all the structures having creep distances below or near what might be considered the lowest safe limit, together with the material upon which they were founded and the weighted creeps for each, with weights for horizontal creep of 0.5, 0.4 and 0.33. At the same time a table was made of the weighted creep ratios which the results of analysis of actual dams indicated to be safe for the various creep ratios. A careful study of these does not show essential superority of one weight over the other.

A careful study of the upward pressure measurements on the Percha*, Island Park*, Pinhook* and Narora** weirs shows that the drop in upward

*Trans. Am. Soc. C. E. Vol. 93, 1929., p. 1717.
**Punjab Engineering Concress, Paper No. 142, plate II

pressure along horizontal concrete surfaces is almost zero, and along a puddle surface somewhat more, indicating that for horizontal concrete surfaces the ratio might be as low as zero. It does not necessarily follow, however, that the safe ratio for horizontal creep in considering piping failure is the same as in considering upward pressure. More study along this line is very desirable. In view of the uncertainity as to how low a ratio to use for horizontal creep, the author believes it is advisable to be conservative. As the present use of unity for this weight has usually produced safe structures (although frequently unnecessarily conservative) it is undesirable to depart too far from the present standards until data definitely establishing the safety of the new ones is available. The author therefore for the present recommends the use of a weight of one-half for the horizontal creep, rather than four-tenths or one third, although there is existing data to indicate that the latter, and perhaps even lower ratios may be better, and experience in the future may establish it conclusively.

Suggested Weighted Creep Ratios

The following table gives a summary of the weighted creep ratios for structures which throw most light on the limits of safety against failure from piping along the creep line.

Summary of Limiting Weighted Creep Ratios

Table & No.	Dam	Foundation Material	Weighted Creep Ratios: Weight of Horizontal Creep $\frac{1}{2}$
Tbl.1:		Clay or Hardpan	$\frac{1}{2}$
1	Woodward (Failed)	Hard material, probably hardpan	0.93
2	Dolgarrog "	Hard blue clay	1.00
3	Fergus Falls "	Hardpan	1.40
11	Dolgarrog (Standing portion)	Hard blue clay	2.00
12	Marysville	Firm clay with boulders	2.60
14	Kettle Creek	Very hard clay	2.00
15	Lake St. Francis		
	Bulkhead section	Stiff clay with small boulders and gravel	1.90
	Spillway section		2.40
20	Whiting Street	Very Compact gravel	1.60
21	North Diversion	Clay with considerable firmly embedded gravelly hardpan	0.64
Tbl.3:		Gravel, cobbles & Boulders:	
1	Puentes (failed)	Sand and gravel	0.86
2	Port Angeles "	Gravel, small grit or coarse sand	1.50
5	Plattsburg "	Boulders, fine gravel and sand	3.10
22	Pinhook	Sand and gravel	4.00

Summary of Limiting Weighted Creep Ratios (Cont'd)

Table & No.	Dam	Foundation Material	Weighted Creep Ratios: Height of Horizontal Creep $\frac{1}{2}$
23	Richmond	Gravel and sand	3.9
26	Puentes (Held)	Sand and gravel	2.6
41	Boise Intake	Very compact gravel and cobbles	2.2
43	Oswegatchie R.	Sand and glacial boulders	2.7
49	Granite Reef	Gravel and boulders	3.1
54	Avigonet	Gravel	3.2
Tbl.5:		Coarse Sand	
13	Riverdale	Torpedo sand	2.9
15	Godavery Weir	Coarse sand	6.9
16	Prairie du Sac		
	Power House	Pure coarse sand	7.2
	Dam (Vents open)	Pure coarse sand	3.9
	Dam (Total creep)	Pure coarse sand	5.1
19	Big Sunflower Dam #1	Sand with a little clay, gravel & clayey sand.	6.6
Tbl.7:		Sand, Fine Sand & Silt	
1	Narora (Failed)		
	Probable	Fine sand	5.4
	Possible	Fino sand	3.5
	Not over	Fine sand	7.2
2	Khanki (Failed)	Fine sand	5.2
3	Corpus Christi (Failed)	Sand	2.6
4	Deoha (Failed)	Fine sand	9.4
5	Flederborn (Failed)		
	Probable	Sharp sand underlaid by fine sand	4.0
	Possible		5.3

Summary of Limiting Weighted Creep Ratios (Cont'd)

Table & No.	Dam	Foundation Material	Weighted Creep Ratios: Weight of Horizontal Creep $\frac{1}{2}$
23	Zifta	Part with running sand & mud	10.2
29	Narora (Reconstructed)	Fine sand.	10.0
30	Khanki (Reconstructed)	Fine sand.	9.8
34	Iron Mountain	Fine sand.	6.3
39	Nepean River	Sand.	9.8
44	Islam	Sand.	9.4

In the following table are given the weighted creep ratios which the analysis of all available data from existing structures and particularly those in the above table indicated as necessary for safety against failure from piping along the contact of the structure and its foundation. In order to use these values with safety the cutoffs must be of solid masonry built in contact with the earth sides of the trench or of interlocking steel piling driven so that the interlock is not broken, and satisfactorily embedded at the top in the masonry structure. They also assume competent supervision during construction and efficient maintenance afterward.

These values are for major structures and are believed to be conservative. Somewhat smaller values may be used for less important structures, ranging down to perhaps 75 per cent of those given for those of minor importance. There are so many types of foundation material that it is impossible to give values for all. Only the usual types are therefore given and the other conditions can be determined by comparison with these.

Suggested Weighted Creep Ratios

Weight of Horizontal Creep $\frac{1}{2}$

Very fine sand or silt	12.0
Fine sand	10.0
Medium sand	9.0
Coarse sand	7.0
Fine gravel	5.5
Medium gravel	4.5
Coarse gravel including cobbles	3.8
Boulders with some cobbles and gravel	3.4
Soft clay	4.0
Medium clay	3.3
Hard clay	2.5
Very hard clay or hardpan	2.1

These values are not the lowest which have been found for structures which have not failed, but rather are believed to be conservative conclusions from the data at hand. Conditions at the various structures differ so much that it is improbable that a definite line of demarkation exists which would be applicable to all structures. It is not improbable that when more data on actual structures has been accumulated, it will be found that somewhat lower values can be safely used, but until that time, the Author does not feel justified in going lower.

The values for medium and soft clay are somewhat uncertain as no record was found of dams founded on these materials. If the requirements of bearing pressures can be met, the values given would seem to be sufficiently conservative.

An examination of minimum weighted creep ratios suggested as a result of this study shows that they are roughly two-thirds to one-half those recommended by Bligh. A comparison is shown in the following table:

Material	Weighted Creep Ratio	Bligh's Value
Very fine sand or silt	12.0	18
Fine sand	10.0	15
Coarse sand	7.0	12
Medium gravel	4.5	
Gravel and sand		9
Boulders with some cobbles and gravel	3.4	
Boulders, gravel and sand		4 to 6

This does not mean that Bligh's values are 50% to 100% too large, or that dams designed according to the weighted creep ratios will have only two-thirds or one-half the percolation distance which dams have had in the past. The difference does not represent as radical change from the present custom as a comparison of the ratios would suggest. The requirements of length to protect against scour, and to secure sufficient bearing pressure will still necessitate considerable horizontal creep. For dams on coarse gravel including cobbles and boulders with some cobbles and gravel, the necessity of conserving all the water may in some cases require greater percolation distances, as the values given may permit too much seepage.

The effect of the adoption of the weighted creep method will be to put the emphasis on deep cutoffs, and lead to their use as far as the limitations of scour prevention and bearing pressure will permit. It will tend to eliminate the wide massonry floor type without substantial cutoffs. For example, to meet the weighted creep requirements for a weir on fine sand with no vertical creep, would require an ordinary creep distance of 2 x 10 = 20, while the ordinary creep distance according to Bligh should be 15 and according to Griffith $12\frac{1}{2}$ to 16.

Because of the emphasis which the weighted creep method puts on deep cutoffs, if the greatest economy is to be secured, it will be necessary to investigate more fully than before the length of apron necessary to prevent scour, and the best shape of dam to accomplish this purpose. The pressure on the designer will be toward using as short an apron as possible and unless careful studies are made there will be a tendency to adopt one which is too short. It should be kept in mind that the weighted creep method deals with piping only, and the requirements of scour may dictate much more expensive structures than safety from piping would necessitate. The recent advances in hydraulic model tests have clearly shown the reliability of this means of determining the conditions necessary to prevent scour. The design of important structures should therefore include a hydraulic model study.

Other Considerations Influencing the Selection of Weighted Creep Ratios

Several factors besides the nature of the foundation material should be considered by the designing engineer in selecting the weighted creep or short path ratio to use in the design of the type of structure under consideration, if the most economical result is to be secured. The safe ratios given on Page 40 may therefore be modified somewhat in view of these factors. No definite rules can be given, as it is a matter of judgment. It is believed, however, that the data

available do not justify more than slight reductions in the ratios given, except for unimportant structures.

Perhaps the most important of these considerations is the damage to life and property which would result from a failure. For a dam so situated that a failure would result in loss of life an unquestionably safe design is required. If extensive property downstream would be endangered, conservatism of design is indicated. For most dams on porous foundations the conditions are such that little damage would result from their failure except to those directly served by them, and the degree of conservatism should be dictated by a comparison of the damage which would result from failure with the cost of the additional factors of safety. One phase necessary to consider in estimating the damage which is likely to result from failure is the time necessary to make repairs or temporarily patch up the structure. A failure which would put a hydro-electric plant out of service for a considerable time, or shut off the supply of irrigation water for a long enough period to cause the loss of a crop, or more especially of orchards or citrus fruit groves, would be very serious and justify larger ratios, while for a structure which could be temporarily patched up in a short time more risk of failure would be justified. For this reason small structures of low head can usually be designed with lower weighted creep ratios for very minor structures perhaps as low as three-fourths as much as those given in the table.

Another consideration is the quality of supervision which the structure would have during and after construction. The weighted creep-head ratios suggested in this paper assume first class inspection during construction. It is doubtful if an engineer should consent to design a dam on porous foundations unless he was satisfied that it would be built with intelligent supervision, but should that not be available, higher values than those suggested should be used.

Most dams on porous foundations require more or less maintenance. Many dams have experienced difficulties from time to time which would have eventually led to failure without competent maintenance. Usually it is necessary occasionally to add riprap below them or repair the flexible apron. If repairs are not made with reasonable promptness scour at the lower edge of the structure may attain such proportions that the conditions assumed in computing the travel paths for the design may no longer hold and the original security against piping no longer exist. Unless a reasonable degree of maintenance can be counted upon, higher values of weighted creep-head ratios than those suggested should be used.

For dams on coarse gravel, a factor which deserves consideration in selecting a creep-head ratio is the money loss which would occur if

some seepage took place beneath the dam without endangering the structure. For example, in dams for power plants, any loss would usually result in a decreased plant output, but for an intake dam for an irrigation system where only part of the water in the stream was diverted and water would, therefore, be continually wasted over the dam, the flow under the dam would be of no consequence.

Reverse Filters, Weep Holes and Drains

Another factor which should be considered in selecting the proper ratio to use in the design of a masonry dam on a porous foundation is the possibility of using reverse filters, weep holes or drains. The reverse filter was probably first developed by J. S. Beresford and seems to have been used extensively about twenty years ago. It was used on Zifta and Assuit barrages on the Nile River and on the Kindia Barrage in Mesopotamia. It has been used in this country on dams Nos. 7 and 13 on the Mohawk River. It consists of a filter placed at the lower edge of the downstream apron, built up of a bottom layer of fine material surmounted by other layers of progressively coarser material. It is called a reverse filter because the position of the layers is the reverse of that in ordinary filters, but the water passes first through the fine and then the coarse material as in other filters. Its only function is to filter the water seeping beneath the dam and prevent the removal of any of the fine foundation material. Although usually covered with riprap, the reverse filter is apt to be damaged or destroyed by the high velocity water passing over the dam.

Holes through the apron of the dam or the wing walls, called weep holes, have been extensively used in America but no examples were found of their use in other countries. Most of the available literature on India and Egypt, however, was published many years ago and it is possible that they have come into use there since then. Their function is usually to relieve upward pressure beneath the apron of the dam and thus permit the use of a thinner apron without danger of its being lifted from under pressure. In recent years pipe drains have been employed in some cases instead of weep holes.

The greatest advantage is secured by combining the functions of the reverse filter and weep hole by constructing a filter behind the weep holes or around the pipe drains. In this position the filter is protected from scour.

Since piping failure results from the removal of foundation material from beneath the dam, if a filter prevents the removal of this material, the dam will be safe, even though large quantities of water are passing beneath it. The quantity which passes through the filter depends upon the head acting on it. In ordinary filtration practice, a filter is frequently subjected to a head of water several times its thickness. If an ordinary reverse filter were subjected to such a

head it would be lifted bodily. Such filters therefore cannot be subject to much head. If however the filter is weighted down by a sufficient thickness of concrete apron it can be subjected to a much higher head than a reverse filter and the dam be made safe for the passage of a greater quantity of water beneath it. With such a filter near the end of the percolation path of the water flowing beneath a dam, a greater velocity of flow in this water is therefore permissible and therefore a shorter percolation path can safely be used.

Weep holes and drains have been used with very short percolation distances between them and the headwater. In one case a flow through the weep holes sufficient to cause geysers several inches high occurred without apparent ill effects.*

*Trans. Am. Soc. C. E. Vol. 88 1925 p. 319

A striking example of the effectiveness of vents came to light at the Coon Rapids Dam. This is an ogee structure having a thin downstream apron with steel sheet piling cutoffs under the heel of the dam and at the downstream edge of the thin apron. Immediately behind the upstream cutoff a drain was constructed consisting of a reverse filter discharging into a small tunnel built in the dam. Due to an accident, one of the steel sheet piles of the upstream cutoff was pushed down far beyond its proper position leaving an opening in the cutoff, and reducing the creep distance to the drain to 0.6 times the head. In spite of this the flow through the drain does not seem to have been sufficient to excite alarm and the defect in the cutoff was not discovered until a large hole washed below the dam was pumped out during repairs to the apron and a blowout occurred. Had the drain not existed it seems very probable that the thin concrete apron below the dam would have been blown up.

Table 9 gives data on dams using weep holes, drains or reverse filters, together with the creep-head, weighted creep-head and short path ratios to the most upstream vent, the foundation material and other pertinent data. The low ratio resulting from the short distance from the headwater to the upstream vent in many cases is striking.

The author does not believe that it is safe to use low ratios for drains or weep holes ecept where combined with a reverse filter. It is probable that the use of a drain without a filter contributed to the failure of the Corpus Christi Dam abutment. The Bureau of Reclamation has for many years used weep holes with filters of gravel beneath with good success. It is the author's opinion that while weep holes and drains are valuable devices, the greatest care must be exercised in their construction. A weep hole or drain which will permit the foundation material beneath the dam to pass through in effect, is a pipe already

partially formed, and certainly no such weak spot in the protection of the dam should be deliberately constructed with the impression that it was adding to the safety of the structure.

Weep holes or drains have been used on dams on practically all classes of material. The necessity of a well constructed filter is especially evident in the case of weep holes or drains with clay foundations, as the fine clay could readily pass out through them unless restrained by an effective filter. In gravel a filter probably would form naturally by the washing out of the fine material near the weep hole or drain, in the same manner as a filter tends to form around the screen of a driven well. However, in view of the small expense in installing a filter behind the weep hole it is believed that their construction is justified even in gravel. They are absolutely necessary in sand or silt, if weep holes are depended upon to reduce the length of the percolation path required.

The location of the weep holes and filters should be given considerable study in dam design. From the standpoint of the prevention of piping, the best location for the filter is probably beneath the apron at the downstream end just upstream from the cutoff wall or row of sheet piling which is usually placed at the downstream edge of the structure, the weep holes discharging through the wall or piling into the riprap or other scour protection material. The downstream wall or row of piling in this case does not act so effectively as a cutoff but is very desirable to prevent undermining. It need therefore be no longer than necessary to accomplish this purpose. To reduce the upward pressure on the apron it may be necessary to place the weep holes farther upstream. They should not be placed farther than necessary for this purpose (unless required by safety from sliding) for to do so shortens the percolation path with no advantage, but brings a greater pressure on the filter and therefore increases the danger of failure. For this reason it is believed that the installation of a drain immediately downstream from the upstream cutoff is not a good practice.

A word of caution also may not be amiss regarding drains beneath the apron of dams where a hydraulic jump is formed. If the drain discharges downstream from the jump while its position under the apron is upstream from the jump, the pressure beneath the apron will be equal to or greater than that where the drain discharges, which, being below the jump, will be greater than the pressure acting downward on the apron, due to the thin sheet of water upstream from the jump. This unbalanced pressure may be enough to lift the apron. In the Iron Mountain Dam the drain discharges at the toe of the ogee section, upstream from any jump which may form. In this case the drain may be downstream from the jump and discharge upstream, which would cause an unbalanced pressure acting downward on the apron, increasing its stability. Weep holes which

discharge directly through the apron equalizes the pressure on the two sides, but they should be constructed in such a manner that the high velocities can not suck foundation material through them.

If it can be conveniently done, it is advisable to have drains discharge where the flow from them can be observed, in order that any failure to function properly may become known. In northern climates reliance should not be placed in weep holes or drains which may freeze up and thus become inoperative.

It is believed that by the use of weep holes or drains constructed with reverse filters which prevent the removal of the foundation material a reduction of 10% in the safe weighted distances given on page 60 may be permitted. This assumes that the vents are of sufficient capacity, are free from freezing and are located a sufficient distance upstream from the lower end of the percolation path that practically all of the underseepage will reach the tailwater through them. No fixed rule can be given on these requirements, but the exercise of reasonable judgment should prevent serious mistakes.

