CONCRETE DAMS
THEORY AND PRACTICE OF CONSTRUCTION

BY

JACKSON HEATH WILKINSON

B. S. University of Illinois, 1915

THESIS

Submitted in Partial Fulfillment of the Requirements for the

Degree of

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IN CIVIL ENGINEERING

IN

THE GRADUATE SCHOOL

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I HEREBY RECOMMEND THAT THE THESIS PREPARED UNDER MY SUPERVISION BY

JACKSON HEATH WILKINSON

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DEGREE OF Master of Science in Civil Engineering

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Committee
on
Final Examination

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THEORY AND PRACTICE IN CONSTRUCTION.

OUTLINE

III ARCH DAMS.

IV. Reinforced Concrete Dams.

V. Failures of Dams.

DIAGRAMS.

Literature consulted.

General.

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Introduction

The steadily increasing demand for the building of dams necessitated by the recent revival of water power development, reclamation projects, flood control, and innumerable city water supply reservoirs, together with the alarming number of disasterous failures has led to a revival of the discussion of the theory and practice in the design of dams. Considerable discussion and investigation on the stresses in, and the design of, concrete dams has been made; but the theory of design has changed but little since De Sazilly. Probably the most important improvement in the design has been the consideration of upward hydrostatic pressure. Another force which has attracted considerable attention is ice pressure. In many dams, especially those in the colder climates, provisions have been made for the resistance of this force.

Although the theory which has been generally accepted among engineers for the last half century has in the main led to successful designs, many of the failures which have occurred have been so disasterous that public opinion as well as engineering reputation requires that the dams which are built in the future be constructed to afford a greater degree of safety and with greater care in guarding against the neglect of well known principles, than in the past. Neither public opinion nor engineering reputation can long withstand such disasterous failures as those of the two Austin dams. Pennsylvania has already placed dams under state supervision. Again, during a single flood in the Southern states four relatively important masonry dams failed. The excuse given in each case was unprecedented floods. The fact is there was not a large enough factor of safety in the design.

From an analysis of the failures of dams it may be said that practically all failures have been due to the neglect of some well known principle. Possibly an exception of this is failure from upward hydrostatic pressure, as the extent of this force was not fully realized until the last few years.

The object of this thesis is to determine, as far as possible, from an analysis of the failures of dams which we have on record, wherein the structures have been faulty and where they have departed from the correct design. Before commencing the analysis of failures it was necessary to collect and present in a concise manner the modern theory and practice in the design of the various types of dams. A considerable research was made in literature on the mathematical analysis of stresses in gravity dams, both straight and curved, to see if anything of value had been developed. The results of this work are given in the discussion of the stresses in the various types of dams. The equation for the "total shear on a vertical section" is developed after Atcherly and Pearson as given in Engineering, March 31, 1905. The equation for the "intensity of shear on a vertical section" is derived from this equation by a method which was suggested by Professor Unwin's work in Engineering April 21, 1905. The equation for "intensity of shear on a horizontal section" is derived from the first equation in a manner similar to that used by Professor Unwin in the paper mentioned above.

Some experiments have been made on model dams at different times which seem to verify these equations. However the stresses as computed by these equations would make no radical change in the present method of design.

Only a limited amount of literature has been published on the design of arch dams. Practice differs radically in their construct-

ion. Many of the bolder designs, while they have proved successful, have met with considerable skepticism. Others which are more conservative in design are hardly mpre economical than straight gravity dams would have been.

The development of the theory of design of gravity dams.

The oldest existing masonry dams are those of Southern Spain which form the large irrigation reservoirs. The great dimensions and massiveness of these dams shows that the designers had no correct conception of the forces to be resisted. In fact, the mass of material in some of these dams is so great that undue stresses are produced in the foundations and their very massiveness becomes a source of weakness rather than of strength.

During the first part of the ninteenth century many dams were built with the batter down-stream so that they would have been stronger if they had been reversed.

The first rational theory of the stresses in masonry dams was advanced by the French engineers and has been proved to be fairly accurate by the satisfactory results in the dams constructed after it. De Sazilly published the first satisfactory literature on the stresses in masonry dams in his memoir which appeared in the "Annales des Ponts et Chaussées" for 1853. According to this writer the stability of masonry dams is dependent upon:

(1) the pressure sustained by the masonry or the foundation.

(2) the possibility of any part of the masonry sliding on that below, or of the whole moving on its foundation. He further states that no dam has been known to fail by sliding and therefore recommends that the dam be wholly designed by the first condition , leaving it to trial to see that the second is fullfilled. He also points out that the dam must be investigated with the reservoir both full and empty.

Delocre, who in 1858 investigated the stresses in dams , advocated using polygonal faces with few changes in slope; while De-Sazilly had favored stepped faces. Delocre also investigated the additional strength added by curved plan. A memoir giving the details

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was published in the "Annales des Ponts et Chasses" for 1866.

In England the first notable work was that of Rankine. He made a mathematical investigation of the best profile for a masonry dam and recommended that the profile be determined by the principles laid down by the French engineers. A report of his work appeared in the "Miscelleaneous Scientific Papers", 1881. However he improved upon the French methods and advanced the theory that no tension should be allowed to exist in the masonry, and that the line of pressure should lie in the middle third.

Le Blanc in 1856 demonstrated that an inclined force, R , on a joint did not produce a compression due to its normal component, R cos a, but that the force to be considered as producing compression is R/ cos a. where a is the angle of inclination of R.

In 1904 Atcherly, assisted by Professor Pearson, carried on an investigation on the stresses in masonry dams, the results of which were published in the "Draper Company's Research Memoir". Their conclusions were:

(1) The current idea that the critical sections of a dam are the horizontal sections is entirely erroneous. A dam collapses first by tension in the vertical sections at the toe.

(2) The shearing of the vertical sections over each other follows immediately on the opening up by tension.

(3) It is possible that the shear on the horizontal sections is far more important than it is usually supposed.

Historical Sketch of Construction Methods.

The first masonry dams were constructed of cut-stone masonry laid in lime mortar. Until the invention of cement this was about the only available method of construction for the lime mortar requires small joints if it sets to any great degree of hardness. Cement was first made on a large scale in New York about 1820, and was then used to lay cut-stone masonry in water. Since that time practice has gradually developed the use of concrete. This first step was the use of rubble masonry with cut-stone faces. This was followed by mass concrete in which large plums were set. At the present time the most economical construction seems to be mass concrete made of large aggregate. This form of construction, which allows the continuous pouring of concrete from large capacity mixers, requires no great care to obtain a good bond between the different materials, as was the case when the large stones were used, and can be performed by unskilled labor, while while the former practice requires a large amount of highly skilled labor. Concrete thus made is an ideal material for dams as it forms a homogeneous structure, free from joints and planes of weakness. Although the amount of cement required is larger than in the previous methods, its cost is more than offset by the cost of the skilled labor in the rubble or cut-stone masonry. Concrete is especially good on dams where rapidity of construction is neceessary to complete the work during the dry season.

Recently many forms of reinforced and arched dams have been developed which are most readily constructed of concrete, as it can be cast in nearly any form and can be reinforced to carry tension.

Types of Concrete Dams.

Concrete dams may be divided into three general classes from the way in which the stability of the structure is derived:

1. Gravity dams, which derive their stability from the weight of the structure.

2. Arch dams, which carry the water pressure by arch action to the abutments.

3. Reinforced dams, which derive their stability against overturning from the weight of the water on the structure.

These general classes are divided into the following types:

Gravity Section	1. Straight plan
	2. Curved plan
Arch Dams	3. Constant radius arch.
	4. Constant angle arch.
	5. Warped arch.
	6. Multiple arch.
Reinforced Dams	7. Cantilever wall.
	8. Buttressed wall.
	9. Cellular.
	10. Hollow section.

GRAVITY DAMS.

Nomenclature- Gravity Dams.

b = the width of any section of the dam at a distance h below crest.

B = the total upward pressure on the base from the foundation.

B_1= upward pressure on the base of the dam for the length, b-x; Fig. 2.

e = distance from the center of the base to the line of action of B.

ΣF_v= summation of vertical forces.

h_x= height of section of dam at distance x from face.

I = moment of inertia of horizontal section of dam.

m=ratio of b to h.

M = bending moment due to eccentricity of B.

P = total pressure on the face of the dam.

S = normal stress at any point in the horizontal section of the dam.

S_o= stress at the toe of the dam as in Fig. 2.

S_1=stress due to B evenly distributed over the base.

S_2=stress due to the eccentricity of B.

S_x=stress at any point distance x from the face of the

w =unit weight of water.

w_1=unit weight of masonry.

W = weight of section of dam.

W_1=weight of section of dam to left of line H--K, Fig. 2.

v_h=intensity of shear on horizontal section at depth h below crest.

v_v=intensity of shear on vertical section.

V_1=total shear on vertical section distance x from face of the dam

V_h=total shear on vertical section to depth h below crest.

V_{h+1}=total shear on vertical section to depth h+1 below crest.

x = distance along the horizontal.

y = distance along vertical.

Gravity Dams. - Straight Plan.

The straight plan gravity dam derives its stability from the weight or inertia of its section. It is simply a block of masonry of such dimensions that it will not be displaced by the water pressure.

In the mathematical analysis of the stresses in such a dam the following assumptions are made:

1. The dam is a homogeneous mass.

2. The dam acts as a vertical cantilever beam.

3. All the forces on the dam act in a plane perpendicular to the dam.

With the above assumptions there are conditions necessary for the stability of a dam:

1. There must be no tensile stress at any point in the dam.

2. The safe compressive stress must not be exceeded at any point in the dam at any stage of water.

3. The shear at any point must not exceed an allowable stress.

The first condition would require that no section would overturn about the toe of any horizontal section.

The maximum compressive stress must be investigated at both high and low water as it often happens, especially in the larger dams, that it is large when the reservoir is empty.

The investigation for shear should include the resistance to sliding on the foundation. This is frequently a point of weakness, but the construction should be such that a plane will not be formed at the foundation.

Stresses in Gravity Dams.

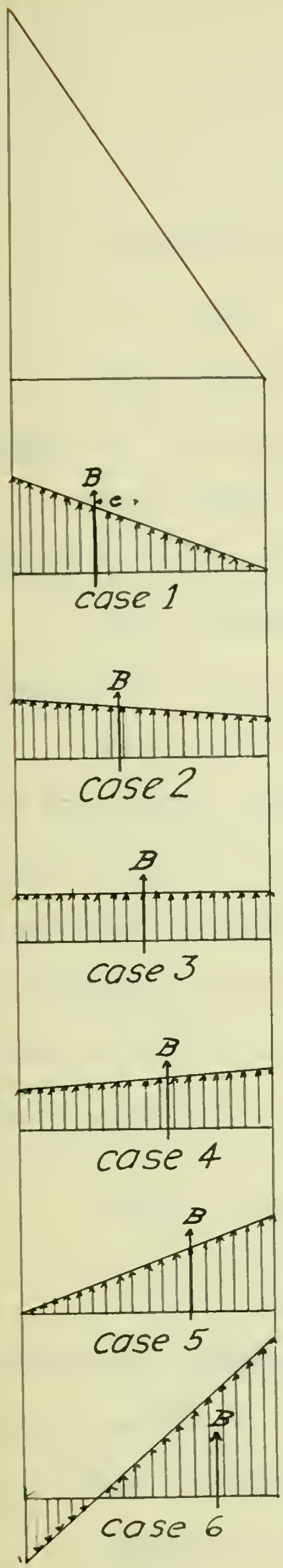


Fig. 1.

Considering a section of the dam one foot in length and assuming that it acts as an undeformable block, the stress at any point will be the sum of the stress due to the weight of the masonry and the stress due to the moment produced by the thrust of the water pressure on the face of the dam.

Figure 1 shows the six possible distributions of stress over any horizontal section, assuming that the stress varies uniformly over the section. In case 1 the reservoir is empty and the stress is due to the weight of the masonry alone. The other five cases show the distribution for different heights of water. In case 2 the resultant of the water pressure and the weight of the masonry cuts the base of the section a distance "e" upstream from the center of the base; in case 3 it cuts exactly at the center; in cases 4,5 and 6 it cuts at a distance "e" downstream from the center. In the case 6 ,the distance is so great that tension is produced at the toe of the section.

From a study of these distributions it may be seen that the stress at any point on the horizontal section will be equal to the sum of that which is produced if the upward pressure, B, were evenly distributed and the stress due to the moment of B about the center of the base.

$$S_1 = \frac{B}{b}$$

$$S_2 = \frac{My}{I} = \frac{Bey}{I} = \frac{Bey\ 12}{b^3}$$

$$S = S_1 \pm S_2 = \frac{B}{b} \pm \frac{Bey\ 12}{b^3} \text{ -------------------(1)}$$

For maximum values of S. y will be b/2.

then $S = \frac{B}{b}\left(1 \pm \frac{6e}{b}\right)$

For e less than b/6. compression will exist over the entire base. as in cases 2.3. and 4. Fig. 1.

