UNITED STATES

DEPARTMENT OF THE INTERIOR

BUREAU OF RECLAMATION

MEMORANDUM TO CHIEF DESIGNING ENGINEER

SUBJECT: SECURITY OF MASONRY DAMS ON EARTH FOUNDATIONS FROM UNDERSEEPAGE OR PIPING

By E. W. LANE, RESEARCH ENGINEER

TECHNICAL MEMORANDUM No. 303

Denver, Colorado,

September 9, 1932

Synopsis

This paper gives the results of an investigation of more than two 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 Beresford. 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

$$Q = K \frac{HA}{L}$$

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 coefficient which depends on the character of the material.

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

$$L = K \frac{H}{V} \tag{2}$$

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 coefficient C equal to $\frac{K}{V_m}$ the expression

$$L_m = CH$$

is obtained where L_m is the minimum safe length of travel path and C is a coefficient 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 experienced 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 dam was blown up, resulting in a breach of the weir. The failure of an important structure followed 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 [illegible].

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 his paper 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 it 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, has led engineers

**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.

generally to give the credit for the idea to Bligh.

In 1911 an article appeared by A. C. Konig*** giving rules for the design of masonry dams on earth foundations, which contains a number of valuable ideas.

***Trans. Am. Soc. C. E., Vol. 73, 1911 p. 175.

Little appeared in literature for many years after these 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 is that used by

Terzaghi*, It seems to have been first used for dam foundations by

*Technical Publication No. 215, Am. Inst. Mining & Metallurgical Engineers, 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 andthat 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 potential, the pressure at any point within the material or beneath the dam may be determined. By drawing lines representing the 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 they 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.

In designing a masonry dam on an earth foundation it is customary to assume that the path of the water flowing beneath it is always normal to the axis of the dam. Analyses are therefore made of cross sections of the dam perpendicular to the axis. If the material on which the structure is founded varies along its length, it may be necessary to investigate more than one section. The assumption of normal flow is sufficiently accurate for most conditions. In unusual cases however, a more accurate analysis may be necessary. In analyzing the security of the dam at the abutments, normal flow cannot be assumed

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 between 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 develops 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, Nos. 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 these 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. This will be discussed more at length on page [illegible]. 40

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 as 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 rigidly 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

	Publication				
	I	II	III	IV	V
Year of publication:-	1910	1910	1913	1916	?
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		
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	

(Cont'd)

	Publication I 1910	II 1910	III 1913	IV 1916	V
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 same 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

*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.

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 of 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-third is given to the horizontal or slightly inclined creep as compared with the other portion of the path.

For example, if a dam sustains a head of 10 ft. and has a creep distance along horizontal or slightly inclined surfaces of 60 ft. and along vertical and steeply inclined surfaces of 10 ft., the weighted creep distance would be 60 x 1/3 plus 10 or 30 ft., and the weighted creep head ratio would be 30 ÷ 10 = 3.0.

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-third 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 path 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 39.

In computing the weighted creep distance it was necessary to determine the division point between steeply sloping and slightly sloping contacts. This division should be at the steepest slope at which a bank of earth under water would be unquestionably unstable; i.e., at which the bank undoubtedly would slip. This was assumed to be on a 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 path 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 39.

In computing the weighted creep distance it was necessary to determine the division point between steeply sloping and slightly sloping contacts. This division should be at the steepest slope at which a bank of earth under water would be unquestionably unstable; i.e., 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. There is some evidence to indicate that the contact of puddle with earth is so close that it would be equivalent to vertical creep. In order to be conservative however, this assumption was not made in computing the ratios used in this paper.

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. In the case of a number of dams of the buttressed concrete type, no floor was provided between the buttresses and the water passing beneath the upstream cutoff could rise in this space and escape thru vents provided in the downstream face of the dam. If these vents were closed, it would have to pass beneath another section of the dam to reach the tailwater. For dams of this type, the distances and ratios for both conditions have been computed altho the shorter creep distance is probably the effective one.

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.

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. Masonry dams on earth where the cutoff was carried to solid rock were not analyzed.

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 dam foundation

*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.

materials however it is objectionable 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 analysis of the weighted creep relations for all structures for which sufficient data could be obtained are given in Tables 1 to 4. The first column gives each dam a number which is useful in identifying it on the figures. The next three columns give the name of the dam, the stream dammed and the state or country in which it is located respectively. The fifth column gives the effective head on the dam, the sixth and seventh columns the vertical and horizontal creep respectively. The eighth gives the weighted creep distance, the horizontal creep having a weight of one-third, and the ninth column gives the weighted creep ratio, or the weighted creep divided by the head. The three following columns give respectively the nature of the foundation material, the source of the information from which the data was secured, and the plate on which the section of the dam is shown.

For those who desire more complete data, Tables 1 to 4 inclusive of Appendix II have been prepared. These give for each dam the plain creep distance and ratio, the reliability of the data on which the figures are based, the results secured with the dam, usually in the nature of a statement of the period of service or date of completion and miscellaneous remarks which give other pertinent information. In regard to the data on "Results," it may be said that for the larger structures 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.

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 headgates.

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

Additional data on the reliability of the data on the various structures, the length of the period of service and other pertinent information, is given in Appendix II. Where the conditions could not adequately be covered in a table, a more complete statement is given in Appendix III. Cross sections of the dams analyzed are given in Plates I to XXII of Appendix IV.

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 many 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 ByeWash 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.

on basis of Bligh's method

Dam	Material	Creep-Head Ratio	Recommended Ratios (approximate)		Actual Ratio in % of Recommended	
			Bligh	Griffith*	Bligh	Griffith
Dam No. 48 Ohio River	Very fine sand	13.2	18	11.6 to 12.8	73	103 to 113
Dam No. 47 Ohio River	Sand, very fine to medium, a little fine gravel	13.5	18	11.6 to 12.8	75	105 to 116
Barre Moss	Fine sand & silt	8.0	18	11.6 to 12.8	44	62 to 69
Iron Mountain	Fine sand	8.6	15	10 to 11.2	57	77 to 86
Riverdale	Coarse sand	4.2	12	8 to 9.6	35	44 to 52
Prairie du Sac	Coarse sand	4.3	12	8 to 9.6	36	45 to 54
		6.2	12	8 to 9.6	52	65 to 77
Columbia	Coarse sand	4.0	12	8 to 9.6	33	41 to 50
Dam No. 1, Big Sunflower Riv.	Sand with a little clay, gravel and clayey sand	6.6	14	8	47	83
Tunuyan Weir	Sand & gravel	6.0	9	6.4	66	94
Granite Reef	Gravel and boulders	3.2	5 to 9	4.8	36 to 64	66
Walkill N.Y.	Gravel and boulders	2.6	5 to 9	4.8	30 to 52	54
Oswegatchie R.	Sand and boulders	3.6	5 to 9	6.4	40 to 72	54
Piedmont	Boulders	2.7	4	3.2	68	85

*These values include the reduction for vertical staunching.

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, not a single case was found where piping unquestionably took place under a deep cutoff, and only one case, the Fiederborn Dam, in which it may have. The data on this failure is so incomplete that the piping may have been behind the abutments, rather than under the dam, but even if it was under the dam it probably took place at ratios much less than recommended in this paper and certainly at ratios somewhat less. 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 plain creep 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. Griffith, 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 Bye 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 the 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 exactly 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.

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 Congress, 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 arriving at the decision to use a weight of one-third 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 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.

In view of the uncertainty 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 recommends a weight of one-third for the horizontal creep, altho there is considerable data to indicate that a higher ratio might be better.

Summary of Limiting Weighted Creep Ratios

The following tables give 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:

Table & Dam No.	Dam	Foundation Material	Wt'd Creep Ratios Wt. of Hor. Creep 1/3
Table 1 Dam No.	Dams on Clay or Hardpan		
1	Woodward (Failed)	Hard material, probably hardpan	0.9
2	Dolgarrog (Failed)	Hard blue clay	0.8
3	Fergus Falls (Failed)	Hardpan	1.1
10	French Landing	Clay	1.6
11	Dolgarrog (Standing portion)	Hard blue clay	1.8
12	Marysville	Firm clay with boulders	2.1
14	Kettle Creek	Very hard clay	1.7
20	Whiting Street	Very compact gravelly hardpan	1.5
21	North Diversion	Clay with considerable firmly embedded gravelly hardpan	0.45
36	Edenville Plant	Clay, gravel and very hard hardpan	2.0
38	Sanford Plant	Clay, gravel and hardpan	2.1
39	Secords Plant	Clay, gravel & hardpan	2.0
40	Smallwood Plant	Clay, gravel & hardpan	1.8
41	Tobacco Plant	Clay, gravel & hardpan	1.8
42	Barton Dam	Gravel & hardpan	1.7
43	Black River Storage Dam	Clay	2.2
47	Lake St. Francis	Stiff clay with small boulders and gravel	
	Bulkhead Section		1.7
	Spillway Section		2.1
50	Baltic Main Dam	Clay and gravel	1.4
51	Baltic Side Dam	Clay and gravel	1.65
56	Logan, Utah	Clay, sand and gravel	1.4
59	New London, Conn.	Clay and gravel	1.5
65	St. Paul, Minn.	Clay with 50% sand and gravel	0.75

Dam	Foundation Material	Wt'd Creep Ratio Wt. of Hor. Creep H/3
	Dams on Gravel, Cobbles and Boulders	
1 Puentes (Failed)	Sand and gravel	6.67 *
2 Port Angeles (Failed)	Gravel, small grit or coarse sand	1.3
3 Plattsburg (Failed)	Boulders, fine gravel and sand	2.8
22 Pinhook	Sand and gravel	3.5
25 Richmond	Gravel and sand	3.3
26 Puentes (Held)	Sand and gravel	2.0
41 Boise Intake	Very compact gravel and cobbles	1.9
43 Oswegatchie R.	Sand and glacial boulders	2.6
49 Granite Reef	Gravel and boulders	2.8
54 Avignonet	Gravel	2.7
66 Flint River, Ga.	Broken limestone	2.7
68 Piedmont, W. Va.	Boulders	1.9
73 Stamford, Conn.	Sand and gravel	2.5
75 Woodstock, Ver.	Loose gravel	2.2
74 Walkill, N. Y.	Boulders and gravel	1.3
Table 3	Dams on Coarse Sand	
Dam No.		
1 Flederborn (Failed)	Sharp sand	3.4
13 Riverdale	Torpedo sand	2.5
15 Godavery	Coarse sand	6.0
18 Big Sunflower Dam No. 1	Sand with a little clay and gravel	6.6
20 Prairie du Sac	Pure coarse sand	
Power house	do	6.5
Dam (Vents open)	do	3.7
Dam (Total creep)	do	4.8
21 Columbia	Coarse sand	2.8
22 Crawford	Coarse sand	2.0
23 Gloucester	Coarse sand	2.3
		3.3
24 Camp Humphreys	Coarse sand to gravel	5.5

Table & Dam No.:	Dam	Foundation Material	Wt'd Creep Ratio Wt. of Hor. Creep H/3
Table 4		Dams on Sand, Fine Sand and Silt	
Dam No.			
1	Narora (Failed)		
	Probable	Fine sand	4.7
	Possible	Fine sand	2.8
	Not over	Fine sand	5.9
2	Khanki (Failed)	Fine sand	4.2
3	Corpus Christi (Failed)	Sand	2.1
4	Deoha (Failed)	Fine sand	6.9
11	Nadral Escape Fall	Light sandy soil	2.9
12	Park Dam	Quicksand & gravel	1.5 1.9
23	Islam Weir	Sand	7.6
25	Khanki (Reconstructed)	Fine sand	7.7
30	Narora (Reconstructed)	Fine sand	8.2
31	Nadral Escape Fall	Light sandy soil	3.9
37	Zifta	Running sand and mud	8.1
49	Iron Mountain	Fine sand	5.5
56	Mottville Plant	Quicksand	4.6
57	Village of Lowell	Sand	4.5
58	Village of Bellaire	Fine, clean sand	2.0
59	Akron, Ohio	Sand and silt	4.2
60	Anadarko, Okla.	Sand	3.1
61	Barre, Mass	Sand, fine sand & silt	2.8 4.8
62	Miami, Ohio	Sandy silt	4.1
63	Okla. Diversion Dam	Sand and shale	6.3

Table & Dam No.:	Dam	Foundation Material	Wt'd Creep Ratio Wt. of Hor. Creep H/3
Table 4	Dams on Sand, Fine Sand and Silt		
Dam No.			
1	Narora (Failed)		
	Probable	Fine sand	4.7
	Possible	Fine sand	2.8
	Not over	Fine sand	5.9
2	Khanki (Failed)	Fine sand	4.2
3	Corpus Christi (Failed)	Sand	2.1
4	Deoha (Failed)	Fine sand	6.9
11	Nadral Escape Fall	Light sandy soil	2.9
12	Park Dam	Quicksand & gravel	1.5 1.9
23	Islam Weir	Sand	7.6
25	Khanki (Reconstructed)	Fine sand	7.7
30	Narora (Reconstructed)	Fine sand	8.2
31	Nadral Escape Fall	Light sandy soil	3.9
37	Zifta	Running sand and mud	8.1
49	Iron Mountain	Fine sand	5.5
56	Mottville Plant	Quicksand	4.6
57	Village of Lowell	Sand	4.5
58	Village of Bellaire	Fine, clean sand	2.0
59	Akron, Ohio	Sand and silt	4.2
60	Anadarko, Okla.	Sand	3.1
61	Barre, Mass	Sand, fine sand & silt	2.8 4.8
62	Miami, Ohio	Sandy silt	4.1
63	Okla. Diversion Dam	Sand and shale	6.3

Suggested Weighted Creep Ratios

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 tables indicate 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 or concrete 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. Somewhat smaller values may be used for less important structures, ranging down to perhaps 80 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.

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

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.

Recommended Values are Conservative

The recommended values have intentionally been made quite conservative. Not a single failure was found where the dam had creep head ratios as large as those given. It is possible that future experience year, and the grouted riprap at the end of this apron was blown up and piping proceeded until the whole structure collapsed. Considering creep under the puddle apron as vertical creep, the weighted creep distance to the point of blowout was 58 feet, giving a weighted creep head ratio of 2.9. The corresponding plain creep and short-path distances and ratios are 109, 84 and 5.5, 4.2 respectively.

Leasburg Dam

The Leasburg Dam was built in 1907. It was founded on sand, about 10 feet below the surface of which there were many boulders. There was a row of 20 ft. tripple lap wooden sheet piling along both upstream and downstream edges. On account of the boulders, it was not possible to drive the sheet piling with close joints throughout. Between the sheet piling were round piles 4 feet on centers. A layer of 4" of broken stone was placed beneath the dam and a reverse filter was placed just upstream from the lower row of piling and discharged thru the concrete, by passing the sheet piling, thru 2-inch pipes at about 4 ft. intervals. There was a flow of hot water up thru the foundation material.

The dam crossed the river channel and extended part way across the higher bottom land. When floods came the water came down the river channel and then spread out to go over the crest of the dam. This gave a higher head near the river channel which caused flow along the upstream face of the crest, causing deep scour. This uncovered the gaps in the sheet piles and possibly scoured below them. Piping developed but the dam did not collapse as it was supported on the piles. It was repaired by throwing riprap and brush on the upstream side where the water had piped thru, which stopped the leaks and allowed the basin upstream to silt up. The dam has been in service since 1906.

Dams on Clay Underlaid by Pervious Materials

Nathaura Escape Head, Sarda Canal, United Provinces, India

The foundation of this structure was excavated in a thick layer of clay underlaid with sand, which inturn 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   ) and was rapidly washed out from beneath the structure, until a channel formed beneath the structure and undermined it.