Effect of Silting of Riverbed above the Dam

The possibility of silting of the riverbed above the dam should be given consideration in determing the permissible percolation ratio, as it may greatly reduce the upward pressure beneath the dam and the probability of failure by piping. The upward pressure on the Island Park Dam was almost entirely removed in two years or less, only the upward pressure from ground water remaining. The upward pressure on the Hamilton Dam was practically eliminated by silt deposit, in spite of the fact that a gravel plant continuously excavated gravel 100 to 200 feet upstream. Both of these dams are on the Miami River, which has a gravel bed (see Figure 1) and does not carry an unusual amount of silt. On the other hand, the upward pressure on the Sherman Island Dam on the Hudson River does not seem to have changed materially over a period of five years.* It is possible, however, that considerable

*Trans. Am. Soc., C. E., Vol. 93, 1929, p. 1576

silting took place when the pond was first filled, which apparently was about 2 months before upward pressure measurements began. The effect of silt in reducing the flow under the Granite Reef Dam has already been noted, and the fact that it probably made this dam safe, when otherwise its security was doubtful. For an intake dam, where the entire width is composed of gates, extending nearly down to the riverbed, silting could not be relied upon to increase the security, as the flow through the gates might remove the impervious silt bed. This effect has been noted at the Grand Valley Dam on the Colorado River.*

*Trans. Am. Soc., C. E., Vol. 93, 1929, p. 1530.

It would not be advisable to count on the silting up above a dam to reduce the necessary travel ratios and at the same time count on vents for further reduction, since if the silting up was effective, the vents would be inoperative. It must be remembered, moreover, that immediately after closure, before the bed has had a chance to silt up, the dam would be subjected to more severe piping conditions, and, therefore, it would be safe to assume in the design only the limited reduction which the dam might stand until it silted up.

Dams on Pervious Material Underlaid by Clay or Hardpan

A number of cases were found of dams resting on pervious material under-laid by clay or hardpan with the cutoffs extending into the impervious material. These cannot be analyzed according to either Bligh's or the short-path method. In fact, no satisfactory method of treating them has been found and therefore the data only is given in Table 6. In determining this data, the shortpath in the clay is assumed to follow the creep line, as it seems unreasonable to consider a path directly through the practically impervious clay.

Consider such a dam founded on a thick layer of pervious material with a sheet pile cutoff extending a few feet into the clay beneath. If well constructed, such a cutoff would nearly entirely stop the flow of the water beneath the dam and result in a head approximately equal to headwater elevation on the upstream side of the piling and to the tailwater level on the downstream side. This condition is strikingly shown by the upward pressure measurements on the Sulemanki Weir in India.*

*Stability of Weir and Canal Works - A. N. Khosla, Punjab Engineering Congress, 1930 - Paper No. 142.

Nearly the entire head of the dam would therefore be concentrated on the few feet of creep along the piling, in the clay. This would be a severe test on this contact and if even a minute passageway existed water would rapidly flow through it and would probably enlarge it to a certain extent. However, unless the pervious material was very coarse, the clay washed out in enlarging this hole would be filtered out in passing through the pervious material and form an impervious coating on the "filter" which would greatly retard flow through it. Also, the loss of head of the water in passing at high velocity through the pervious material in the immediate vicinity of the upstream and down stream ends of the "pipe" through the clay would decrease the velocity and reduce the scour in the"pipe." This action is somewhat analoguous to the flow through a nearly closed valve. Under ordinary conditions, therefore, no great enlargement of the opening could be expected.

Another reason why this would be the case is that the flow coming through the small opening upon entering the pervious material would spread out into the greater flow area of the pervious material and lose velocity. By the time it had reached the surface where movement of the particles of pervious material was possible it would have too low a velocity to cause movement. No significant movement of the particles near the bottom of the sheet piling would be possible because they would be surrounded by material which was not moved. Therefore, unless there were enough of these openings to permit sufficient flow to cause piping velocities at the downstream edge of the dam, no danger to the structure would result from them.

It is evident, therefore, that cutoffs into impervious material beneath dams greatly increase the safety of the dam against piping even if the contact between the cutoff and the impervious material is not watertight. No satisfactory method has been worked out for estimating how much reduction of travel path can be allowed under these conditions, and since the data at hand is slight, it must remain for the present largely a matter of judgment. For this sort of foundation however, great care must be exercised to prevent the concentration of flow around the end of the pile cutoff from forming a pipe under or behind the abutment.

Dams on Clay Underlaid by Pervious Materials

Structures which rest on a thin layer of clay underlaid by pervious material seem especially likely to cause trouble. Apparently the water frequently enters the pervious layer over a large area and seeks to escape at the downstream end of the structure in a comparatively small area, giving rise to flows of considerable magnitude which are apt to endanger the structure and possibly may blow up the impervious layer due to excessive upward pressure. The pressure in the pervious layer is sometimes due to groundwater conditions of the surrounding terrain and only remotely connected with the local headwater conditions.

The Pittsfield Dam seems to have had this kind of foundation, its failure being probably due to a blow up of the impervious layer. The Nathaura Escape Head was another case of difficulty and final failure under these conditions. As the details of this failure are not accessable to most engineers, a summary is given here.

Nathaura Escape Head, Sarda Canal, United Provinces, India.

Data from Note on the Failure of the Nathaura Escape Head - Department of Public Works, United Provinces, India.

Technical Library
Bureau of Reclamation
Denver Colorado

The foundation of this structure was excavated in a thick layer of clay underlain with sand, which in turn was probably underlain with a lower stratum of clay. The floor level was nearly to the bottom of the clay layer. It seems to have been designed for a creep-head ratio of about 13 and had no sheet piling cutoff and very little vertical creep. The sand layer carried water under pressure, the spring level being about 5 feet above the downstream escape floor. A strong spring developed downstream which functioned even when the canal was closed. After throwing up a sand crater it appeared to stop removing foundation material. A water cushion three feet deep was maintained over it and later disappeared. With a head of 7.5 feet, giving a creep-head ratio of about 17 the structure failed suddenly from undermining.

The immediate cause of this failure is somewhat obscure. It is probable however, that the pressure built up beneath the clay layer downstream from the structure until it suddenly raised a considerable area of it, breaking the contact with the downstream apron of the structure and permitting the water within the sand layer to escape. With the sudden release in pressure, the sand changed to a dilatant state (see page 50) and was rapidly washed out from beneath the structure, until a channel formed beneath the structure and undermined it.

In the report describing the Nathaura failure mention is made of difficulties at the Udra, Narwa and Jabaria culverts of the same canal under similar conditions. Mr. Khosla* has described

*Punjab Engineering Congress 1930. Papers Nos. 138 and 142

in detail the difficulties with the Dugri No. 1 and Jauryan siphons in the Punjab and the remedies unsuccessfully and successfully used to remedy the trouble. Although success is sometimes achieved under these conditions* the number of failures and near failures should serve to

*The Design and Construction of the Bassano Dam - Engineering News, Vol. 72 8/27/14, p. 425.

warn one contemplating the construction of a dam on such a foundation to look into the matter very carefully. As the conditions may vary so greatly it is probably better not to attempt to prescribe a general rule.

Limitations of Creep Analysis

As has already been pointed out, piping may occur as a result of percolation along the lines of contact of the dam with its foundations, or by flow through the foundation material itself. In a given case failure would result by travel along the path which offered the least resistance.

In order to determine experimentally the length of percolation path necessary to prevent piping from flow through the material Mr. W. M. Griffith performed a series of experiments with a masonry trough $1\frac{1}{2}$ x $1\frac{1}{2}$ ft. in section, filled with fine micaceous sand, and covered with earth, to determine the head of water necessary* to cause a blow-out to occur

*Data from an unpublished portion of Mr. Griffiths paper previously mentioned.

The results showed that a blowout did not occur until the ratio of length of creep to head reached values of 5.33 to 2.5 although experience had shown that creep-head ratios of about $14\frac{1}{2}$ were necessary to secure safety in dams built on this sand. He found that the flow of water through the sand column increased directly with the head until failure occurred, which came suddenly, without the previous formation of springs to form a channel and carry away the material. He also found that loading the sand decreased the tendency to blow out, and that water brought up suddenly on dry sand increased the tendency.

Much the same results were obtained by Terzaghi*, who found that the water percolating through the sand increased with the head until a point was reached at which the permeability of the sand suddenly decreases, and the velocity through it consequently increases, and a blowout takes place.

*Technical Publication No. 215 Pg. 36 Am. Inst. of Min. and Met. Engrs.

This increase in permeability is the result of a change of position of the sand grains to a less compact arrangement, giving them an increased bulk and greater voids. The effect of the added weight in Griffiths experiments was probably to resist this change of volume and therefore increase the head necessary to cause a blowout. This phenomenon has also been studied by Osborn Reynonds.* and by Dr. Warren J. Mead**

*Osborn Reynodds Scientific Papers, Vol II (1901) p. 217
**The Geologic Role of Dilatancy - Journal of Geology, Vol. XXXIII, No. 7, October, November 1925.

In his paper "On Percolation Under Aprons of Irrigation Works" S. Leliavsky discusses a number of experiments to establish the head ratio at which blowout occurs and explains the difference in the results obtained by different experimenters. This is largely due to the slope of the surface of the material where the water emerges. If the surface is inclined, less pressure is necessary to move the particles than if the surface is level, as gravity assists in the movement and blowout will therefore take place with lower head.

The data is insufficient to define closely the safe limit of length from the standpoint of flow directly through the foundation material. It does indicate however in a rough way that the assumption is justified that the flow takes place directly through the material when the distance is less than half the weighted creep distance. On this basis one might assume that the limit of safe short path would be one-half that of safe weighted creep. There is another factor which enters here however. Even with well designed and maintained dams scour may occasionally take place at the downstream edge. Under these conditions the material on the side of the hole is more easily moved by percolating water than if it were on a flat surface. For this reason it is believed advisable to limit the short path ratios to three-quarters of those given for weighted creep.

As previously stated, there is a defect in the creep analysis when rows of sheet piling are too close together. Bligh recognized this and stated that the percolation followed the line of creep as long as rows of sheet piling were not closer together than twice their depth.*

*The Practical Design of Irrigation Works, Bligh 2nd. Ed. p. 168.

Some such limitation of sheet pile spacing is necessary, for it is obvious that two rows of long sheet piling would offer little more resistance to piping than one row, while if the rows were a great distance apart they would offer considerably more resistance than one row. Bligh did not suggest any reason for using the limit of twice the depth rather than some other value, nor did he discuss what would take place if piling rows were spaced closer than his limiting value.

As the flow follows the path of least resistance Bligh's limit seems to be equivalent to stating that when the piles are spaced twice their depth the resistance along the creep line is equal to that directly through the material and when closer than twice their depth the resistance directly through the material is less than along the line of creep. When the piling are spaced twice their depth apart the creep distance between their lower ends is twice the short path distance and since for this condition the resistance along these two paths are equal, the resistance per unit length along the short path is twice that per unit length along the creep line. Wherever there are two points on the creep path so close together that a path exists directly through the foundation material which is less than half the weighted creep distance between the points, the flow would take place directly through the material with a resistance of twice that along the creep path. Where several overlapping short paths are thus possible, the one which gives the smallest combined creep and weighted short path distance for the entire structure is the one on which the travel would take place.

For example, two lines of steel sheet piling were driven for the Middle Loup River Dam, 31 feet apart. One extends 29 feet below the bottom of the dam and the other 39 feet below. One possible short path which fulfills the above conditions extends horizontally from the end of 29 foot piling to the longer piles. Other possible paths extend from various points on the shorter row of piling to various points along the longer row. The path extending between the lower ends of the sheet piling rows gives the lowest weighted creep for the structure of all the possible paths, and should therefore be adopted as the path to be used. Where the water follows the short path, the path connecting the ends of the two adjacent sheet piling rows will usually give the lowest weighted creep for the structure unless the bottom of the longest row is more than one-half the distance between the rows below the bottom of the other row, in which case the short path extends from the bottom of the shorter row to a point having an elevation one-half the row spacing lower on the other row.

The above reasoning applies also to the weighted creep. In this case the horizontal creep is considered to have a weight of one-half, the vertical creep a weight of one and the short path a weight of two. In computing the plain and weighted creeps for existing dams, these rules have been used wherever applicable.

The rule that the flow follows the short path if its length is less than one-half the weighted creep distance between any two points in the creep path permits somewhat smaller pile spacing than Bligh's rule, since the horizontal creep, which in Bligh's rule has a weight of one in computing the creep distance, has a weight of only one-half in computing the weighted creep distance.

If boring records indicate a very pervious layer extending beneath the dam and coming near the surface near the downstream toe, more conservatism in the short path distance will be necessary, for as Terzaghi* has pointed out this condition is conducive to high velocities where the water emerges and therefore to danger of piping.

*Technical Publication No. 215, Am. Inst. Min. & Met. Engrs. p. 31

In masonry dams upward pressure is important because of its tendency to reduce the factor of safety against sliding. Sliding is not so important a factor in the design of masonry dams on earth foundations. The principal danger from upward pressure on a dam on earth foundations is that it will be sufficient to lift up some portion of the dam, breaking a new outlet, opening for the percolation which will give such a short percolation path that the dam will fail. Upward pressure is therefore a factor in the failure of dams from piping.

Upward Pressure

As previously shown, dams may fail from piping by flow along the line of contact of the structure with its foundation or by flow directly through the foundation material. In all dams some flow takes place directly through the foundation material and no doubt the usual condition is that most of the flow occurs in this way. If a structure is designed so as to be safe against piping, the upward pressure under practically all of it will probably depend more upon the conditions of flow through the foundation material than along the line of contact. Even if a pipe did form it would cover only a small area and the foundation masonry would bridge over it. Therefore although it is necessary to give creep along the contact the preeminent place in determining the safety against piping, it may have less bearing in regard to upward pressure.

Under conditions where the foundation material and its distribution is accurately known, it seems probably that the upward pressure to be provided for could be obtained with considerable accuracy by the flow net or electro-hydraulic analogy, although these results would be modified somewhat by the case of creep along horizontal surfaces. The Author hopes in the near future to investigate this further by means of a comparison of observed upward pressures on actual structures and the results obtained by the electrical analogy method. Where the foundation conditions are complicated or unknown, some approximate rule must must be used. Measurement of upward pressure on actual dams and research by model experiment, flow net, and electro-hydraulic analogy methods seem to indicate that the drop in pressure caused by a cutoff is greater in proportion to the creep distance than for the horizontal contacts. Leliavsky*, after a study of the works of Coleman,

*On Percolation Under Aprons of Irrigation Works, p. 46

Forschheimer, Terzaghi and Pavlovsky states, "Thus in calculating the hydraulic gradient the depth of the sheet piling should be multiplied by a coefficient = 2, (sic) whereas there is already sufficient evidence to show that this coefficient should be three or four or more." In other words, the horizontal creep should be weighted three-fourths, one-half or less. Until more definite data is obtained therefore it is suggested that where it is necessary to use an empirical rule, the upward pressure be assumed to vary along the contact line in proportion the weighted creep; ie, that the pressure drop along the vertical or steeply sloping contacts per unit of length be assumed to be three times that along horizontal contacts or slightly sloping contacts. This is believed to be a conservative rule as upward pressure experiments frequently indicate that a weight for horizontal creep of zero would give reasonably close results.

Where sheet piling is so close together that the flow may be considered to follow the short path between the piling rather than the line of creep, the upward pressure along the creep line between the ends of the short path may be considered to vary according to the weighted creep distance between these points.

Since the weighted creep rule gives less slope of the piezometric line for horizontal creeps than the method advocated by Bligh, and since ordinarily the cutoff walls, which constitute the greater part of the vertical creep are located in the upstream portion of the dam, the weighted creep rule usually indicates less upward pressure under the downstream portion of the dam than the Bligh rule. Measurements of upward pressures on existing dams indicate results nearer to the weighted creep method. In other words, for the portion of the dam where the magnitude of the upward pressure is important, the Bligh rule gives results which are too large. This shows why that rule has given satisfactory results, although frequently the thickness of masonry used has been little if any greater than the theory would indicate necessary. The factor of safety therefore has been in the theory and none was therefore necessary in applying it. It would be much more logical therefore to apply a more correct theory with a reasonable factor of safety. It is suggested therefore that a factor of safety of at least 25% be used in determining the thickness of masonry required, to take care of the variations of the pressure from that indicated by the theory.

Special conditions sometimes cause unusual distribution of upward pressure. A dam founded on pervious material underlaid with impervious material and with a cutoff extending nearly to impervious material, will have a lower uplift downstream from the cutoff. A dam with such a foundation where the cross-section of pervious material under the downstream end of the apron is less than that at the upstream edge will have a greater upward pressure over the entire base than one founded on pervious material of great depth. This condition was noted in the upward pressure measurements on the Grand Valley Dam*. A contracting of the space through

*Trans. Am. Soc. C. E. Vol. 93, 1929, p. 1532.

which the water flow beneath the dam acts as a partially closed valve and raises the pressure upstream from it and lowers it downstream.

In computing upward pressure it should be remembered that the conservative assumptions may be different from those for safety from piping. In the latter case the creep under flexible aprons and similar construction downstream from the solid portion of the dam, should be neglected in estimating the security of the dam against piping, as their effect is uncertain and cannot be relied upon. In computing the upward pressure they should be given consideration however, as they may cause greater upward pressure to occur at the downstream edge of the solid portion of

the dam than the other assumption would indicate. The upward pressure experiments on the Pinhook Dam,* for example, showed a considerable up-

*Trans. Am. Soc. C.E., Vol. 93, 1929, p. 1551

ward pressure at the downstream edge of the solid portion of the dam, due no doubt to the flexible apron downstream, although if the ordinary assumption for the design of such structures for safety against piping were used in determining the position of the end of the creep path, there would be no upward pressure at that point.

It is perhaps worthwhile at this point to suggest that the upward pressure be assumed to be applied at the bottom of the foundation as in fact it really is, and has a magnitude equal to the difference in elevation between this point and the piezometric line. The total weight of the masonry (in air) and of any water which may be above it, can then be considered as resisting this pressure. This will eliminate any uncertainty as to whether or not the masonry ought to be considered as submerged, and therefore as having lost weight, and prevent a mistake sometimes made, of measuring the upward pressure by taking the difference between the elevation of the top of the apron and the piezometric line, while still assuming the full weight of the masonry available to resist it. Experiments by Hde B. Parsons* and others have shown the necessity of assuming that the upward pressure acts over the entire area of the base of the dam.

*Transactions, Am. Soc. C.E., Vol 93, 1929, p. 1317

Miscellaneous Causes of Failure

Although a dam may apparently be designed with sufficient length of percolation path, it may still fail by piping, due to causes which the designer may not forsee. Failure may also occur from lack of care in construction. A number of cases in both classes are described in the accounts of dam failures previously given in this paper, but it is believed to be desirable, by way of emphasis, ~~and~~ to assemble this data in convenient form and to group these points together.