For e equal to b/6. the stress will be zero at one end.as in cases 1 and 5. Fig. 1.

For e greater than b/6. tension will exist over part of the base.as in case 6. Fig. 1.

As it is not permissible that tension should exist according to the first condition requisite for stability. a stress distribution such as case 6 must not be allowed to occur. or the resultant of the water pressure on the face of the dam and the weight of the structure must cut the base of the section at a point not farther from the center of the section than b/6, or the " resultant must fall within the middle third."

Width of Base of any Horizontal Section in Terms of Height.

$B = W$; for $\Sigma F_v = W - B = 0$

and for a triangular section . $x = \frac{2b}{3}$;

and $W = bhw_1/2$; $P = wh^2/2$;

Taking moments about the toe. 0;

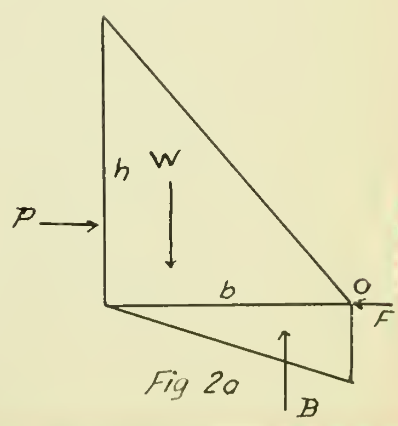


Fig 2a

$$\frac{bhw_1\, 2b}{6} - \frac{bhw_1\, b}{6} = \frac{b^2 hw_1}{6} \geqq \frac{wh^3}{6}$$

$$b = h\sqrt{\frac{w}{w_1}} \text{ ---------------------(2)}$$

Width of Base of Any Horizontal Section When Full Hydrostatic Pressure Exists over the Entire Base. This section must be designed so that the downward pressure at the toe will be equal to the hydrostatic pressure at that point.

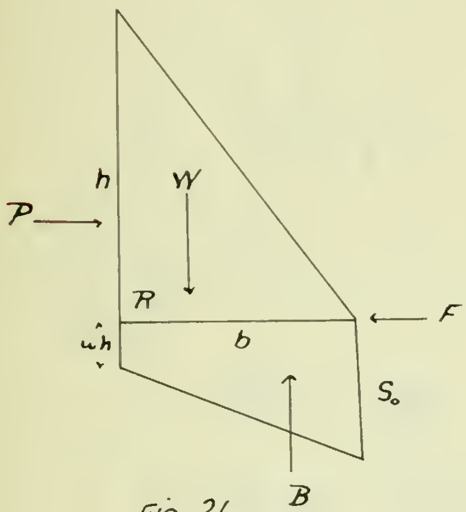


Fig. 2b.

$$W = \frac{bhw_1}{2}; \qquad P = \frac{wh^2}{2}; \quad B = (wh + S_0)\frac{b}{2}$$

$$wh = \frac{w_1 h}{2} - \frac{6w_1 h}{2}\,\frac{e}{b}$$

$$2bw = bw_1 - 3we$$

$$e = \frac{b}{6} - \frac{bw}{3w_1}$$

$$S_o = \frac{hw_1 b}{2b} + \frac{6w_1 bh}{2b\ 6} - \frac{6w_1 bhw}{2b3w_1}$$

$$= hw_1 - hw$$

Taking moments about R:

$$\frac{Ph}{3} + \frac{Wb}{3} = B\left(\frac{b}{2} + e\right)$$

$$\frac{wh^3}{6} + \frac{w_1 b^2 h}{6} = \frac{w_1 bh}{2}\left(\frac{2b}{3} - \frac{bw}{3w_1}\right) = \frac{w_1 b^2 h}{3} - \frac{wb^2 h}{6}$$

$$wh^2 + w_1 b^2 - 2w_1 b^2 + wb^2 = 0$$

$$b = h\sqrt{\frac{w}{w_1 - w}}$$

Total Shear on a Vertical Section.

(After Atcherly and Pearson, Engineering, March 31, 1905.)

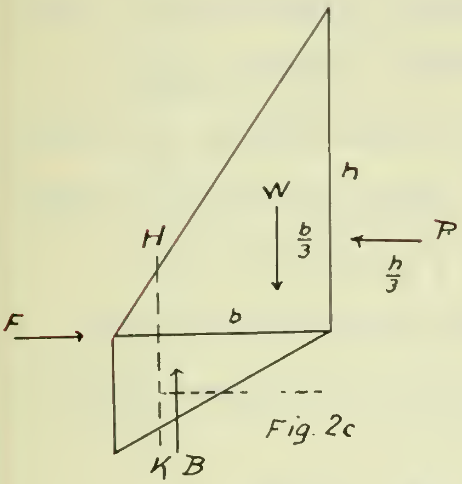


Fig. 2c

This dam has been designed so that the resultant will cut the edge of the middle third when the reservoir is full. Then the stress distribution will be triangular on the base.

Let $b = mh$.

Then $h_x = \dfrac{h(mh - x)}{mh}$

$$W_1 = h_x(mh - x)\frac{w_1}{2} = (mh - x)^2 \frac{w_1}{2m} .$$

$$S_x = \frac{S_o\, x}{mh} = \frac{w_1 h x}{mh} = \frac{w_1 x}{m}$$

$$B_1 = \frac{mh - x}{2}\left(S_o + S_x \right) = \frac{mh - x}{2}\left(w_1 h + \frac{w_1 x}{m}\right).$$

$$\therefore\ V_1 = B_1 - W_1 = \frac{mh - x}{2}\left(w_1 h + \frac{w_1 x}{m}\right) - \frac{w_1}{2m}(mh - x)^2$$

$$= (mh - x)\frac{w_1}{2}\left(h + \frac{x}{m} - h + \frac{x}{m} \right) = (mh - x)\,\frac{w_1 x}{m}$$

$$= w_1 h x - \frac{w_1 x^2}{m}$$ which is a parabola.

Intensity of shear on vertical section.

From equation 4. $V_h = w_1 hx - w_1 x^2/m$

Now if h+1 is substituted for h in the above equation, the shear on a vertical section of h + 1 height will result.

$$V_{h+1} = w_1 hx + w_1 x - w_1 x^2/m$$

And if v_v is the shear on a vertical section one unit high at a depth h below the crest.

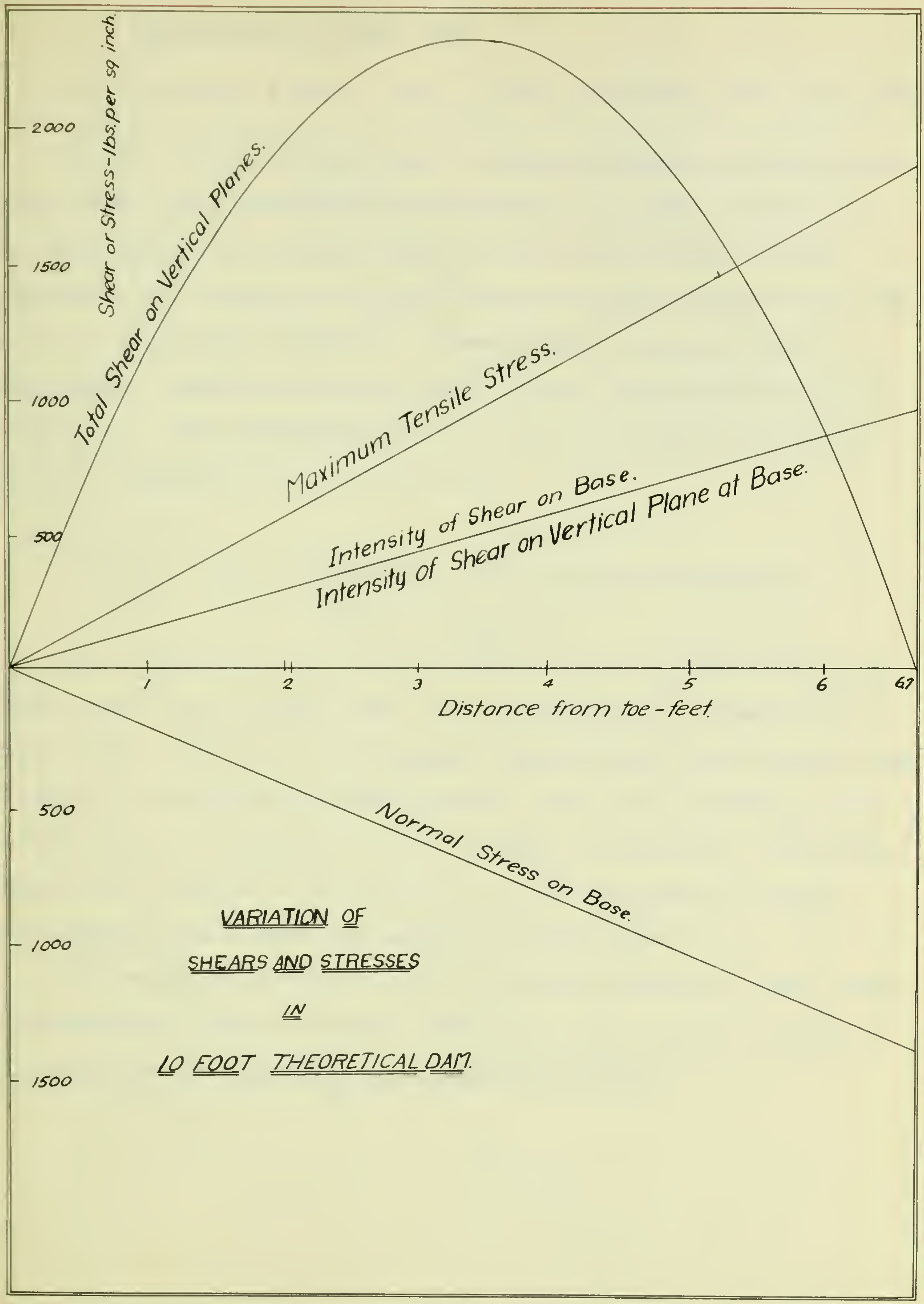
$$v_v = -V_h + V_{h+1} = + w_1 x; \quad \text{-----------------------} (5)$$

This equation is independent of h. Therefore the shear on a vertical section is constant from the base to the top of the dam.

Intensity of shear on a horizontal section.

It is proven in mechanics that the horizontal shear at any point is equal to the vertical shear at that point. Therefore the intensity of shear on a horizontal section at any point is equal to

$$v_h = w_1 x \quad \text{-------------------------} (6)$$



Shear or Stress - lbs. per sq inch.
2000
1500
1000
500
500
1000
1500
Total Shear on Vertical Planes.
Maximum Tensile Stress.
Intensity of Shear on Base.
Intensity of Shear on Vertical Plane at Base.
1
2
3
4
5
6
6.7
Distance from toe - feet.
Normal Stress on Base.
VARIATION OF
SHEARS AND STRESSES
IN
10 FOOT THEORETICAL DAM.

Experiments on model dams.

J.S.Wilson and W.Gore. Inst. of Civil Engineers. Vol. 172 page121.

Messers Wilson and Gore made a series of tests on india-rubber model dams. and measured the deformations in the dam sections by the deformation of ellipses drawn on the surface. Each ellipse represents the stress at the point where its axes intersect. The semi-axes. in length and direction. represent the principle stresses. The maximun shearing stress is equal to half the difference in the semi-axes. The diagrams on page 16 show the variation of the different stresses as thus determined.

J.W.Ottley and A.W.Brightmore. Inst. of Civil Engineers. Volume 172 page 89.

These experiments were made on clay models, 30 inches high. 26 inches wide and 12 inches long. They were moulded in frames with glass sides. Vertical and horizontal scratches were made on both the glass and the clay and the deformations under load determined from the difference in position of the two sets of scratches. The water pressure was applied to the face of the dam by water in a thin rectangular india-rubber bag made to fit the frame.

The graphs on page 16 show the results obtained. These curves are remarkably similar to those obtained by plotting the results of the equations develpped for the different stresses.