- - - -

APPENDIX III

PLAIN CREEP AND SHORT PATH RATIOS

[illegible]							APP. III, TABLE NO. 1.
No.	Name	Head	Plain Dist.	Creep Ratio	Reliability of Data	Results	Remarks
	DAMS WHICH FAILED						
1	[illegible] Dam	20	22	1.1	Fair	In service 4 or 5 years.	
2	Dolgarrog Dam	10	18	1.8	Good	In service more than 14 years.	Drying of foundation during low reservoir stage may have caused failure.
3	Fergus Falls Dam	22	104	4.8	Fair	In service 11 months.	Boring under power house reduced the weighted creep ratio from 3.8 to 1.8.
	DAMS OF WHICH NO FAILURE IS RECORDED						
11	Dolgarrog Dam	10	20	2.6	Fair	In service more than 14 years.	Portion of dam which did not fail.
12	Marysville Dam	22	90	4.2	Good	Built about 1920.	Downstream apron light; if it blows up the weighted creep ratio will be 1.32; has stood 29' head for short time; ratio .88.
13	Fergus Falls Dam	22	104	4.6	Good	In service 11 months.	Failure under power house caused washing out of this section.
14	Kettle Creek Dam	20.5	66	3.2	Good	Built about 1922.	Downstream apron light; if lifted up the weighted creep ratio would become 1.0.
15	[illegible] Weir, India	9	71	7.9	Good	Built in 1921.	
16	[illegible]	11.5	116	10.05	Good	Built in 1922.	Spillway section.
17	[illegible] Dam	40	368	9.1	Good	Built about 1912.	[illegible]
18	[illegible]	9	150	16.6	Uncertain	Built prior to 1888.	
19	[illegible] Dam	18	120	9.2	Good	Successful operation since 1920.	Creep does not include rubble masonry apron 36' wide.
20	[illegible]	18	[illegible]	2.0	Good	Built about 1920.	
21	North [illegible]	4.7	5.0	1.1	Good	Successful since 1912.	
22	[illegible]	6	30	4.7	Good	Successful since 1924.	
23	[illegible] Weir	7.5	81	10.8	Good		
24	Dam No. 2, Ouachita	14.4	94	6.5	Good	Successful since 1919.	
25	Dam No. 5, Ouachita	6.9	111	16.1	Good	Successful since 1924.	Creep ratio given is for normal head; for maximum head it is 7.7.
26	Dam No. 8, Ouachita						
	Original condition	10.5	63	6.0	Good	Satisfactory service 11 years for the first and since 1924 for the second.	
	Revised condition	13.5	131	9.7			Dam increased in height 3 feet after 11 years' use.
27	Dam No. 5, [illegible]	12	140	9.3	Good	Satisfactory service since 1912; some leakage stopped by grouting and additional sheet piles.	
28	Dam No. 6, [illegible]	15	140	9.3	Good	Satisfactory since 1911.	
29	Junction Dam						
	Power house section	20	204	7.3	Fair	Completed about 1915.	Power house.
	Spillway section	20	[illegible]	6.6	Fair		
30	[illegible] (County Line)	24	[illegible]	6.1	Good		
31	Lake Washington	20	200	11.7	Good	Satisfactory service since 1916.	Long, thin apron; 90% of creep under apron 2' thick or less.
32	Austin Dam	17.7	141	8.0	Good	Satisfactory service since 1915.	
33	[illegible] River Irr. Sys., [illegible]	7.25	[illegible]	11.44	Good	Completed in 1925.	Creep distances do not include flexible block floor 51 feet wide.
34	[illegible]	8.9	181.5	20.5	Good	Completed in 1922.	Complete creep distance does not include 52' apron.
35	[illegible] Plant	20	164.6	6.87	Good	Completed in 1926.	Does not include apron 20' wide.
36	[illegible] Plant						
	Section through dam	40	169.3	3.77	Good	Completed in 1926.	Flexible apron 12' wide not included in creep distance.
	Section through power house	40	182.3	4.12			
37	[illegible] Plant	14	[illegible]	4.7	Good		
38	[illegible] Plant						
	Typical section	27.5	201.75	3.7	Good		Width of apron 20' not included in weighted creep.
	Section through power house	27.5	110	3.8			
39	[illegible] Plant						
	Section through dam	40.5	274.[illegible]	5.04	Good	Completed in 1926.	Creep distance taken to turbine tailrace; apron 35' not included in creep distance.
	Section through power house	40.5	184.[illegible]	[illegible]			
40	[illegible] Plant						
	Section through dam	27.5	121.75	4.40	Good	Completed in 1924.	Width of flexible apron 12' not included in creep distance.
	Section through power house	27.5	[illegible]	3.44			
41	[illegible] Plant	40.4	[illegible]	3.7			
42	[illegible] Dam	[illegible]	[illegible]	5.7	Good	Completed in 1922.	Section through dam.
		[illegible]	[illegible]	1.9	Good		Spillway section; creep includes downstream section.
43	[illegible] Dam	15	[illegible]	7.4	Good	Built in 1920.	Creep does not include downstream section.
		15	[illegible]	2.9			Creep includes downstream section.
44	Dam built by the [illegible] Co.	[illegible]	[illegible]	2.70	Good	Built in 1927.	Creep does not include downstream section.
		[illegible]	[illegible]	6.8	Good	Built in 1927.	Creep does not include downstream section.
45	Dam built for City of [illegible]	[illegible]	[illegible]	6.5	Good	Built in 1926.	Creep includes downstream section.
46	[illegible], Alberta	40	[illegible]	4.16		Completed in 1911.	Spillway section.
47	[illegible]						
	[illegible] section	27	[illegible]	[illegible]	Fair	Successful since 1925.	[illegible] section.
	Spillway section	27	[illegible]	[illegible]	Good		Spillway section.
48	[illegible] Dam	[illegible]	[illegible]	[illegible]	[illegible]	[illegible] about 1922.	
49	[illegible] Dam	20	100	[illegible]	Uncertain	Probably in successful use since 1887.	Cutoff wall goes to fairly good hardpan at early [illegible] dam.
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	Good	Completed in [illegible].	[illegible] section.
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]		Completed in [illegible].	[illegible] section.
[illegible]	[illegible], New York	[illegible]	[illegible]	[illegible]	Good	Completed in [illegible].	Spillway section.
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	Good	Completed in [illegible].	Spillway section.
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	Good	Completed in [illegible].	Spillway section.
[illegible]	[illegible], Michigan	[illegible]	[illegible]	[illegible]	[illegible]	Completed in [illegible].	Spillway section.
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	Completed in [illegible].	Spillway section.
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	Completed in [illegible].	
[illegible]	[illegible], Ohio	[illegible]	[illegible]	[illegible]	[illegible]	Completed in [illegible].	Spillway section.
[illegible]	[illegible], Conn.	[illegible]	[illegible]	[illegible]	[illegible]	Completed in [illegible].	Spillway section.
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	Completed in [illegible].	Spillway section.
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	Good	Completed in [illegible].	Spillway section.
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	Good	Completed in [illegible].	Spillway section.
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	Completed in [illegible].	[illegible] spillway section.
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	Completed in [illegible].	Spillway section.
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	Completed in [illegible].	[illegible] section.
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	Completed in [illegible].	[illegible] section.
[illegible]	[illegible], Ohio	[illegible]	[illegible]	[illegible]	[illegible]	Completed in [illegible].	[illegible] section.
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	Completed in [illegible].	Spillway section.
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	Completed in [illegible].	Spillway section.
[illegible]	[illegible] Plant	[illegible]	[illegible]	[illegible]	Good	Built in [illegible].	Spillway section; creep includes downstream section. Creep does not include downstream section.

[illegible] App. III, Table No. [illegible]

Name	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	Performance	Remarks
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	Failed [illegible].	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	Satisfactory since [illegible].	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	Built in [illegible].	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	Completed in [illegible].	Spillway section.

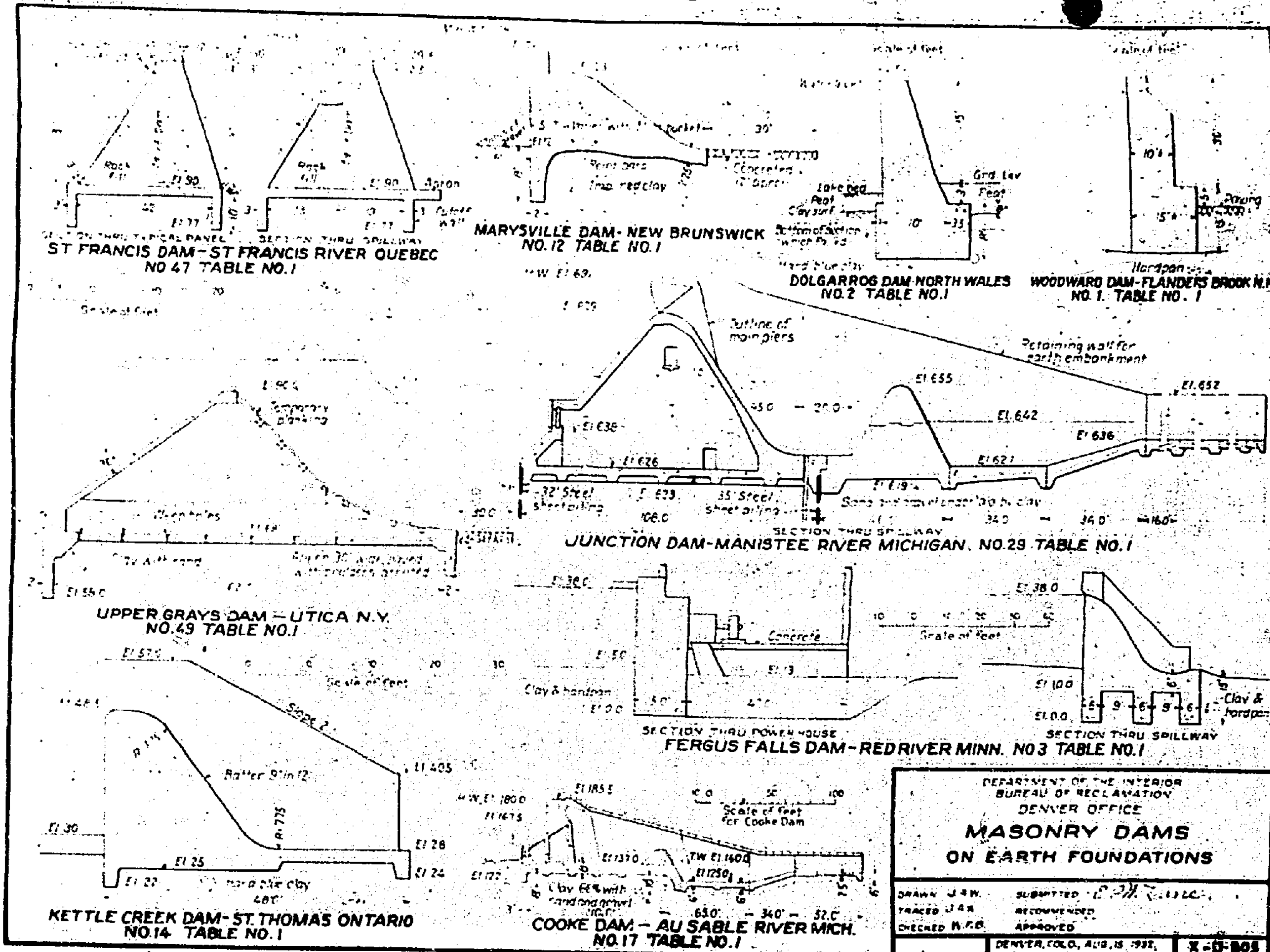
No.	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	[illegible]								
1	[illegible] (a) [illegible] (b) [illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]		[illegible]
	[illegible]								
11	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
12	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]		
13	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	
14	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]		
15	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
16	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]		
17	[illegible]	[illegible]	170	[illegible]	112	10.7	[illegible]	Satisfactory since 1922.	
18	[illegible] No. 1, [illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	Satisfactory since 1917.	
19	[illegible] (1) [illegible] (2) [illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
20	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	Satisfactory since 1912.	[illegible]
21	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	Completed in [illegible]	[illegible]
22	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	Completed in [illegible]	[illegible]
23	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	Completed in 1907.	[illegible]
24	[illegible]	[illegible]	[illegible]	[illegible]	41.9	[illegible]	[illegible]	Completed in 1919.	[illegible]
25	[illegible]	11	[illegible]	[illegible]	[illegible]	19	[illegible]	Built in 1932.	[illegible]

Name	Data	History	Remarks
[illegible]	[illegible]	Built in 1901.	[illegible]
[illegible]	[illegible]	Satisfactory for 40 years.	[illegible]
[illegible]	[illegible]	Completed in 1906.	[illegible]
[illegible]	[illegible]	[illegible]	Failure [illegible]
[illegible]	[illegible]	Completed in 1908	[illegible]
[illegible]	[illegible]	Built in 1890.	[illegible]
[illegible]	[illegible]	Reconstructed about 1900.	[illegible]
[illegible]	[illegible]	Completed in 1912	[illegible]
[illegible]	[illegible]	Rebuilt in 1900.	Section as rebuilt in 1900.
[illegible]	[illegible]	Built about 1906.	[illegible]
[illegible]	[illegible]	Built about 1907.	[illegible]
[illegible]	[illegible]	Built about 1907.	[illegible]
[illegible]	[illegible]	Built about 1915.	[illegible]
[illegible]	[illegible]	Satisfactory since about 1917.	[illegible]
[illegible]	[illegible]	Completed in 1921.	[illegible]
[illegible]	[illegible]	[illegible] completed in 1927.	[illegible]
[illegible]	[illegible]	Built in 1907.	[illegible]
[illegible]	[illegible]	Completed in 1907.	[illegible]
[illegible]	[illegible]	Completed in 1907.	Maximum head 11 feet.
[illegible]	[illegible]	Built about 1916.	[illegible]
[illegible]	[illegible]	Built about 1916	[illegible]
[illegible]	[illegible]	Satisfactory since 1900.	Failed, probably due to [illegible]
[illegible]	[illegible]	About 10 years.	[illegible]
[illegible]	[illegible]	Satisfactory at least 7 years	[illegible]
[illegible]	[illegible]	Satisfactory since 1908.	Used only in floods.
[illegible]	[illegible]	Not used yet.	[illegible]
[illegible]	[illegible]	In service since 1906.	[illegible]
[illegible]	[illegible]	Built in 1907.	[illegible]
[illegible]	[illegible]	Built in 1907.	[illegible]
[illegible]	[illegible]	Built in 1906.	[illegible]
[illegible]	[illegible]	Completed in 1914.	Grillage section.
[illegible]	[illegible]	Completed in 1903.	Grillage section.
[illegible]	[illegible]	Completed in 1906.	[illegible] section.
[illegible]	[illegible]	Completed in 1917.	Grillage section.
[illegible]	[illegible]	Completed in 1911.	Grillage section.
[illegible]	[illegible]	Completed in 1912.	Grillage section.

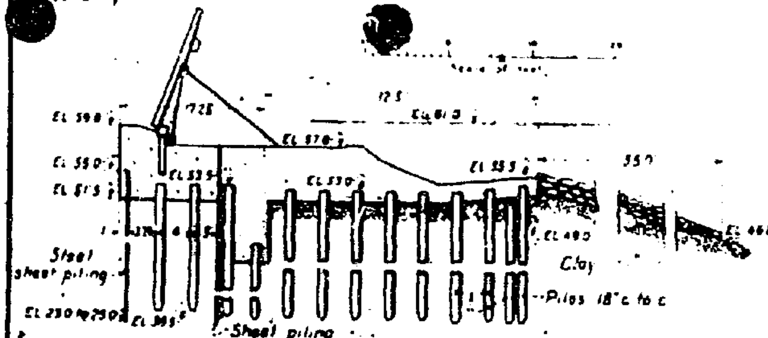
APPENDIX IV

PLATES I TO XXII

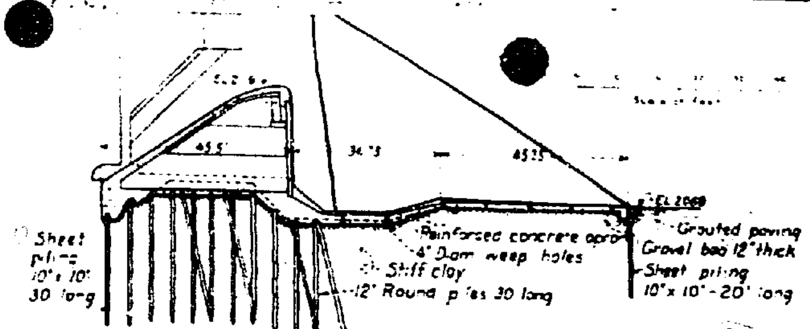
SECTIONS OF MASONRY DAMS ON EARTH FOUNDATIONS



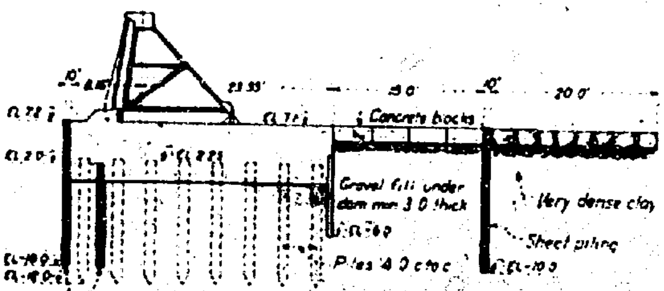
SECTION THRU TYPICAL PANEL
SECTION THRU SPILLWAY
ST FRANCIS DAM - ST FRANCIS RIVER QUEBEC
NO 47 TABLE NO.1
MARYSVILLE DAM - NEW BRUNSWICK
NO.12 TABLE NO.1
DOLGARROG DAM-NORTH WALES
NO.2 TABLE NO.1
WOODWARD DAM-FLANDERS BROOK N.H.
NO.1. TABLE NO. 1
Outline of main piers
Retaining wall for earth embankment
SECTION THRU SPILLWAY
JUNCTION DAM-MANISTEE RIVER MICHIGAN. NO.29 TABLE NO.1
UPPER GRAYS DAM - UTICA N.Y.
NO.49 TABLE NO.1
SECTION THRU POWER HOUSE
SECTION THRU SPILLWAY
FERGUS FALLS DAM - RED RIVER MINN. NO 3 TABLE NO.1
KETTLE CREEK DAM - ST. THOMAS ONTARIO
NO.14 TABLE NO.1
COOKE DAM - AU SABLE RIVER MICH.
NO.17 TABLE NO.1
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
DENVER OFFICE
MASONRY DAMS
ON EARTH FOUNDATIONS
DRAWN J.A.W.
TRACED J.A.W.
CHECKED W.G.B.
SUBMITTED
RECOMMENDED
APPROVED
DENVER, COLO., AUG. 15, 1932.



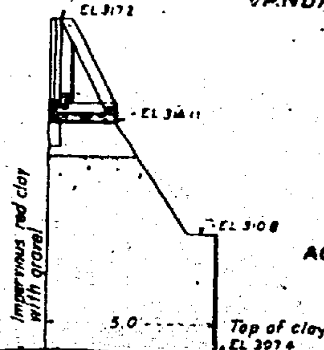
SECTION THRU NAVIGABLE PASS
OUACHITA RIVER DAM (NO 8)-NO.26 TABLE NO.1



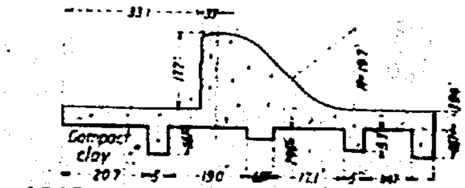
VANDALIA DAM-NO.48 TABLE NO.1



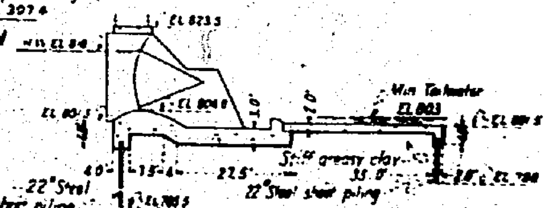
OUACHITA RIVER DAM (NO.2)-NO.24 TABLE NO.1



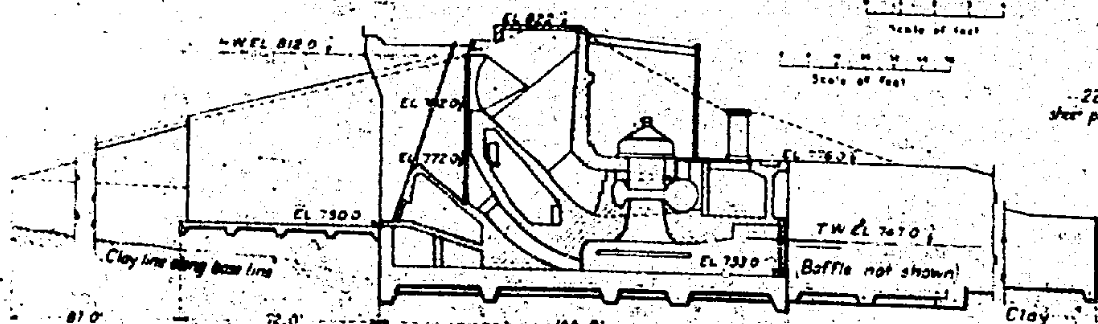
NORTH DIVERSION
ORLAND PROJECT
NO.21 TABLE NO.1