Too great emphasis cannot be given to the necessity of having the cutoff walls absolutely tight, tied into the main masonry of the dam and in intimate contact with the foundation material. A single pile missing caused the failure at the Coon Rapids Dam and leakage between the piles and between their tops and the masonry was a large factor in the failure of the Deoha Barrage. Even with interlocking steel sheet piling, rigid inspection is necessary to see that there are no breaks in the interlocking and that their tops are properly imbedded in the masonry of the main structure. The cutoffs must also be tied into the structures in

Designed creep distances can be materially shortened by a blow-up of the base of the dam from upward pressure. This action was probably a material factor in the failure of the Puentes, Narora and Deoha dams. The apron must be heavy enough to resist raising.

Although not connected with piping, it may be worthwhile to mention here several other causes of difficulty with masonry dams on earth foundations which should be avoided. In many streams there is a constant "flow" of solid material composing the river bottom. If the backwater from the dam prevents this material from continuing to come down, while the flow downstream from the dam continues to carry material away, a considerable lowering of the river bed often results. This may cause greatly increased scour at the downstream edge of the dam, and in the case of the Islam Weir was sufficient to cause a failure.

In computing the uplift beneath the apron of dams where a hydraulic jump is formed on the apron, the fact that the downward pressure of the thin stream of water upstream from the jump is less than that below the jump, should be given consideration. The upward pressure on the bottom of the apron is influenced by the depth below the jump and the downward pressure on top of a portion of the apron depends upon the lesser depth above the jump. Failure to provide sufficient weight of apron to overcome this difference in addition to the normal upward pressure has resulted in failure in several cases.

In designing a dam on a porous foundation it is necessary to be sure that the hydraulic jump will be formed on the structure where it is expected. A case has occurred where a dam was placed at the top of a steep section of the river and the tailwater level was so low that the jump occurred below the downstream end of the apron. In a short time it scoured out the riprap and destroyed the structure.

In several structures difficulty has been experienced with aprons formed in layers, due to the tendency of the swiftly flowing water to get between them and "peel off" the upper ones. If possible, aprons should be cast in blocks rather than layers, but if layers are necessary they should be thoroughly bonded together.

Although the bed of a stream may be considerably higher near the banks than in the center, the downstream apron of the dam should be level all the way across. The construction of the dam is usually such that the discharge is spread nearly uniformly over the entire width of the river, and if the apron is higher at the ends the flow over the dam at the sides of the river, after passing off the apron tends to run along the downstream edge of the apron toward the middle of the river. If the slope in this direction is considerable, sever scour

is likely to be set up. This result has been observed at several dams. Even if the downstream apron is level, it is advisable to plan the earth excavation below the dam in such a way that severe currents are not set up along the toe from the ends of the dam toward the main channel.

Conclusions

The ordinary method of analysis of a masonry dam on an earth foundation to secure safety against piping, which is usually ascribed to Bligh, is faulty in that it does not consider the ~~greater~~ lesser probability of creep along vertical and steeply sloping contact surfaces as compared with slightly sloping ones.

The flow net and electrical analogy methods are faulty in neglecting the lesser resistance along the line of contact of a dam with its foundation as compared with that directly through the foundation material.

Piping may occur in two distinct ways, (1) by flow along the line of contact of the structure and its foundation or (2) by flow directly through the foundation material. Flow ordinarily takes place along both of these paths in inverse proportion to their relative resistances.

Considerable light on the probability of failure from the first of these causes can be obtained by experimental methods, but the second must be studies largely by an analysis of the action of actual dams.

From the result of an analysis of the action of more than one hundred dams, it has been found that creep along contact surfaces having slopes with the horizontal of less than 45 degrees should be considered to offer only one-half the resistance to piping as those with slopes of 45 degrees or over.

Analysis on this basis may be called the weighted creep analysis, the creep along surfaces under 45 degree slopes called horizontal creep and that of 45 degrees or over vertical creep. It should be noted that these slopes are the slopes of the surface of contact. The slope of the path taken by the water may be different. The weighted creep-head ratio is the weighted creep divided by the effective head.

The following values are suggested as the safe weighted creep-head ratios for use in the design of major structures:

SUGGESTED WEIGHTED CREEP RATIOS
WEIGHT OF HORIZONTAL CREEP $\frac{1}{2}$

Very fine sand or silt	12.0
Fine sand	10.0
Medium sand	9.0
Coarse sand	7.0
Fine gravel	5.5
Medium gravel	4.5
Coarse gravel including cobbles	3.8
Boulders with some cobbles and gravel	3.4
Soft clay	4.0
Medium clay	3.3
Hard clay	2.5
Very hard clay or hardpan	2.1

These values can only be used for solid masonry cutoffs built directly against the earth or interlocking ~~sheet~~ sheet piling driven so that the interlock is not broken and with the top of the piling satisfactorily imbedded in the masonry of the dam. Competent supervision during construction and efficient maintenance are assumed. If all these do not exist higher values must be used.

For less important structures these ratios may be reduced somewhat down to perhaps 75% of the values given for very minor structures.

Additional factors which should be considered in selecting the proper ratio are (1) the loss of life and property which would result from a failure, (2) the quality of supervision during construction and of maintenance after completion, (3) the loss resulting from percolation under a dam, (4) the difficulty in mending a breach if it occurs, (5) the possibility of the river bed upstream silting up.

Reverse filters, weep holes and drains are an aid to security and weighted creep-head ratios may be reduced up to 10% if they are used. The best form is a weep hole with a reverse filter behind it. For best results vents should be located far enough from the end of the travel path to insure that most of the flow passes through them. Except to reduce upward pressure, they should not be located farther upstream than necessary to accomplish this purpose. Usually the best location, from the standpoint of piping, is just above the downstream cutoff, venting through the cutoff. The position of the hydraulic jump should be considered in locating vents. Great care must be exercised in constructing vents or drains.

The security of dams on pervious foundations underlaid by impervious material will be greatly increased if a cutoff is carried into the impervious material. No satisfactory analysis for this condition has been developed. Dams founded on impervious material underlaid by pervious material are apt to give trouble and should be designed with care.

In all cases care must be exercised that cutoffs are properly tied in at the ends, so that the water will not outflank them, and that there is no short route behind or under the abutments through which a channel may be formed.

In order to prevent failure of dams by percolation directly through the foundation material the short path-head ratios should not be less than three-fourths of those recommended for the weighted creep.

If any two points on the creep line are so close together that the short path between them is less than half the weighted creep distance between them, flow may be considered to take place directly through the material, the length of this travel being given a weight of two. Where more than one short path is possible between the same portions of the creep path, the one which gives the smallest total weighted creep for the structure should be used.

The upward pressure to be used in design may be estimated by assuming that the drop in pressure from headwater to tailwater along the contact line of dam and foundation is proportional to the weighted creep distance. Between sheet pile lines which are so close together that the short path limitations apply the total pressure drop may be computed as proportional to the short path distance with its weight of two and distributed between the two ends of the short path in proportion to the weighted creep between these points.

Miscellaneous causes of difficulty or failure which can be avoided by proper design and construction are clay foundations or aprons which are located where they can dry out, construction on dry sand, control of springs during construction, apron blowups due to insufficient allowance for upward pressure, apron uplift due to the hydraulic jump, scour below the dam due to jump forming off the apron, peeling of apron due to construction in layers, retrogression of the river bed after construction, scour below apron due to it not being level across the stream.

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96 - 1-7-26 - p. 12 - Failure of Dam in Wales Due to Washout under Foundation.

94 - 5-7-25 - p. 762 - Low Head Power Plant Designed for Typical Mid-West River.

93 - 12-18-24 - p. 1002 - Repairing the Laguna Dam on the Colorado.

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82 - 6-12-19 - p. 1166 - Balanced Automatic Sluice Gate for Park Dam.

81 - 9-12-18 - p. 491 - Sixty-One-Foot Hydraulic-Fill Dam Rests on Earth Foundation.

80 - 4-25-18 - p. 831 - Uplift from Hydraulic Jump Neglected in Dams on Porous Foundation.

79 - 8-2-17 - p. 217 - Island Lake Storage Dam Has Immense Log Sluice.

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75 - 5-25-16 - p. 974 - Failure of Diversion Dam on Salt River Project.

75 - 6-8-16 - p. 1070 - Irrigation Headworks Repair and Dam Failure.

75 - 6-8-16 - p. 1106 - Plattsburg Dam Goes Out.

75 - 6-15-16 - p. 1113 - Building Power House and Dam on Sand Foundations.

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71 - 2-5-14 - p. 300 - Construction Work for East St. Louis Flood Protection ((Cahokia Creek Dam).

70 - 12-18-13--p. 1258 - Reconstructing the Foundation of the Elwha River Dam.

69 - 1-9-13 - p. 83 - The Sheet Pile Cutoff on the Port Angeles Dam.

69 - 2-6-13 - p. 266 - Lessons from the Failure of a Weir and Sluices on Porous Foundations.

68 - 12-5-12 - p. 1072 - Foundation Failure of a High Concrete Dam - Port Angeles, Washington.

68 - 12-26-12--p. 1232 - The Accident to the Dam at Port Angeles, Washington.

67 - 4-4-12 - p. 621 - Irrigation on the Royal Murgab Estate - Turkestan, Russia.

65 - 4-13-11 - p. 444 - Weirs on Porous Foundations and with Pervious Floors.

65 - 1-26-11 - p. 109 - Sheet Piles as a Means of Decreasing Permeability of Porous Foundations.

64 - 12-8-10 - p. 611 - Movable Dams on the New York State Barge Canal.

64 - 12-29-10--p. 708 - Dams, Barrages and Weirs on Porous Foundations. (See also Vol. 65, 1-12-11, p. 52.)

62 - 10-14-09--p. 393 - The Failure of the Fergus Falls, Minnesota, City Dam.

62 - 11-4-09 - p. 497 - The Failure of the Dam at Fergus Falls, Minnesota.

61 - 1-7-09 - p. 1 - The Granite Reef Dam and Gate Mechanism.

61 - 4-1-09 - p. 345 - The Undermining of a Reinforced Concrete Dam at Pittsfield, Massachusetts.

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60	10-1-08	p. 366	-	The Granite Reef Diversion Dam of the Salt River Project of the U. S. Reclamation Service, Arizona.
59	2-27-08	p. 213	-	The Construction of the Laguna Dam, Colorado River, Arizona.
59	4-30-08	p. 491	-	The Break in the Hauser Lake Dam, Montana.
58	11-14-07	p. 507	-	The Hauser Lake Steel Dam in the Missouri River near Helena, Montana.
54	10-12-05	p. 374	-	Water Power Plants on the Apple River, Wisconsin.
54		p. 100	-	Pile Foundation for Movable Weir.
53	2-9-05	p. 146	-	The Laguna Dam, Yuma Irrigation Project, California.
53	2-9-05	p. 147	-	Grouted Rubble Core Walls for the Weirs of the Delta Barrage, Egypt.
40	7-7-98	p. 2	-	The New U. S. Government Needle Dam at Louisa, Kentucky, on the Big Sandy River.
36	8-13-96	p. 98	-	Locks and Dams, Great Kanawha River.
36	12-31-96	p. 426	-	Movable Dams, Great Kanawha River.
33	3-14-95	p. 166	-	New Masonry Dam at Lonsdale, Rhode Island.
31	5-24-94		-	The Chenab Weir.

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71	1-2-15	p. 24	-	Hindia Dam on Euphrates River.

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	69	3-28-14	p. 372	- Reconstruction of the Elwha River Dam.
	66	8-31-12	p. 246	- Water Power from the Au Sable River.
	66	11-30-12	p. 600	- Washout of Base of Port Angeles Dam.
	64	10-21-11	p. 481	- A Small Hydroelectric Plant on the Oswegatchie River.
	63	5-20-11	p. 560	- Granite Reef Diversion Dam, Arizona.
	62	10-8-10	p. 396	- The Las Vegas Irrigation Project, New Mexico.
	61	4-9-10	p. 487	- Failure of Canal Headworks, Egypt.
	61	1-1-10	p. 24	- The Failure of a Small Concrete Dam.
	61	6-4-10	p. 726	- Compensation Weirs, Nepean River.
	58	11-21-08	p. 572	- The Hydro-Electric Plant of the Uncas Power Company at Scotland, Connecticut.
	58	12-5-08	p. 645	- A Successful Low Timber Dam for Broad, Sandy River Channels.
	58	12-26-08	p. 728	- A Low Head Hydro-Electric Development at Berrien Springs, Michigan.
	56	7-20-07	p. 75	- The Upper Grays Dam, Consolidated Water Company, Utica, New York.
	56	10-19-07	p. 418	- Three Low Head Hydro-Electric Developments in Michigan.
	56	10-26-07	p. 462	- Three Low Head Hydro-Electric Developments in Michigan.
	52	10-14-05	p. 431	- The Riverdale Plant of the Apple River Power Company.
	51	4-15-05	p. 428	- The Asyut Barrage Across the Nile.

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	44 -	9-8-15 -	p. 193	- Considerations of Sliding, Overturning, Crushing and Blowouts in Dam Design by the Ohio River Board.
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	39 -	3-19-13 -	p. 315	- Unique Construction Methods and Devices Employed at Lock and Dam No. 1, Mississippi River Improvement.
	39 -	5-7-13 -	p. 533	- Methods of Constructing a Hydro-Electric Power House and Dam on Sand Foundations.
	37 -	6-5-12 -	p. 639	- The Design and Methods Employed in Constructing the Cooke Water Power Plant on the Au Sable River, Michigan.
	37 -	6-[illegible]	. 646	- Three Approved Designs for Weir Dams for Irrigation Canals.
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	33 -	2-2-10 -	p. 117	- What Caused the Failure of the Dansville Dam.
	33 -	3-2-10 -	p. 207	- The Dam Failure at Dansville, New York.
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	65 -	June '26 -	p. 269	- A New Type of Spillway.
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DAMS ON CLAY AND MATERIAL

No.	Name	Stream	Location	Head	Creep Dist. Vert.	Creep Dist. Horz.	Weighted Creep Distance	Weighted Creep Ratio	Data	Condition	Material	Remarks
										FAILURES NOW EXISTING		
1	Woodward	Flanders Brook	Hill, N. Y.	30	24	8	28	.93	Fair	In service 4 or 5 years.	Hard material, probably hardpan. May have had sand layers.	
2	Delamarre	Lake Elrian	North Wales	10	3	15	10	1.0	Good	In service more than 14 years.	Hard blue clay containing boulders.	Drying of foundation during low reservoir stage may have caused failure.
3	Pemos Falls	Red River	Minnesota	23	17	28	31	1.4	Fair	In service 11 months.	Hardpan.	Spring under power house reduced creep distance from 2.1 to 1.4.
										NON-FAILURES		
4	Delamarre	Lake Elrian	North Wales	10	13	15	20	2.0	Fair	In service more than 14 years.	Hard blue clay containing boulders.	Portion of dam which did not fail.
5	Marysville	Nashwaak River	New Brunswick	22	24	58	56	2.6	Good	Built about 1921.	Firm, impermeable red clay with at least 50% boulders.	Downstream apron light. If it blows up ratio will be 2.0. Has stood 25' head for short time. Ratio 1.2.
6	Pemos Falls	Red River	Minnesota	22	70	34	67	3.0	Good	In service 11 months.	Hardpan.	Failure under power house caused washing out of this section.
7	Kettle Creek	Kettle Creek	Ontario	20.5	18	46	42	2.0	Good	Built about 1922.	Very hard blue clay.	Downstream apron light. If lifted up ratio would become 1.2.
8	Lake St. Francis	St. Francis River	Quebec	27 27	46 43	46 64	70 87	1.9 2.4	Fair Good	Successful since 1924.	Stiff clay with small boulders and gravel.	Bulkhead section. Spillway section.
9	Hebgen	Milk River	Montana	27	113	191	209	7.7	Good	In successful use since about 1912.	Stiff clay and clay with fine gravel.	
10	Tocco	Au Sable River	Michigan	60	76	239	271	5.5	Good	Built about 1912.	Clay.	
11	Tower Drive	Black Creek	China, N. Y.	30	45	110	100	3.2	Uncertain	Probably in successful use since 1927.	Good clay, fairly tempered with sand, underlain by hardpan.	Cutoff wall goes to fairly deep hardpan. An early Ambursen dam.
12	Wausau	Trib River	Wisconsin	15	57	81	98	6.5	Good	Successful operation since 1923.	Stiff, greasy clay.	
13	Smith Street		Holyoke, Mass.	18	20	17	29	1.6	Good	Built about 1890.	Very compact and unyielding gravelly hardpan.	
14	North Tuolumne, Priest Project		California	4.7		5	3	0.64	Good	Successful since 1917.	Rather impervious red clay with considerably firmly compacted gravel.	
15	Big Dodge	Big Dodge	Montana	6	12	15	21	3.5	Good	Successful since 1922.	Clay.	
16	Bhakra	Sutlej River	Punjab, India	7.6	25	46	54	7.7	Good		Clay.	
17	Red River No. 2	Ouachita River	Louisiana	14.4	63	31	79	5.5	Good	Successful since 1919.	Very dense clay.	
18	Red River No. 3	Ouachita River	Louisiana	6.9	77	34	94	13.6	Good	Successful since 1924.	Hard clay.	Creep ratio given is for normal head. For maximum head it is 4.9.
19	Red River No. 4	Ouachita River	Arkansas	10.5 13.5	41 97	22 34	52 114	5.0 8.5	Good Good	Satisfactory service 11 years for the first and since 1934 for the second.	Clay predominating, some sand.	Dam increased in height 2 feet after 11 years' use.
20	Red Rock No. 2 Rotterdam	Mohawk River	New York	16	63	77	102	6.5	Good	Satisfactory service since 1912. Some leakage stopped by grouting and additional sheet piles.	On clay. (A portion also on clay overlaid by gravel and sand.)	
21	Red Rock No. 4 "Prospectville"	Mohawk River	New York	15	63	77	102	6.8	Good	Satisfactory since 1912.	On clay. (A portion also on clay overlaid with thin layer of gravel.)	
22	Hamilton	Manistee River	Michigan	50 50	199 205	165 225	282 318	5.6 6.4	Fair Fair	Completed about 1918.	Hard clay.	Power house.
23	Otsego County Line	Manistee River	Michigan	65	144	351	335	5.2	Good		Clay.	
24	Lake Washington		Seattle, Washington	24	89	191	185	7.7	Good	Satisfactory service since 1915.	Hard clay. (Piles driven to refusal at 8'.)	Long thin apron. 62% of creep under apron of thirty or less.
25	Mexico		Mexico	17.7	51	97	56	3.4	Good	Satisfactory service since 1934.	Compact clay.	