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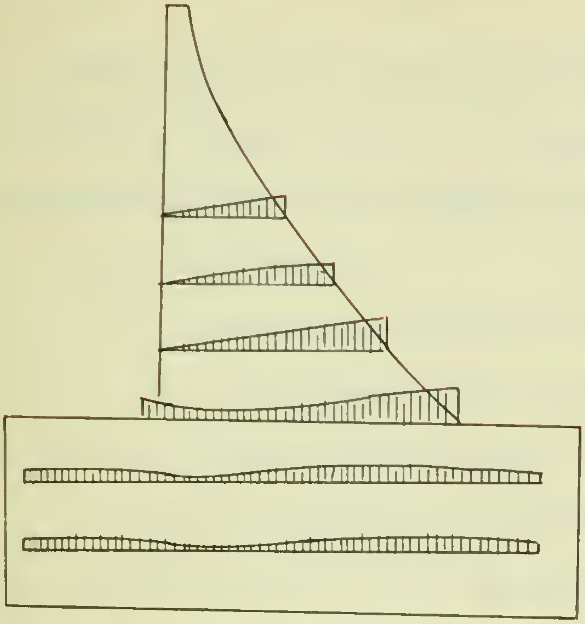
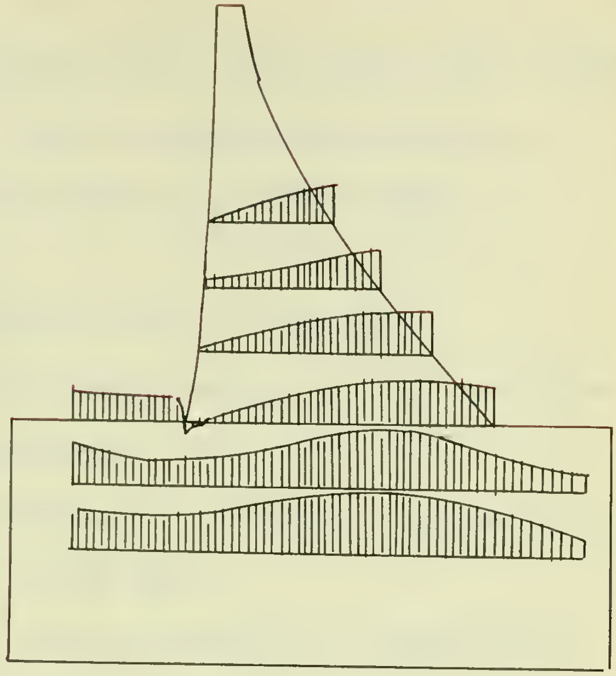
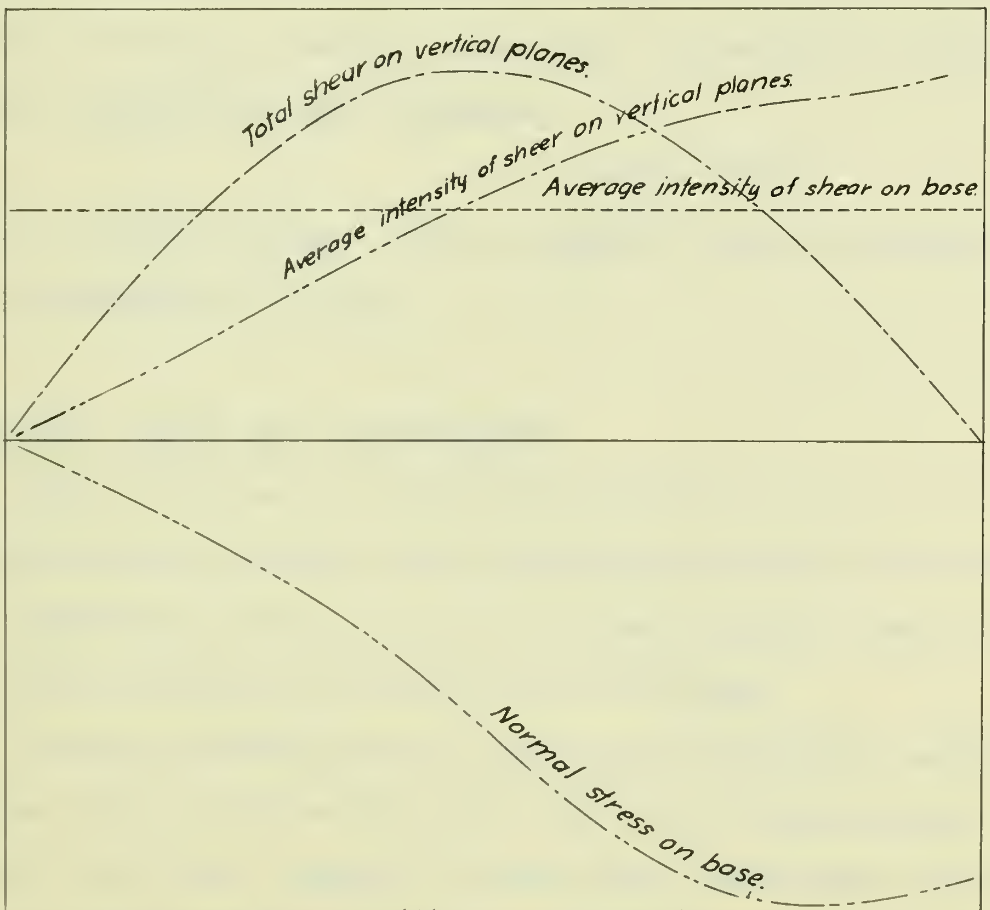


Shearing stresses on horizontal planes. Ends of foundation supported.



Vertical pressure on horizontal planes.

From experiments by Wilson and Gore.



From paper of Ottley and Brightmore.

Practical profile in gravity dams.

The practical profile in masonry dams is one which will contain the least volume and at the same time be sufficiently strong to resist all the forces which may come upon the actual dam.

These forces are

(1) The water pressure upon the face of the dam,

(2) the impact from floating bodies, waves and the pressure from ice on the surface of the reservoir.

(3) the upward hydrostatic pressure of water percolating in the foundation or in the masonry of the dam.

In the theoretical dam only the first force was considered, and it was found that to resist this force it was necessary that

(a) the line of pressure fall within the middle third,

(b) the pressure at any point must not exceed a certain safe stress when computed by the formulae,

$$S = \frac{W}{b}\left(1 \pm \frac{6e}{b}\right).$$

(c) the shear at any section must not exceed a certain safe stress as computed by $v_h = w_1 x$.

Floating bodies and ice pressure.

The force which may be exerted by expanding ice or an ice field under a high wind is equal to the crushing strength of the ice. The possible magnitude of this force will depend largely upon the altitude and latitude of the location of the dam.

The crushing strength of ice as given by different authorities varies from 100 to 1000 pounds per square inch depending upon the condition of the ice. Trautwine states that the thrust which may be exerted by a field of ice on a reservoir is probably not less

than 30,000 pounds per square foot. However few failures have been recorded as being caused by ice pressure and it is seldom considered except to make the top width of the dam large enough to resist the shearing effect of the thrust.

The thrust which may result from floating bodies striking the dam may be larger in intensity than the ice thrust , but will not be as large in extent, so that if provision for ice thrust is made, other forces of this character may disregarded.

The general practice in providing against these forces is to make the top width of the dam a certain width which is strong enough to resist the shearing effect of whatever ice thrust that may occur, and no account is taken of the overturning effect, except in the higher latitudes.

It is also necessary for the top of the dam to extend above the high water level so that the waves will not wash over the crest. This is chosen arbitrarily, depending somewhat upon the extent of the reservoir.

Another force of this chaacter which must be considered on dams located in positions where high winds occur, is the wind pressure upon the down-stream face of the dam when the reservoir is empty.

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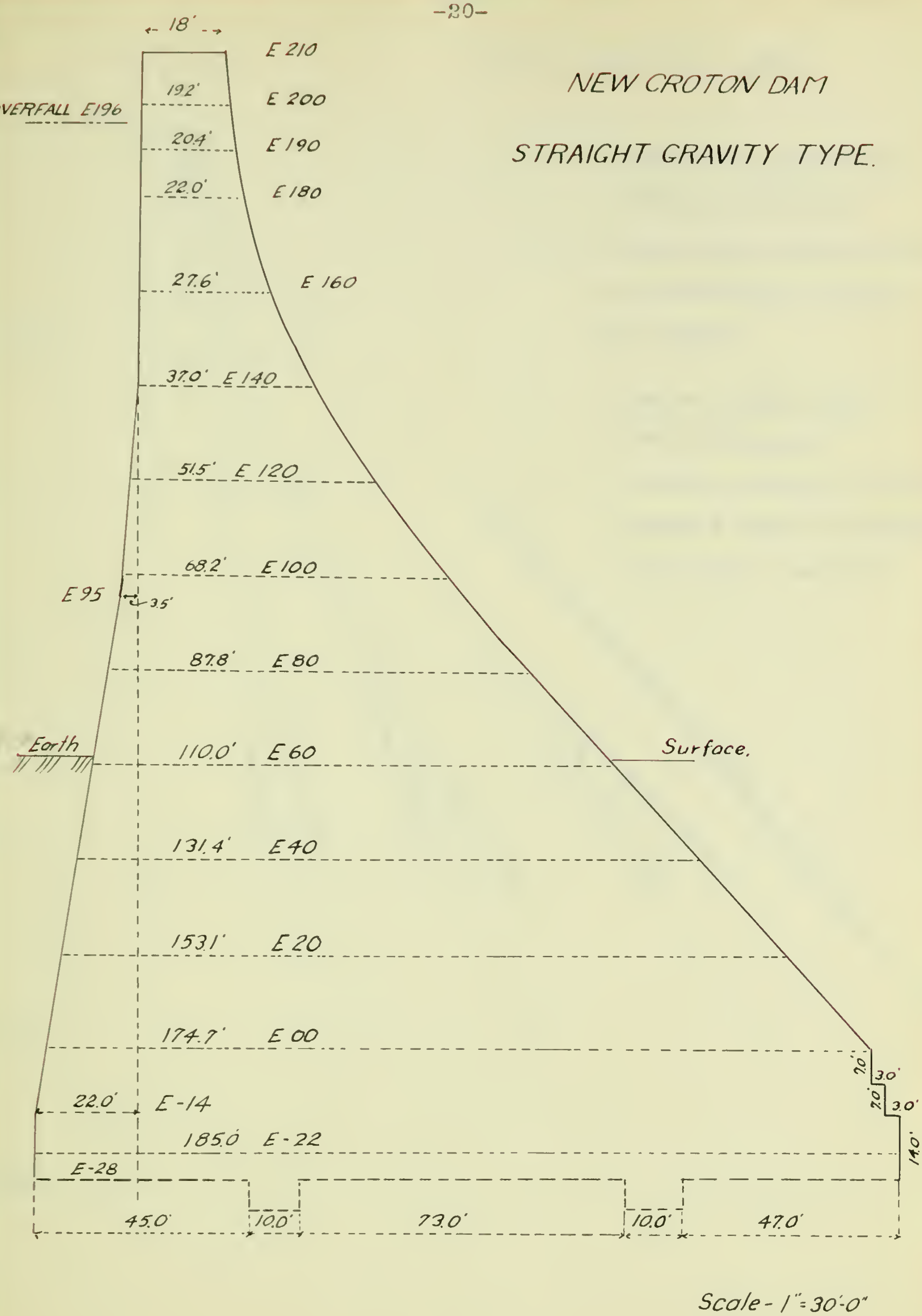
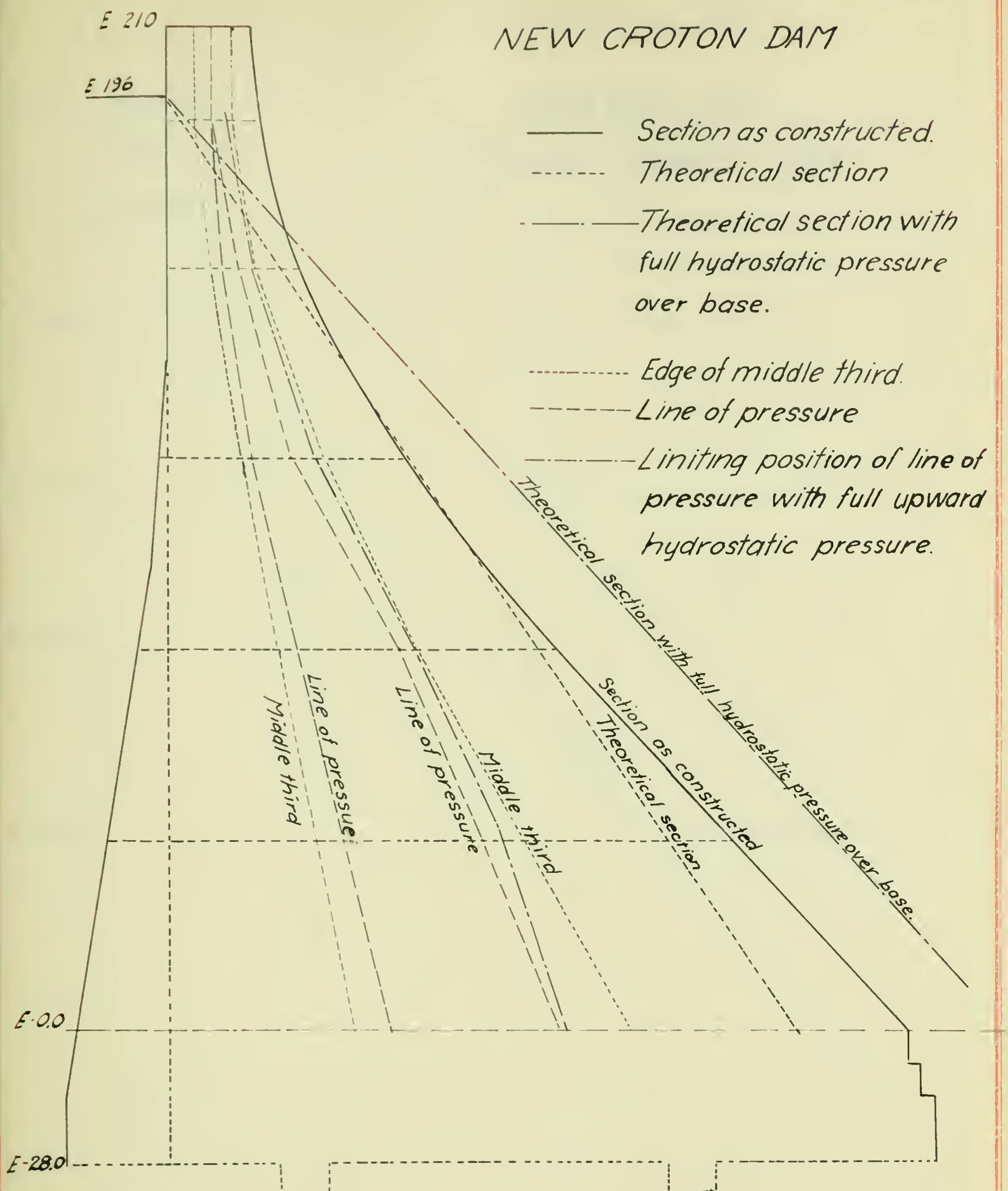
Hydrostatic pressure upon horizontal sections.