ACATLAN DAM-NO.32 TABLE NO.1



SHAWANO DAM-NO.19 TABLE NO.1



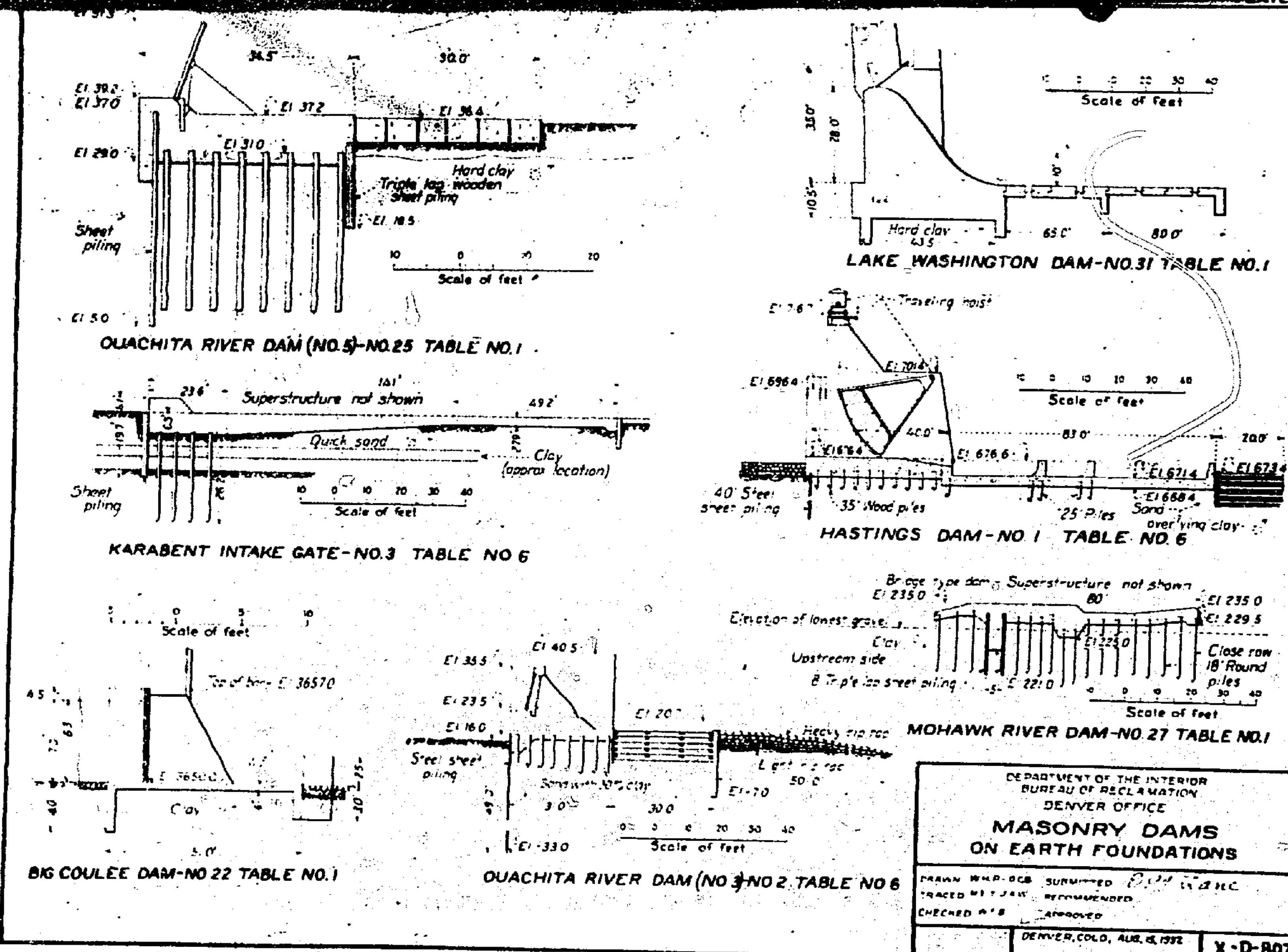
SECTION THRU POWER HOUSE
HODENPHYL (COUNTY LINE) DAM-NO.30 TABLE NO.1

DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
DENVER OFFICE

MASONRY DAMS
ON EARTH FOUNDATIONS

DRAWN W.K.R.-O.C.B. SUBMITTED
TRACED J.R.W. RECOMMENDED
CHECKED R.F.B. APPROVED

DENVER, COLO. AUG. 15, 1932. X-D-806


OUACHITA RIVER DAM (NO.5)-NO.25 TABLE NO.1
El 39.2
El 37.0
El 37.2
El 36.4
El 31.0
El 29.0
Hard clay
Triple lap wooden sheet piling
El 18.5
Sheet piling
El 5.0
Scale of feet
KARABENT INTAKE GATE-NO.3 TABLE NO 6
Superstructure not shown
Quick sand
Clay (approx location)
Sheet piling
Scale of feet
BIG COULEE DAM-NO 22 TABLE NO.1
Scale of feet
Clay
OUACHITA RIVER DAM (NO 3)-NO 2 TABLE NO 6
El 40.5
El 35.5
El 23.5
El 16.0
Steel sheet piling
El -33.0
Scale of feet
LAKE WASHINGTON DAM-NO.31 TABLE NO.1
Scale of feet
Hard clay
HASTINGS DAM-NO 1 TABLE NO. 6
Traveling hoist
El 701.4
El 696.4
El 676.6
El 671.4
El 673.4
El 668.4
40' Steel sheet piling
35' Wood piles
25' Piles
Sand overlying clay
Scale of feet
MOHAWK RIVER DAM-NO.27 TABLE NO.1
Bridge type dam; Superstructure not shown
El 235.0
El 229.5
El 225.0
El 221.0
Elevation of lowest gravel
Clay
Upstream side
Close row 18' Round piles
Scale of feet
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
DENVER OFFICE
MASONRY DAMS
ON EARTH FOUNDATIONS
DENVER, COLO, AUG. 15, 1932
X-D-807

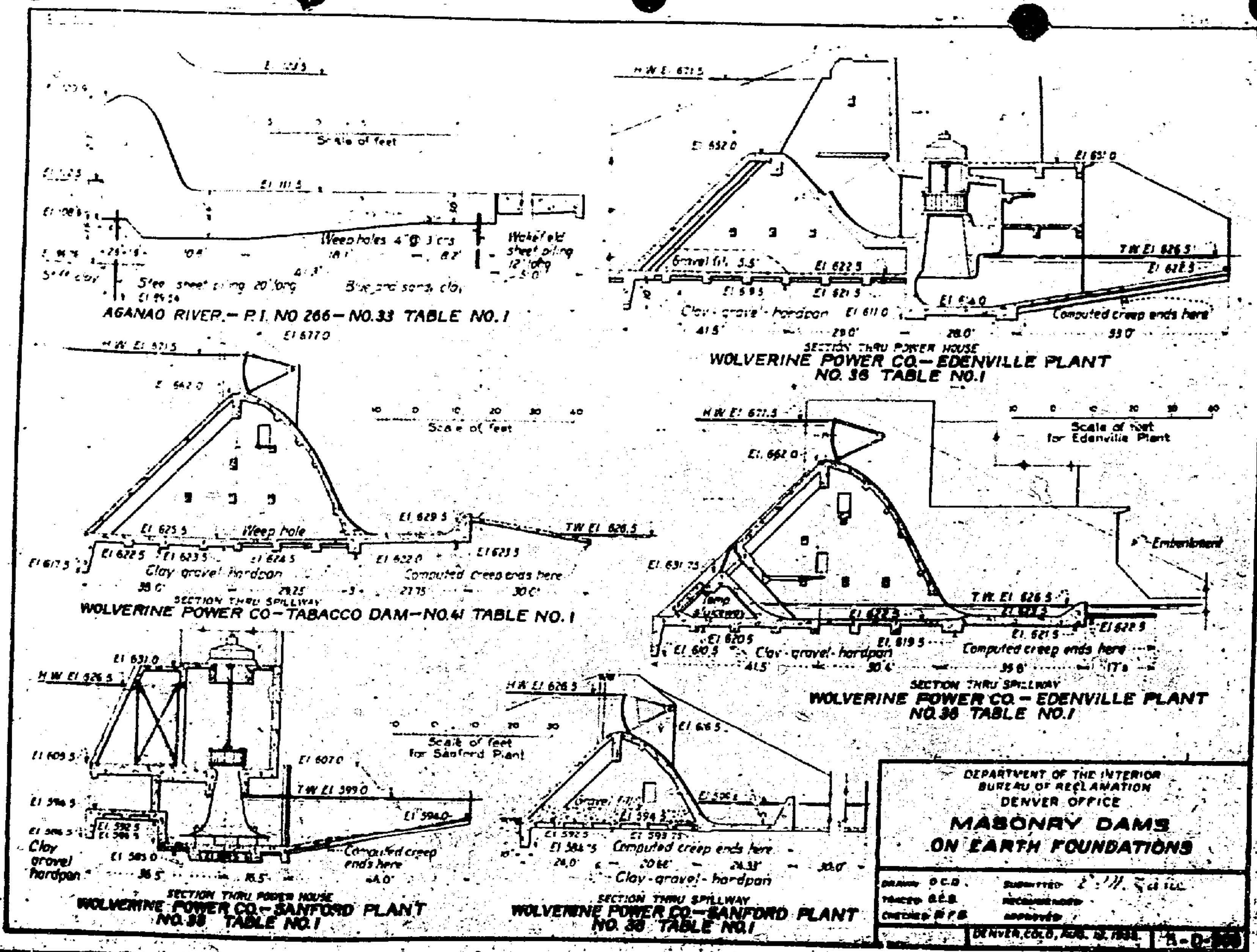
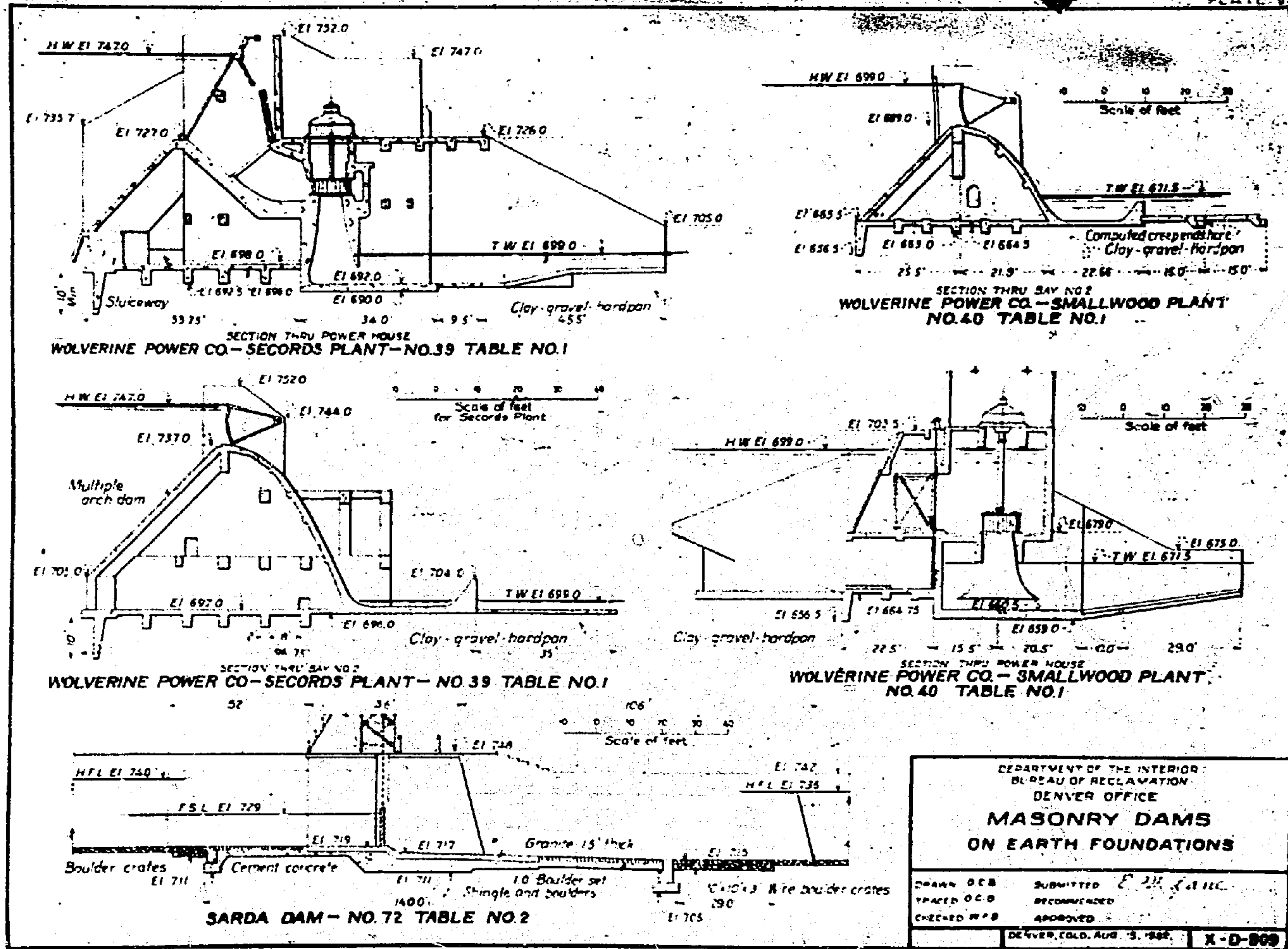
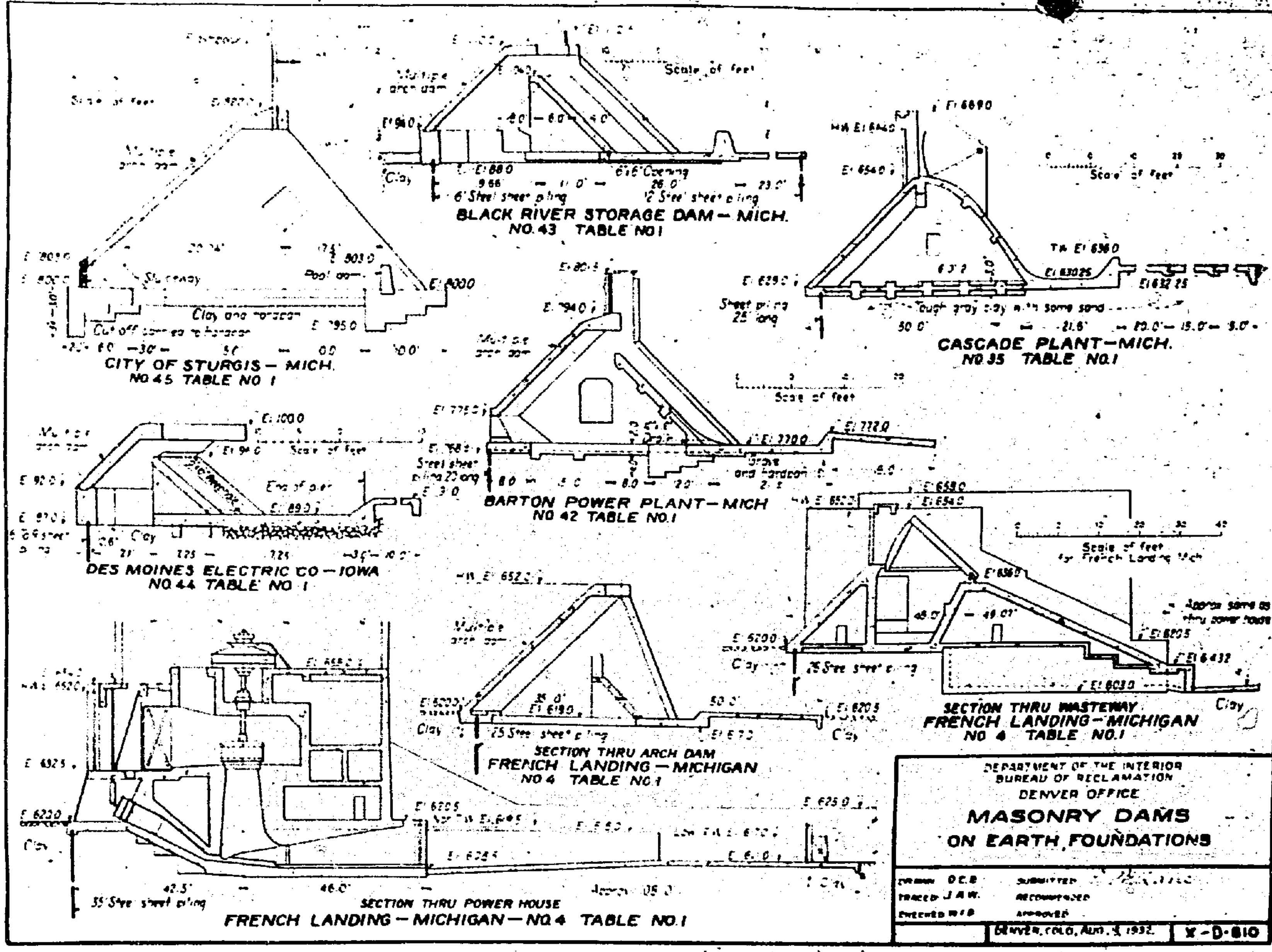
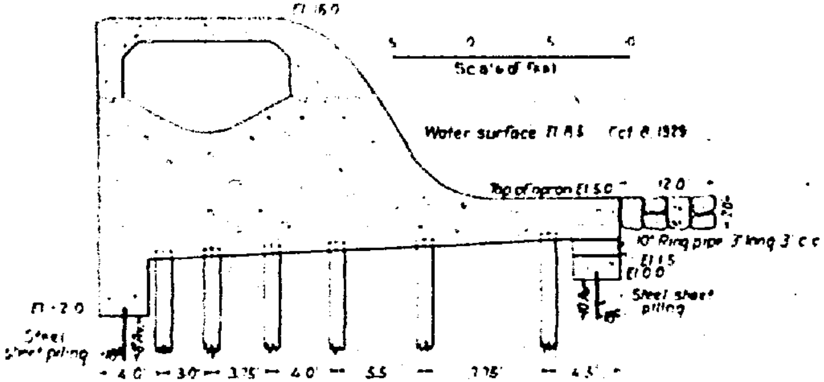

AGANAO RIVER.– P.I. NO 266 – NO.33 TABLE NO.1
Weep holes 4" @ 3 ctrs
Wakefield sheet piling 12' long
Steel sheet piling 20' long
Scale of feet
WOLVERINE POWER CO. – EDENVILLE PLANT NO. 36 TABLE NO.1
SECTION THRU POWER HOUSE
Clay-gravel-hardpan
Computed creep ends here
WOLVERINE POWER CO – TABACCO DAM – NO.41 TABLE NO.1
SECTION THRU SPILLWAY
Weep hole
Scale of feet for Edenville Plant
Embankment
WOLVERINE POWER CO. – EDENVILLE PLANT NO.36 TABLE NO.1
WOLVERINE POWER CO. – SANFORD PLANT NO.38 TABLE NO.1
Scale of feet for Sanford Plant
Clay gravel hardpan
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
DENVER OFFICE
MASONRY DAMS
ON EARTH FOUNDATIONS
DENVER, COLO.