DAMS ON CLAY AND HARDPAN (Continued)

TABLE NO. 2.

No.	Name	Head	Plain Dist.	Creep Ratio	
					DATA FROM DAMS WHICH FAILED
1	Woodward Dam	30	32	1.1	Engineering News-Record, Vol. 80, No. 23, June 6, 1918, p. 1104.
2	Dolgarrog Dam	10	18	1.8	Engineering News-Record, Vol. 96, No. 1, Jan. 7, 1926, p. 12; Engineering, Vol. 122, Oct. 15, 1926, p. 484.
3	Fergus Falls Dam				Engineering News, Vol. 62, Oct. 14, 1909, p. 393 and Vol. 62, Nov. 4, 1909, p. 497.
4					
5					
6					
7					
8					
9					
10					
					DAMS OF WHICH NO FAILURE IS RECORDED
11	Dolgarrog Dam	10	28	2.8	Engineering News-Record, Vol. 96, No. 1, Jan. 7, 1926, p. 12; see No. 2 above.
12	Marysville Dam	22	92	4.2	Canadian Engineer, Vol. 38, June 3, 1920, p. 517.
13	Fergus Falls Dam	23	104	4.5	Engineering News, Vol. 62, Oct. 14, 1909, p. 393, and Vol. 62, Nov. 4, 1909, p. 497.
14	Battle Creek Dam	20.5	66	3.2	Canadian Engineer, Vol. 42, No. 18, May 2, 1922, p. 444.
15	Lake St. Francis				
	Bulkhead section	37	94	2.5	Engineering News, Vol. 74, No. 22, Nov. 25, 1915, p. 1032; O. Lefebvre, Chief Engineer, Quebec Streams Commission.
	Spillway	37	121	3.3	
16	Vandalia Dam	27	27	3.04	Milk River Project History, 1913, Vol. 4 (pocket in back).
17	Cooke Dam	40	365	9.1	Engineering Record, Vol. 66, No. 9, Aug. 31, 1912, p. 246, and Engineering and Contracting, Vol. 37, No. 23, June 5, 1912, p. 625.
18	Upper Grays Dam	30	155	5.2	Engineering Record, Vol. 56, No. 3, July 20, 1907, p. 75.
19	Shawano Dam	15	138	9.2	D. W. Mead.
20	Whiting Street	18	361	2.0	Engineering News, Sept. 6, 1890, p. 204.
21	North Diversion	4.7	5.0	1.1	Bureau of Reclamation Files. (Letter from Supt., Jan. 3, 1920.)
22	Big Coulee Intake	6	28	4.7	Printed specification No. 395.
23	Sidhani Weir	7.5	81	10.8	
24	Dam No. 2 Ouachita	14.4	94	6.5	U. S. War Department.
25	Dam No. 5 Ouachita	6.9	111	16.1	U. S. War Department.
26	Dam No. 8 Ouachita				
	Original condition	10.5	63	6.0	U. S. War Department.
	Revised condition	13.5	131	9.7	
27	Dam No. 5 Mohawk	15	140	9.3	Wm. M. Acheson.
28	Dam No. 6 Mohawk	15	140	9.3	Wm. M. Acheson.
29	Junction Dam				
	Power house section	50	364	7.3	Engineering News-Record, Vol. 81, Sept. 12, 1918, p. 491.
	Spillway section	50	430	8.6	
30	Hodenphyl (County Line)	65	525	8.1	William W. Tefft.
31	Lake Washington	24	280	11.7	A. L. Alin.
32	Acatlan Dam	17.7	141	8.0	W. H. Fraser.

TABLE OF GRAVEL, BOULDERS AND APRONS

TABLE NO. 3.

	Name	River	Location	Feet	Creep Hist. Vert.	Creep Hist. Horz.	Estimated Creep Ratio Hist.	Estimated Creep Ratio Unit	Reliability of Data	Results	Foundation Material	Remarks
									DAMS WITHOUT FAILURE			
	[illegible]	Swietlactin River	Spain	148	44	197	123	.56	Fair	Failed immediately.	Sand and gravel classification somewhat uncertain.	No failure at creep-head ratio 1.4. Apron blew up reducing ratio to .56.
	Port Angeles	Elwha River	Washington	79	72	97	121	1.3	Fair	Held 9 days.	Gravel, small cut or coarse sand.	Head may have been 104 feet.
	Littleville	West Branch	Massachusetts	22	29	47	53	1.6	Good	Failed on first filling.	Layer of gravel and fine material, underlain by impervious layer, underlain in turn by pervious fine material and boulders.	Failed probably through seepage from lower layer causing slumpout. Cutoffs extended into impervious layer.
	Twin Buttes	Mississippi River	Minnesota	48					Good	Failed soon after head applied.	Sand and boulders.	Creep distance cannot be computed.
	Jamesburg		New York	22	49	38	68	3.1	Uncertain	Held probably 2 months.	Boulders, fine gravel and sand, not more than 1% clay. See sieve analysis, Fig. 1.	4 sections having a creep ratio of 2.4 short-path ratio 2.6 not failured.
									DAMS OF WHICH NO FAILURE IS REPORTED			
1	Horse Creek Diversion	Horse Creek	Wyoming	6.4	18	17	27	4.2	Good	Satisfactory since 1923.	Coarse sand and gravel to 3-inch maximum size.	
2	Vecilla	Rio Grande River	New Mexico	9.7	24	40	44	4.1	Good	Satisfactory since 1917.	Medium sand with fair sprinkling of gravel.	
3	Dam No. 6	Dumentia River	Arkansas	6.2	47	22	44	10.3	Good	Satisfactory service 6 years first time, and since 1923 second time.	Sand and gravel with much fine sand.	After 6 years, height of dam increased 3.5 feet.
4	Dam No. 42	Ohio River	Indiana-Kentucky	9.0	87	29	102	11.6	Good	Satisfactory since 1927.	Sand and gravel, much small gravel.	Navigable pass.
5	Dam No. 46	Ohio River	Indiana-Kentucky	9.0	97	35	101	11.6	Good	Satisfactory since 1926.	Sand and small gravel.	Navigable pass.
6	Dam No. 51	Ohio River	Illinois-Kentucky	8.0	87	35	102	13.1	Good	Satisfactory since 1928.	Sand and gravel, much of it fine below part of fine sand.	Navigable pass.
7	Dam No. 53	Ohio River	Ohio-Kentucky	6.4	47	36	61	10.1	Good	Satisfactory since 1918.	Sand and gravel.	Navigable pass.
8	Dam No. 56	Ohio River	Ohio-Kentucky	7.9 7.9 7.9	84 89 80	35 80 20	102 129 90	12.9 16.3 11.4	Good Good Good	Satisfactory since 1925. Satisfactory since 1925. Satisfactory since 1925	Sand and gravel. Sand and gravel. Sand and gravel.	Navigable pass. Lower trap. Wicket only.
19	Dam No. 62	Ohio River	Indiana-Kentucky	9.0	88	40	104	11.8	Good	Satisfactory since 1928.	Sand and gravel.	Navigable pass.
20	Dam No. 52	Ohio River	Illinois-Kentucky	12.0	85	40	102	8.9	Good	Satisfactory since 1929.	Sand and gravel, some clay.	Navigable pass.
21	Dam No. 50	Ohio River	Illinois-Kentucky	13.4	87	35	105	7.9	Good	Satisfactory since 1929.	Sand and gravel, some clay.	Navigable pass.
22	Hinkson	Nojaoseta River	Iowa	25	68	81.5	99	4.0	Good	Built in 1924.	Sand and gravel; see sieve analysis, Fig. 1.	Power house slope concrete apron not included.
23	Richmond		Indiana	5	11	16	19	3.8	Good	Probably in service about 7 years.	Gravel and sand.	Failed during flood, not by underseepage.
24	Lower Intake	Salt River	Arizona	6.5	43	18	52	8.0	Good	In service about 10 years.	Gravel and sand.	Failed from erosion of apron.
25	Island Park	Miami River	Ohio	7.5	37	19	51	6.8	Good	Satisfactory since 1917.	Gravel and sand; see sieve analysis, Fig. 1.	
26	Puente	Guadalantin River	Spain	75	51	283	195	2.6				Height reduced season previous to failure.
27	Willow Creek	Willow Creek	Oregon	5.0	14	14	31	6.2	Good	Satisfactory since 1928.	Sand, gravel and earth.	
28	Sardarab	Aras River	Russia	16.7	57	108	111	6.6	Good	Completed about 1930.	Coarse sand and gravel.	
29	Cherrecassy	Arpachay River	Armenia	13.3	35	59	64	4.8	Fair	Completed about 1930.	Coarse sand and gravel.	
30	Limmerg	Yssel River	Holland	12	84	131	100	12.5	Good	Completed about 1932.	Coarse gravel and sand.	
31	Kerr Island	Allegheny River	Pennsylvania	7	45	48	69	9.9	Good		Gravel.	
32	Firowita	Moldau River	Czechoslovakia	12.8	80	33	77	6.0	Good		Gravel.	
33	Vacilles	Maule	Chile	7.1	31	31	46	6.1	Good	Built about 1923.	Gravel.	
34	Fremont Weir	Sacramento River	California	3.0	21	36	39	10.9	Good		Gravel.	For flood control, in use only at intervals.
35	Amsterdam (No. 7)	Mohawk River	New York	12	32	63	64	5.3	Fair	Satisfactory since 1912.	Gravel.	Reverse filter used.
36	Fort (No. 13)	Mohawk River	New York	8	34	52	51	6.4	Fair	Satisfactory since 1912.	Gravel.	Reverse filter used.
37	Dam No. 5	Ohio River	Pennsylvania	7.8	43	32	59	7.9	Good	Satisfactory since 1904.	Gravel.	
38	Dam No. 15	Ohio River	Ohio-West Virginia	8.4	49	35	67	7.9	Good	Satisfactory since 1915.	Gravel.	
39	Vandalay Underslinge	Madaya River	Burma	11.2	10	78	49	4.3	Good		Gravel and shingle.	
40	Grand Valley	Colorado River	Colorado	10.25	52	85	74	5.2	Good	Satisfactory since 1915.	Coarse gravel and cobbles with only small percentage of sand.	
41	Milner Diversion	Snake River	Idaho	33	48	48	72	2.2	Good	Satisfactory since 1906.	Very compact gravel and cobbles.	
42	Sherman Island	Hudson River	New York	53	182	181	263	5.0	Good	Built about 1923.	Sand and boulders.	
43	---	Susquehanna River	New York	23	45	37	64	2.8	Fair	Constructed about 1911.	Sand and glacial boulders.	
44	[illegible]	Rio Grande River	New Mexico	11	89	73	96	8.7	Good	Satisfactory since 1919.	From sand with small per cent of gravel at surface to heavy boulders with some sand and gravel at bottom of sheet piles.	
45	Farmer Diversion	Malheur River	Oregon	12	29	57	58	4.8	Good	Satisfactory since 1930.	Fairly well graded sand and gravel with a few boulders.	
46	Twin City	Mississippi River	Minnesota	30.8	57	140	127	4.2	Good	Probably satisfactory since 1913.	Sand, gravel and broken limestone slabs.	
47	St. Mary's Intake	(San River Project)	Montana	5.5	20	39	39	7.1			Sand, gravel and boulders.	
48	Dam No. 44	Ohio River	Indiana-Kentucky	9.0	47	25	106	11.7	Good	Built 1929.	Sand, gravel and boulders.	
49	Granite Reef	Salt River	Arizona	20	45	32	61	3.1	Good	Satisfactory since 1909.	Gravel and boulders.	Seepage 2 to 3 second-feet from 300-feet length at first. Silted up.
50	[illegible] Island	Housatonic River	Connecticut	24.4 24.4	83 82	154 102	180 133	6.2 5.5	Good Good		Gravel and small boulders.	Railway section. Sluice gate section, 20' concrete block apron not included in travel distance.
51	Weber-Provo Diversion	Weber River	Utah	4.5	26	36	44	9.4	Good	Constructed about 1930.	Gravel and boulders.	
52	Strawberry	Spanish Fork River	Utah	18	42	62	74	4.9	Good	Satisfactory since 1908.	Gravel and boulders.	
53	Greeley Intake	Cache la Poudre	Colorado	8	34	50	59	11.2	Good	Satisfactory since 1917.	Sand, gravel and boulders.	
54	Arlesmont	Oise	France	56	95	171	180	3.2	Fair	Built about 1902.	Gravel.	
55	Wheelwright Company		West Dudley, Mass.	7.6	64	19	74	9.7	Good	Satisfactory since 1919.	Gravel with occasional boulders.	Some sheet piling not driven to depth used in this data. Effect of vent neglected.
56	Radis		Germany	9.7	39	46	62	6.4	Good		Fine gravel.	

Dams on Gravel, Cobbles, and Boulders (Continued). Table No. 4.

No.	Name	Head	Plain Method Dist.	Plain Method Ratio	Short Path Dist.	Short Path Ratio	
							Data from Dams Which Failed.
1	Quanten Dam	143	200	1.4	195	1.4	Design and Construction of Dams - Wegmann, 6th Ed., p. 57, Plate XXIV; Reservoirs for Irrigation, Water Power, and Domestic Water Supply - Schuyler, 1st Ed., p. 253; Control of Water - Parker, p. 389.
2	Fort Angeles Dam	79	169	2.1	150	1.9	Engineering News, Vol. 68, No. 23, Dec. 5, 1912, p. 1072; Vol. 68, No. 26, Dec. 26, 1912, p. 1232; Vol. 69, No. 2, Jan. 9, 1913, p. 83; Vol. 70, No. 26, Dec. 18, 1913, p. 1258; Engineering Record, Vol. 66, No. 22, Nov. 30, 1912, p. 600; Vol. 69, No. 13, Mar. 28, 1914, p. 372.
3	Pittsfield Dam	32	76	2.3	63	1.9	Engineering News, Vol. 61, No. 13, April 1, 1909, p. 345; S. W. Stewart.
4	Corn Rapids Dam	48					Engineering News-Record, Vol. 81, July 25, 1918, p. 186; Journal of the Western Society of Engineers, Vol. 19, 1914, p. 979.
5	Plattsburg Dam	22	87	4.0	70	3.2	Engineering News, Vol. 75, Jan. 20, 1916, p. 106; Vol. 75, June 8, 1916, p. 1106; S. W. Stewart.
							Dams of Which No Failure is Recorded.
11	Horse Creek Div. Dam.	6.4	36	5.6	29	4.5	Drawing 20-C-168.
12	Perilla Dam	8.67	67	7.7	51	5.9	Drawing 22-37(1) and correspondence.
13	Dam No. 6 Guachita	6.2	80	12.9	60	9.7	U. S. War Department.
14	Dam No. 45 Ohio	9	122	13.6	101	11.2	U. S. War Department.
15	Dam No. 46 Ohio	9	122	13.6	101	11.2	U. S. War Department.
16	Dam No. 51 Ohio	8	122	15.2	101	12.6	U. S. War Department.
17	Dam No. 35 Ohio	6.4	82	12.8	67	10.5	U. S. War Department.
18	Dam No. 36 Ohio	7.9	119	15	98	12.4	U. S. War Department. (1) Navigable Pass.
			169	21.4	140	17.7	U. S. War Department. (2) Bear Trap.
			100	12.6	85	10.8	U. S. War Department. (3) Fixed Weir.
19	Dam No. 42 Ohio	9	126	14	103	11.4	U. S. War Department.
20	Dam No. 52 Ohio	12	125	10.4	102	8.5	U. S. War Department.
21	Dam No. 53 Ohio	13.4	122	9.1	101	7.5	U. S. War Department.
22	Kinnock Dam	25	130	5.2	92	3.7	Engineering News-Record, Vol. 94, No. 19, May 7, 1925, p. 762; Trans., A. S. C. E., Vol. 93, 1929, p. 1551.
23	Richmond Dam	5	27	5.4	20	4.0	Engineering Contracting, Vol. 25, No. 23, June 6, 1906, p. 155.
24	Power Intake	6.5	61	9.4	41	6.3	Engineering News, Vol. 75, No. 21, May 25, 1916, p. 974.
25	Island Park Dam	7.5	66	8.8	45	6.0	Engineering News-Record, Vol. 84, No. 21, May 20, 1920, p. 1014.
26	Quenten Dam	75	339	4.5	334	4.6	Design and Construction of Dams - Wegmann, 6th Ed., p. 57, Plate XXIV. See No. 1 above.
27	Miller Creek	5	38	7.6	25	5.0	Drawing 12-P-297 and H. S. Lieurance.
28	Kardorab Dam	16.7	165	9.9	115	6.8	Irrigation Herald of Middle Asia Reclamation Bureau, Nov.-Dec., 1928, p. 95.
29	Cherreskaky Dam	13.3	93	7.0	68	5.4	Irrigation Herald of Russian Reclamation Service, Jan., 1928, p. 29.
30	Limbourg Dam	12	215	17.9	165	13.8	Le Genie Civil, Vol. 81, No. 2, July 8, 1922.
31	Herr Island Dam	7	52	12.3	74	10.6	Improvement of Rivers - Thomas and Watt, 2d Ed., p. 657.
32	Mirowitz Bridge	12.8	93	7.3	73	5.7	Improvement of Rivers - Thomas and Watt, 2d Ed., p. 629.
33	Hamilton Dam	7.5	62	8.3	45	6.0	Engineering News-Record, Vol. 90, May 24, 1923, p. 926.
34	Fremont Weir	3.5	56	16.0	43	12.3	U. S. War Department.
35	Dam No. 7 Mohawk	12	95	7.9	67	5.6	Engineering News, Vol. 64, Dec. 8, 1910, p. 617; Improvement of Rivers - Thomas and Watt, 2d Ed., p. 693; Book of Plans, New York State
36	Dam No. 13 Mohawk	8	76	9.5	59	7.4	Barge Canal, Plates Nos. 1 and 55.
37	Dam No. 5 Ohio	7.5	75	10.0	62	8.3	U. S. War Department.
38	Dam No. 10 Ohio	8.4	84	10.0	69	8.2	U. S. War Department.
39	Mandalay Undersluices	11.3	88	7.8	84	7.4	Irrigation Works of India - Buckley, 2d Ed., p. 121.
40	Grand Valley Dam	18.25	137	7.5	100	5.5	Transactions, A. S. C. E., Vol. 93, 1922, p. 1529.
41	Intake Dam Boise	33	96	2.9	92	2.8	Specifications No. 68.
42	Sherman Island	57	247	4.3	242	4.6	Transactions, A. S. C. E., Vol. 88, 1925, p. 1257.
43	[illegible]	2.5	80	3.0	60	2.6	Engineering Record, Vol. 64, No. 17, Oct. 21, 1911, p. 481.
44	Verona Dam	11	132	12	86	7.8	Transactions, A. S. C. E., Vol. 93, p. 1535 - U. S. B. R. Drawing 23-3-26.
45	Harper Diversion	12	86	7.2	69	5.8	New Reclamation Era, March, 1930, p. 40; Engineering News-Record, Vol. 105, No. 26, Dec. 25, 1930, p. 1009.
46	Twin City Dam	30.5	197	6.5	165	5.4	Engineering & Contracting, Vol. 39, No. 12, Mar. 19, 1913, p. 315 - U. S. District Engineer.
47	St. Mary's Canal Intake Dam	5.5	57	10.4	43	7.8	Drawing No. 6, Specification No. 255.
48	Dam No. 44 Ohio	9	122	13.5	101	11.2	U. S. War Department.
49	Granite Reef Dam	20	77	3.8	58	2.9	Engineering News, Vol. 60, Oct. 1, 1908, p. 366; Engineering News-Record, Vol. 85, August 19, 1920, p. 355.
50	Scotland Dam						Engineering Record, Vol. 58, No. 21, Nov. 21, 1908, p. 572.
51	Weber-Provo Diversion Dam	8.5	64	7.5	50	5.9	Specification No. 501.
52	Intake Dam, Strawberry Project	15	106	7	84	5.6	Drawing No. 27-*27(b).
53	Greeley Intake	5	84	16.8	64	12.8	City Engineer, Greeley, Colorado.
54	Avignonet Dam	56	266	4.8	204	3.6	Design and Construction of Dams - Wegmann, p. 74.
55	Burrows Paper Co.	7.6	83	10.9	66	8.7	John H. Gregory; and Waterworks Handbook, 3d Ed., p. 156.
56	Madin Dam	9.7	85	8.8	77	7.9	Schweizerische Bauzeitung, Vol. 89, p. 248.