The maximun force which may result from the hydrostatic pressure of water percolating into the concrete at the face of the dam is full hydrostatic pressure over the entire base of the section. It is necessary that the downward pressure from the weight of the masonry above be equal to this pressure or the concrete will be placed in tension and be ruptured. Also if the downward pressure is not great enough to equalize this uplift. an enormous overturning moment will result which is likely to be large enough to overthrow the dam.

Several recent dams have been designed with provisions for resisting uplift. The most conservative provision thus far has been to assume full upward hydrostatic pressure over the entire base of the horizontal sections. Another condition which is sometimes assumed is that the magnitude of the uplift is equal to the full hydrostatic pressure at the face of the dam and diminishes to zero at the toe. In either case it is necessary that the downward pressure from the masonry be equal to the full hydrostatic pressure at the face.

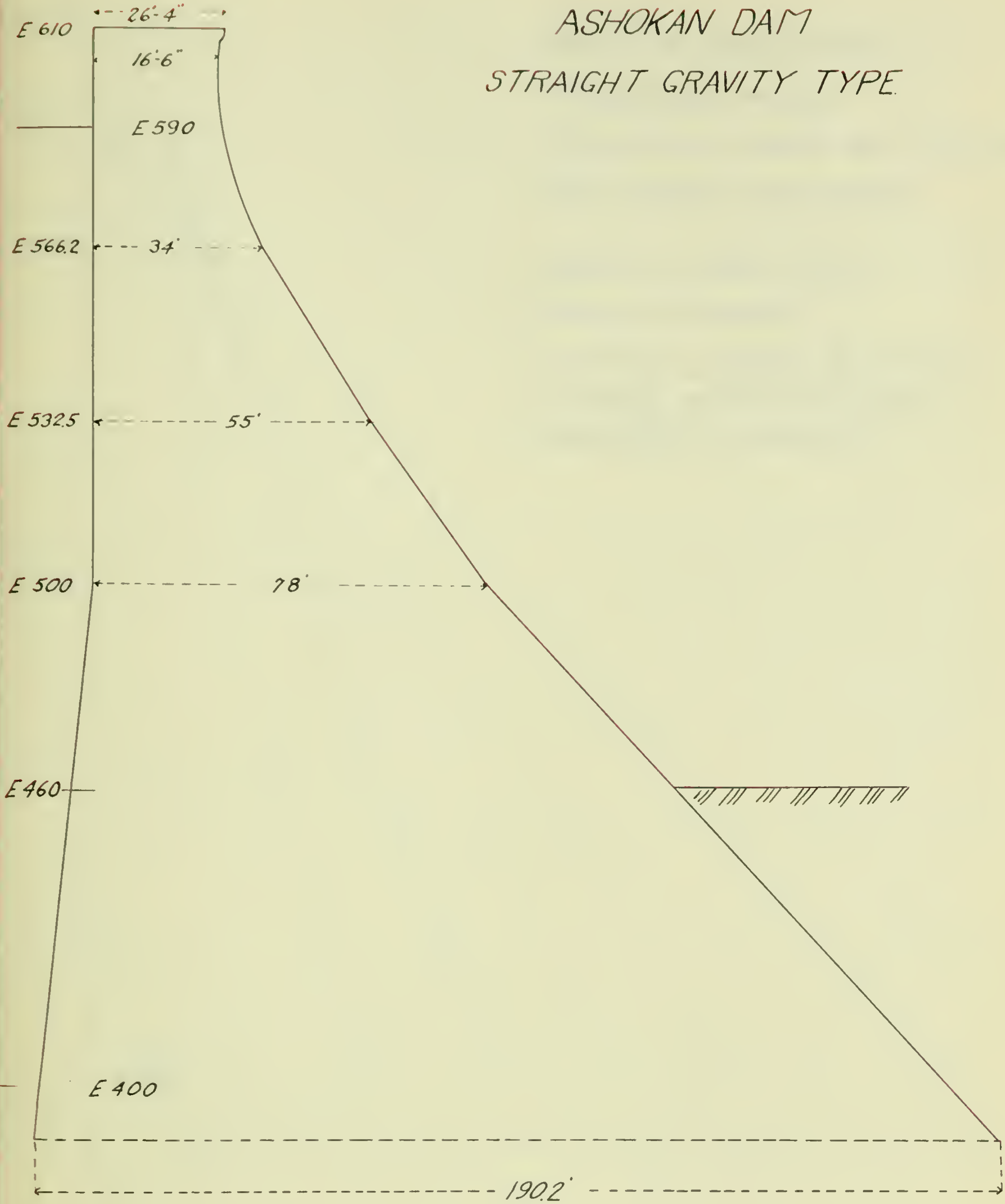
Whatever may be the actual pressure from the uplifting effect of water in the masonry, it is very desirable that some provision be made for the force. The present knowledge of the action of water in masonry indicates that it is only safe to assume that the uplift at the face of the dam is equal to full hydrostatic pressure and that the downward pressure from the masonry should be equal to this amount.


NEW CROTON DAM
STRAIGHT GRAVITY TYPE.
18'
E 210
19.2'
E 200
OVERFALL E196
20.4'
E 190
22.0'
E 180
27.6'
E 160
37.0' E 140
51.5' E 120
68.2' E 100
E 95
3.5'
87.8' E 80
Earth
110.0' E 60
Surface.
131.4' E 40
153.1' E 20
174.7' E 00
7.0'
3.0'
7.0'
3.0'
22.0' E -14
185.0 E -22
14.0'
E -28
45.0'
10.0'
73.0'
10.0'
47.0'
Scale - 1" = 30'-0"


E 210
NEW CROTON DAM
E 196
Section as constructed.
Theoretical section
Theoretical section with full hydrostatic pressure over base.
Edge of middle third.
Line of pressure
Limiting position of line of pressure with full upward hydrostatic pressure.
Theoretical section with full hydrostatic pressure over base.
Section as constructed
Theoretical section
Middle third
Line of pressure
Line of pressure
Middle third
E·0.0
E-28.0

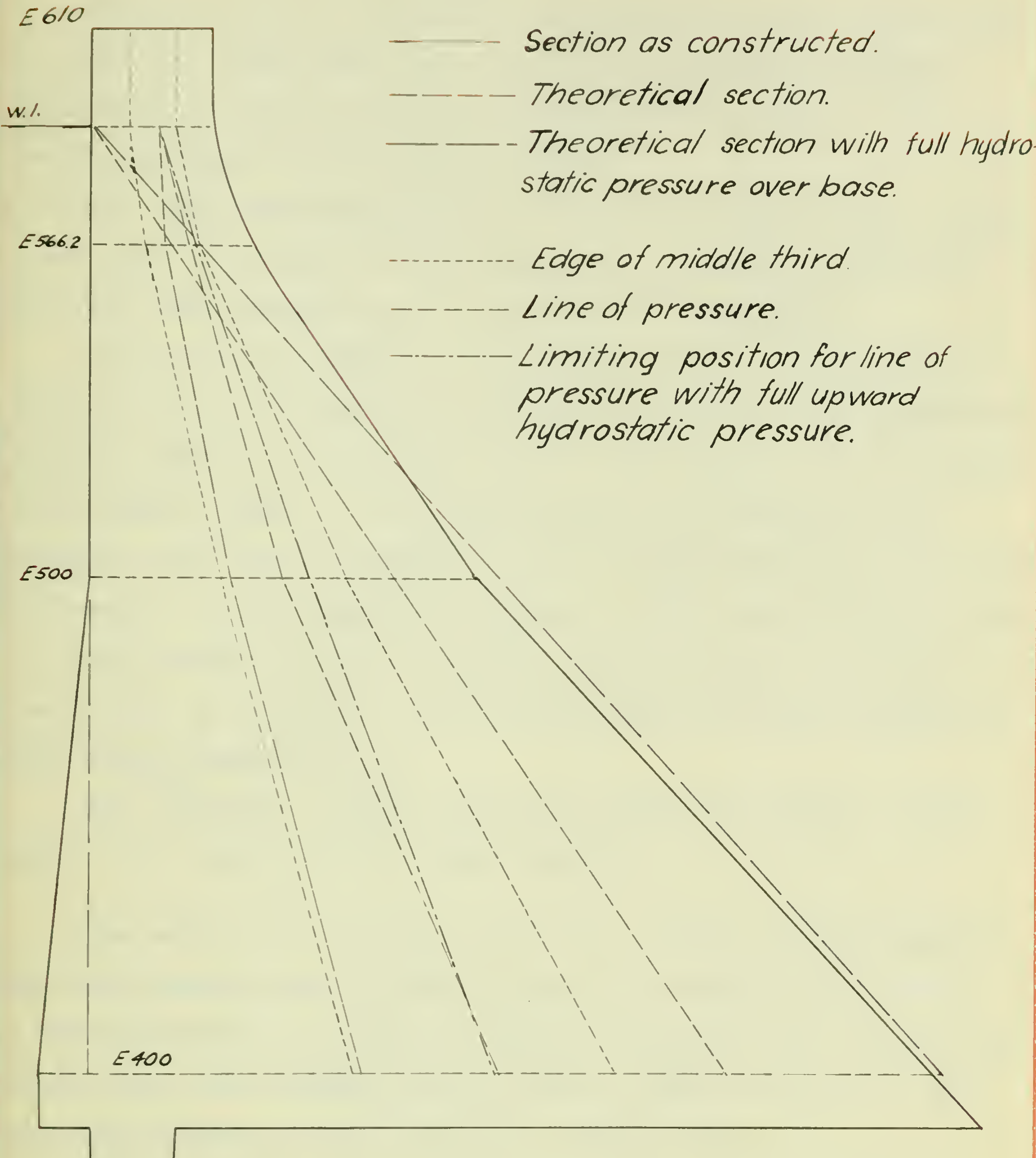
ASHOKAN DAM

STRAIGHT GRAVITY TYPE



Scale- 1"= 30'-0"

ASHOKAN DAM



E 610
W. L.
E 566.2
E 500
E 400
Section as constructed.
Theoretical section.
Theoretical section with full hydrostatic pressure over base.
Edge of middle third.
Line of pressure.
Limiting position for line of pressure with full upward hydrostatic pressure.

Examples of Straight Gravity Dams.

Probably the most thoroughly investigated and best designed dam of this type is the New Croton. This dam was designed by Edward Wegmann and is given a complete description in his book, "The Design and Construction of Masonry Dams."

This dam, the maximum section of which is shown on sheet 20,has a top width of 18 feet, a super-elevation of 14 feet, a height of 238 feet above foundations and a base width of 185 feet.