PLATE V

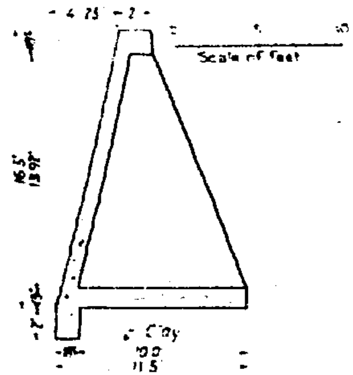


H W El 742.0
El 752.0
El 747.0
El 735.7
El 727.0
El 726.0
El 705.0
T W El 699.0
El 698.0
El 692.0
El 690.0
El 692.5
El 696.0
Sluiceway
Clay-gravel-hardpan
33.75'
34.0'
9.5'
45.5'
SECTION THRU POWER HOUSE
WOLVERINE POWER CO.—SECORDS PLANT—NO.39 TABLE NO.1
H W El 699.0
El 689.0
Scale of feet
T W El 671.5
El 665.5
El 656.5
El 663.0
El 664.5
Computed creep endshere
Clay-gravel-hardpan
25.5'
21.9'
22.66
15.0'
15.0'
SECTION THRU BAY NO.2
WOLVERINE POWER CO.—SMALLWOOD PLANT NO.40 TABLE NO.1
H W El 742.0
El 752.0
El 744.0
El 737.0
Scale of feet for Secords Plant
Multiple arch dam
El 701.0
El 704.0
T W El 699.0
El 697.0
El 696.0
Clay-gravel-hardpan
96.75'
35'
SECTION THRU BAY NO.2
WOLVERINE POWER CO—SECORDS PLANT—NO.39 TABLE NO.1
El 703.5
H W El 699.0
Scale of feet
El 679.0
El 675.0
T W El 671.5
El 656.5
El 664.75
El 660.5
El 659.0
Clay-gravel-hardpan
22.5'
15.5'
20.5'
12.0'
29.0'
SECTION THRU POWER HOUSE
WOLVERINE POWER CO.—SMALLWOOD PLANT NO.40 TABLE NO.1
52'
36'
106'
Scale of feet
El 748
H F L El 740
El 742
H F L El 735
F S L El 729
El 719
El 717
Granite 15' thick
El 715
Boulder crates
El 711
Cement concrete
El 711
1.0 Boulder set
Shingle and boulders
1400'
Wire boulder crates
29.0'
El 705
SARDA DAM—NO.72 TABLE NO.2
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
DENVER OFFICE
MASONRY DAMS
ON EARTH FOUNDATIONS
DRAWN O.C.B.
TRACED O.C.B.
CHECKED W.F.B.
SUBMITTED
RECOMMENDED
APPROVED
DENVER, COLO. AUG. 5, 1936.
X-D-809

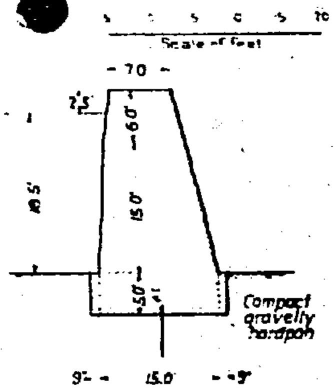

BLACK RIVER STORAGE DAM — MICH.
NO.43 TABLE NO.1
6' Steel sheet piling
12' Steel sheet piling
Scale of feet
CASCADE PLANT — MICH.
NO.35 TABLE NO.1
Tough gray clay with some sand
Sheet piling 25' long
CITY OF STURGIS — MICH.
NO.45 TABLE NO.1
Clay and hardpan
Cut off carried to hardpan
Pool dam
BARTON POWER PLANT — MICH
NO.42 TABLE NO.1
Steel sheet piling 22' long
Gravel and hardpan
DES MOINES ELECTRIC CO — IOWA
NO.44 TABLE NO.1
End of pier
SECTION THRU WASTEWAY
FRENCH LANDING — MICHIGAN
NO.4 TABLE NO.1
Scale of feet for French Landing Mich
25' Steel sheet piling
SECTION THRU ARCH DAM
FRENCH LANDING — MICHIGAN
NO.4 TABLE NO.1
SECTION THRU POWER HOUSE
FRENCH LANDING — MICHIGAN — NO.4 TABLE NO.1
35' Steel sheet piling
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
DENVER OFFICE
MASONRY DAMS
ON EARTH FOUNDATIONS
DRAWN D.C.B.
TRACED J.A.W.
CHECKED W.F.B.
SUBMITTED
RECOMMENDED
APPROVED
DENVER, COLO., AUG. 5, 1932.
X-D-810



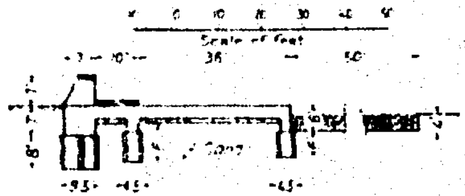
SCIOTO DAM - COLUMBUS OHIO



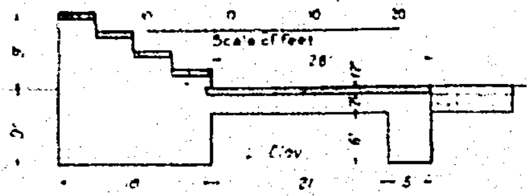
SCRANTON PA
NO.63 TABLE NO.1



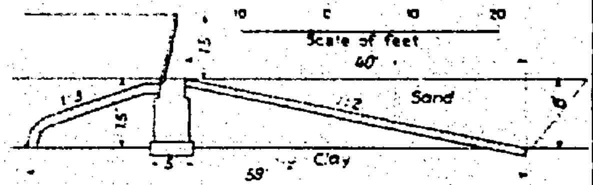
WHITING STREET DAM
HOLYOKE MASS.
NO. 20 TABLE NO.1



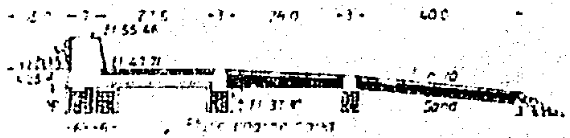
PALAR WEIR - INDIA



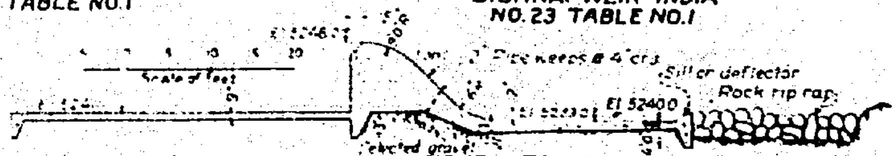
SHATIATOPE WEIR - INDIA
NO.15 TABLE NO.1



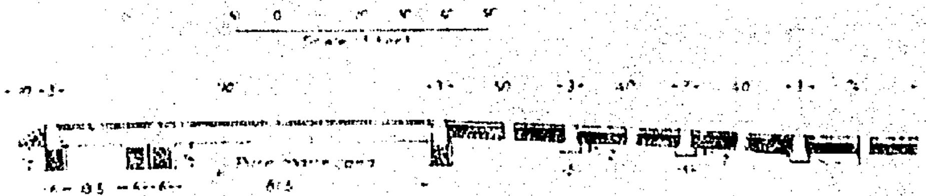
SIDHNAI WEIR - INDIA
NO.23 TABLE NO.1



BAITURNEE WEIR - INDIA



COCHITI DAM - N.M. NO.19 TABLE NO.3



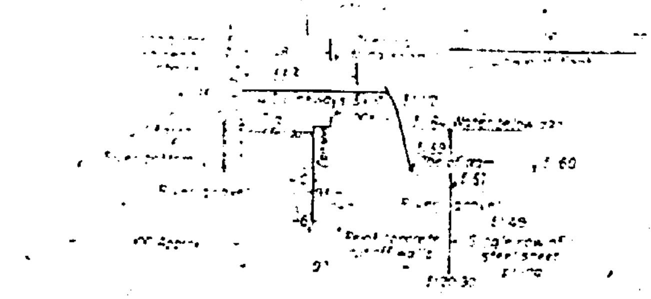
BAITURNEE WEIR - INDIA

DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
DENVER OFFICE

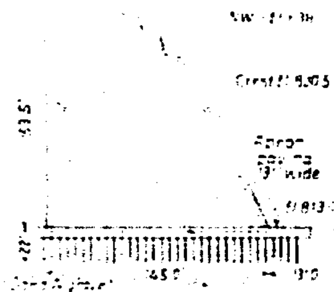
MASONRY DAMS
ON EARTH FOUNDATIONS

DRAWN	SUBMITTED
TRACED	RECOMMENDED
CHECKED	APPROVED

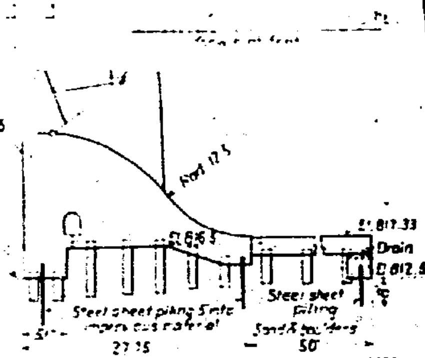
DENVER COLO. AUG. 15, 1938 — X-D-811



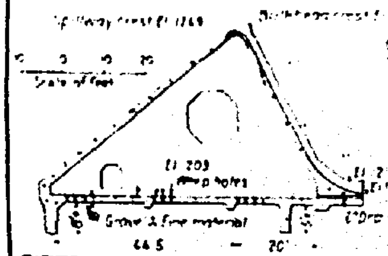
PORT ANGELES DAM - ELWHA RIVER - WASHINGTON
NO. 2 TABLE NO. 2



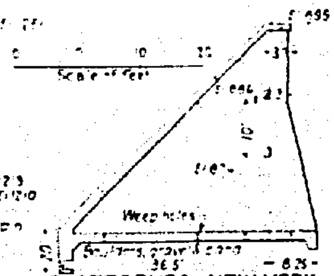
PUENTES DAM
GWADALANTIN RIVER - SPAIN
NO. 1 TABLE NO. 2



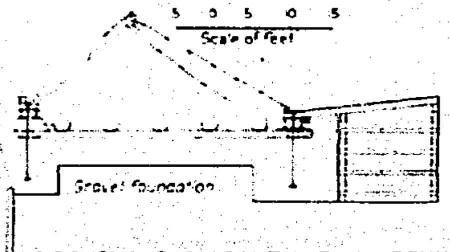
COON RAPIDS - MISSISSIPPI RIVER - MINN.
NO. 4 TABLE NO. 2



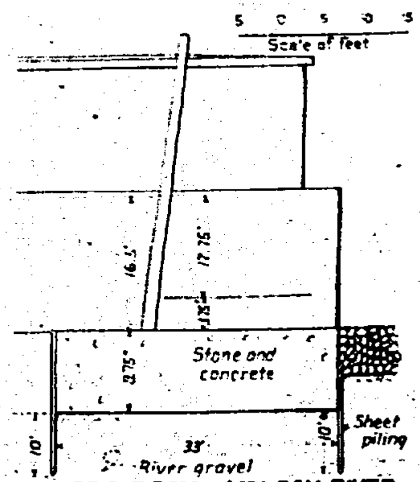
PITTSFIELD - WEST BROOK - MASS.
NO 3 TABLE NO. 2



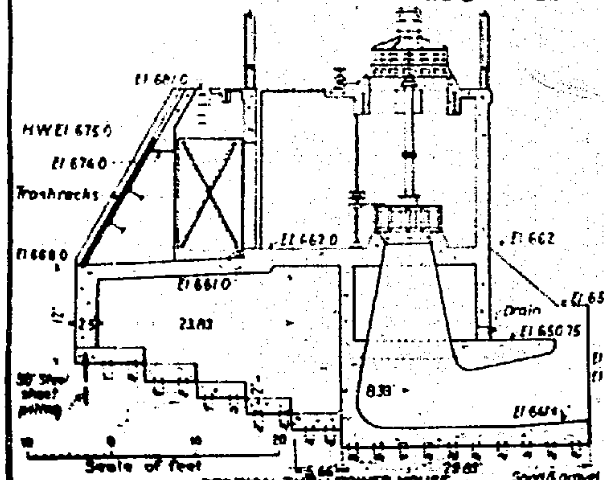
PLATTSBURG - NEW YORK
NO. 5 TABLE NO. 2



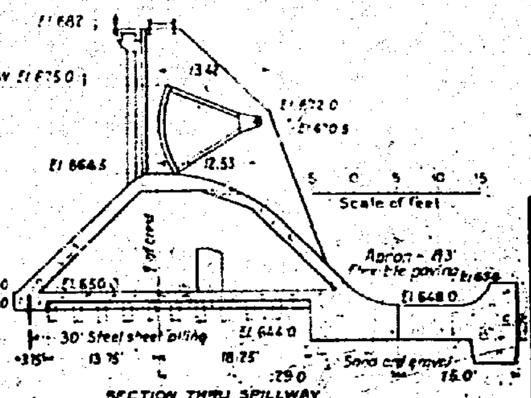
HERR ISLAND - ALLEGHENY RIVER - PENN.
NO. 31 TABLE NO. 2



MIROWITZ BRIDGE DAM - MOLDAU RIVER
BOHEMIA - NO. 32 TABLE NO. 2



SECTION THRU POWER HOUSE
PINHOOK - MAQUOKETA RIVER - IOWA
NO. 22 TABLE NO. 2

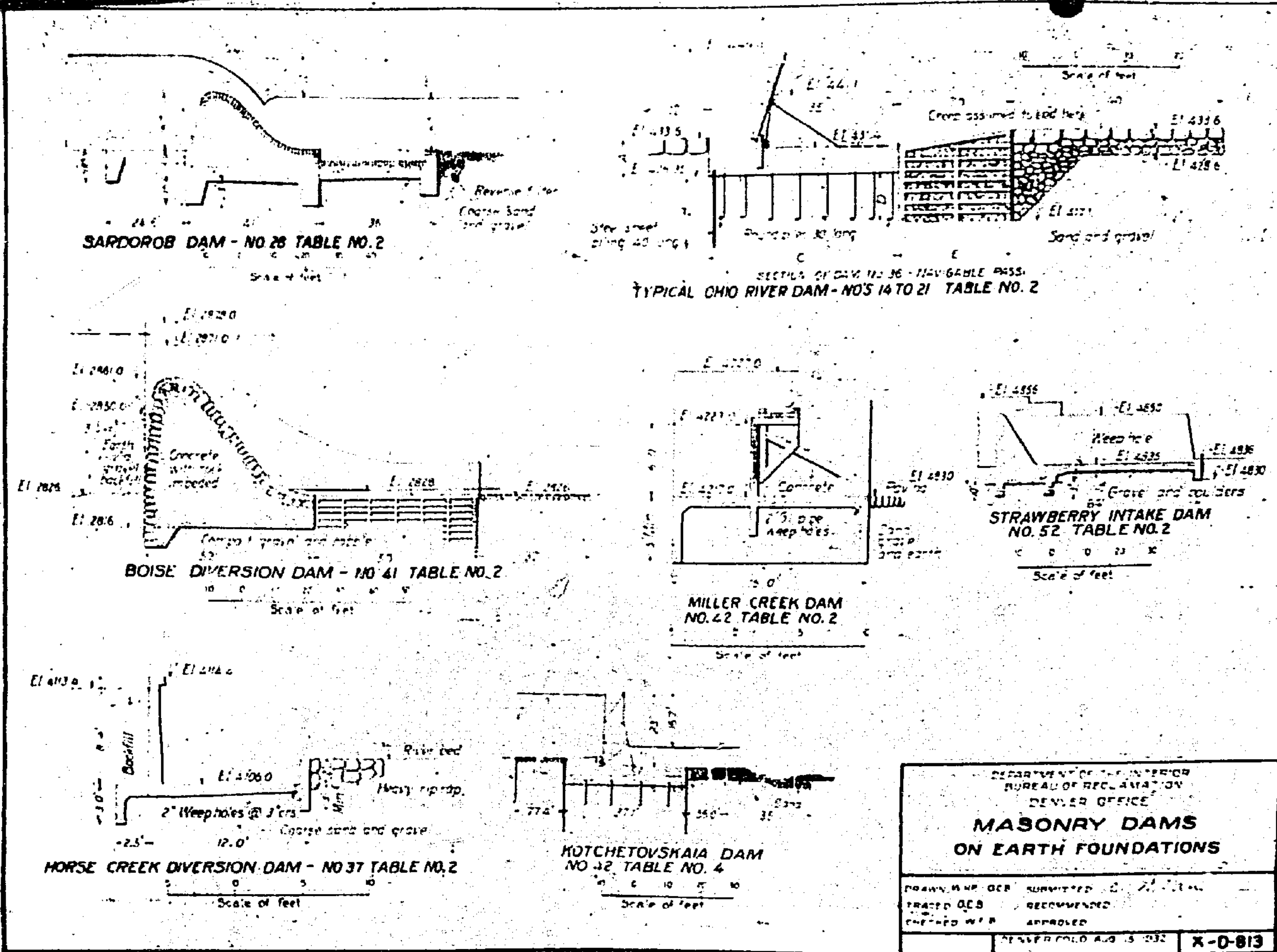


SECTION THRU SPILLWAY
PINHOOK - MAQUOKETA RIVER - IOWA
NO. 22 TABLE NO. 2

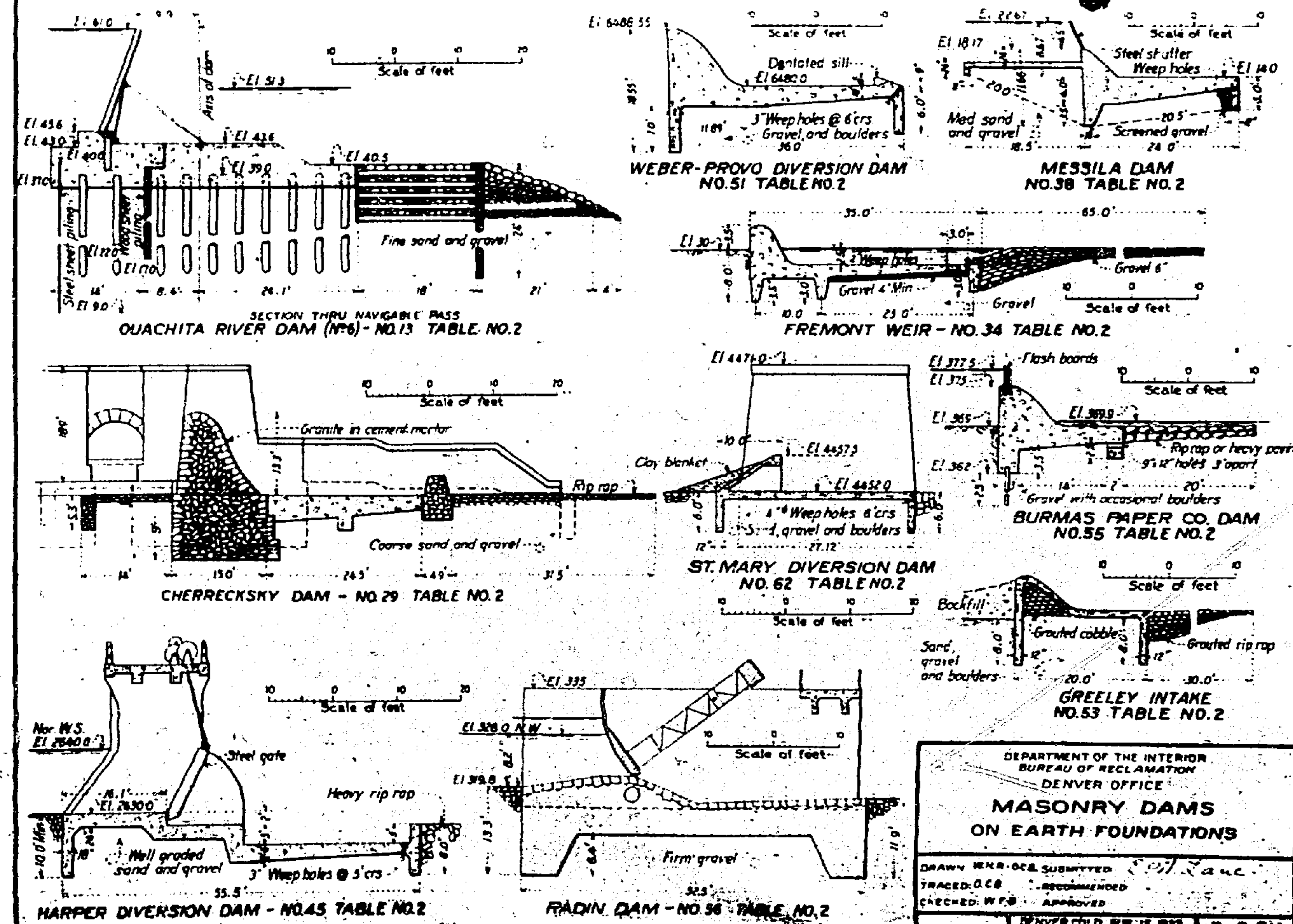
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
DENVER OFFICE