DAMS ON COARSE SAND

TABLE NO. 5.

No.	Name	Stream	Location	Head	Creep Dist. Vert.	Creep Dist. Horz.	Weighted Creep $\frac{1}{2}$ Dist.	Weighted Creep $\frac{1}{2}$ Ratio	Reliability of Data	Results	Foundation Material	Remarks
1	Wiederborn	Buddow	Germany	23 23	49 76	89 89	95 121	4.0 5.3	Good		Sharp sand underlain by fine sand.	Probable value. Possible value.
12	Upper Coleroon	Coleroon River	India	5.3	24	44	46	8.7	Fair	Stood more than 50 years.	Pure, coarse sand.	Reconstructed condition.
	Nira	Mahanuddee	India	12	40	69	75	6.2	Uncertain			
13	Riverdale	Apple River	Wisconsin	14	39	62	70	2.9	Good	Held at least 5 years.	Coarse sand.	
14	Sirhind	Nari River	Punjab, India	7.5	49	53	76	10.1	Good		Torpedo sand.	
	Godavery	Godavery River	Madras, India	10.8	48	52	74	6.9	Uncertain	Held at least 20 years.	Coarse sand.	
16	Fort Irwin U. Dam	Wisconsin River	Wisconsin	32 29 32	130 100 160	68 24 141	164 112 231	5.1 3.9 7.2	Good Good Good	Satisfactory since 1912.	Coarse sand. Pure coarse sand; see mechanical analysis, Fig. 1.	Coefficients may be lower. Dam vents open. Dam vents closed. Power house.
17	Ware Shoals	Oconee River	Florida	11.5	159	40	179	15.6	Good			
18	Tempe	Gila River	Arizona	10.5	69	102	119	11.3	Good	Satisfactory since 1922.	Stratified sand from very fine to very coarse.	
19	Dam No. 1	Big Sunflower River	Mississippi	16.9	111		111	6.6	Good	Satisfactory since 1917.	Grading from silt and fine sand at surface to coarse sand at bottom of sheet piling.	
20	Tochiti	Rio Grande	New Mexico	8 8	17 8	80 80	57 48	7.1 6.0	Good	Under construction.	Sand with a little clay, gravel and clayey sand. Coarse sand and large gravel.	Dam section. Scouring sluice.

TABLE NO. 6

DAMS ON COARSE SAND (Continued)

No.	Name	Head	Plain Method Dist.	Plain Method Ratio	Short Path Dist.	Short Path Ratio	Data from Investigation Failures
9	Fleiersorn						
	(a) Probable	23	138	6.0	117	5.1	Wasserkraft und Wasserwirtschaft, March 15, 1930, page 81.
	(b) Possible	23	165	7.2	128	5.6	
							Dams with No Failure Recorded
11	Upper Coleroon	5.3	69	12.8	56	10.6	Irrigation Works of India - Buckley, 2nd Ed., p. 149. Control of Water - Parker, p. 672. Practical Design of Irrigation Works - Bligh, 2nd Ed., p. 178.
12	Tirra Weir	12	109	9.1	100	8.3	Practical Design of Irrigation Works - Bligh, 2nd Ed., p. 175. Irrigation Works of India - Buckley, 2nd Ed., p. 141.
13	Riverside Dam	24	101	4.2	76	3.2	Engineering News, Vol. 54, Oct. 12, 1905, p. 374. Engineering Record, Vol. 52, Oct. 14, 1905, p. 431.
14	Sidhnai Weir	7.0	102	13.6	74	9.9	Irrigation Works of India - Buckley, 2nd Ed., p. 138. Practical Design of Irrigation Works - Bligh, 2nd Ed., p. 195.
15	Godavery Weir	10.0	100	9.3	77	7.1	Irrigation in India - Wilson, W. S. P. #87, p. 116. Irrigation Works of India - Buckley, 2nd Ed., p. 141. Practical Design of Irrigation Works - Bligh, 2nd Ed., p. 197.
16	Prairie Du Sac (Power house)						Control of Water - Parker, p. 675. Engineering News, Vol. 75, June 15, 1916, p. 1113. Engineering and Contracting, Vol. 39, May 7, 1913, p. 523.- D. W. Reed.
	Dam - Total creep	32	301	9.4	216	6.8	
	Dam - Creep to inside of dam	32	198	6.2	149	4.7	
		29	124	4.3	104	3.6	
17	Ross Bluff Dam	11.[illegible]	199	17.2	125	10.9	U. S. War Department. C. C. Esher.
18	Florence Dam	10.[illegible]	170	16.2	112	10.7	Engineering News Record, Vol. 89, No. 20, Nov. 16, 1922, p. 824.
19	Dam No. 1, Big Sunflower River	16.9	111	6.3	106	6.2	U. S. War Department.
20	Cocnitt Diversion						
	(1) Dam	8	97	12.1	83	10.4	J. L. Burkholder.
	(2) Scouring sluice	8	88	11.1	83	10.4	

DAMS ON SAND, FINE SAND AND SILT

TABLE NO. 7.

No.	Dam	Stream	Location	Head	Creep Dist. Vert.	Creep Dist. Horz.	Modified Creep Dist.	Modified Creep Ratio	Reliability of data	Results	Foundation Material	Remarks
									DAMS WHICH FAILED			
1	Barora	Jamuna	India	12		90	70	5.6	Good	Failed after 20 years.	Fine sand.	5.6 probable value, certainly not over 7.2, and possibly as low as 3.9. Failure probably not caused by piping.
2	Khanki (Lower Chenab)	Chenab River	Punjab, India	11	15	99 54	94 45	7.2 3.5	Fair	Failed after 2 years.	Fine sand.	Failure probably not caused by piping. Head may have been 14 feet.
3	Jarpua Chariti	Hindon	Pazar	37	29	115	97	1.6	Good	Failed in less than 1 year.	Sand.	
4	Deoha	Deoha	India	12	23	191	112	9.4	Good		Fine sand.	
5	Kitsna Eye Wash Weir	Rohilkhand	India	8.0	22	54	40	5.0	Uncertain	Immediate failure.	Dry sand.	
9	Bulli Dam	Rohilkhand	India						Uncertain	Failed after 10 years.	Fine sand.	
10	Bulli Eye Wash	Rohilkhand	India						Uncertain	Failed after 3 years.	Fine sand.	
									DAMS OF WHICH NO FAILURE IS RECORDED			
21	Rockwood Heading	Colorado River	California	4.1	56	42	79	17.5	Fair	Built about 1912.	Colorado River silt, very fine.	Maximum head 11 feet.
22	Sultan Wand	Purnah River	Pakistan	24	189	207	271	11.2	Uncertain	Satisfactory since about 1910.	Probably founded on loess.	
23	Sifta	Nile River	Egypt	13	34	156	152	10.7	Good		Layer of clay below most of it but running sand and sand below part.	Reversed filter used.
24	Hindia	Euphrates	Mesopotamia	18.4	63	292	215	12.1	Good	Built about 1915.	Fairly hard silt, force engineer to stand vertically when excavated in the dry.	Lower value assumes path to end at filter section in center of apron.
15	Dam No. 48	Ohio River	Indiana-Kentucky	9	54	35	102	11.3	Good	Built in 1922.	Very fine sand.	Lower value assumes path to end at filter section in center of apron.
26	Assuan	Nile River	Egypt	8.2	53	138	122	14.9	Good	Built about 1904.	Water-laid fine sand.	Has reverse filter.
27	Tema (Hema)	Nile River	Egypt	8.2	52	139	122	14.9	Good	Built about 1927.	Fine sand.	
28	Jamrao	--- ---	Sind, India	8	42	111	98	12.2			Fine sand.	
29	Barora	Ganges River	India	13	54	142	130	10.0	Good		Fine sand.	As repaired after failure.
30	Khanki (Lower Chenab)	Chenab River	Punjab, India	13 13	44 75	160 158	128 143	9.8 11.0	Good Good		Fine sand. Fine sand.	Weir as repaired after failure; head may be 14 ft. Underslices: head may be 14 feet.
31	Marala	Chenab River	Punjab, India	12.6	121	121	191	15.2	Good		Fine sand.	
32	Lower Jhelum	Jhelum River	India	10. 10	87 92	196 134	154 129	15.4 15.9	Uncertain Uncertain		Fine sand.	Underslices. Weir.
33	--- ---	Middle Loup River	Nebraska	10	134	13	141	14.1	Fair	Built about 1916.	Sand, probably fine.	
34	Iron Mountain	Menominee River	Michigan	30	115	150	198	6.3	Good	Satisfactory since 1926.	Fine sand.	
35	Dam No. 51	Ohio River	Illinois-Kentucky	8	87	36	105	11.7	Good	Completed 1929.	Part on fine sand. (Part on sand and gravel.)	
36	Dam No. 47	Ohio River	Indiana-Kentucky	9	87	76	105	12.1	Good	Completed 1927.	Sand predominating, very fine to medium. A little gravel, mostly fine.	
37	Nag Hammadi	Nile River	Egypt	14.7	72	196	169	11.6	Good	Built about 1927.	Fine sand predominating, coarse sand and pebbles.	
38	Mendota Headgate	San Joaquin River	California	10	40	100	92	9.0	Uncertain	Stood 18 years.	Drifted sand.	Failed, probably due to escape of sand through hole in structure.
39	--- ---	Kapan River	New South Wales	6	28	42	59	9.8	Good	Satisfactory at least 7 years.	Sand.	
40	Sandy Lake	--- ---	Minnesota	12	129	115	187	15.6	Fair	Satisfactory since 1898.	Sand.	
41	Verset	Ems River	Germany	8.2	76	89	92	10.5	Good		Sand.	
42	Rotenstovsnie	Don River	Russia	11.2	90	156	148	13.2	Fair	Completed 1921.	Sand.	
43	Lloyd		India	18.3	114	190	209	11.3	Good	Constructed about 1900.	Sand and silt.	
44	Islam Weir	Sutlej	Punjab	18	79	175	147	9.3		Failed in 8 years.	Sand.	Failure not due to piping.
45	Su Chuang (Regulator) (Sluice)	Pei Ho	China	11.5 13.1	83 92	91 115	129 140	11.1 10.7	Good	Completed about 1924.	Loess and sand.	
46	Bonnet Carre	Mississippi	Louisiana	14.5	143	103	200	13.8	Good	Not used yet.	Loam, silt and clay.	Used only in floods.

Dams on Sand, Fine Sand, and Silt (Continued).

Table No. 8.

No.	Name	Sand	Plain Method Dist.	Plain Method Ratio	Short Path Dist.	Short Path Ratio	
							Data from Dams which Failed
1	Narora Weir						Irrigation Works of India - Buckley, 24 Ed., p. 162; Practical Design of Irrigation Works - Hligh, 24 Ed., p. 169, 34 Ed. preface, pp. 152, 156 and 185; Dams and Weirs - Hligh, p. 167;
	[Probable]	18	98	7.3	80	6.2	Control of Water - Parker, p. 488; Unpublished Data - W. F. Griffith, United Provinces Public Works Dept., 1931 [Notes on Failure of Narora Weir in 1930]; United Provinces Public Works
	[Not over]	15	143	11.0	113	8.7	Dept., 1932 [Accident to Narora Weir, Narora Division, Lower Ganges Canal].
	[Possible]	13	70	5.4	67	5.2	
2	Khanki Weir	12	119	9.9	110	9.2	Irrigation Works of India - Buckley, 24 Ed., 169; Control of Water - Parker, pp. 676 and 685; Practical Design of Irrigation Works - Hligh, 24 Ed., p. 179, 34 Ed. preface, pp. 162 and 193;
	[Lower Chenab]						Dams and Weirs - Hligh, p. 170.
3	Corpus Christi	37	154	4.2	145.5	3.9	Engineering News-Record, Vol. 105, Oct. 2, 1930, p. 520, Nov. 27, 1930, p. 861, Dec. 4, 1930, p. 897, Dec. 18, 1930, p. 974; Vol. 106, Jan. 1, 1931, p. 40, Jan. 22, 1931, p. 183,
4	Dooba Barrage	12	208	17.0	183	15.2	Feb. 12, 1931, p. 286.
5							Note on Dooba Barrage, Dept. of Public Works, United Provinces, India.
6							
7							
8	Kitcha Sye Wash Weir	8.0	58	7.2	47	5.9	Reports on Damage Done to the Kitcha Sye Wash, Rohilkhand Canals Division, Dept. of Public Works, United Provinces, India.
9	Kulli Bar			14-16			Mr. Griffith.
10	Kulli Sye Wash			14-16			Mr. Griffith.
							Dams of which No Failure Is Recorded
11	Rockwood Heading	4.5	101	22.4	76	16.9	Transactions, A. S. C. E., Vol. 97, 1922, p. 262, W. J. Gault.
12	Sultan Wad Dam	24	276	11.6	248	10.3	Engineering News, Vol. 67, April 4, 1912, p. 621.
13	Elfin Barrage	13	211	16.3	160	12.3	Proceedings, Institution of Civil Engineers, Vol. 156, 1903, p. 322; Irrigation Works of India - Buckley, 24 Ed., p. 174.
14	Hindia Barrage	16.4	341	22.0	309	18.9	Dams and Weirs - Hligh, p. 194; Engineering Record, Vol. 71, No. 1, Jan. 2, 1915, p. 24.
15	Dam No. 48, Ohio	9	119	13.2	98	10.9	U. S. War Department.
16	Tarbell Barrage	8.2	191	23.3	148	18.0	Dams and Weirs - Hligh, p. 193; Proceedings, Institution of Civil Engineers, Vol. 158, 1904, p. 26; Engineering Record, Vol. 81, No. 15, 1915, p. 428; Engineering Works of India -
17	Isma Barrage	8.2	191	23.3	149	18.2	Buckley, 24 Ed., p. 179.
18	Jamrao Weir	8	155	19.1	128	16.0	The Engineer, Vol. 144, Sept. 2, 1927, p. 251.
19	Narora Weir	13	201	15.4	168	12.9	Practical Design of Irrigation Works - Hligh, 24 Ed., p. 181.
20	Sadai						For source of data, see No. 1 above.
	[Weir]	17	274	16.1	182	14.5	Irrigation Works of India - Buckley, 24 Ed., p. 172; Control of Water - Parker, p. 696; Practical Design of Irrigation Works - Hligh, 24 Ed., p. 181; Dams and Weirs - Hligh, p. 155.
	[Underslung]	13	210	16.1	168	12.9	
21	Verala	12.5	262	20.2	186	14.9	Dams and Weirs - Hligh, p. 172.
22	Lower Chenab [Khanikhoma]	10	212	21.2	216	21.6	Irrigation Works of India - Buckley, 24 Ed., pp. 171 to 173; Practical Design of Irrigation Works - Hligh, 24 Ed., pp. 190 and 218; Control of Water - Parker, pp. 682 and 692.
	[Weir]	10	224	22.4	149	14.9	
23	Middle Loup Dam	10	147	14.7	102	10.2	Engineering Record, Vol. 74, No. 24, Dec. 9, 1916, p. 716.
24	Iron Mountain	30	257	8.6	180	6.0	J. W. Wood.
25	Dam No. 47, Ohio	5	122	13.5	101	11.2	U. S. War Department.
26	Dam No. 11, Ohio	8	122	15.2	101	12.6	U. S. War Department.
27	[illegible]	14.75	207	14.1	221	15.0	Engineer [London], Vol. 144, Sept. 2, 1927, p. 251.
28	[illegible] Municipal	14	140	14.0	90	9.0	Engineering Record, Vol. 74, No. 16, Oct. 28, 1916, p. 541.
29	[illegible] River	6	90	15.	67	11.2	" " 61, No. 27, June 4, 1910, p. 726.
30	[illegible] Lake	12	244	20.3	141	11.8	U. S. War Department.
31	Verson	8.6	104	12.1	70	8.1	Improvement of Rivers - Thomas and Watt, 24 Ed., p. 579.
32	Fritchetosmale	11.2	200	18.4	145	13.3	Advances Made in the Study and Construction of Dams - S. V. Wilinian and L. C. Pomerantzev - International Navigation Congress, 1926.
33	Lloyd Barrage	18.5	304	16.4	174	9.4	Bombay Public Works Department.
34	Islam Weir	18	254	14.1	197	11.0	Report [Dec. 13, 1929] by Islam Engineering Committee, Public Works Dept., Punjab.
35	Ju Chuang						
	(a) Regulator	11.5	174	15.1	118	10.3	General Report, Chihli River Commission, 1923, and R. D. Goetrich.
	(b) Sluice	12.12	198	16.1	137	10.7	" " " " " " "
36	Bonnet Carre	16.5	252	17.4	149	10.3	War Department. W. H. Wormorter. Engineering News-Record, Vol. 103, August 15, 1929, p. 242.