On sheet 21 is shown: (1) the actual section, (2) the theoretical section.( ) the theoretical section required if full hydrostatic pressure exists over the entire base of every section, (4) the edges of the middle third,(5) the lines of resultant pressure for the reservoir both full and empty. (6) the limiting position of the line of pressure if full hydrostatic pressure exists over the entire base.

The position of these lines was determined using the weight of the masonry as 140 pounds per cubic foot and the weight of water as 62.5 pounds per cubic foot.

The theoretical profile falls well within the actual. but the profile for uplift is not so well included.

The Ashokan is a later and also well designed straight gravity dam. This dam,as shown on sheet 22, has a top width of 16.5 feet, a super-elevation of 20 feet. a height of 220 feet. and a base width of 190 feet. The diagrams for this dam are shown on sheet 23. The same unit weights as in the previous dam were used.

The two designs are relatively the same in their relation to the theoretical. except that the Ashokan is somewhat more conservative.

[illegible]

Curved Gravity Dams.

Curved gravity dams resist water pressure by a combination of vertical cantilever and arch action. Many dams of this type have been designed to be stable as vertical cantilevers and then built on a curved plan to add to their stability. This is decidedly uneconomical because the increase in volume of the dam caused by the increase in length is greater in proportion than the increase in stability.

However in some case it is advisable, or even necessary, to use a curved plan, as was the case with the Lake Cheesman dam. In this dam the foundation materials were such that a curved plan had to be used to secure foundations which were at all satisfactory. An extensive investigation as to the relative proportion of the load carried by the vertical cantilever and the arch was carried out during the design of thisstructure. A complete report of this investigation is given in the Transactions of the American Society of Civil Engineers. Volume 53, page 90.

The chief difficulty encountered in determining the strength of a curved gravity dam is to determine the relative proportion of the total water pressure carried by each action. In the investigation above mentioned formulae were developed which proved satisfactory. but the amount of computations required is enormous. A more convenient formula and method is suggested by Mr. R.Shirreffs in his discussion of the preceding paper.

These formulae are too lengthy to repeat here and will only be refered to.

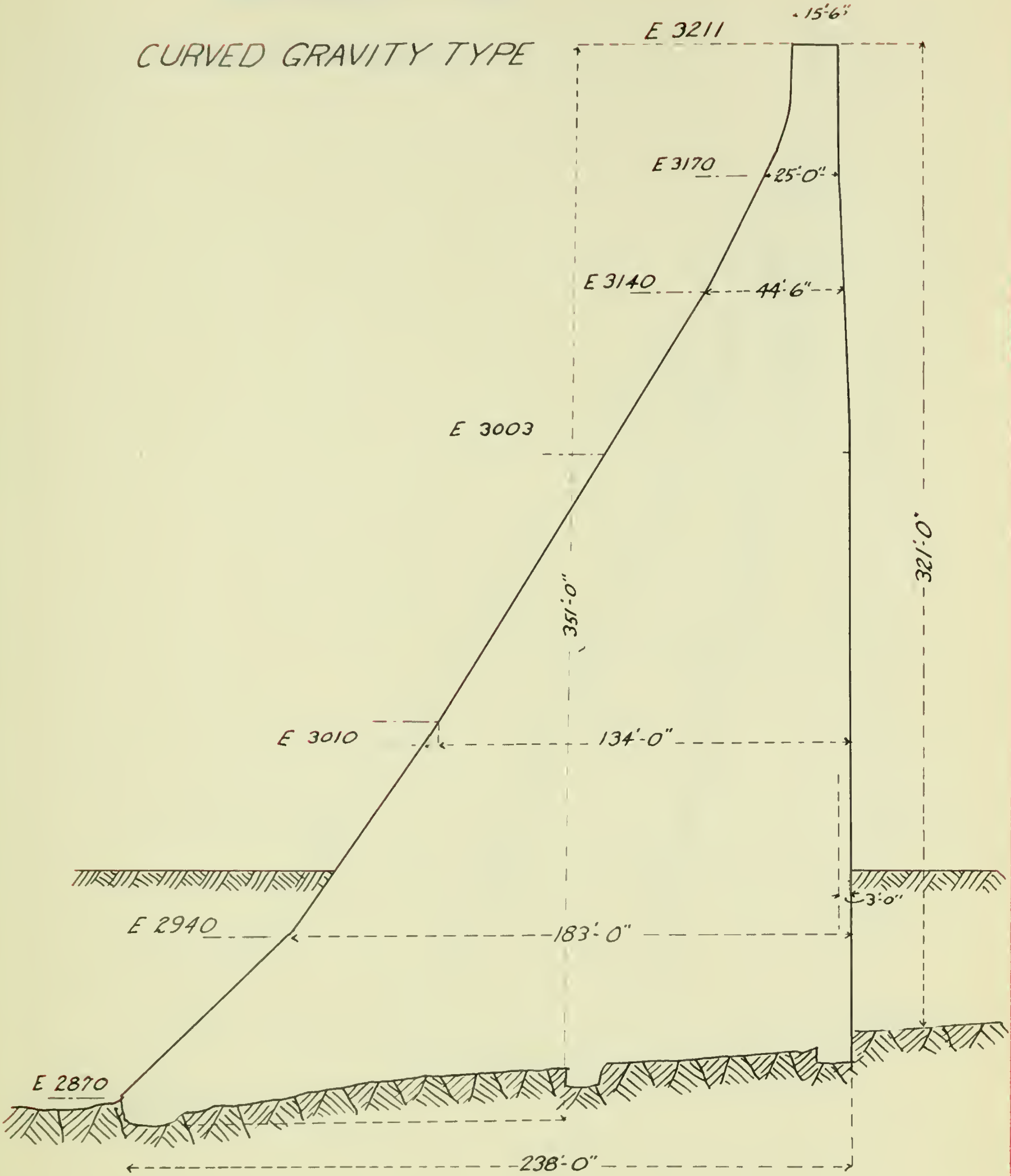
THE ARROWROCK DAM-CURVED GRAVITY TYPE.

A typical example of the curved gravity type is the Arrowrock dam built by the United States Reclamation Service on the Boise project in Idaho. When the reservoir is full, this dam would be on the verge of distruction as a straight gravity dam. With the water line below the high mark, the dam is perfectly safe as a straight gravity structure.

The strength of the structure as a pure arch, in the lower sections, is more than sufficient to carry the water pressure.

ARROWROCK DAM.

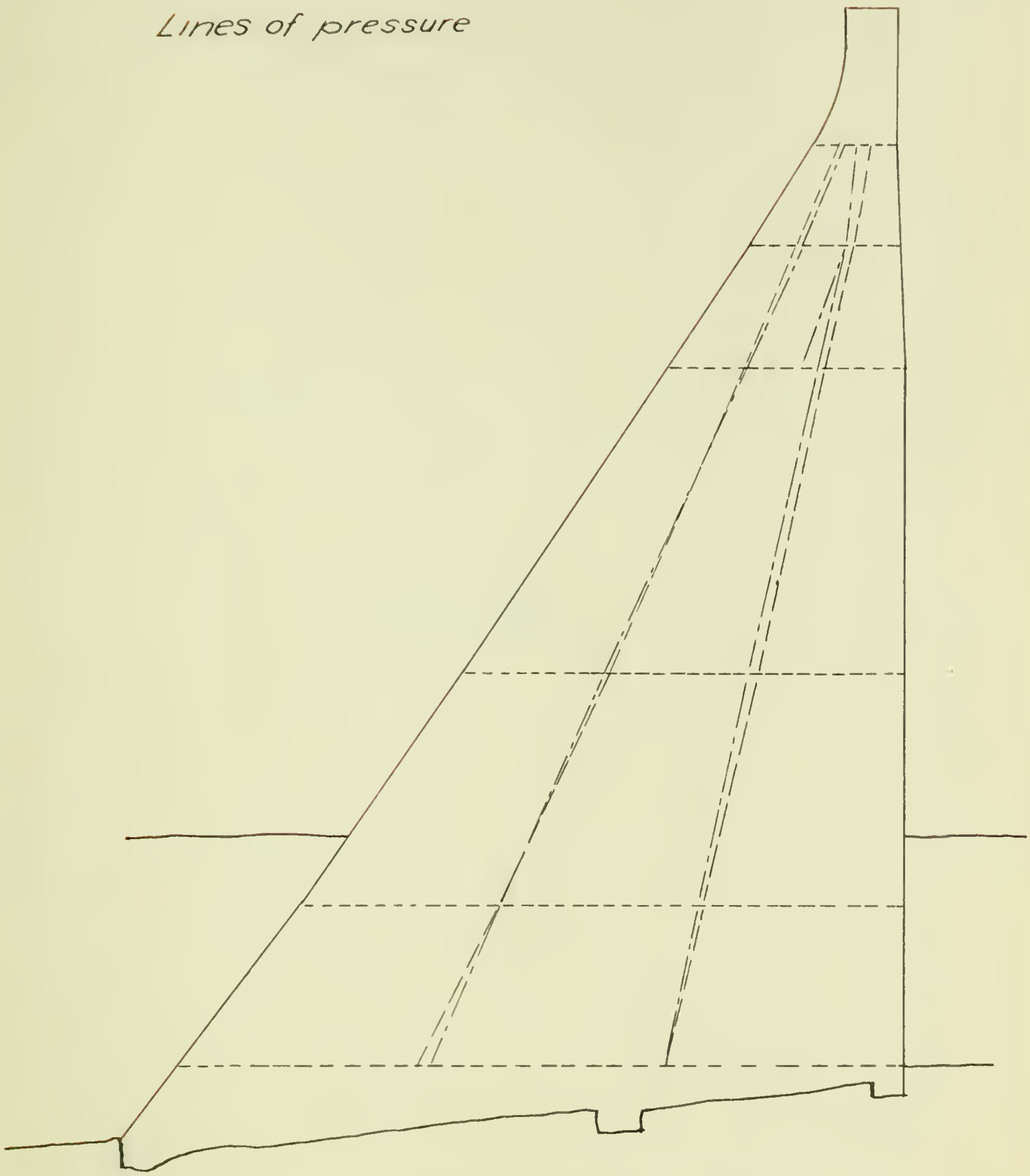
CURVED GRAVITY TYPE



Scale: 1" = 50'-0"

ARROWROCK DAM

Lines of pressure



ARCH DAMS.

Arch Dams.

Arched dams carry the horizontal thrust of the water by arch action to the abutments at the ends of the arch. These abutments are usually the canyon walls in dams consisting of a single arch and are artificial piers in multiple arch dams.

Theoretically the arch acts as a ring. The water pressure is prependicular to the face of the dam at all points and will always be radial so that the resulting stress in the arch will be axial compression. Such a stress is especially favorable for the use of concrete and a much higher working stress may be used in design than if there were likelihood of tensile stresses occuring.

The Theoretical Arch Dam.

Assuming that the dam offers no resistance as vertical cantilevers, and that each arch lamina supports the water pressure against itself independent of the other lamina, the axial compressive stress in a lamina of unit depth will be

$$T = w\ h\ R_m \text{ ---------------------- } (7)$$

Where T = the axial compression in the lamina.

w = weight of water per cubic foot.

h = depth of the lamina below the surface of the water.

R_m = the mean radius of curvature of the lamina.

Then if "s" is the allowable compressive stress in the concrete, and "b" is the thickness of the arch lamina at a depth "h".

$$b = \frac{w\ h\ R_m}{s} \text{ -------------------- } (8)$$

In the above formulae, "b" varies as "h", and the resulting dam will be a symmetrical triangle in vertical section.

The Economic Radius.

From equation (8), the bottom width of a dam of H height will be

$$B = \frac{w\ H\ R_m}{s} \text{ -------------------- } (9)$$

Then the area of a vertical section will be

$$A = \frac{B\ H}{2} = \frac{w\ H^2\ R_m}{2s}$$

And the volume, where "L" is the mean length of the center line of the dam and θ is half the subtended angle,

$$V = A\ L = \frac{(w\ H^2\ R_m)(R_m\ 2\theta)}{2s}$$

Now if "C" is the mean width of the canyon,

$$R_m = \frac{C}{2 \sin \theta}$$

$$V = \frac{wH^2C^2\theta}{4s \sin^2\theta} = \frac{K\theta}{\sin^2\theta}$$

If a curve is plotted between values of θ and $\frac{\theta}{\sin^2 \theta}$. the value of θ when $\frac{\theta}{\sin^2 \theta}$ .and therefore the volume.is a minimum will be found to be 66 degrees 47 minutes.

Then the equation for the mean radius for least volume

$$R_m = \frac{C}{2 \sin \theta} = .548\ C' \text{ ------------------ } (9)$$

In this formula C' is the width of the canyon at depth h.

The Practical Profile in Arch Dams.

The practical profile of arched concrete dams will be required to resist the following forces:

1. The hydrostatic pressure on the face of the structure,
2. Impact from floating bodies, and the thrust from ice,
3. The weight of the structure.