MASONRY DAMS
ON EARTH FOUNDATIONS

DRAWN: O.C.S.	SUBMITTED: E. W. [illegible]
TRACED: U.A.H.	RECOMMENDED
CHECKED: H.W.B.	APPROVED
	DENVER, COLO., AUG. 15, 1933.
	X-D-212



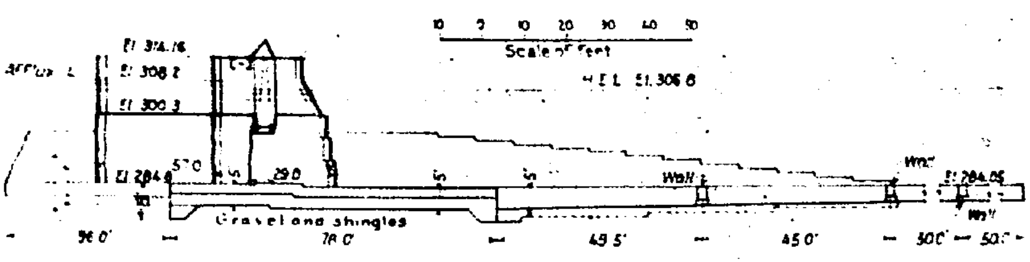
SARDOROB DAM - NO 28 TABLE NO. 2
Scale of feet
Reverse filter
Coarse sand and gravel
TYPICAL OHIO RIVER DAM - NO'S 14 TO 21 TABLE NO. 2
Scale of feet
Crest assumed to end here
El 433.6
El 428.6
El 441.1
El 433.5
Sand and gravel
Round piles 30' long
Steel sheet piling 40' long
BOISE DIVERSION DAM - NO 41 TABLE NO. 2
Scale of feet
El 2929.0
El 2861.0
El 2826
El 2816
Concrete with rocks imbedded
Earth gravel backfill
Compact gravel and cobble
MILLER CREEK DAM
NO. 42 TABLE NO. 2
Scale of feet
Concrete
Weep holes
El 4830
Sand gravel and earth
STRAWBERRY INTAKE DAM
NO. 52 TABLE NO. 2
Scale of feet
El 4856
El 4850
Weep hole
El 4836
El 4835
El 4830
Gravel and boulders
HORSE CREEK DIVERSION DAM - NO 37 TABLE NO. 2
Scale of feet
El 4106.0
Backfill
2" Weepholes @ 3' crs.
Coarse sand and gravel
Heavy riprap
River bed
12.0'
KOTCHETOVSKAIA DAM
NO 42 TABLE NO. 4
Scale of feet
Sand
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
DENVER OFFICE
MASONRY DAMS
ON EARTH FOUNDATIONS
DRAWN W.H.F. OCB
TRACED OCB
CHECKED W.F.B.
SUBMITTED
RECOMMENDED
APPROVED
DENVER COLO AUG 15 1932
X-D-813



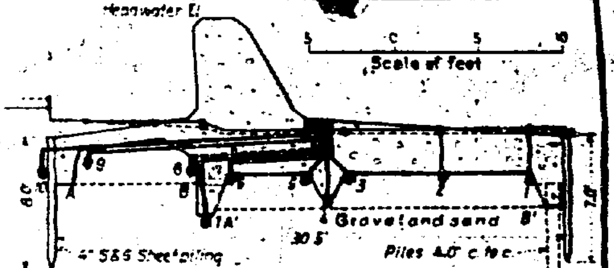
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
DENVER OFFICE

MASONRY DAMS ON EARTH FOUNDATIONS

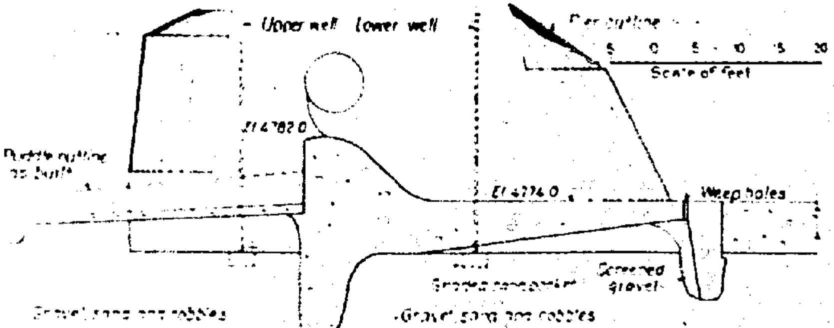
DRAWN: W.K.B.-O.C.B.	SUBMITTED:
TRACED: O.C.B.	RECOMMENDED:
CHECKED: W.F.B.	APPROVED:
DENVER, COLO., AUG. 15, 1932.	X-D-814



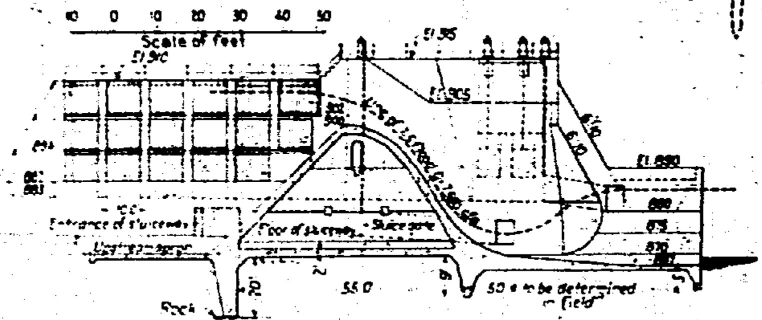
MANDALAY UNDER SLUICES – MADAYA RIVER · BURMA NO. 39 TABLE NO. 2



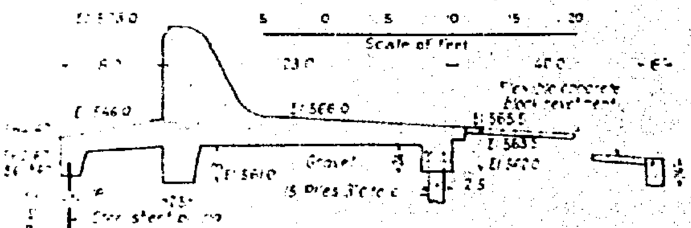
ISLAND PARK · MIAMI RIVER · OHIO
NO. 25 TABLE NO. 2



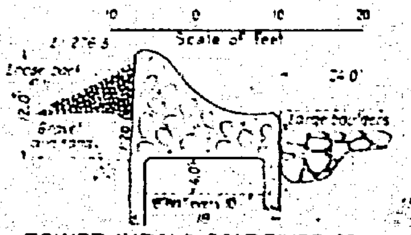
GRAND VALLEY · COLORADO RIVER · COLO. NO. 40 TABLE NO. 2



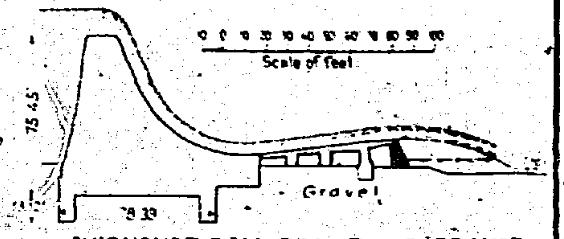
UPPER ALAMEDA CREEK DAM – SAN FRANCISCO CALIF.



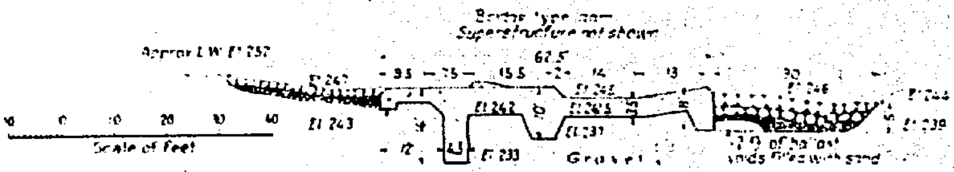
HAMILTON · MIAMI RIVER · OHIO
NO 33 TABLE NO. 2



POWER INTAKE · SALT RIVER · ARIZ.
NO. TABLE NO.



AVIGNONET DAM · DRAC RIVER · FRANCE
NO. 54 TABLE NO. 2



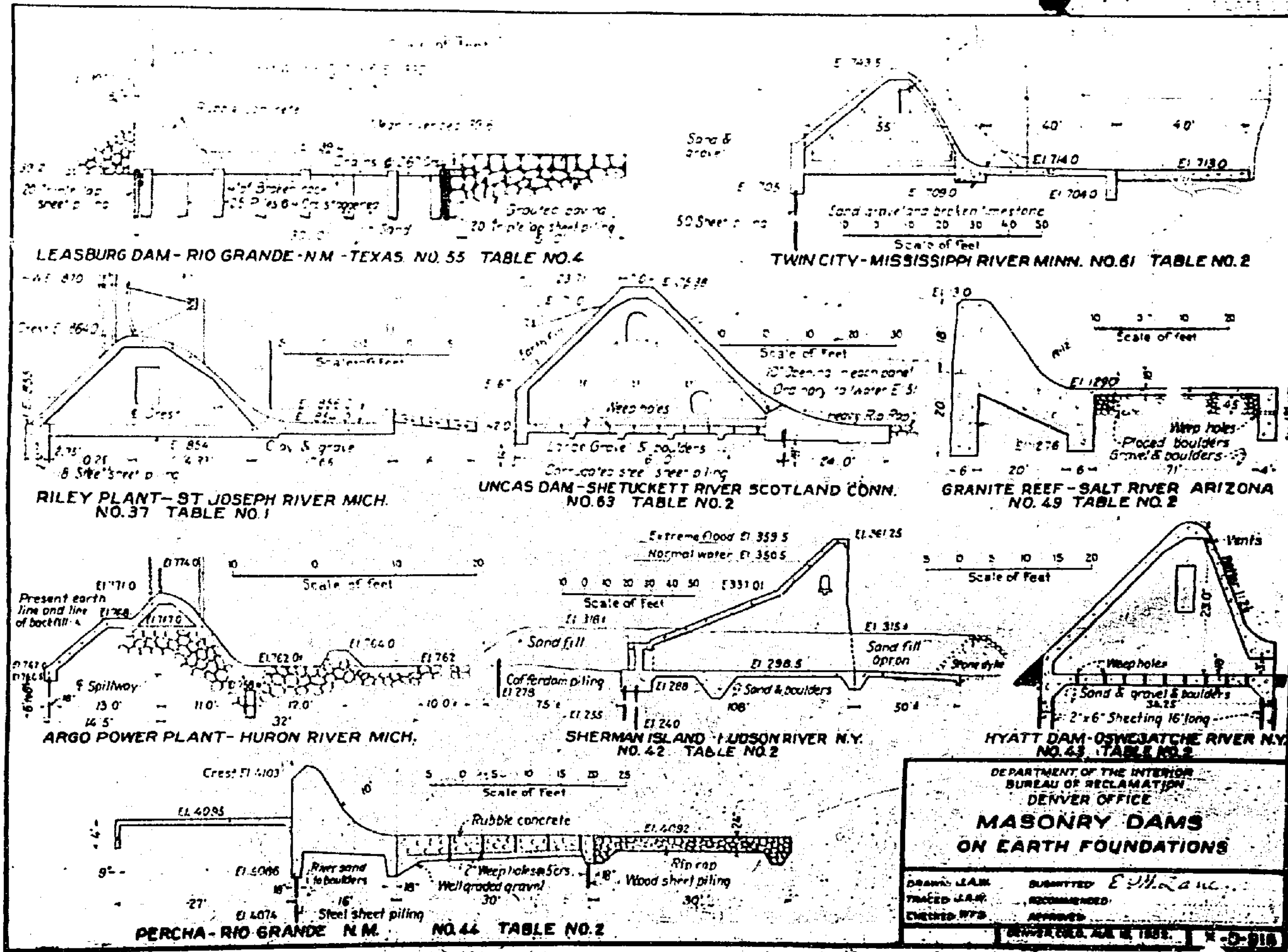
AMSTERDAM · NO. 7 · MOHAWK RIVER · N.Y. NO. 35 TABLE NO 2

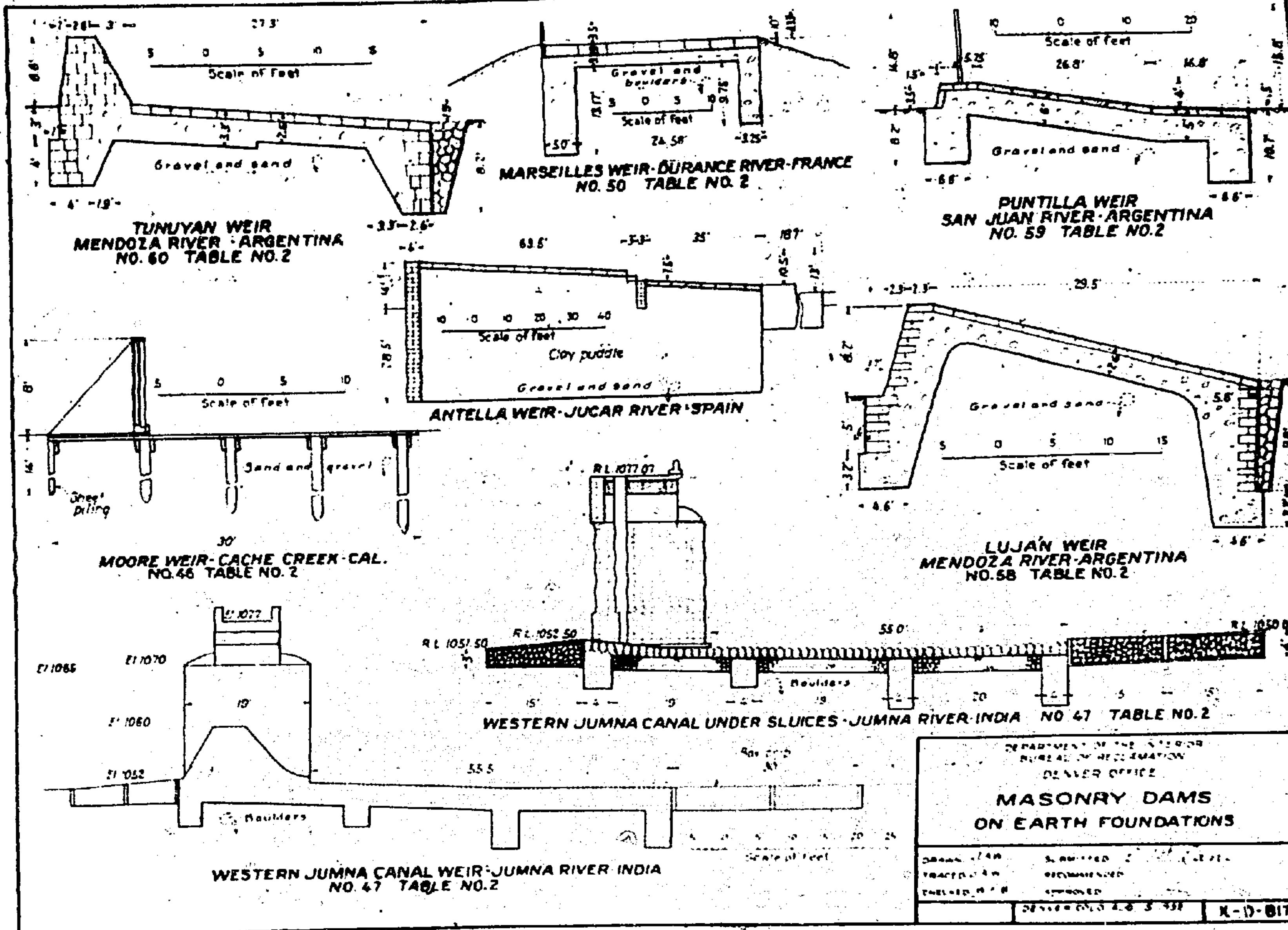
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
DENVER OFFICE

MASONRY DAMS
ON EARTH FOUNDATIONS

DRAWN J.A.W. SUBMITTED C.H. Lane
TRACED J.A.W. RECOMMENDED
CHECKED W.F.R. APPROVED

DENVER COLO., AUG. 13, 1932. X-D-815


LEASBURG DAM - RIO GRANDE - N.M. - TEXAS. NO. 55 TABLE NO. 4
TWIN CITY - MISSISSIPPI RIVER MINN. NO. 61 TABLE NO. 2
RILEY PLANT - ST JOSEPH RIVER MICH. NO. 37 TABLE NO. 1
UNCAS DAM - SHETUCKETT RIVER SCOTLAND CONN. NO. 63 TABLE NO. 2
GRANITE REEF - SALT RIVER ARIZONA NO. 49 TABLE NO. 2
ARGO POWER PLANT - HURON RIVER MICH.
SHERMAN ISLAND - HUDSON RIVER N.Y. NO. 42 TABLE NO. 2
HYATT DAM - OSWEGATCHIE RIVER N.Y. NO. 43 TABLE NO. 2
PERCHA - RIO GRANDE N.M. NO. 44 TABLE NO. 2
Scale of Feet
Weep holes
Sand & gravel
Rubble concrete
Rip rap
Wood sheet piling
Steel sheet piling
Well graded gravel
Cofferdam piling
Sand fill
Sand & boulders
Sand fill apron
Placed boulders
Gravel & boulders
Vents
Present earth line and line of backfill
℄ Spillway
Crest El. 4103
Extreme flood El. 359.5
Normal water El. 350.5
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
DENVER OFFICE
MASONRY DAMS
ON EARTH FOUNDATIONS
DRAWN J.A.M.
TRACED J.A.M.
CHECKED W.F.S.
SUBMITTED
RECOMMENDED
APPROVED



DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
DENVER OFFICE

MASONRY DAMS
ON EARTH FOUNDATIONS

X-D-817

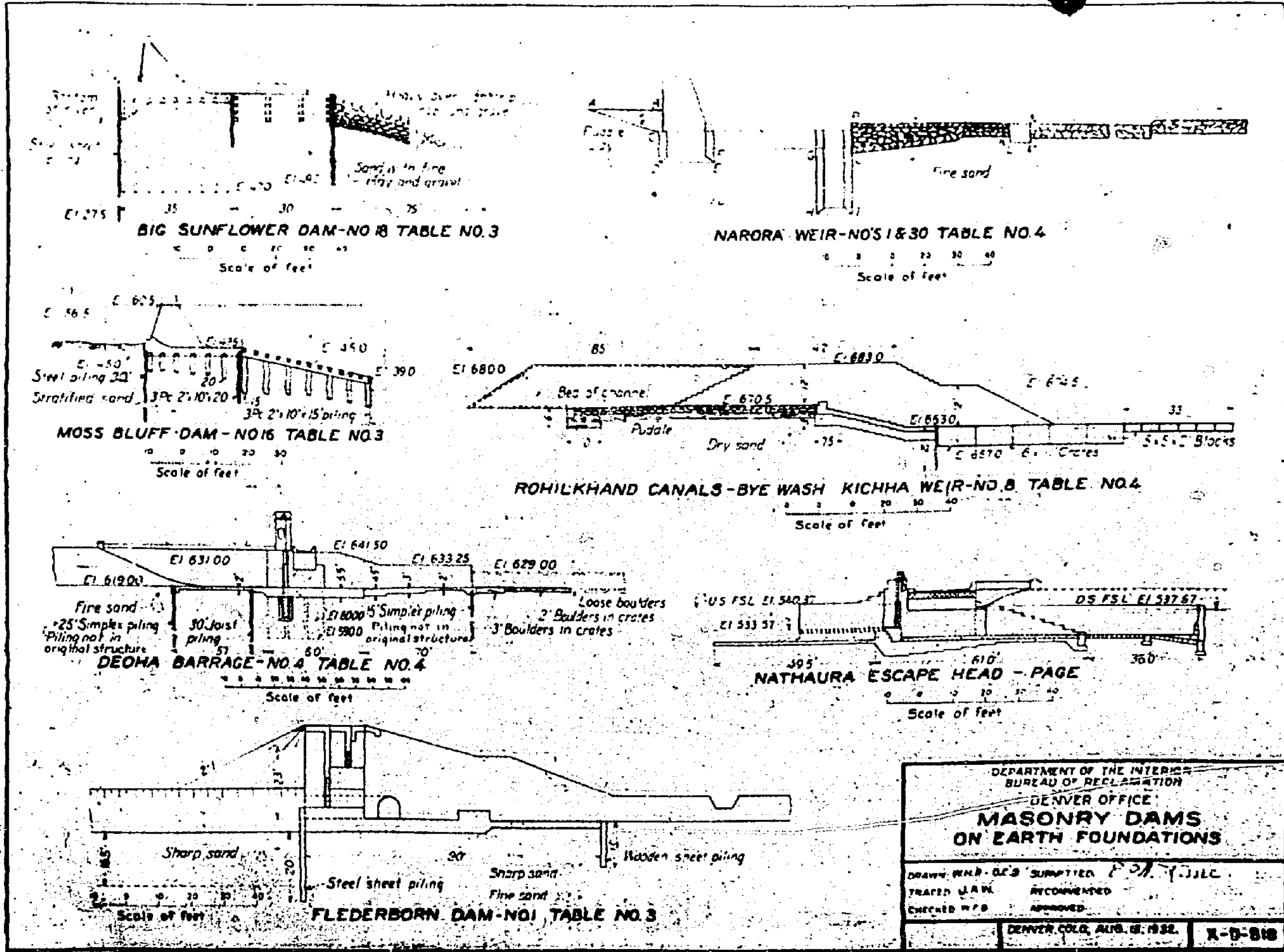
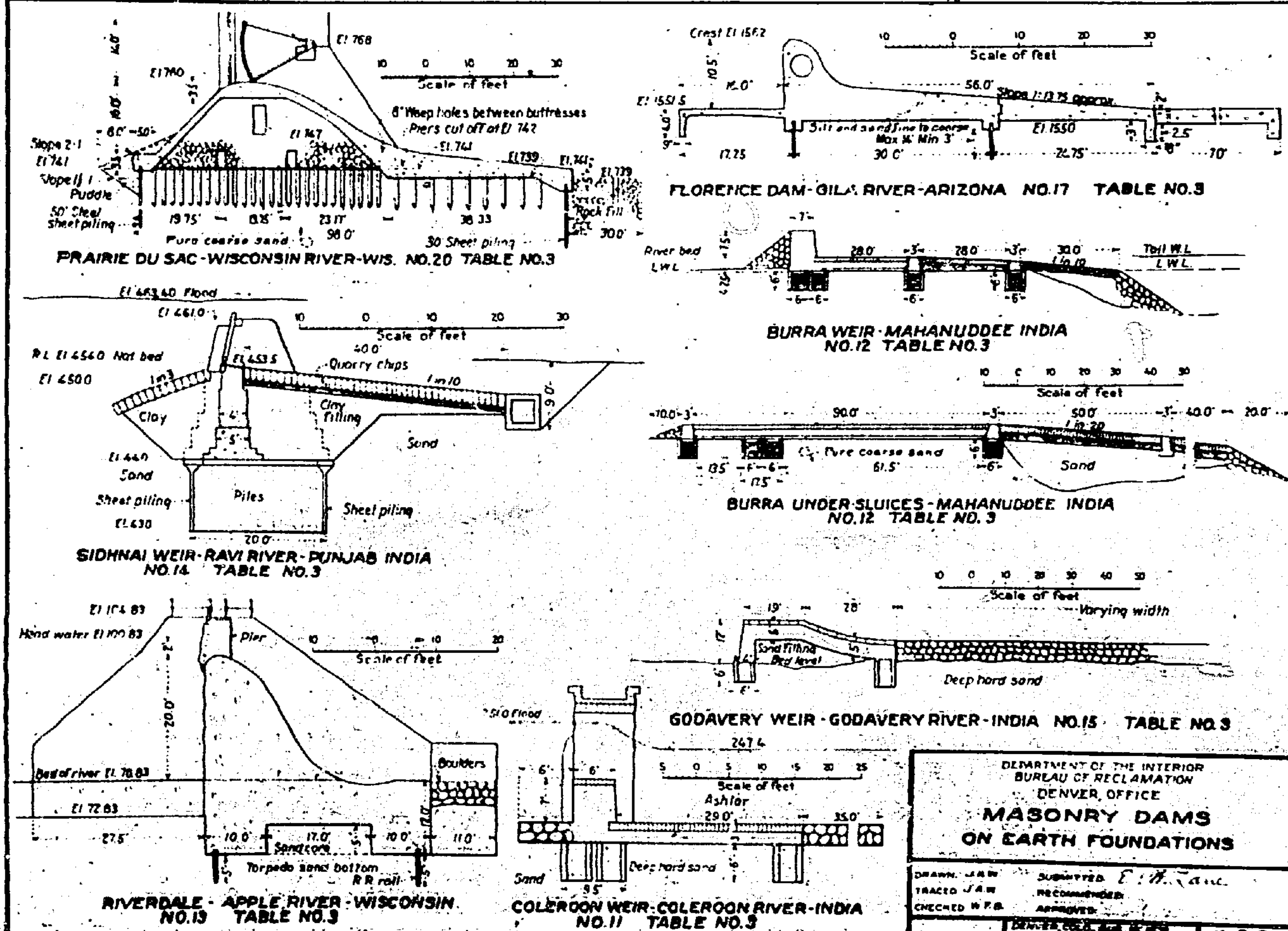
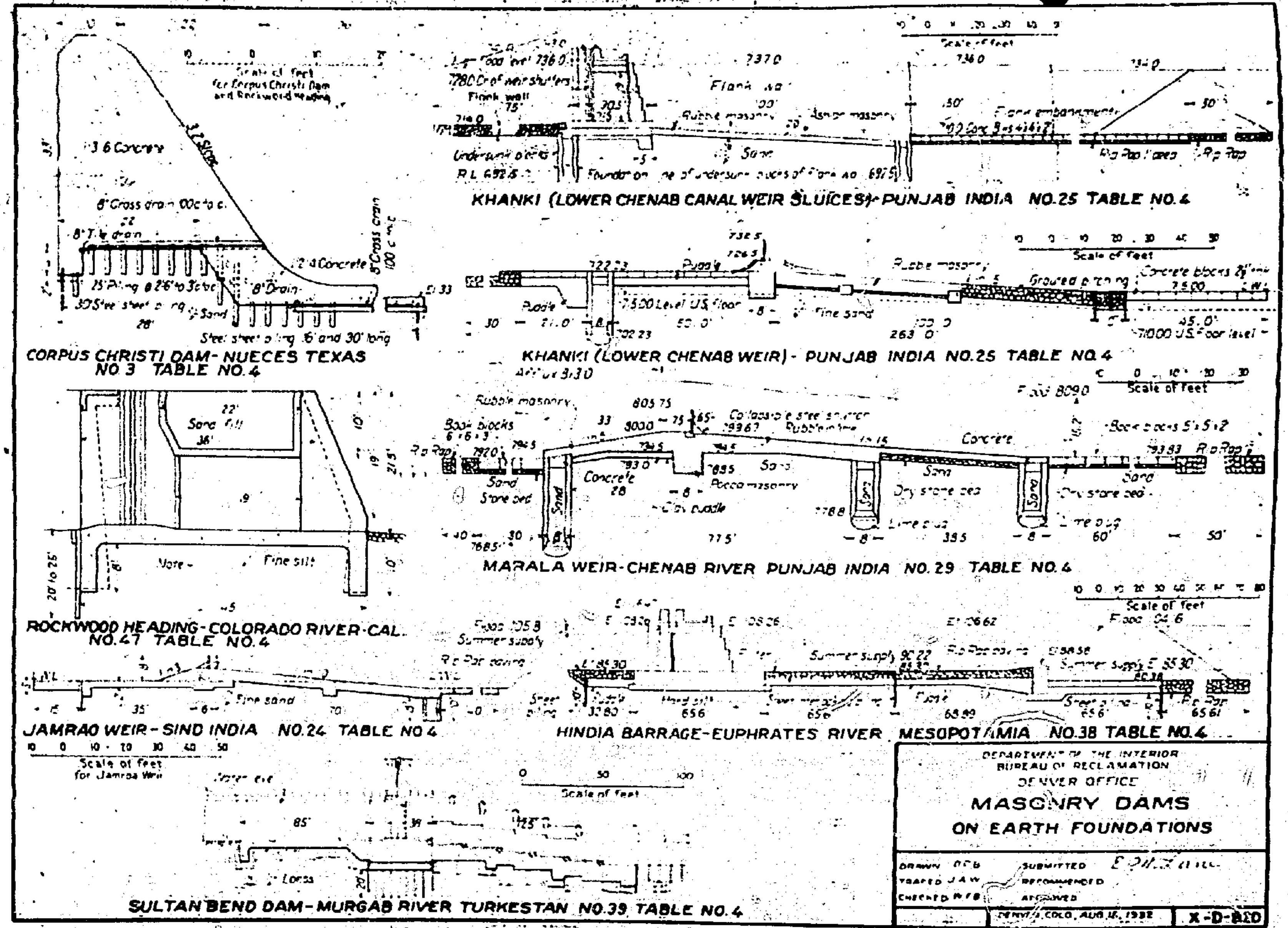

BIG SUNFLOWER DAM-NO 18 TABLE NO.3
Scale of feet
El 275
Sand with fine clay and gravel
NARORA WEIR-NO'S 1 & 30 TABLE NO.4
Fine sand
Scale of feet
MOSS BLUFF DAM-NO16 TABLE NO.3
Steel piling 20'
Stratified sand
3Pc 2"x10"x20'
3Pc 2"x10"x15' piling
El 390
Scale of feet
El 6800
Bed of channel
Puddle
Dry sand
El 6830
El 6705
El 6530
S'x5'x2' Blocks
Crates
ROHILKHAND CANALS-BYE WASH KICHHA WEIR-NO.8 TABLE NO.4
Scale of feet
El 619.00
El 631.00
El 641.50
El 633.25
El 629.00
Fine sand
+25' Simplex piling
Piling not in original structure
30' Joist piling
El 600.0
El 590.0
5' Simplex piling
Piling not in original structure
Loose boulders
2' Boulders in crates
3' Boulders in crates
DEOHA BARRAGE-NO.4 TABLE NO.4
Scale of feet
US FSL El 540.37
El 533.57
DS FSL El 537.67
NATHAURA ESCAPE HEAD - PAGE
Scale of feet
Sharp sand
Steel sheet piling
Sharp sand
Fine sand
Wooden sheet piling
Scale of feet
FLEDERBORN DAM-NO1 TABLE NO.3
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
DENVER OFFICE
MASONRY DAMS
ON EARTH FOUNDATIONS
DRAWN W.H.R.-O.C.S.
TRACED J.A.W.
CHECKED W.F.B.
SUBMITTED
RECOMMENDED
APPROVED
DENVER, COLO., AUG. 18, 1932.
X-D-818

PLATE XV


PRAIRIE DU SAC - WISCONSIN RIVER - WIS. NO.20 TABLE NO.3
FLORENCE DAM - GILA RIVER - ARIZONA NO.17 TABLE NO.3
BURRA WEIR - MAHANUDDEE INDIA NO.12 TABLE NO.3
SIDHNAI WEIR - RAVI RIVER - PUNJAB INDIA NO.14 TABLE NO.3
BURRA UNDER-SLUICES - MAHANUDDEE INDIA NO.12 TABLE NO.3
GODAVERY WEIR - GODAVERY RIVER - INDIA NO.15 TABLE NO.3
RIVERDALE - APPLE RIVER - WISCONSIN NO.13 TABLE NO.3
COLEROON WEIR - COLEROON RIVER - INDIA NO.11 TABLE NO.3
Scale of feet
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
DENVER, OFFICE
MASONRY DAMS
ON EARTH FOUNDATIONS
DRAWN J.A.W.
TRACED J.A.W.
CHECKED W.F.B.
SUBMITTED
RECOMMENDED
APPROVED
X-D-819


KHANKI (LOWER CHENAB CANAL WEIR SLUICES) - PUNJAB INDIA NO.25 TABLE NO.4
CORPUS CHRISTI DAM - NUECES TEXAS NO.3 TABLE NO.4
KHANKI (LOWER CHENAB WEIR) - PUNJAB INDIA NO.25 TABLE NO.4
MARALA WEIR - CHENAB RIVER PUNJAB INDIA NO.29 TABLE NO.4
ROCKWOOD HEADING - COLORADO RIVER - CAL. NO.47 TABLE NO.4
JAMRAO WEIR - SIND INDIA NO.24 TABLE NO.4
HINDIA BARRAGE - EUPHRATES RIVER MESOPOTAMIA NO.38 TABLE NO.4
SULTAN BEND DAM - MURGAB RIVER TURKESTAN NO.39 TABLE NO.4
Scale of feet
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
DENVER OFFICE
MASONRY DAMS
ON EARTH FOUNDATIONS
DRAWN
TRACED J.A.W.
CHECKED
SUBMITTED
RECOMMENDED
APPROVED
DENVER, COLO., AUG. 16, 1932
X-D-820

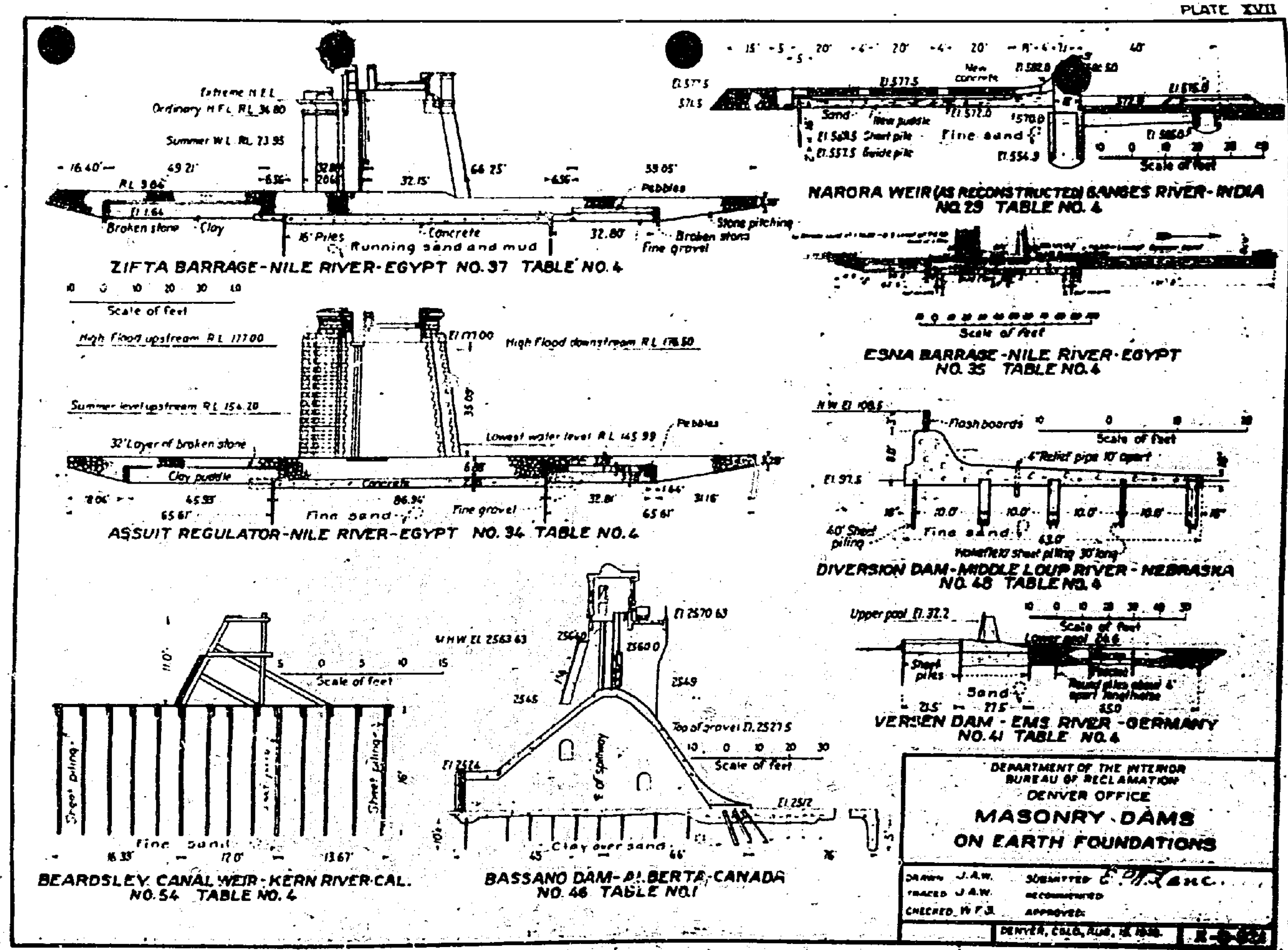
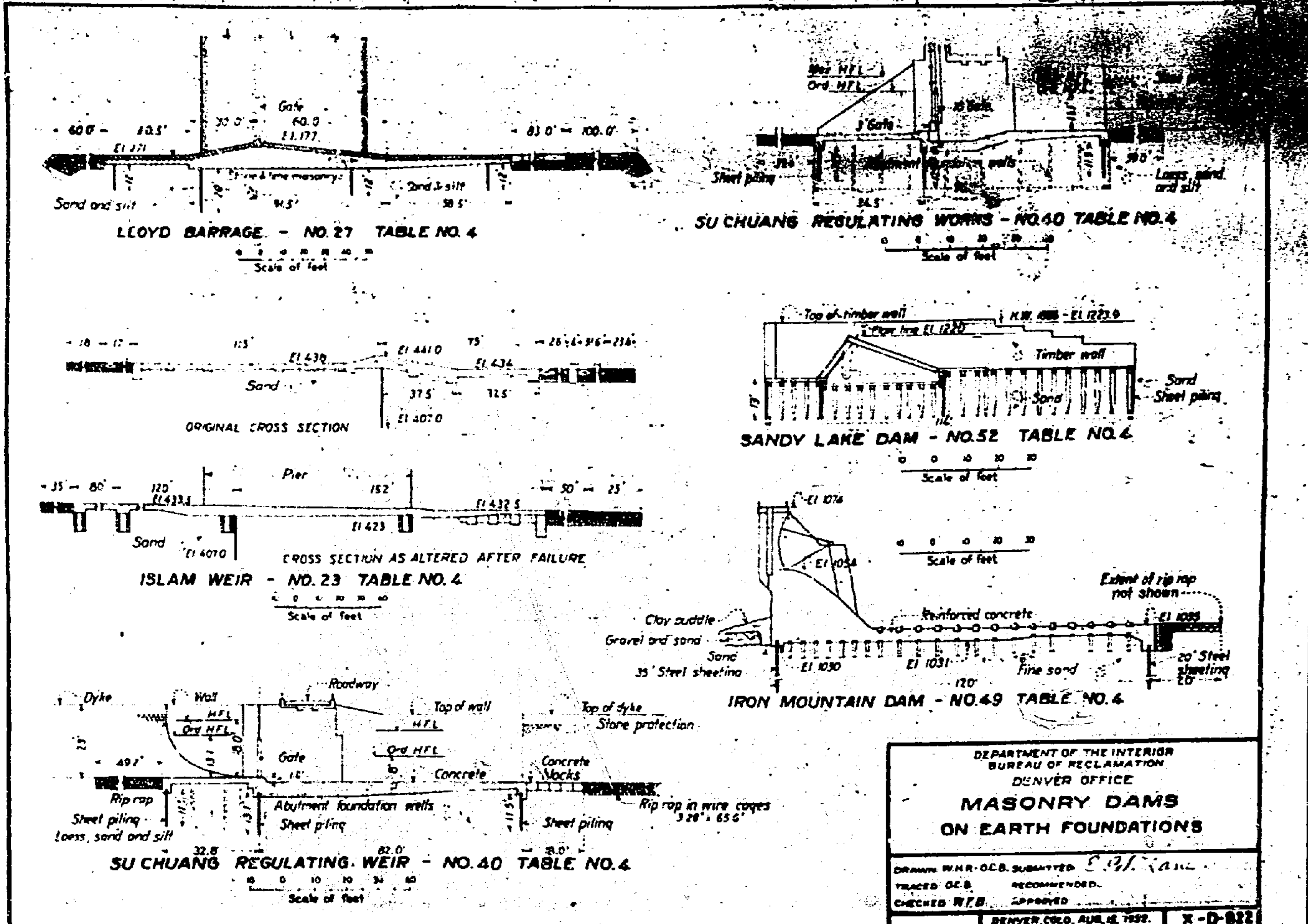