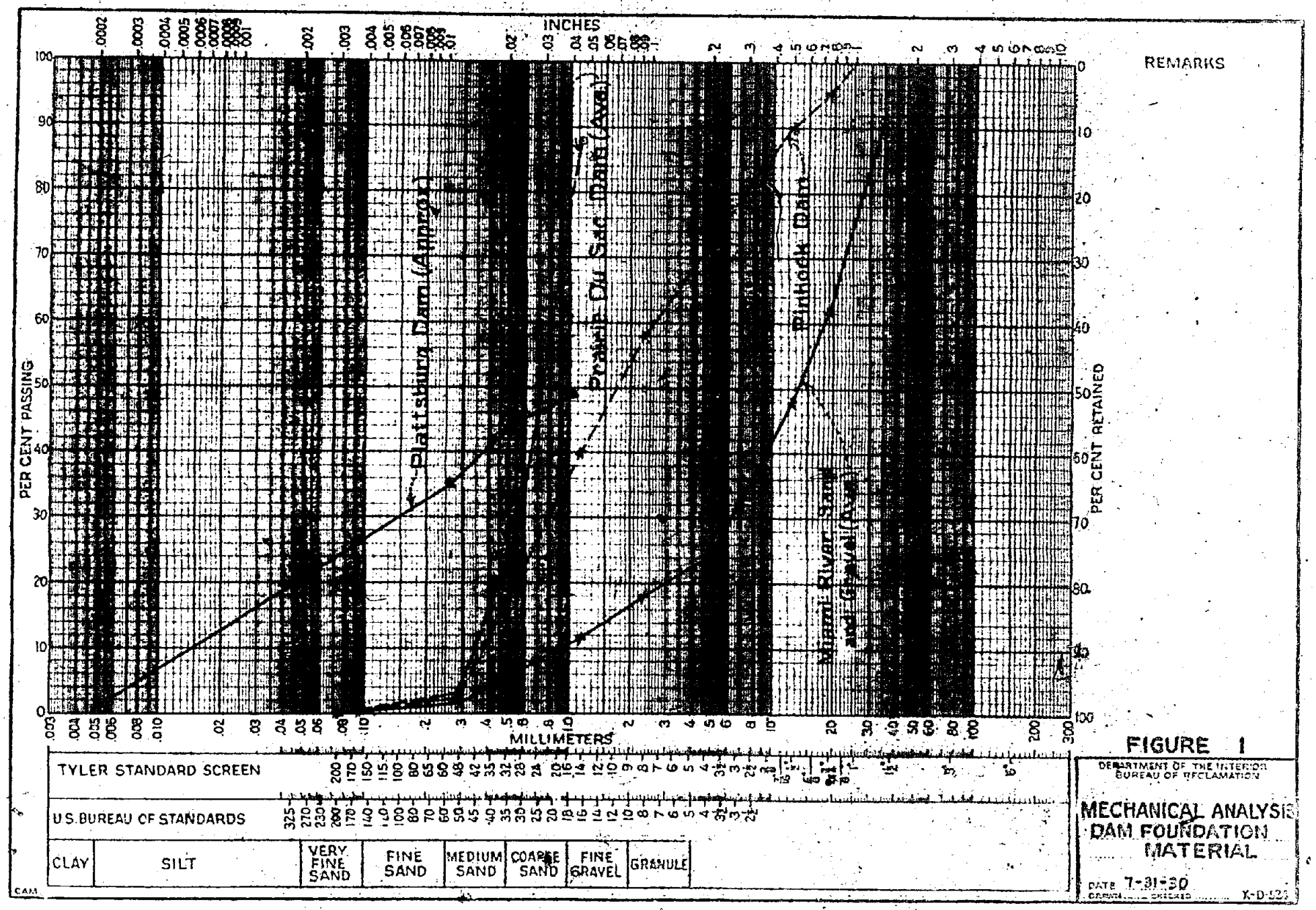
Data on Weep-Holes, Drains, and Reverse Filters.

Table No. 9.

Dam	No.*	Type	Foundation Material	Head	Plain Weths Total Dist.	Plain Weths Creep Ratio	Plain Weths Av. Dist.	Plain Weths Feet Ratio	Tailored Weths Total Dist.	Tailored Weths Creep	Tailored Weths Av. Dist.	Tailored Weths Feet Ratio	Short Path Weths Total Dist.	Short Path Weths Creep Ratio	Short Path Weths Av. Dist.	Short Path Weths Feet Ratio	Remarks
			Weep Holes														
Farmville	1 - 16	Ackersen	Clay	27	304	11.3	151	5.6	209	7.8	112	4.2					
Silver Springs	1 - 18	"	"	30	188	6.2	30	1.0	100	3.3	28	.93					
Littlefield	2 - 3	"	Fine material, gravel and boulders	33	76	2.3	24	.73	53	1.6	21	.64	63	1.9	21.5	.65	
Horse Creek	3 - 11		Coarse sand and gravel to 1/2 inch maximum size	6.4	86	6.6	29.7	4.6	30	4.	23	3.6	29	4.5	28.7	4.2	
Tenaille	2 - 12	Ogee, crest gates and apron	Sand and gravel	6.67	67	7.7	87	6.6	46	5.4	37	4.3	51	5.9	45	5.2	
Miller	2 - 27	Reinforced concrete step-lock	Sand, gravel, and clay	5	88	7.6	28	5.6	31	6.2	21	4.2	25	5.0	20	4.0	
Fremont	3 - 34	Ogee, crest and apron	Gravel	9.5	86	16.0	48	13.7	38	10.9	23	9.6	43	12.3	39	11.1	
Grand Valley	3 - 40	Ogee, with roller crest	Coarse gravel and cobbles with little sand	18.25	137	7.5	117	6.4	94	5.2	76	4.2	100	5.5	92	5.0	
Greenwatchie	3 - 43	Ambursen	Sand and boulders	23													
Tarsha	3 - 44	Ogee, with apron	Sand, gravel, and boulders	11	132	12.	100	9.1	96	9.7	74	6.8	86	7.8	64	5.8	
Harper Diversion Dam	3 - 46	Gate and apron	Sand, gravel and boulders	12	86	7.2	78	6.5	58	4.8	50	4.2	69	5.8	63	5.3	
Twin City Dam	3 - [illegible]	Ambursen	Sand, gravel, and limestone slabs	30.5	197	6.5	134	4.4	127	4.2	94	3.1	165	5.4	107	3.5	
St. Marys Intake Dam	3 - 47	"	Sand, gravel, and boulders	8.9	87	10.4	48	8.6	39	7.1	29	5.3	43	7.8	39	7.1	
Granite Reef	3 - 49	Ogee, with apron	Gravel and boulders	20	77	3.8	58	2.9	61	3.1	45	2.3	58	2.9	47	2.4	
Lochland Dam	3 - 50	Ambursen	Gravel and boulders														
Tower-Grove	3 - 51	Ogee, with apron	Gravel and boulders	8.66	64	7.5	54	6.1	46	5.4	34	4.0	50	5.9	45	5.5	
Strawberry	3 - 52	" " "	Gravel with occasional boulders	15	105	7.0	39	2.9	74	4.9	46	3.1	86	5.6	42	2.9	
Burnis Paper Company Dam	3 - 56	" " "	Silt and sand	7.6	82	10.9	74	9.7	74	9.7	64	8.4	66	8.7	59	7.8	
Florence	3 - 18			10.5	170.	16.2	143	13.6	119	11.3	103	10.	112	10.7	85	8.1	
			Drains														
Jackson	1 - 19	Ogee, crest gates, and apron	Clay	18	188.	9.2	76	5.1	98	6.5	54.	3.6					Gravel drain.
Jackson Mathias	2 - 4	Ogee, crest gates, and apron	Sand and boulders	48									130	2.7			With filter.
				20			12	.60			10	.50			12	.60	"
Iron Mountain	7 - 24	Ogee, crest gates, and apron	Gravel and boulders	30	257	8.6	171	5.7	188	6.3	123	4.1	190	6.0	129	4.3	"
			Reverse Filters														
Indian Barrage	7 - 24	Stoney gate	Fairly hard silt	11.5			115	10.0			69.0	6.0			102	8.9	
Sanderson	2 - 28	Ogee with apron	Coarse sand and gravel	16.4	361	22.0			215	13.1			303	18.9			
Dam #7 Toback (Masterings)	3 - 35	Weirs	Gravel	16.7	165	9.9			111	6.6			112	6.5			Filter at end of dam.
Dam #31 Tonnes (Post)	3 - 36	"	Gravel	12	95	7.9			63.0	5.3			67	5.6			" " " " "
Fifth Barrage	7 - 23	Stoney gate	Fine sand and mud	8	76	9.5			51	6.4			59	7.4			" " " " "
Assault Barrage	7 - 26	" "	Fine sand	13	211	16.			132	10.2			160	12.3			" " " " "
				8.2	191	23.3			122	14.9			148	18.			

*First figure refers to the table number; second, to dam number.

[illegible]


INCHES
MILLIMETERS
PER CENT PASSING
PER CENT RETAINED
Pittsburg Dam (Ambrose)
Pecatonica Div. Sta. (Dam Area)
Pinhook Dam
Mineral River Sand and Gravel Co.
TYLER STANDARD SCREEN
U.S. BUREAU OF STANDARDS
CLAY
SILT
VERY FINE SAND
FINE SAND
MEDIUM SAND
COARSE SAND
FINE GRAVEL
GRANULE
REMARKS
FIGURE 1
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
MECHANICAL ANALYSIS
DAM FOUNDATION
MATERIAL
DATE 7-31-30
X-D-125

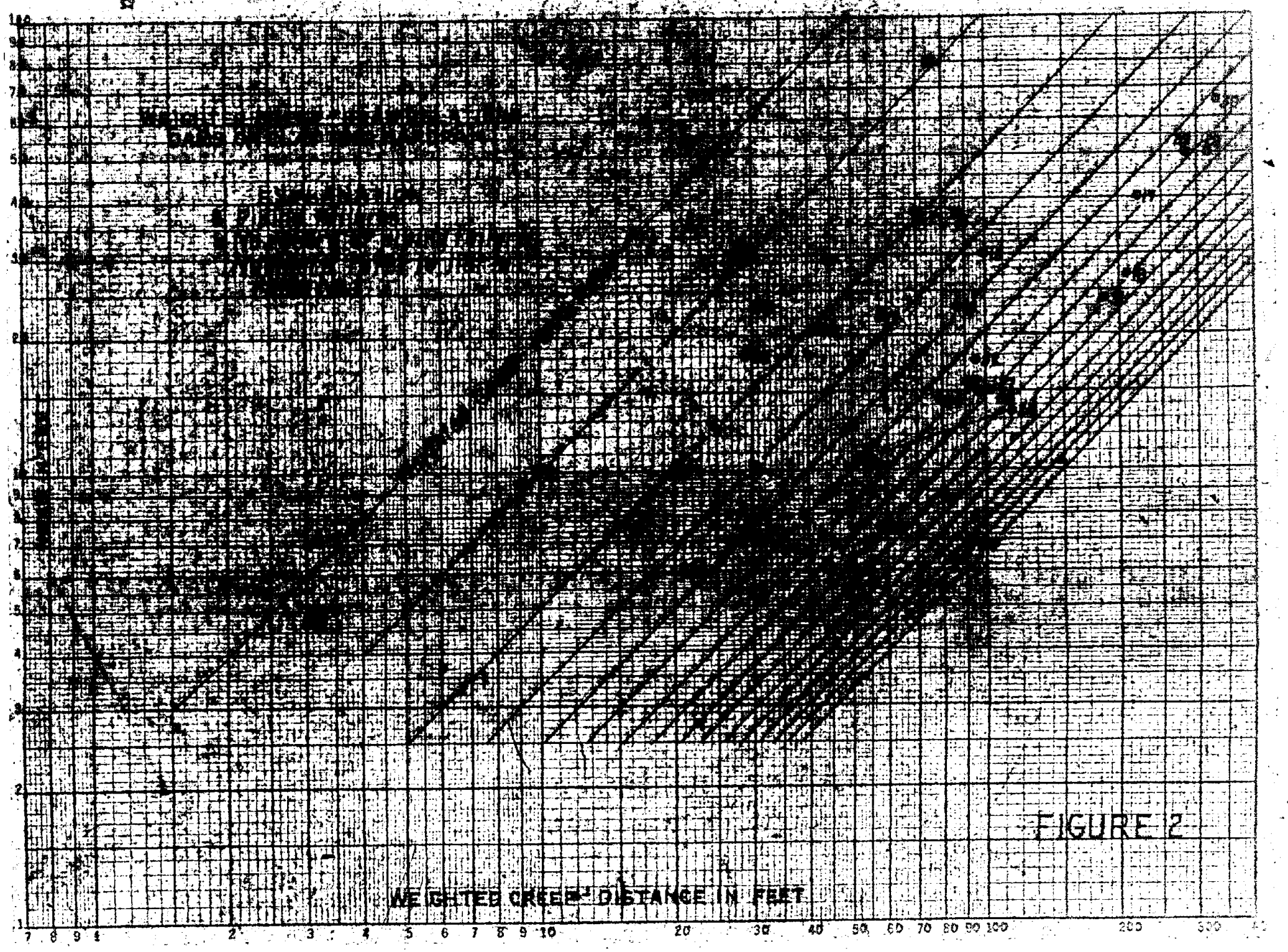

WEIGHTED CREEP DISTANCE IN FEET
7 8 9 1 2 3 4 5 6 7 8 9 10 20 30 40 50 60 70 80 90 100 200 300

FIGURE 2

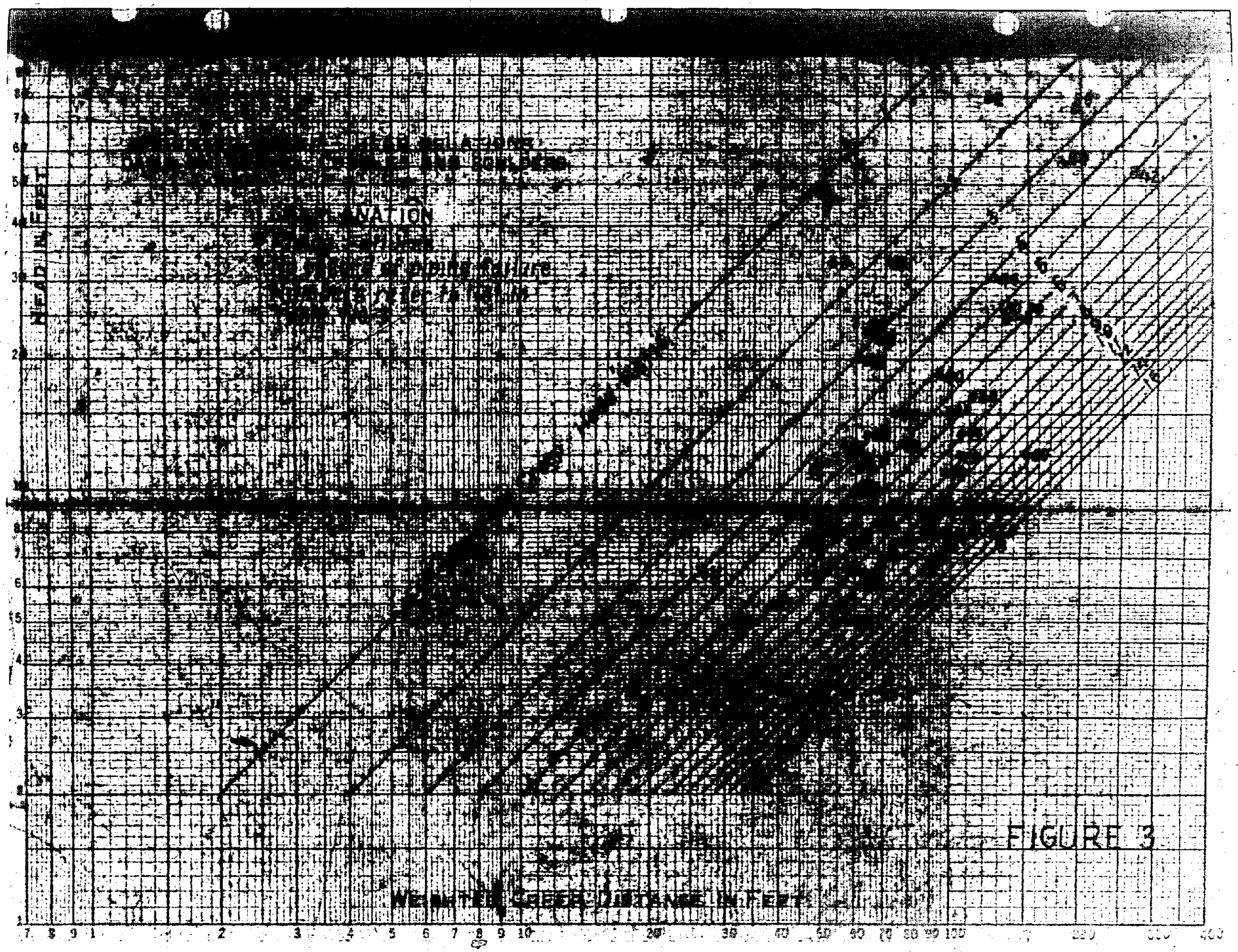
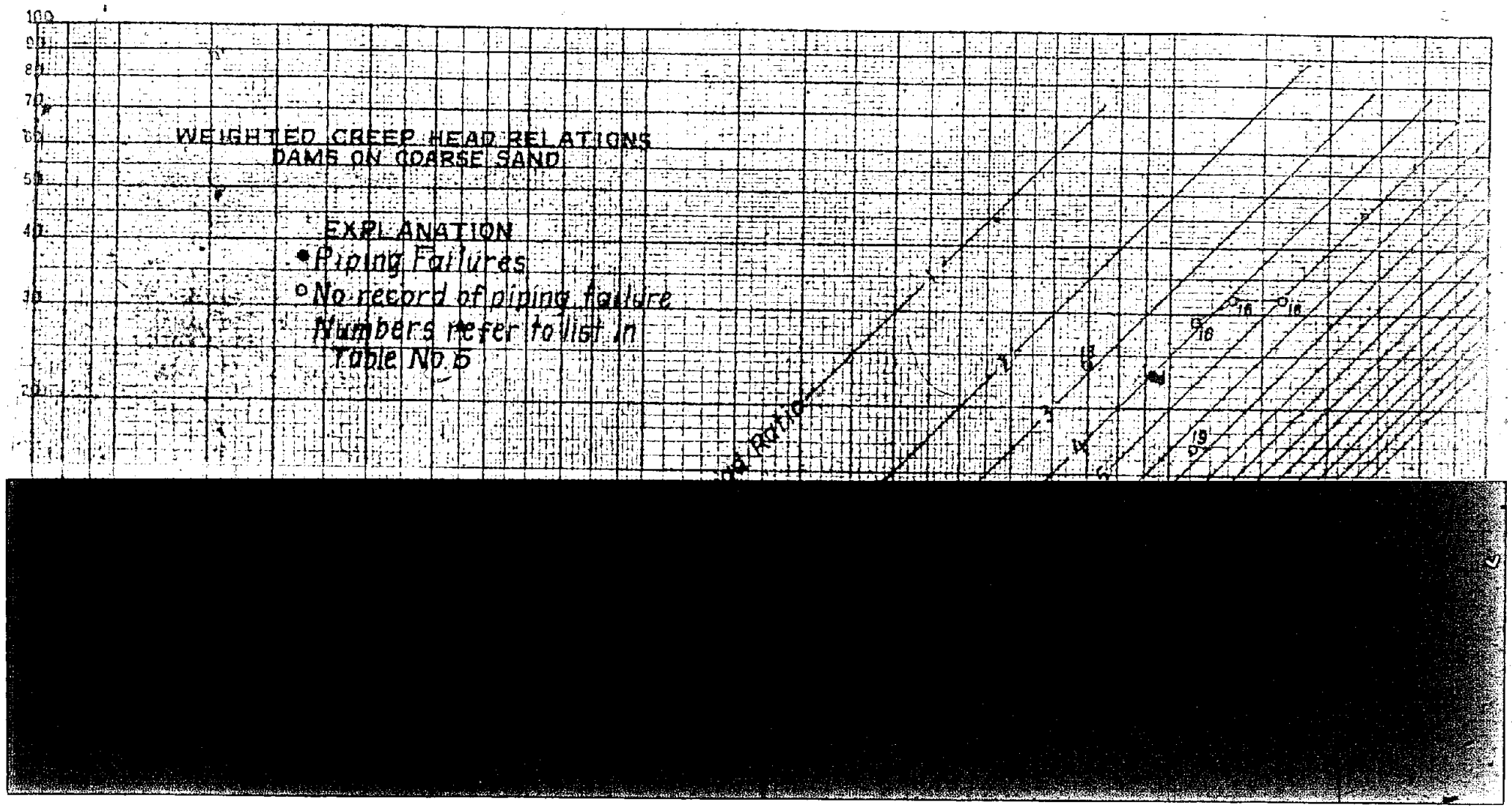