The thickness of the arch necessary to resist the hydrostatic pressure will be determined from the formula:

$$b = \frac{R_m wh}{s} \quad \text{-------------------}(10)$$

The top thickness and the super-elevation will be about the same as required by a gravity dam. While these dimensions are not usually made as large as in the former structure, they are determined by the same factors. The arch has somewhat the advantage over the gravity dam in resisting these stresses in that they do not affect the design of the structure below the crest to any great extent.

The dimensions required to support the weight of the structure may control the design of the lower sections in some special cases. This would only be the case with dams of very short radii where the thickness necessary to support the axial compression is small. This stress is much more important in arch dams than in gravity dams because the width of the base is small with respect to the height.

Arch dams are designed in two general forms, with a straight vertical axis, and with a curved vertical axis.

These two forms are subdivided into the following types.

I Straight vertical axis.

1. Constant-angle arch dam.
2. Constant-radius arch dam,

As proposed by Lars Jorgensen.

II Curved vertical axis.

1. Warped arch

As designed by Mr. Williams for the Six-mile Creek dam at Ithica, N.Y.

Principles of design.

The first arch dams were designed with constant or nearly constant radii, similar to the theoretical dam. The chief objection to this type is that the base is attached to the foundation so that no arch action can exist at this point and all the water thrust must be carried by vertical cantilever action.

The two latter types, the constant-radius and the constant-angle arches, were developed in an attempt to remedy this defect. With the constant-angle dam in a V-shaped canyon the radius will shorten as the bottom is approached, thus giving the arch a sharper curvature than the constant-radius dam. The shorter radius will decrease the stress to be carried and also the deflection necessary to carry the load, so that more arch action can take place. In the warped arch the lower portion is made spherical so that the water thrust can be carried wholly by arch action. This dam resulted in a great saving of material but was rather radical.

The Constant-Radius Arch Dam.

Design.

1. Determine the mean width of the canyon, C, and the radius, R_m, from the formula

$$R_m = .548\ C.$$

Determine the width of arch required at different elevations by the formula

$$b = \frac{.548\ w\ h\ C'}{s}$$

where C' is the width of the canyon at the depth h below the crest.

3. Assume a top width between 3 and 12 feet, depending upon the probably magnitude of ice thrust and wave impact, and a supre-elavation depending upon the probably size of waves.

4. Investigate the stresses in the base of the structure for the reservoir both full and empty.

If "C" is greater than 150 feet the dam will have considerable stability as a gravity dam and should be designed as a curved gravity structure.

THE SHOSHONE DAM- CONSTANT-RADIUS TYPE.

The Shoshone dam is one of the best examples of modern practice in constant-radius arch dam design. The upstream face is battered .25 feet per foot, and the downstream face is battered .15 feet per foot, to the elevation of the stream bed. Below this point both faces are vertical. The radius of curvature of the center line is 150 feet.

Although the structure has considerable stability as a gravity dam, its strength as an arch is more than sufficient to carry the entire water pressure.

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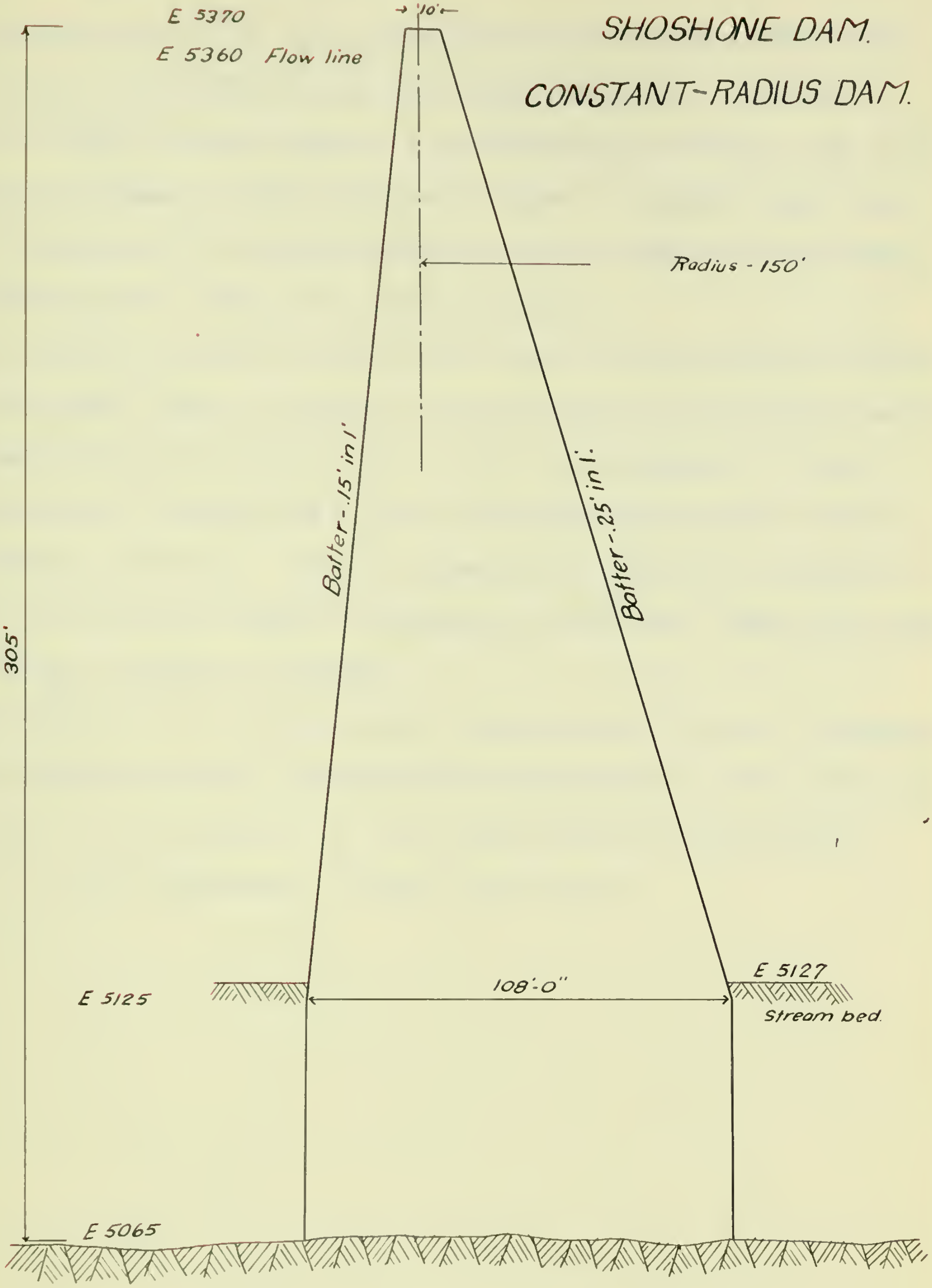

SHOSHONE DAM.
CONSTANT-RADIUS DAM.
E 5370
E 5360 Flow line
10'
Radius - 150'
Batter - .15' in 1'
Batter - .25' in 1'
305'
E 5125
108'-0"
E 5127
Stream bed.
E 5065

The Constant-Angle Arch Dam.

In the theoretical determination of the economic radius for arched dams it was proved that the least volume for a dam in a canyon of width C occurs when θ = 66 degrees,47 minutes. Mr. Lars Jorgensen proposed to build the dam to have a constant angle from top to bottom and to vary the radius with the width of the canyon, thus obtaining th greatest economy possible.

The method of design is similar to that of the constant radius arch dam except that the radius is varied for the succeeding lamina. each lamina being made from 20 to 30 feet in thickness. There is one objection to having the value of 2θ exactly constant; the top will overhang the base. This is remedied by varying θ between 65 degress at the top to about 70 degrees at the base, which will make but a slight increase in volume.

No dams of this type have been constructed, but Mr. Jorgensen gives a design of such a structure in his article on "The Constant-Angle Dam," which appeared in the Transactions of the American Society of Civil Engineers, Volume 78,page 685.

[illegible]

[illegible]

[illegible]

[illegible]

[illegible]

The Warped Arch Dam.

This type of dam was developed for the "Six-Mile Creek Dam" near Ithica.N.Y. ,where a dam of this type with a slight modification was constructed. This dam is fully described by its designer, Mr. G.S.Williams, in the Transactions of the American Society of Civil Engineers. Volume 57,page 182.

"One of the chief criticisms directed against arch dams has been that, by reason of the rigidity of the base, the arch action could not be developed in their lower part, and------ to overcome this objection, and to overcome as far as possible stresses of opposite sign acting at right angles to each other, a condition which certainly weakens the material's ultimate capacity to resist either one, recourse was had to a design similar to that introduced in an egg-shaped boiler.and the base was made the shape of a torus or ring."

The crest was sloped at an angle of 45 degrees to allow ice to slide over it and also to discharge water away from the foundation of the base.

Computation of stresses.

For the preliminary computations, the formula $T = pR$, was used. Where the faces are inclined as in this dam a more exact formula is $T = pR \sec. i$. The following formulae were used in the final computation of stresses.

$$H = \frac{T}{R} \text{ -------------------- } (11)$$

$$H = (p_w - (P_n + P_{n-1}) \sin \frac{\theta}{2}) \sec i + \Delta G \tan i$$

$$= 2T \sin \frac{\psi}{2} \text{ ----------------- } (12)$$

$$t = \frac{T}{F} = \frac{HR}{F} \quad \text{------------------}(13)$$

$$s = \frac{P_n}{F_b} \quad \text{------------------}(14)$$

In making the investigation of the stresses the dam was divided into 31 sections and the values of "t" and "s" were computed for both full reservoir and a 10-foot flood.

Nomenclature.

P_w = water pressure acting normal to the face of the block.

P_n = component of the total pressure normal to the base.

P_{n-1} = normal component to the top of the block.

ΔG = weight of the block.

T = horizontal thrust in the arch ring.

θ = the angle between the top and bottom faces of the block.

ψ = the angle between the sides of the block.

i = the angle between the normal to the upstream face and the horizon.

H = the total force carried by the horizontal arch.

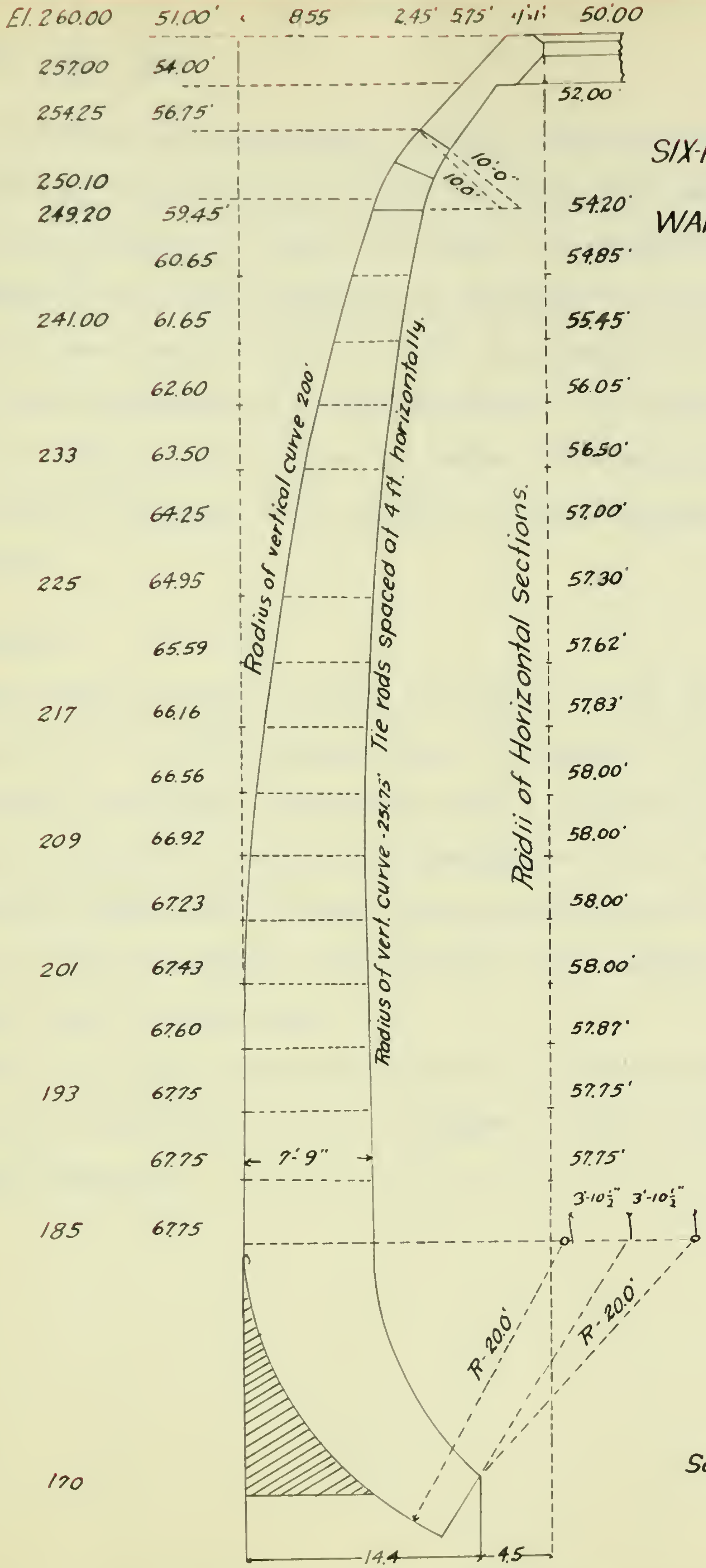
F_v = the area of the vertical face.