ZIFTA BARRAGE-NILE RIVER-EGYPT NO. 37 TABLE NO. 4
ASSUIT REGULATOR-NILE RIVER-EGYPT NO. 34 TABLE NO. 4
NARORA WEIR (AS RECONSTRUCTED) GANGES RIVER-INDIA NO. 29 TABLE NO. 4
ESNA BARRAGE-NILE RIVER-EGYPT NO. 35 TABLE NO. 4
DIVERSION DAM-MIDDLE LOUP RIVER-NEBRASKA NO. 48 TABLE NO. 4
VERSEN DAM-EMS RIVER-GERMANY NO. 41 TABLE NO. 4
BEARDSLEY CANAL WEIR-KERN RIVER CAL. NO. 54 TABLE NO. 4
BASSANO DAM-ALBERTA-CANADA NO. 46 TABLE NO. 1
Scale of feet
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
DENVER OFFICE
MASONRY DAMS
ON EARTH FOUNDATIONS
DRAWN J.A.W.
TRACED J.A.W.
CHECKED W.F.B.
SUBMITTED
RECOMMENDED
APPROVED
DENVER, COLO.

PLATE XVII

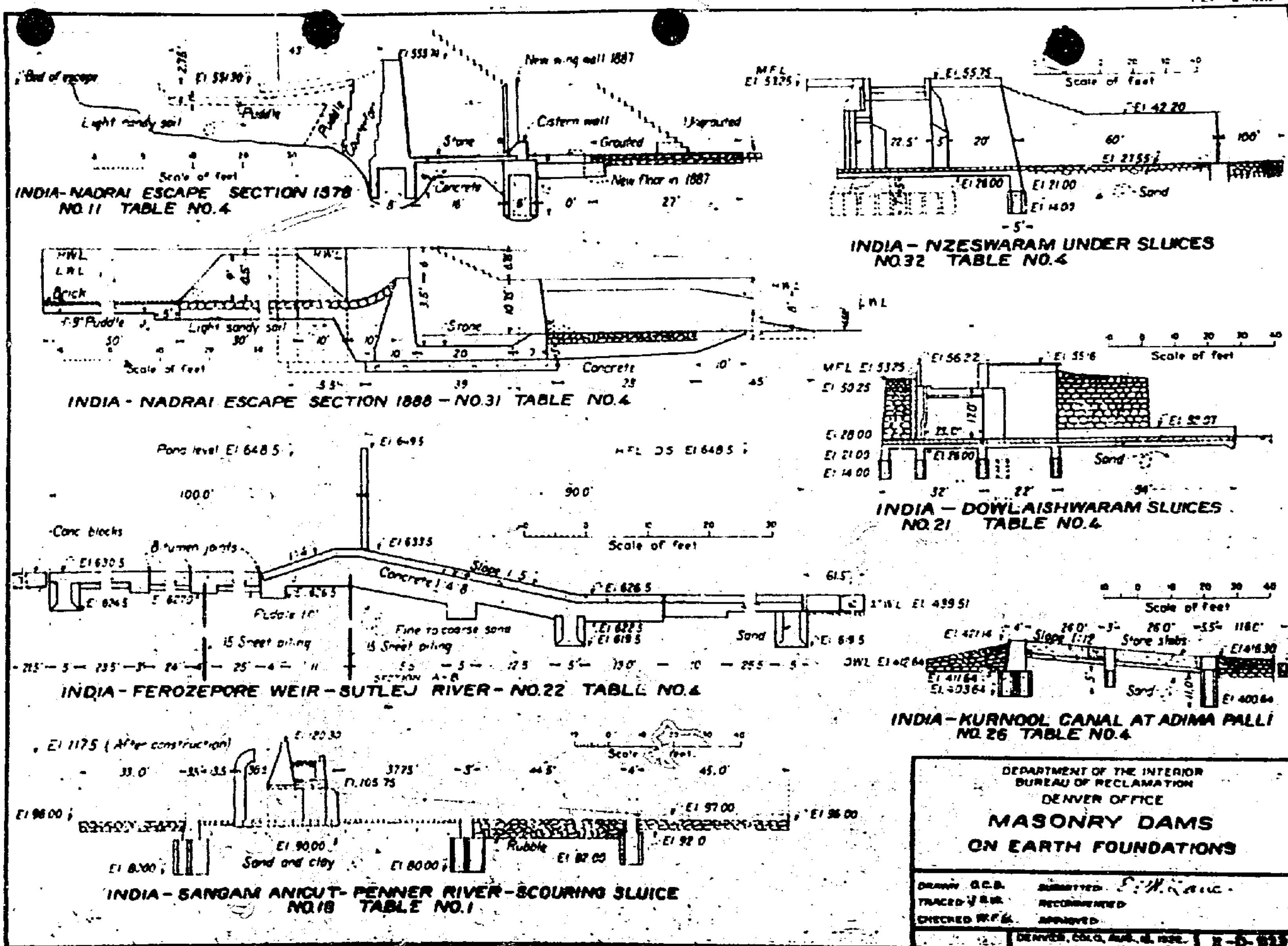


DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
DENVER OFFICE

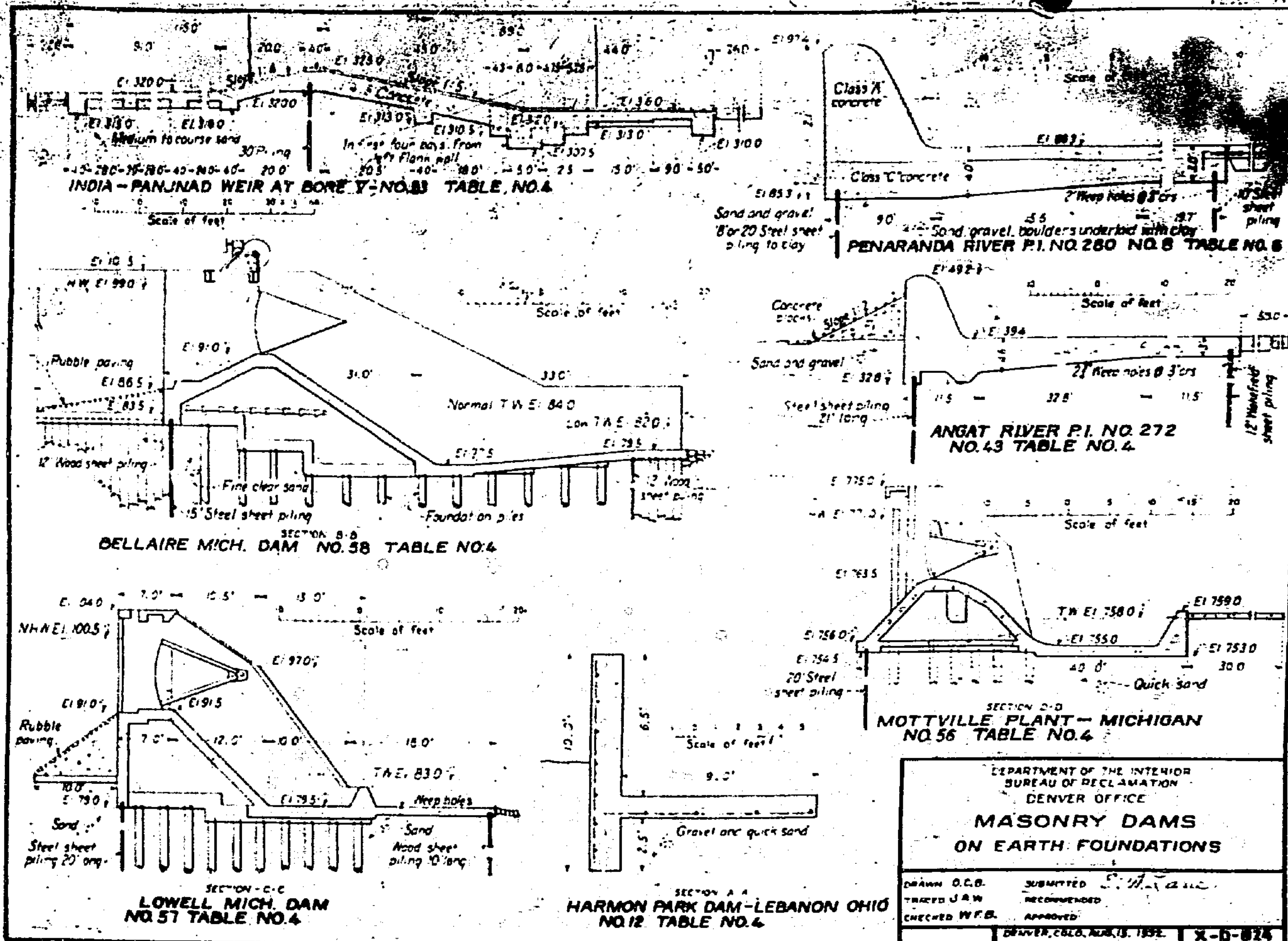
MASONRY DAMS
ON EARTH FOUNDATIONS

DRAWN W.H.R.-O.C.B. SUBMITTED
TRACED O.C.B. RECOMMENDED
CHECKED W.F.B. APPROVED
DENVER, COLO. AUG. 18, 1932. X-D-822

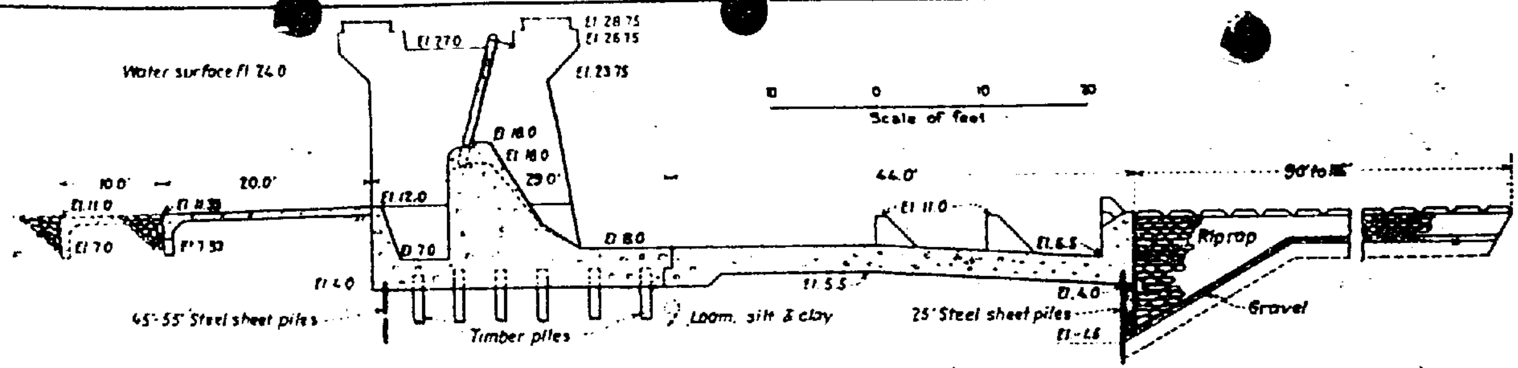
PLATE XIX



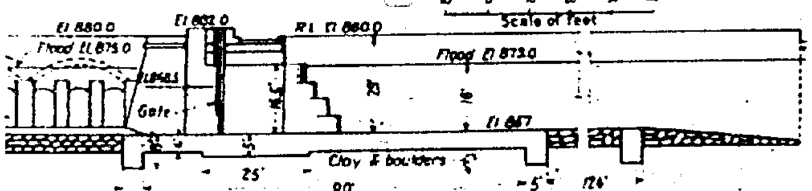
INDIA-NADRAI ESCAPE SECTION 1878
NO.11 TABLE NO.4
Bed of escape
Light sandy soil
Puddle
New wing wall 1887
Cistern wall
Stone
Grouted
Ungrouted
Concrete
New floor in 1887
Scale of feet
INDIA - NZESWARAM UNDER SLUICES
NO.32 TABLE NO.4
M.F.L. El 53.25
El 55.75
El 42.20
El 21.55
El 28.00
El 21.00
El 14.00
Sand
Scale of feet
INDIA - NADRAI ESCAPE SECTION 1888 - NO.31 TABLE NO.4
H.W.L.
L.W.L.
Brick
Puddle
Light sandy soil
Stone
Concrete
Scale of feet
INDIA - DOWLAISHWARAM SLUICES
NO.21 TABLE NO.4
M.F.L. El 53.25
El 56.22
El 55.6
El 52.07
Sand
Scale of feet
INDIA - FEROZEPORE WEIR - SUTLEJ RIVER - NO.22 TABLE NO.4
Pond level El 648.5
M.F.L. D.S. El 648.5
Conc blocks
Bitumen joints
Concrete 1:4:8
Slope 1:5
Puddle
15 Sheet piling
Fine to coarse sand
Sand
Scale of feet
INDIA - KURNOOL CANAL AT ADIMA PALLI
NO.26 TABLE NO.4
Slope 1:12
Stone slabs
Sand
Scale of feet
INDIA - SANGAM ANICUT - PENNER RIVER - SCOURING SLUICE
NO.18 TABLE NO.1
El 117.5 (After construction)
Sand and clay
Rubble
Scale of feet
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
DENVER OFFICE
MASONRY DAMS
ON EARTH FOUNDATIONS
DRAWN G.C.B.
TRACED
CHECKED
SUBMITTED
RECOMMENDED
APPROVED
DENVER, COLO.



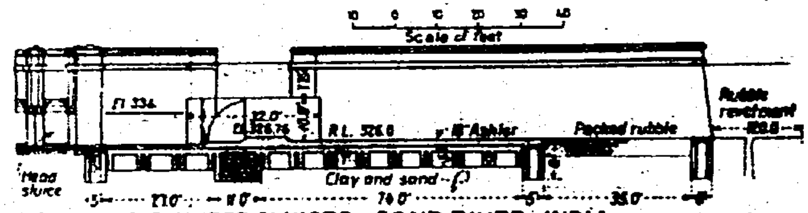
INDIA – PANJNAD WEIR AT BORE Y-NO.83 TABLE NO.4
Scale of feet
Medium to course sand
In first four bays from left flank wall
PENARANDA RIVER P.I. NO. 280 NO. 8 TABLE NO. 6
Class 'A' concrete
Class 'C' concrete
2" Weep holes @ 3' crs
Sand and gravel
8' or 20' Steel sheet piling to clay
Sand, gravel, boulders underlaid with clay
10' Steel sheet piling
ANGAT RIVER P.I. NO. 272 NO. 43 TABLE NO. 4
Concrete blocks
Sand and gravel
Steel sheet piling 21' long
12' Wakefield sheet piling
BELLAIRE MICH. DAM NO. 58 TABLE NO. 4
SECTION B-B
Rubble paving
12' Wood sheet piling
Fine clear sand
15' Steel sheet piling
Foundation piles
Normal T.W. El. 84.0
MOTTVILLE PLANT – MICHIGAN NO. 56 TABLE NO. 4
SECTION D-D
20' Steel sheet piling
Quick sand
LOWELL MICH. DAM NO. 57 TABLE NO. 4
SECTION C-C
Rubble paving
Steel sheet piling 20' long
Weep holes
Wood sheet piling 10' long
HARMON PARK DAM – LEBANON OHIO NO. 12 TABLE NO. 4
SECTION A-A
Gravel and quick sand
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
DENVER OFFICE
MASONRY DAMS
ON EARTH FOUNDATIONS
DRAWN O.C.B.
TRACED J.A.W.
CHECKED W.F.B.
SUBMITTED
RECOMMENDED
APPROVED
DENVER, COLO., AUG. 15, 1932.
X-D-824



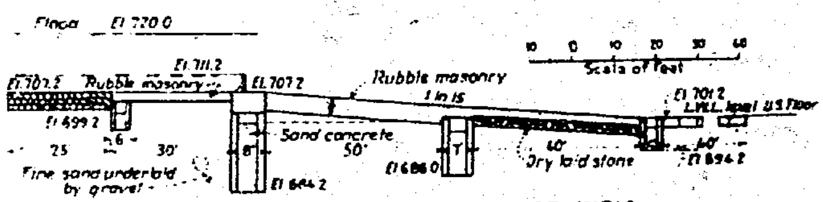
BONNET CARRE SPILLWAY-MISSISSIPPI RIVER L.A. NO.53 TABLE NO.4