FIGURE 3


WEIGHTED CREEP HEAD RELATIONS
DAMS ON COARSE SAND
EXPLANATION
• Piping Failures
o No record of piping failure
Numbers refer to list in
Table No. 5
100
90
80
70
60
50
40
30
20
10
Ratio

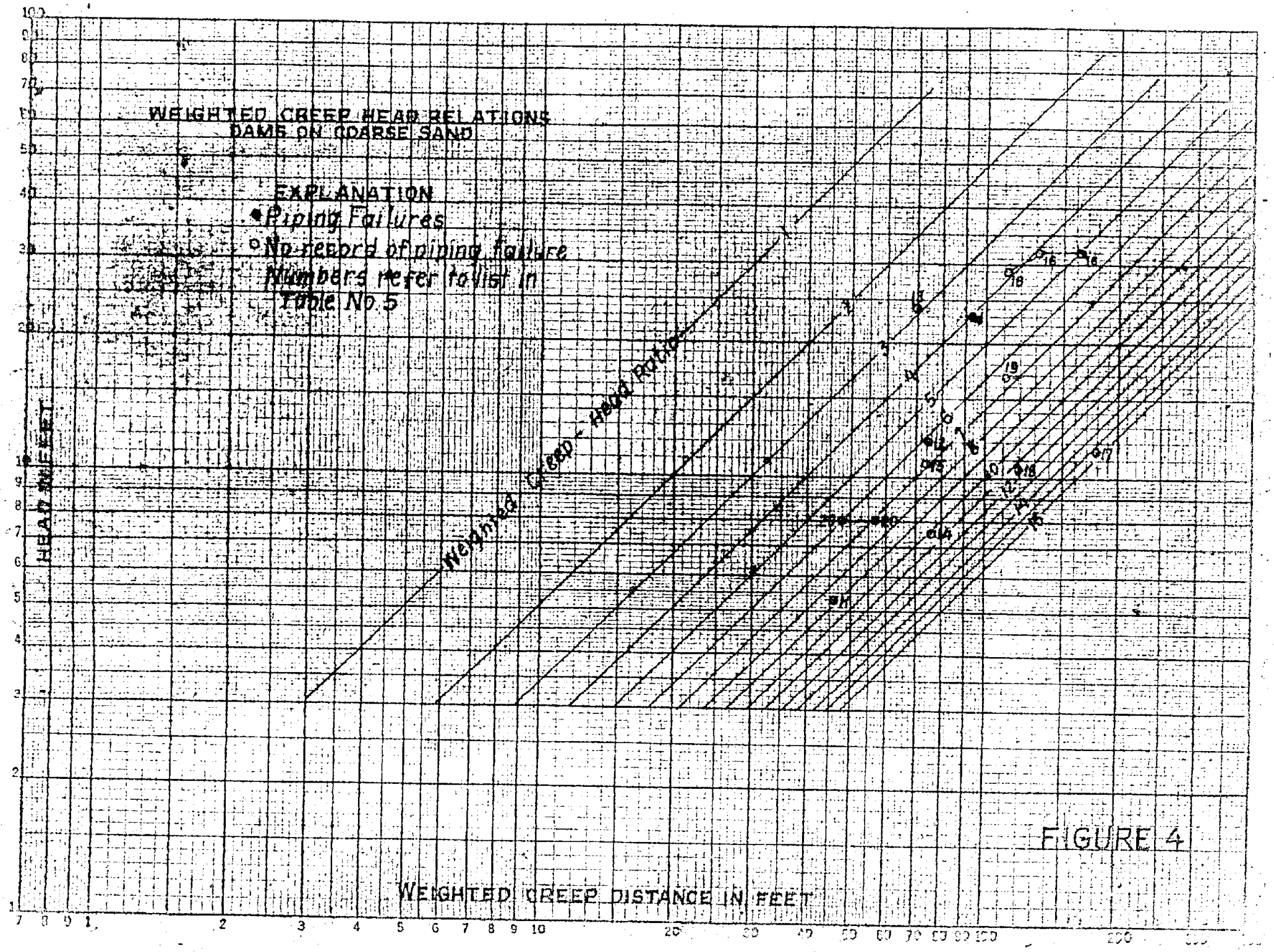

WEIGHTED CREEP HEAD RELATIONS
DAMS ON COARSE SAND
EXPLANATION
Piping Failures
No record of piping failure
Numbers refer to list in Table No. 5
Weighted Creep - Head Ratio
HEAD IN FEET
WEIGHTED CREEP DISTANCE IN FEET

FIGURE 4

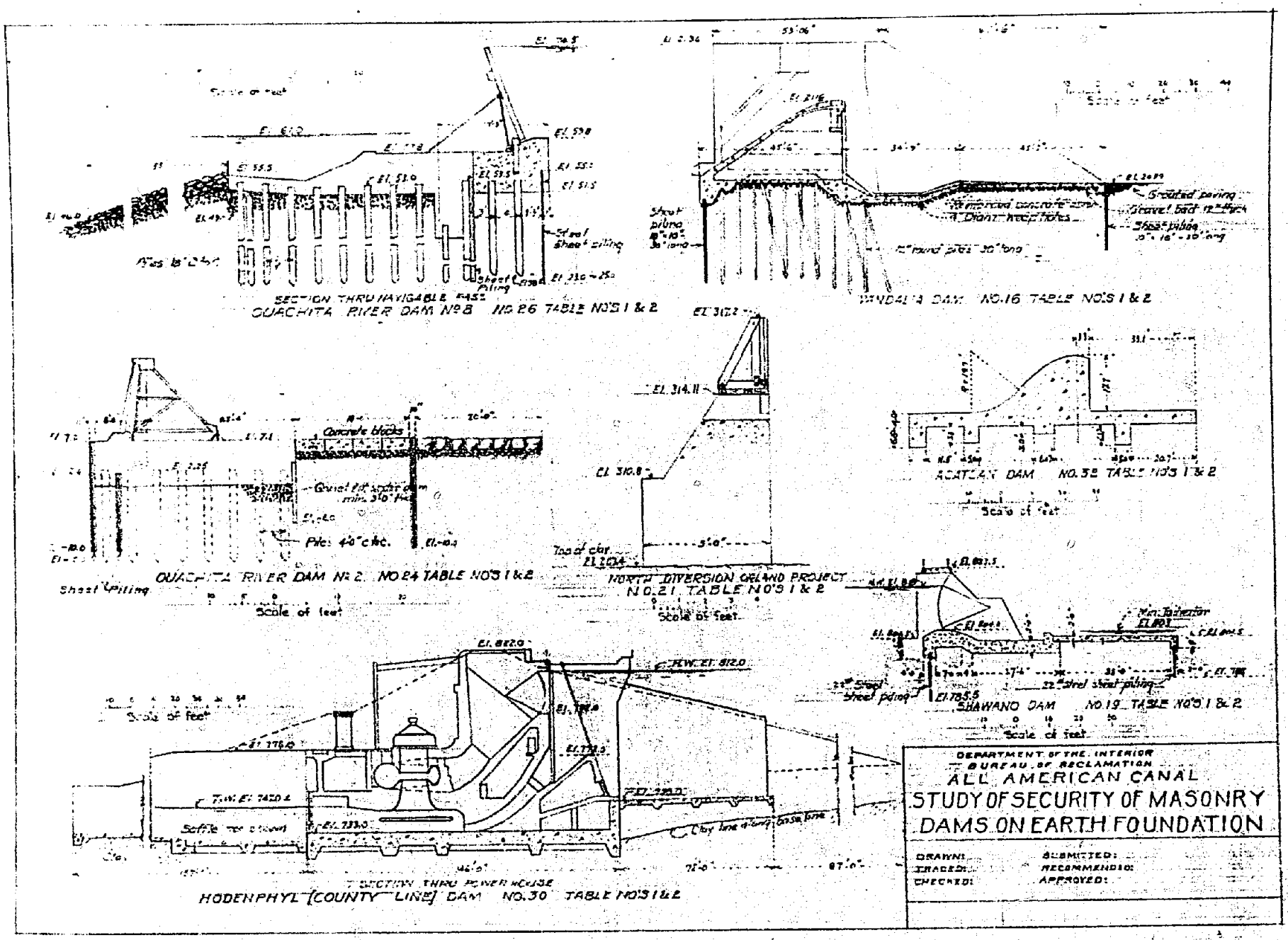

SECTION THRU NAVIGABLE PASS
OUACHITA RIVER DAM Nº 8 NO. 26 TABLE NO'S 1 & 2
VANDALIA DAM NO. 16 TABLE NO'S 1 & 2
OUACHITA RIVER DAM Nº 2 NO. 24 TABLE NO'S 1 & 2
NORTH DIVERSION ORLAND PROJECT
NO. 21 TABLE NO'S 1 & 2
ACATCAN DAM NO. 32 TABLE NO'S 1 & 2
SHAWANO DAM NO. 19 TABLE NO'S 1 & 2
SECTION THRU POWER HOUSE
HODENPHYL (COUNTY LINE) DAM NO. 30 TABLE NO'S 1 & 2
Scale of feet
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
ALL AMERICAN CANAL
STUDY OF SECURITY OF MASONRY
DAMS ON EARTH FOUNDATION
DRAWN:
TRACED:
CHECKED:
SUBMITTED:
RECOMMENDED:
APPROVED:

PLATE I

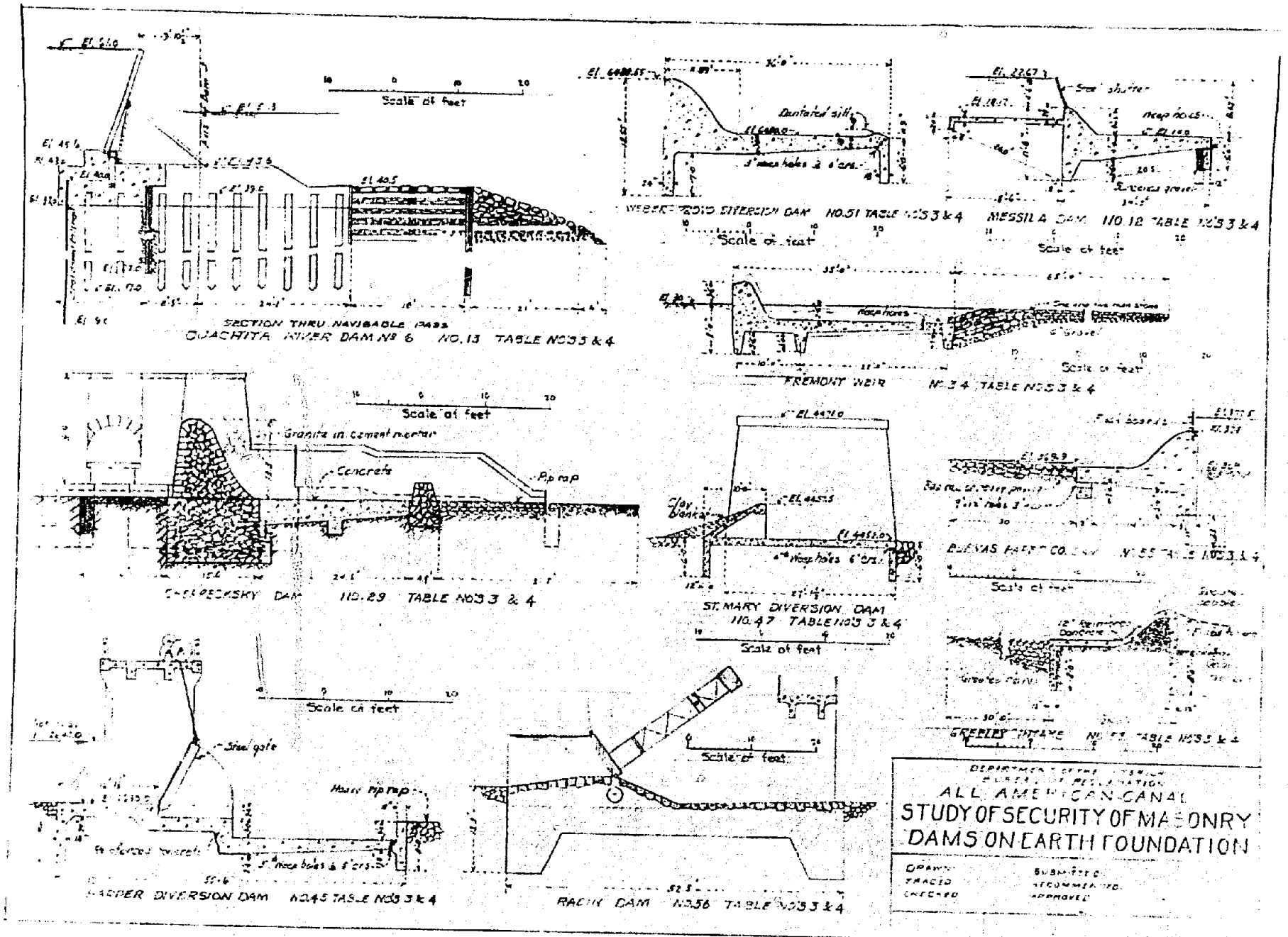
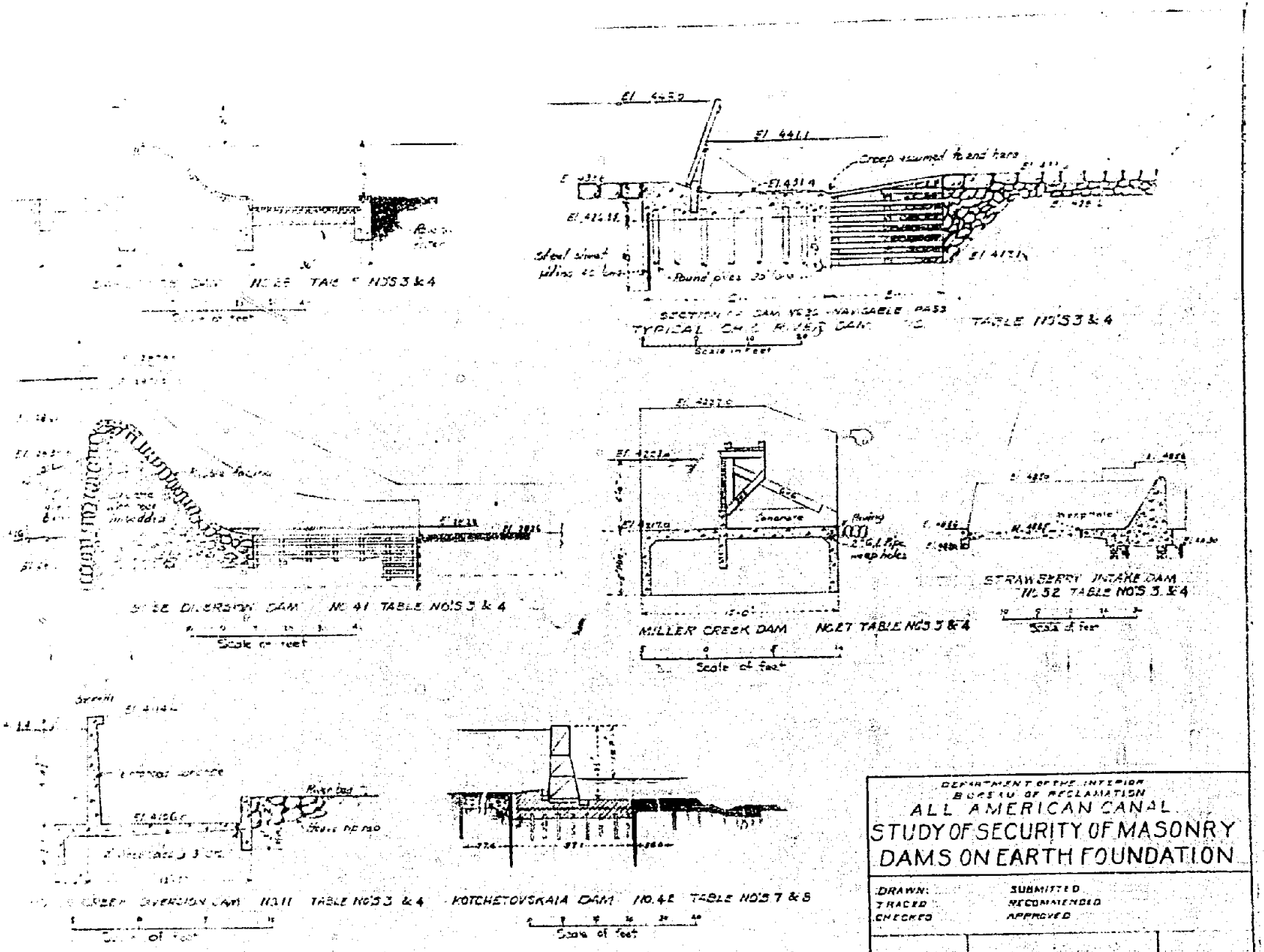