F_b = the area of the normal face.

t = the unit pressure on the horizontal arch at the center of the area.

s = the unit pressure on the vertical arch at the center of the base of the area.

R = the horizontal radius of the upstream face.



SIX-MILE CREEK DAM

WARPED ARCH TYPE.

Scale-1" = 10'-0"

MULTIPLE ARCH DAMS.

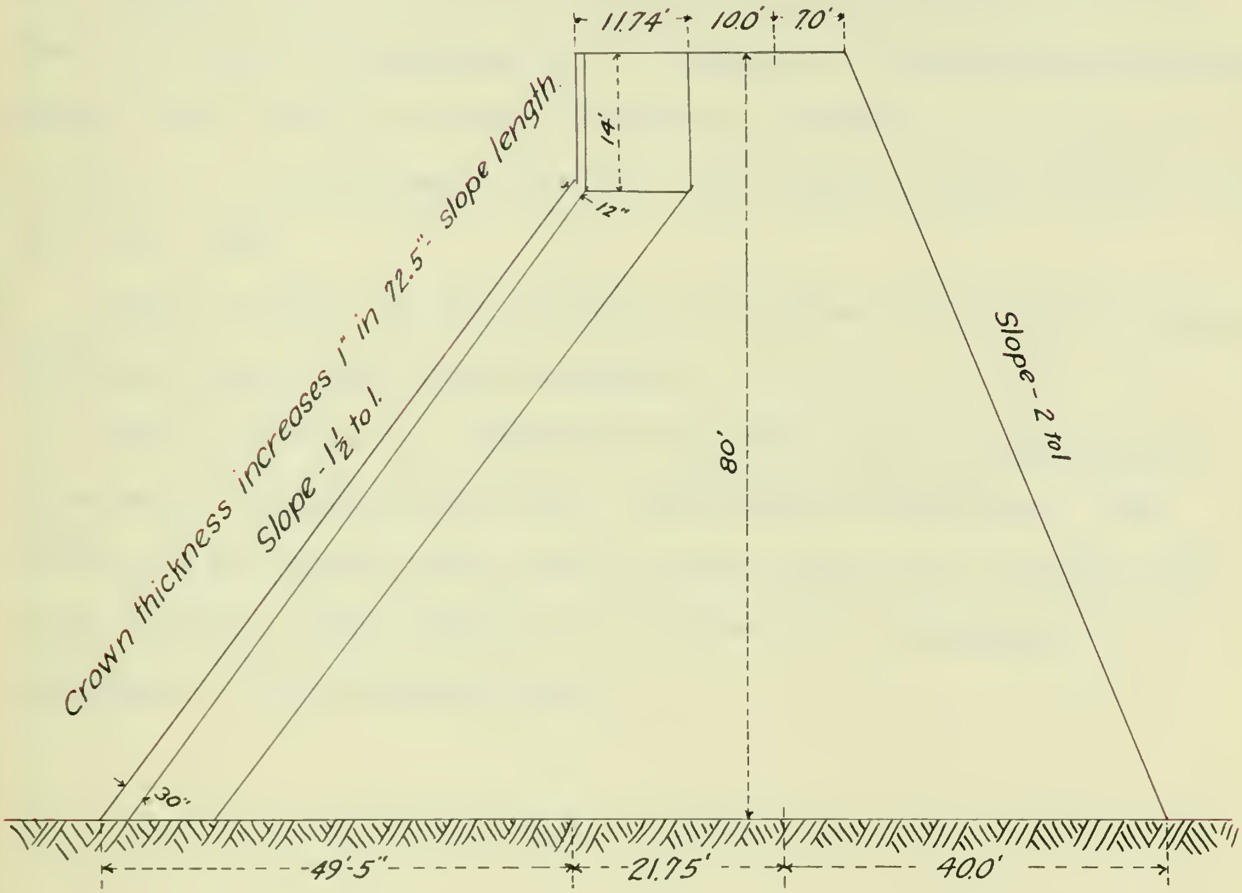
In an arch dam the thrust, $T = pR$, varies directly with R, and becomes very large when the radius is long. This makes a thick arch ring necessary to carry the load. In the multiple arch dam the thickness of arch ring required is decreased by shortening the span and therefore the radius. This is accomplished by placing artificial buttresses at intervals across the valley and then spanning the openings by a series of arches. The resulting dam is very economical of material but expensive of construction per unit of volume.

The Design.

The arched face is usually inclined so that the vertical component of the water pressure may be utilized to increase the resistance to sliding. The first step in design is to select a spacing for the buttresses. The dimensions of the buttress are then determined. These must be such that the buttress will resist overturning, shear, crushing, and sliding, when carrying the water pressure from one arch span.

The arch proper is then designed to carry the water pressure between the buttresses. It is customary to make the top portion of the arch vertical.

NEW BEAR VALLEY DAM.
MULTIPLE ARCH TYPE.



The New Bear Valley Multiple Arch Dam.

The New Bear Valley dam is one of the best examples of Multiple Arch dam construction. It consists of a series of 10 arches, each 32 feet clear span, abutting on 11 butresses. The total length is 363 feet and the maximum height 92 fe-t.

The slope of the downstream face up to with 14 feet of the top is 1 1/2 to 1 or 36 degrees 52 minutes. The downstream slope of the butress is 2 to 1. The butress is 1.5 feet thick at the top and increases in thickness with a batter of .016 feet per foot of height or 1 to 60. on each side for all heights. The arc of the extrados is 140 degrees 8 minutes, the radius 17 feet and the rise is 11.22 feet.

The top radius of the intrados is 16 feet, the arc 145 degrees 8 minutes, and the rise 11.74 feet

The butresses are supported by strut members consisting of T-beams and a supporting arch, both heavily reinforced. The T-beam is 12 inches thick and 2.5 feet wide with a 12 inch stem on an arch 12 inches square at the crown and thickening to 15 inches at the springing line.

REINFORCED CONCRETE DAMS.

Reinforced Concrete Dams.

Reinforced concrete dams, although allowing a great saving in concrete, are only practicable for the lower dams, because of the large amount of steel required in the higher structures. Dams of this type are most economical as earth retaining walls, but have been used with considerable success, especially the hollow type, for dams of low and medium height. The great advantage of this type of dam is that the tension reinforcement can be placed wherever it is needed. For this reason the structure may be built with comparative safety on gravel and other pervious foundations.

Cantilever and Buttresses Wall Dams.

These types are designed similarly to the earth retaining walls. The principle of design is to support a vertical wall on a horizontal floor by either a buttress or cantilever reinforcing, depending upon the type. The entire structure must be stable against sliding, overturning and excessive stresses on both the concrete and the foundation. In these types the amount of steel required is enormous for the higher structures.

The Cellular Reinforced Concrete Dam.

Dams of this type are designed as a series of vertical compartments to be filled with some solid material to provide the weight necessary for the stability of the structure. The advantages of this type is the small amount of material required; the disadvantages are the high unit cost of construction and the liability of a total failure if a single small crack occurs and allows the filling material to escape.

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HOLLOW REINFORCED CONCRETE DAMS.

The hollow reinforced concrete dam consists of an inclined buttressed wall carried on a horizontal floor. The face is inclined upstream so that the vertical component of the water pressure will add to the stability of the structure. This type is the most practicable of the reinforced concrete dams and has been used successfully on many large structures.

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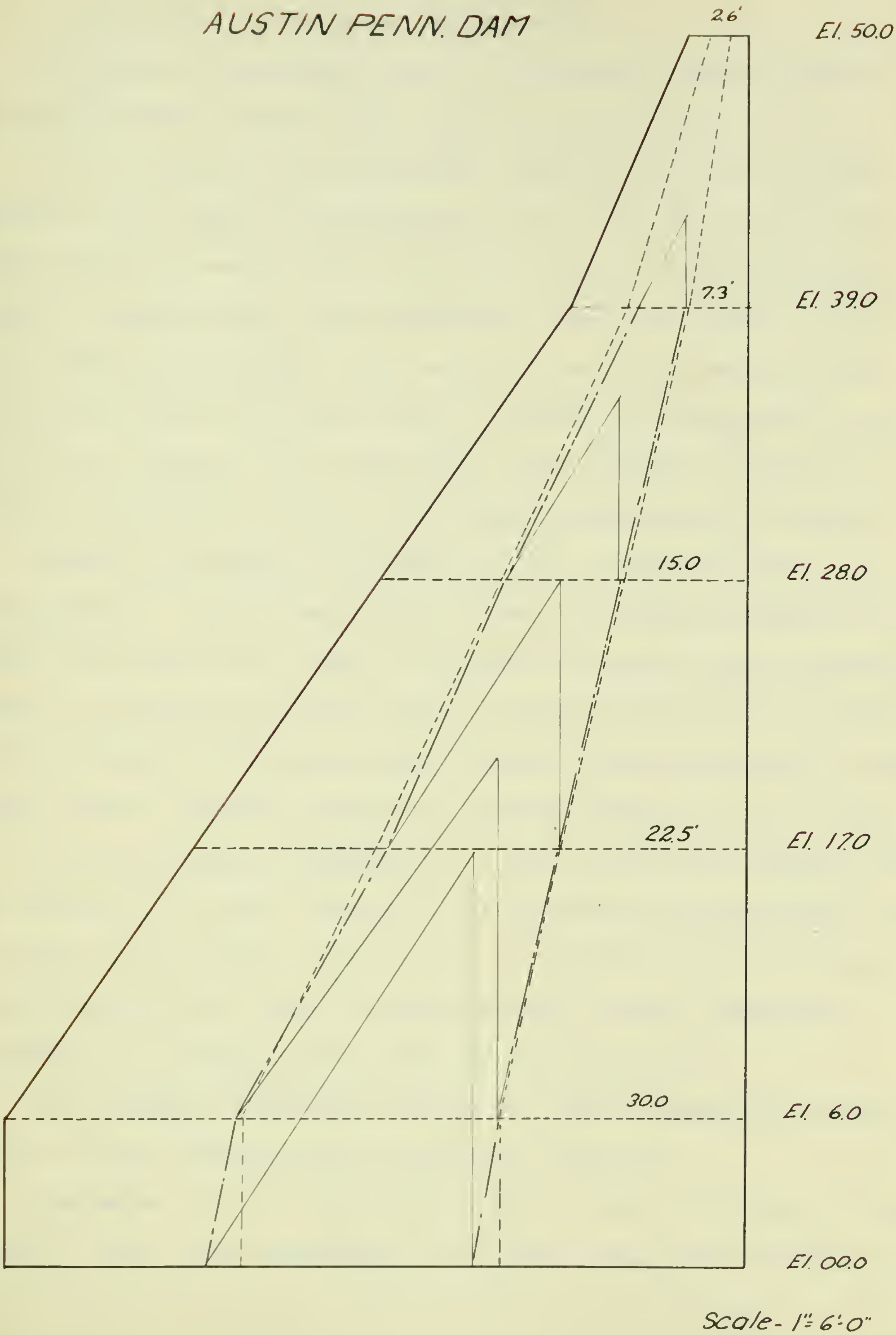
FAILURES OF DAMS.


AUSTIN PENN. DAM
2.6'
El. 50.0
7.3'
El. 39.0
15.0
El. 28.0
22.5'
El. 17.0
30.0
El. 6.0
El. 00.0
Scale - 1" = 6'-0"

The Austin Dam Failure.

(From paper by Professor McKibben.)#

The Austin Pennsylvania dam is a straight gravity concrete structure across Freemans Run a short distance above the town.

At the time of the preliminary failure in 1910 two cracks appeared and a land slide occurred at the east end of the dam. A large amount of water passed under the dam and came up thru the natural groundseveral feet downstream. Later the cracks became larger and more numerous. The reservoir was then drained. This failure was due to the water getting under the foundations, softening up the strata of clay and shale lying between two layers of rock and permitting one layer to slide foward about 18 inches.