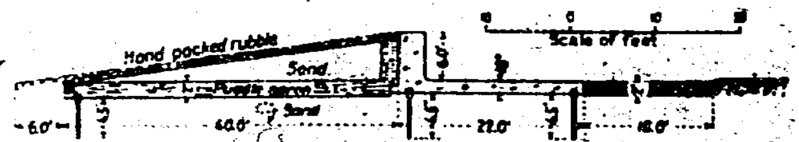
RUPAR WEIR SLUICES - SIRHIND CANAL - SUTLEJ RIVER - INDIA
NO.16 TABLE NO.1



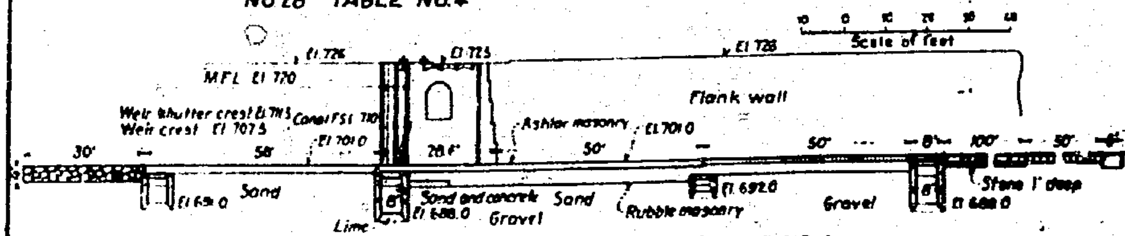
DELHI WEIR UNDER SLUICES - SONE RIVER - INDIA
NO.68 TABLE NO.1



JHELUM WEIR-JHELUM CANAL - PUNJAB INDIA
NO 28 TABLE NO.4



CAMBDIAN-NEPEAN RIVER - NEW SOUTH WALES.
NO.51 TABLE NO.4



JHELUM WEIR SLUICES - JHELUM CANAL - INDIA NO. 28 TABLE NO.4

DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
DENVER OFFICE

MASONRY DAMS
ON EARTH FOUNDATIONS

DRAWN J.A.R. SUBMITTED E. W. Lane
TRACED J.A.R. RECOMMENDED
CHECKED W.F.R. APPROVED

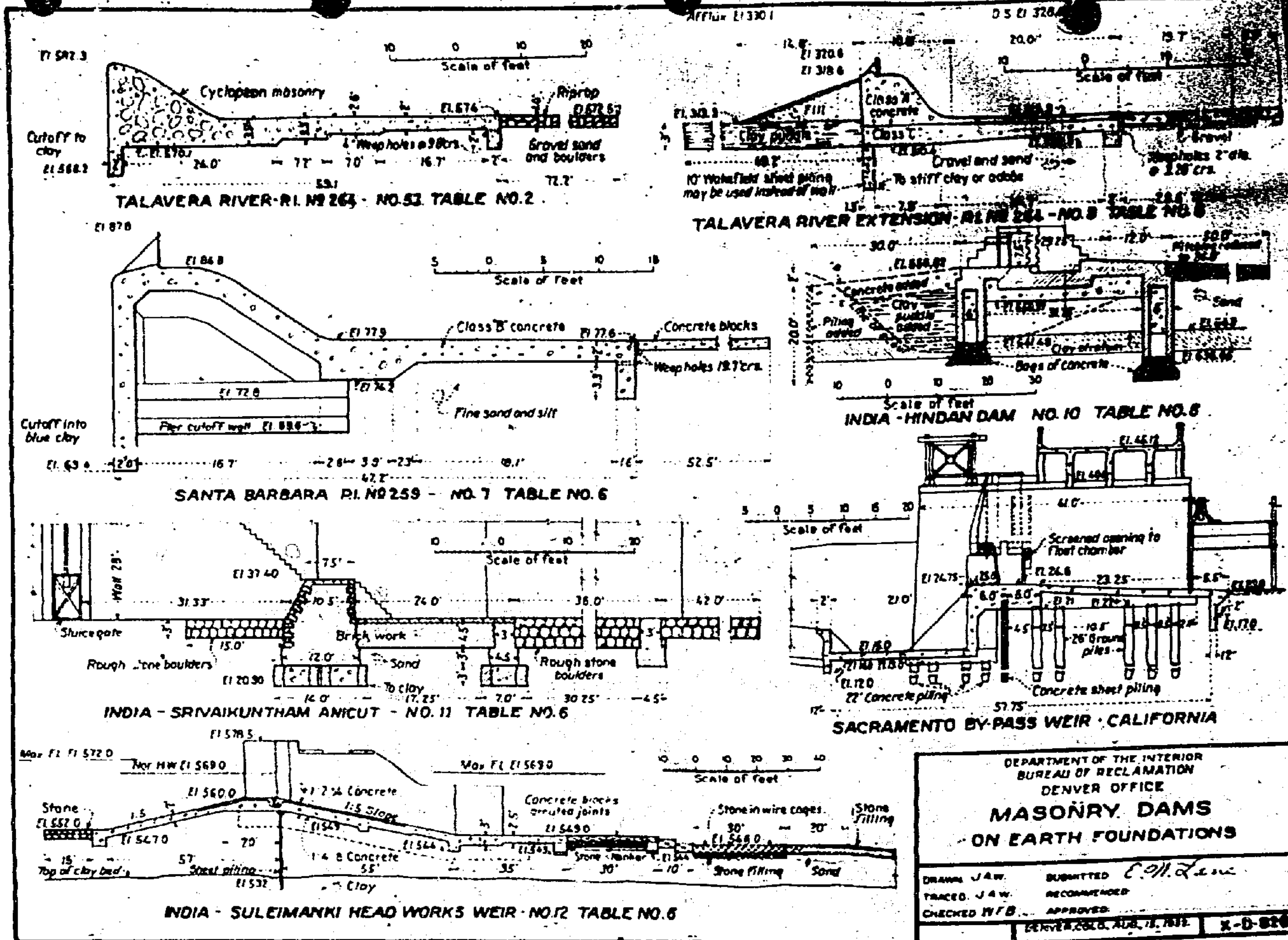


PLATE
TALAVERA RIVER-R.I. Nº 263 - NO.53 TABLE NO.2
TALAVERA RIVER EXTENSION
SANTA BARBARA P.I. Nº 259 - NO.7 TABLE NO.6
INDIA - HINDAN DAM NO.10 TABLE NO.6
INDIA - SRIVAIKUNTHAM ANICUT - NO.11 TABLE NO.6
SACRAMENTO BY-PASS WEIR · CALIFORNIA
INDIA - SULEIMANKI HEAD WORKS WEIR · NO.12 TABLE NO.6
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
DENVER OFFICE
MASONRY DAMS
ON EARTH FOUNDATIONS
DRAWN J.A.W.
TRACED J.A.W.
CHECKED W.F.B.
SUBMITTED
RECOMMENDED
APPROVED

APPENDIX I

Bibliography
Masonry Dams on Earth Foundations

The Irrigation Works of India - Buckley - 2nd Edition, pp. 107 to 227.

The Practical Design of Irrigation Works - Bligh - 2nd Edition, pp. 162 to 266.

Dams and Weirs - Bligh - pp. 151 to 198.

The Control of Water - Parker - pp. 656 to 706.

The Improvement of Rivers - Thomas and Watt - 2nd Edition, Part II, pp. 579, 629, Plates 54, 67, 69, 72 and 73.

Book of Plans, New York State Barge Canal, Plate 55.

Madras Irrigation Manual - Mullins.

Irrigation Practice and Engineering, Vol. III - Etcheverry.

On Percolation under Aprons of Irrigation Works - S. Leliavsky.

Erdbaumechanik - Dr. Ing. Karl Terzaghi.

Uplift Pressure on Dams - Warren Weaver.

Journal of Mathematics and Physics - Vol. XI, No. 2, June, 1932.

<u>Engineering News-Record - Through June 1934.</u>

Vol. 110 - 6-15-33 - p. 780 - Concrete Wall Replaces Levee at Helena, Arkansas.

108 - 1-28-32 - p. 130 - Depressed Apron Checks Erosion below Hamilton Dam.

107 - 10-8-31 - p. 595 - Upper Alameda Creek Dam for San Francisco Water.

106 - 3-5-31 - p. 407 - Corpus Christi Dam Failure.

106 - 2-12-31 - p. 286 - The Corpus Christi Dam Failure.

105 - 12-18-30 - p. 974 - Corpus Christi Dam Failure.

105 - 12-4-30 - p. 897 - Cause of Corpus Christi Dam Failure Still Undetermined.

105 - 11-27-30 - p. 861 - Corpus Christi Dam Fails with Loss of North Abutment Wall.

Engineering News-Record

Vol. 105 -	10-2-30 -	p. 520 -	Building New Dam for Corpus Christi Water Supply.
103 -	10-31-29 -	p. 685 -	Automatic Roof Weirs Used for Reservoir Control on Guadalupe River.
103 -	8-15-29 -	p. 242 -	Planning a Huge Diversion Weir, Bonnet Carre Spillway.
102 -	2-14-29 -	p. 284 -	Progress of Tests and Design of Bonnet Carre Spillway.
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.
90 -	5-24-23 -	p. 826 -	Building a Gravel Retaining Dam by Dragline.
89 -	11-16-22 -	p. 824 -	Floating Concrete Dam Built on the Gila River.
85 -	8-5-20 -	p. 244 -	Improvement Work on River, Murray River in South Australia.
85 -	8-19-20 -	p. 355 -	Floods Wash Out Part of Apron of Granite Reef Dam.
84 -	4-15-20 -	p. 782 -	An Indian Engineer on Dam Design (Book Review).
84 -	5-20-20	- p.1014 -	Measuring Upward Pressure under a Masonry Dam.
82 -	4-24-19 -	p. 818 -	Panels of Movable Weir Collapse Automatically.
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.

Engineering News

Vol. 75 -	1-20-16 -	P. 106 -	Reservoir and Concrete Dam in Glacial Drift.
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.

Engineering News

Vol.	Date	Page	Title
Vol. 75	6-15-16	p. 1113	Building Power House and Dam on Sand Foundations.
74	11-25-15	p. 1032	$101,000 Hollow Concrete Dam, St. Francis River.
72	7-16-14	p. 118	A Low Head Water Power Plant.
72	8-27-14	p. 425	The Design and Construction of the Bassano Dam.
72	9-3-14	p. 484	The Design and Construction of the Bassano Dam.
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.
67	6-20-12	p. 1196	Blowout of a Canal Retaining Wall at Ansonia, Conn.
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.
61	6-10-09	p. 615	Turning the Colorado River and Completing the Laguna Dam, Arizona-Colorado.

Engineering News.

Vol. 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.

24 - 9-6-90 - p. 204 - Whiting St. Dam, Holyoke, Massachusetts.

Engineering Record

Vol. 74 - 12-9-16 - p. 716 - Diversion Weir - Canal Type Used for Pioneer Hydroelectric Development in Sandy Soil.

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

69 - 1-17-14 - p. 77 - Coon Rapids Hydroelectric Development.

69 - 3-28-14 - p. 372 - Reconstruction of the Elwha River Dam.

66 - 12-7-12 - p. 640 - Connecting Lake Washington Seattle with Puget.

66 - 10-5-12 - p. 376 - Failure of Intake at Bow River Dam.

66 - 11-30-12 - p. 600 - Washout of Base of Port Angeles Dam.

66 - 8-31-12 - p. 246 - Water Power from the Au Sable River.

Engineering Record

Vol. 65 - 6-15-12 - p. 650 - Break in the Power Canal Wall at Ansonia.

65 - 4-13-12 - p. 401 - The Failure of Dam at Oswego, N. Y.

64 - 10-21-11 - p. 481 - A Small Hydroelectric Plant on the Oswegatchie River.

64 - 7-29-11 - p. 141 - Pueblo-Rocky Ford Irrigation Project.

63 - 5-27-11 - p. 589 - Bow River Dam, Southern Alberta Land Co.

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.

Engineering and Contracting

Vol. 53 - 2-11-20 - p. 147 - New Storage Dams in Illinois (Decatur Dam) (See also Engineering News-Record, Vol. 91, August 16, 1923, p. 264).

51 - 3-12-19 - p. 261 - Conditions of Stability and Suggested Design for Wooden Dam Built on Sand.

44 - 9-8-15 - p. 193 - Considerations of Sliding, Overturning, Crushing and Blowouts in Dam Design by the Ohio River Board.

Engineering and Contracting

Vol. 41 - 5-20-14 - p. 584 - Structural Features and Construction Methods, Hydro-Electric Plant at Elkhart, Indiana.

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 - 6-5-12 - p. 646 - Three Approved Designs for Weir Dams for Irrigation Canals.

35 - 3-8-11 - p. 290 - Dams on Sand Foundations.

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.

25 - 6-6-06 - p. 155 - Cost of a Small Reinforced Concrete Dam at Richmond, Indiana.

Water Works (Formerly Part of Engineering and Contracting)

Vol. 65 - Feb. '26 - p. 65 - Oliver Dam on the Southern Okanogan Irrigation Project.

65 - June '26 - p. 269 - A New Type of Spillway.

59 - March '23 - p. 605 - Concrete Barrier Dam for Intercepting Drifting Gravel and Sand.

Roads and Streets (Formerly Part of Engineering and Contracting)

Vol. 62 - Aug. '24 - p. 321 - Frictional Resistance of Concrete on Various Sub-Bases.

The Engineer (London) Through Vol. 156, 1933

Vol. 156 - 8-4-33 - p. 105 - The Flow of Water in Sand.

153 - 1-22-32 - p. 105 - The Lloyd Barrage at Sukkur on the Indus.

151 - 1-30-31 - p. 123 - The Nag Hammadi Barrage.

2-6-31 - p. 150

148 - 10-4-29 - p. 366 - The Sukkur Barrage.

The Engineer (London)

Vol. 145 - 2-10-28 - p. 147 - Irrigation in Siam.

144 - 9-2-27 - p. 251 - The Nag Hammadi Barrage

142 - 12-24-26 - p. 680 - The Sukkur Barrage Irrigation Project, 1920.

11-3-93 - p. 431 - Movable Waste Weirs, Mira Canal, India.

Engineering (London) Through 1933

Vol. 136 - 10-13-33 - - Automatic Radial Sluice Gate on the River Ouse.

133 - 1-29-32 - - The Lloyd Barrage at Sukkur.

131 - 2-2-31 - p. 20 - The Nag Hamandi Barrage.

129 - 6-27-30 - p. 821 - Zuyder Zee Reclamation Works. Main Enclosing Embankment and Sluices.

122 - 10-15-26 - p. 484 - The Dolgarrog Dam Failure.

119 - 3-20-25 - p. 371 - The Regulation of the Murray River.

119 - 4-3-25 - p. 405 - " " " " " "

119 - 4-17-25 - p. 469 - " " " " " "

119 - 5-1-25 - p. 532 - " " " " " "

119 - 5-8-25 - p. 565 - " " " " " "

114 - 8-25-22 - - Sukkur Barrage.

The Canadian Engineer

Vol. 44 - 5-2-22 - p. 444 - Construction of Kettle Creek Dam, St. Thomas.

38 - 6-3-20 - p. 517 - Dam That Withstood Unusual Severe Test.

- 10-30-08 - - Diversion Weirs on Sand.

Transactions of the American Society of Civil Engineers - Through 1933

Vol. 93 - 1929 - p. 262 - Headworks of the Imperial Canal.

93 - 1929 - p.1317 - Hydrostatic Uplift in Pervious Soils.

Transactions of the American Society of Civil Engineers (Continued)

Vol. 93 - 1929 - p. 1717 - Upward Pressures under Dams. Experiments by the United States Bureau of Reclamation.
92 - 1928 - p. 1001 - Lake Washington Ship Canal, Seattle.

89 - 1926 - p. 1181 - The Oradell Dam of the Hackensack Water Co.

88 - 1925 - p. 1257 - Sherman Island Dam and Power-House.

86 - 1923 - p. 148 - Locks and Movable Dams on Ohio River.

81 - 1917 - p. 1 - Designing an Earth Dam Having a Gravel Foundation.
80 - 1916 - p. 421 - Action of Water under Dams.

77 - 1914 - p. 111 - Prewitt Reservoir Proposition.

77 - 1914 - p. 932 - Diversion of Irrigating Water from Streams.

76 - 1913 - p. 121 - A Western Type of Movable Weir Dam.

76 - 1913 - p. 1264 - Irrigation and River Control in the Colorado River Delta (Sharpe's Headgate).
73 - 1911 - p. 175 &
p. 190 - Dams on Sand Foundations.

32 - 1894 - p. 389 - The Dunnings Dam.

15 - 1886 - p. 887 - The Southington Dam.

Civil Engineering

2 - No. 1 - Jan. 1933 - p. 9 - Profile for a Low Dam Determined by Models.
No. 6 - June 1933 - p. 322 - Temiscouata Storage Dam on Sand Foundation.

Journal of Electricity, Power and Gas

Vol. 35 - 10-16-15 - p. 297 - Port Angeles Dam.

Professional Memoirs, Corps of Engineers, U. S. Army.

Vol. 7 - 1915 - p. 32 - Percolation and Upward Pressure of Water.

U. S. Army Engineering School, Occasional Papers

#21-30 - Report on Construction of Lock and Dam No. 4, Ohio River.

Western Construction News

Vol. 7 - 3-10-32 - p. 137 - Roof - Weir on Guadalupe River, Texas.

7 - 5-25-32 - p. 287 - Cochiti Diversion Dam on the Rio Grande.

Proceedings of the Institution of Civil Engineers - Through Vol. 233, 1931-32

Vol. 233 - 1931-32 - p. 140 - The Design and Construction of the Sarda Canal.

232 - 1930-31 - p. 340 - The Nag Hammadi Barrage, Upper Egypt.

197 - 1914 - p. 221 - The Stability of Weir Foundations on Sand and Soil Subject to Hydrostatic Pressure.

158 - 1904 - p. 26 - The Barrage Across the Nile at Asyut.

157 - 1904 - p. 278 - The Jamrao Canal.

156 - 1903 - p. 323 - The Zifta Barrage and Subsidiary Works.

113 - 1893 - p. 314 - The Chenab Weir.

Journal of the Western Society of Engineers

Vol. 19 - 1914 - p. 979 - Coon Rapids Low Head Hydro-Electric Development.

Le Genie Civil

Vol. 92 - 5-5-28 - p. 486 - Canalization of the Meuse in Holland.

81 - 7-8-22 - p. 40½ - Canalization of the Meuse in Holland.

Journal of the Association of Engineering Societies

Vol. 16 - June '96 - p. 210 - Bear Trap, Sandy Lake Reservoir.

16 - June '96 - p. 208 - Bear Trap, Davis Island Dam.

American Institute of Mining and Metallurgical Engineers.

Technical Publication No. 215 - p. 31.

Die Bautechnik

Vol. 37 - 8-7-26 - p. 526 - Die Kanaliserung der Hollindschen Maas.

De Ingenieur

Vol. 40 - pp. 713,717 - Weirs of Canalfied Meuse.

Punjab Engineering Congress, 1930.

Paper No. 138 - Hydraulic Gradients in Subsoil Water Flow in Relation to Stability of Structures Resting on Saturated Soils.
Paper No. 142 - Stability of Weirs and Canal Works - An Application of the New Theory of Hydraulic Gradient.

Punjab Irrigation Research Institute

Research Publication Vol. II, Jan. 1933, No. 2 - Studies on Subsoil Hydraulics. Investigation of Observational Methods for Models.
Research Publication Vol. II, No. 3, 1934 - A Study of the Flow of Water Under Works on Sand Foundations by Means of Models - Part I
Research Publication Vol. II, No. 4, 1934 - A Study of the Flow of Water Under Works on Sand Foundations by Means of Models - Part II
Research Publication Vol. II, No. 5, Feb. 1934 - An Investigation of the Pressures on Works on Sand Foundations.
Research Publication Vol. II, No. 6, Feb. 1934 - An Investigation of the Flow of Water Under Khanki Weir and the Pressures on the Floor.