Scale of feet
SECTION THRU NAVIGABLE PASS
OUACHITA RIVER DAM No 6 NO. 13 TABLE NOS 3 & 4
Dentated sill
WEBER DIVERSION DAM NO. 51 TABLE NOS 3 & 4
MESSILA DAM NO. 12 TABLE NOS 3 & 4
FREMONT WEIR NO. 34 TABLE NOS 3 & 4
Granite in cement mortar
Concrete
Riprap
CHELPECKSKY DAM NO. 29 TABLE NOS 3 & 4
Clay Banks
ST. MARY DIVERSION DAM
NO. 47 TABLE NOS 3 & 4
Steel gate
Main Riprap
HASPER DIVERSION DAM NO. 45 TABLE NOS 3 & 4
RACIN DAM NO. 56 TABLE NOS 3 & 4
ALL AMERICAN CANAL
STUDY OF SECURITY OF MASONRY
DAMS ON EARTH FOUNDATION

PLATE II



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BUREAU OF RECLAMATION

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TRACED:	RECOMMENDED
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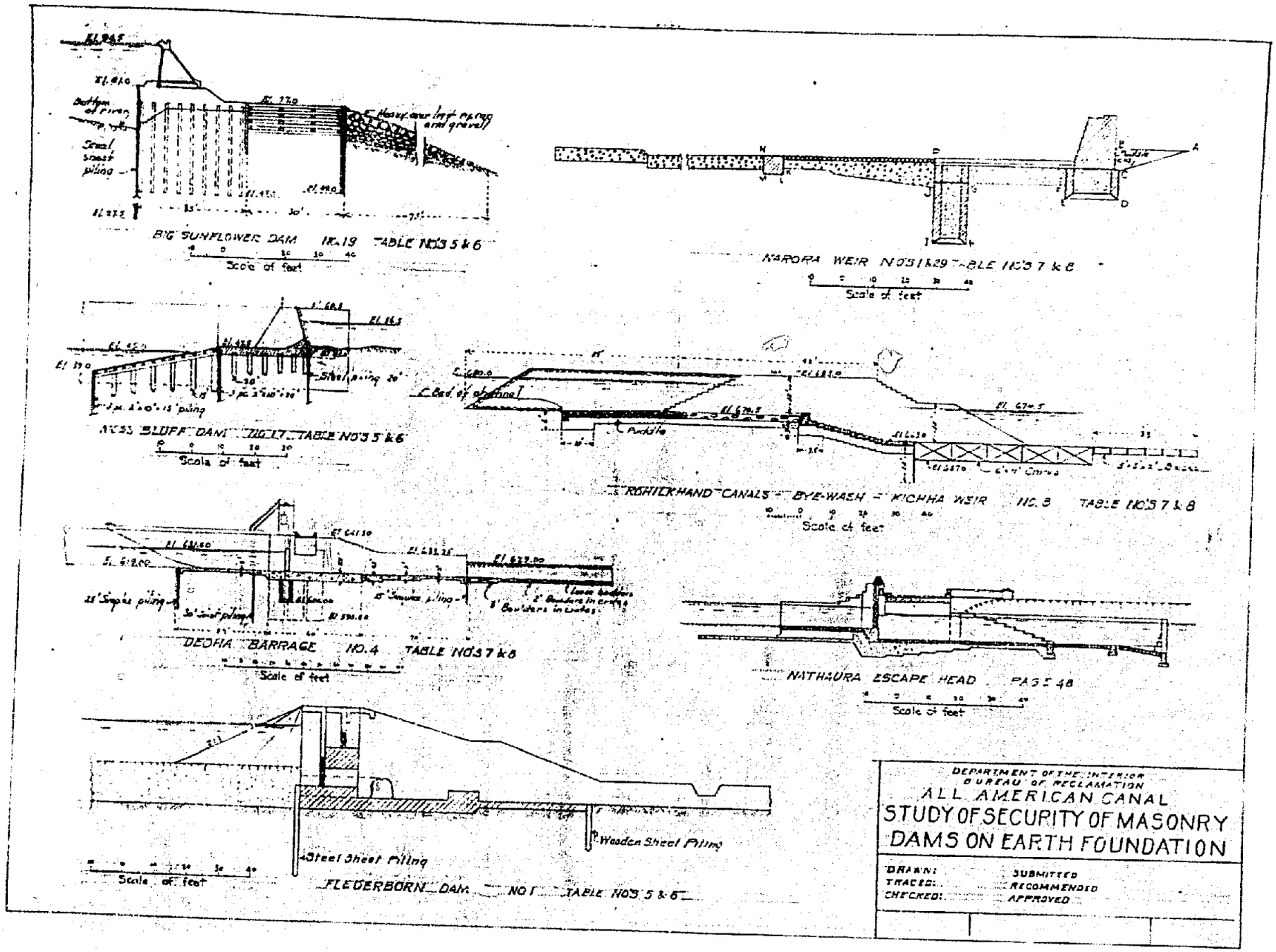
PLATE III


BIG SUNFLOWER DAM NO. 19 TABLE NOS 5 & 6
Scale of feet
NAROPA WEIR NOS 1 & 29 TABLE NOS 7 & 8
Scale of feet
MOSS BLUFF DAM NO. 17 TABLE NOS 5 & 6
Scale of feet
RGHILKHAND CANALS BYE-WASH KICHHA WEIR NO. 8 TABLE NOS 7 & 8
Scale of feet
DEOHA BARRAGE NO. 4 TABLE NOS 7 & 8
Scale of feet
NATHAURA ESCAPE HEAD PAGE 48
Scale of feet
Steel Sheet Piling
Wooden Sheet Piling
Scale of feet
FLEGERBORN DAM NO. 1 TABLE NOS 5 & 6
DEPARTMENT OF THE INTERIOR
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ALL AMERICAN CANAL
STUDY OF SECURITY OF MASONRY
DAMS ON EARTH FOUNDATION
DRAWN:
TRACED:
CHECKED:
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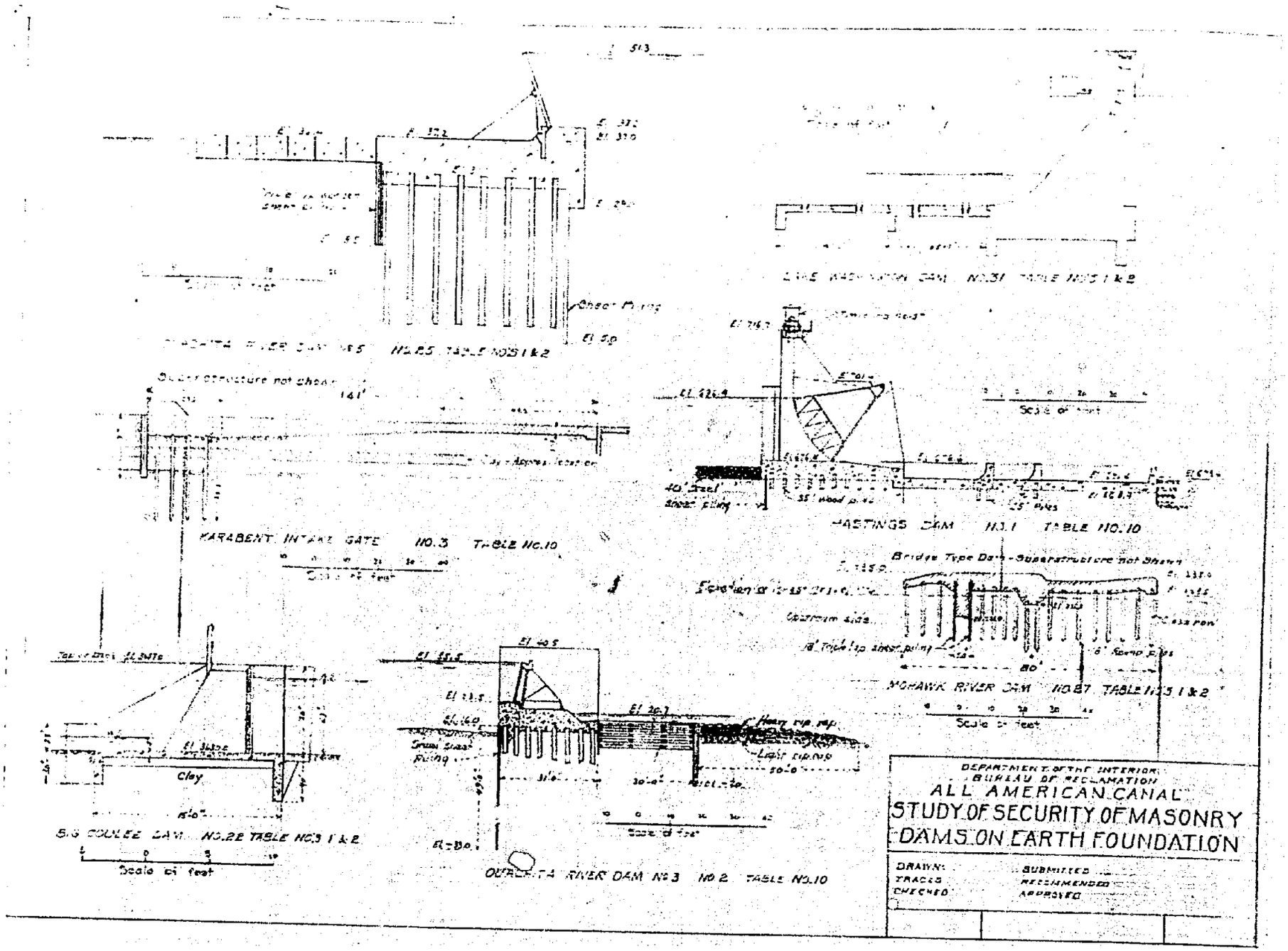
PLATE IV


Sheet Piling
Scale of feet
HASTINGS DAM NO.1 TABLE NO.10
KARABENT INTAKE GATE NO.3 TABLE NO.10
Bridge Type Dam - Substructure not Shown
MOHAWK RIVER DAM NO.87 TABLE NO.3 1 & 2
Clay
BIG COULEE DAM NO.2E TABLE NO.3 1 & 2
Scale of feet
Heavy rip rap
Light rip rap
OUTLETTA RIVER DAM NO.3 NO.2 TABLE NO.10
DEPARTMENT OF THE INTERIOR
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ALL AMERICAN CANAL
STUDY OF SECURITY OF MASONRY
DAMS ON EARTH FOUNDATION
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TRACED
CHECKED
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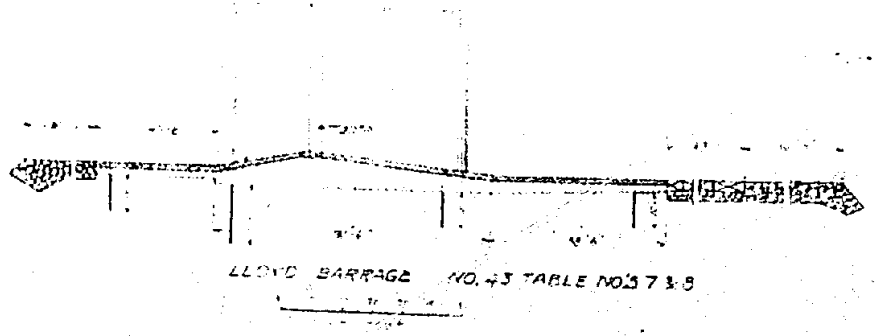
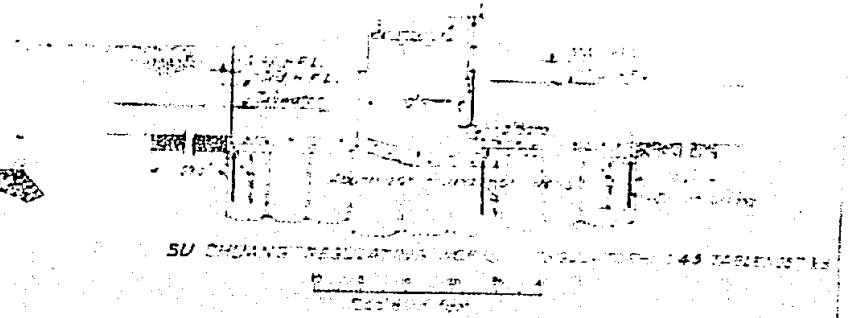
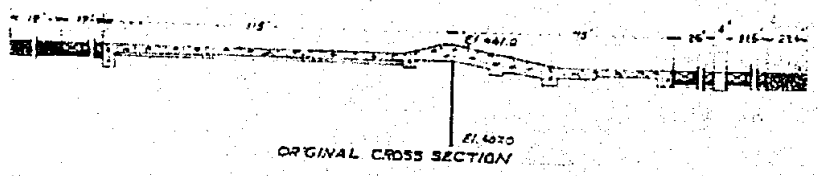
PLATE V



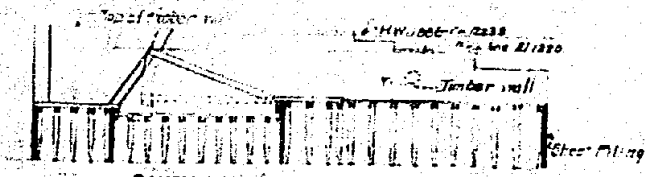
LLOYD BARRAGE NO. 43 TABLE NO'S 7 & 8



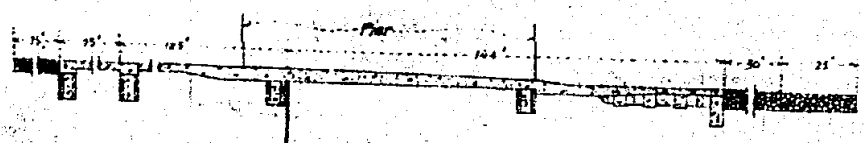
SU CHUANG REGULATING WEIR NO. 45 TABLE NO'S 7 & 8



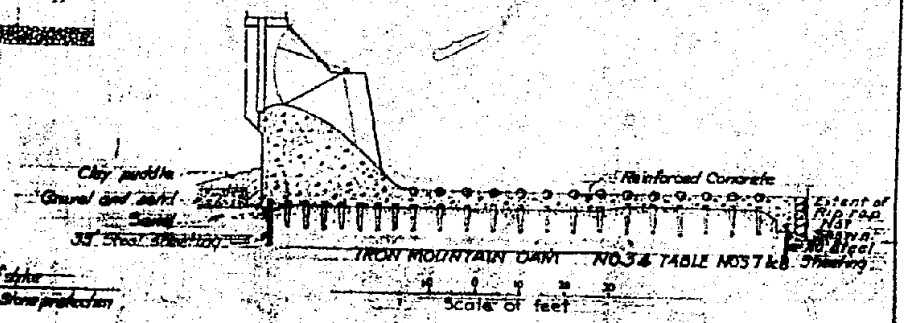
ORIGINAL CROSS SECTION



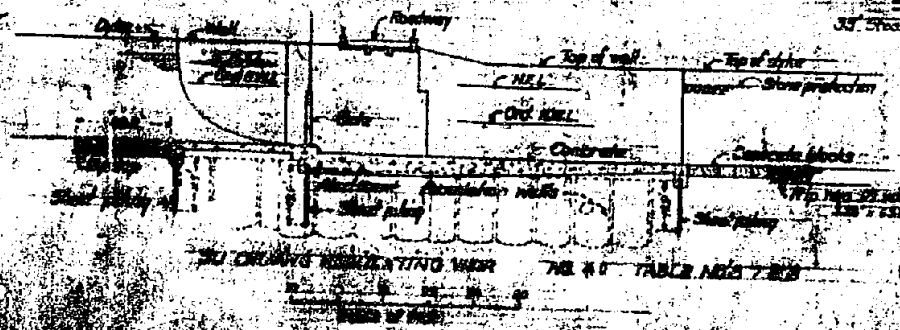
SANDY LAKE DAM NO. 40 TABLE NO'S 7 & 8



CROSS SECTION AS ALTERED AFTER FAILURE
ISLAM WEIR NO. 44 TABLE NO'S 7 & 8



IRON MOUNTAIN DAM NO. 34 TABLE NO'S 7 & 8



SU CHUANG REGULATING WEIR NO. 41 TABLE NO'S 7 & 8

DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
ALL AMERICAN CANAL
STUDY OF SECURITY OF MASONRY DAMS ON EARTH FOUNDATION

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