After the structure had thus failed, nothing was done to strengthen it, but the water was allowed to collect behind it. In March 1910 there was a depth of 36 feet of water in the reservoir. Later in the month the water rose to within 2 feet of the overflow. and 2 streams were passing under the dam, appearing about 10 feet below the toe. Another stream was passing around the end of the dam.

On the mornign of September 30, 1911, water was issuing from the cracks in the dam . Early in the afternoon the water was flowing over the spillway. At 2:20 P.M. the portion of the structure at the bottom near the west end burst foward and was immediately followed by a complete demolition of the structure.

The failure of the dam was due to faulty foundation, faulty design, faulty construction and faulty operation.

The materials upon which the dam was placed consisted of thin layers of shale and sandstone. In at least one place covering an area of about 100 square feet this shale is very poor.

Association of Engineering Socities, Volume 48,page 285.

[illegible]

[illegible]

[illegible]

Calculations as well as examination of the wrecked structure show that that the design of the base was faulty. It was not stepped as it should have been but was practically level. It was not wide enough and the proper precausions were not taken to prevent the percolation of water under the dam. The structure shows that in its design no provisions had been made for upward pressure of water. in the joints or under the base, hence the thickness of the dam was entirely too small. In view of the character of the foundation this is a serious mistake.

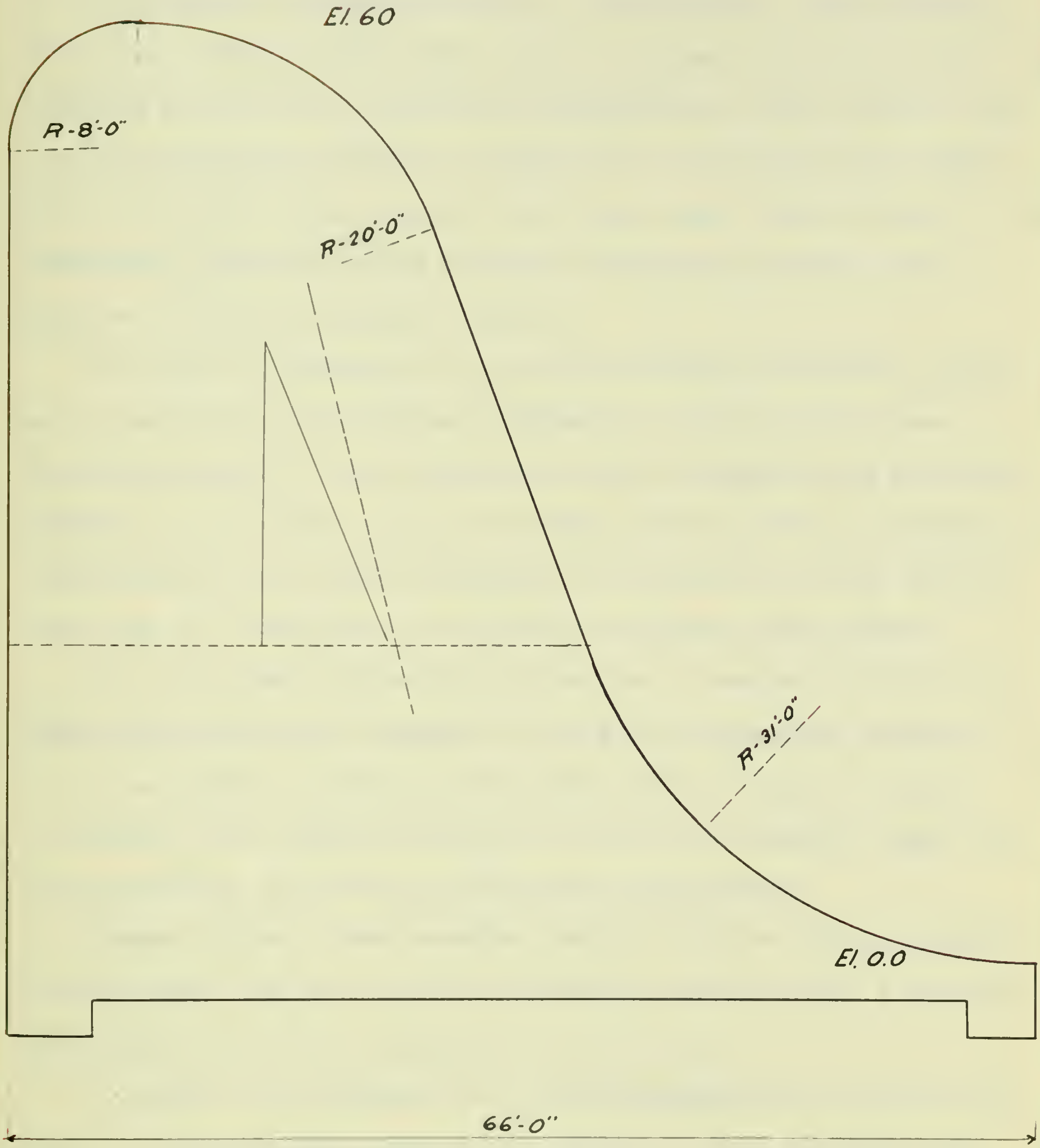
Faulty construction in the dam is shown in many places, especially near the west end where large fairly smooth joints passed horizontally thru the structure.

References- Eng. News, Volume 68, page 605.

Penn. Water Supply Report, 1912

AUSTIN, TEXAS, DAM

Failed under 11 feet on crest



Scale - 1" = 10'-0"

The Austin Texas Dam Failure.

The Austin Texas dam was a limestone rubble masonry straight gravity structure. The total length of the structure was 1275 feet, 1125 feet of which was spillway.

The failure occurred on April 7,1900 after a heavy rainfall. The flood overtopped the dam 11 feet. This was 1.27 feet above the previous flood level. The rupture occurred 300 feet from the east end and the current pushed two sections of the dam, one at each side of the gap, and each about 300 feet long, bodily about 61 feet downstream. These sections were not overturned and were left parallel to their original postions.

The line of pressure lies entirely within the middle third, so it seems that the failure occurred by sliding on the base. The total weight of the structure is 273,000 pounds and the water pressure on the face is 112,300 pounds, which gives the tangent of inclination of the line of pressure at the base as .443. This is less than the coeficient of friction of masonry upon stone .

If full upward hydrostatic pressure is assumed over the 50 feet of the base, the tangent of the line of pressure becomes 1.33,

If an upward pressure varying from full to zero at the toe is assumed, the tangent becomes .627 which is slightly less than the coeficient of friction of masonry upon masonry.

There had been considerable leakage thru the dam by seepage. and the flow from the tailrace had been allowed to cut a channel along the foot of the dam parallel to. . .

A study of the formation of the foundation rocks shows that they are considerably faulted and folded so that crevices are formed thru which water under high head would easily be forced.

The failure seems to have been due to sliding on the found-

tion. The dam would have been on the verge of sliding if no upward pressure had existed, so it is probable that this force was a large factor in the failure.

The dam was of fair if not good construction with the exception that greater precautions should have been taken to prevent seepage. However the operation was very poor for the tailrace should never have been allowed to undermine the structure or even approach it.

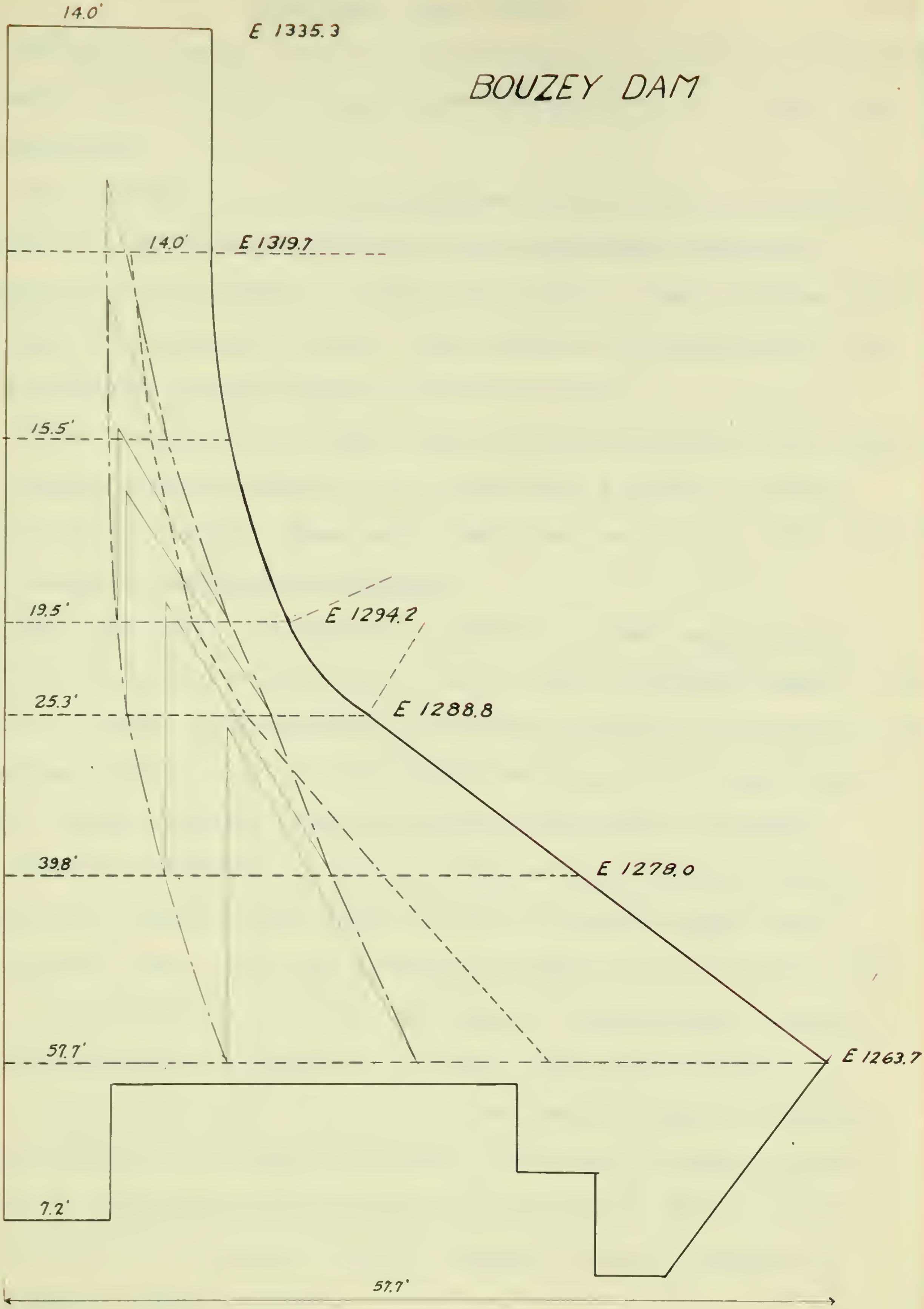
This failure is one of the best examples of failure by upward hydrostatic pressure.

References-Engineering Record, Volume 63.

Engineering News. Volume 45. page 393.

Engineering News. Volume 44. page 64.

Engineering News. Volume 43.


14.0'
E 1335.3
BOUZEY DAM
14.0'
E 1319.7
15.5'
19.5'
E 1294.2
25.3'
E 1288.8
39.8'
E 1278.0
57.7'
E 1263.7
7.2'
57.7'

The Bouzey Dam. France.

The Bouzey dam is a straight gravity masonry structure, 1700 feet in length and 72 feet in height above foundations, or 49 feet above the river bed.

The foundation is of red-sandstone which is quite fissured and permeable. Considerable difficulty was experienced during the construction from springs. A guard wall about 6 feet 6 inches thick was built at the up-stream face from bedrock to the riverbed, but the foundation was not carried down to bed-rock.

After completion the water rose to within 33 feet of the crest and springs appeared below the dam which had a volume of over 2 cubic feet per second. These were partly due to cracks in the cutoff wall caused by temperature stresses.

When the water had reached a level 10.5 feet below the top of the dam, a portion of the wall 444 feet long was shoved foward so as to form a curve. Four additional fissures appeared in the wall a t the same time and the flow of the springs increased to 8 cubic feet per second. These fissures opened in winter and closed in summer.

Later an abutment was built to prevent the dam from sliding further and drains placed under the wall. When the water rose to its highest level , the dam overturned about a plane 33 feet below the top. The fissure is almost horizontal longitudinally and is level transversly for about 12 feet and then dips to the face.

At the point where the failure occurred the line of pressure falls considerably outside the middle third and the dam obviously failed by tension at the toe and then overturned. This accounts for the horizontal rupture dipping towards the toe, as this is the typical tension failure.

Reference-Proc. Inst. of Civil Engineers, Volume 125. page 461.

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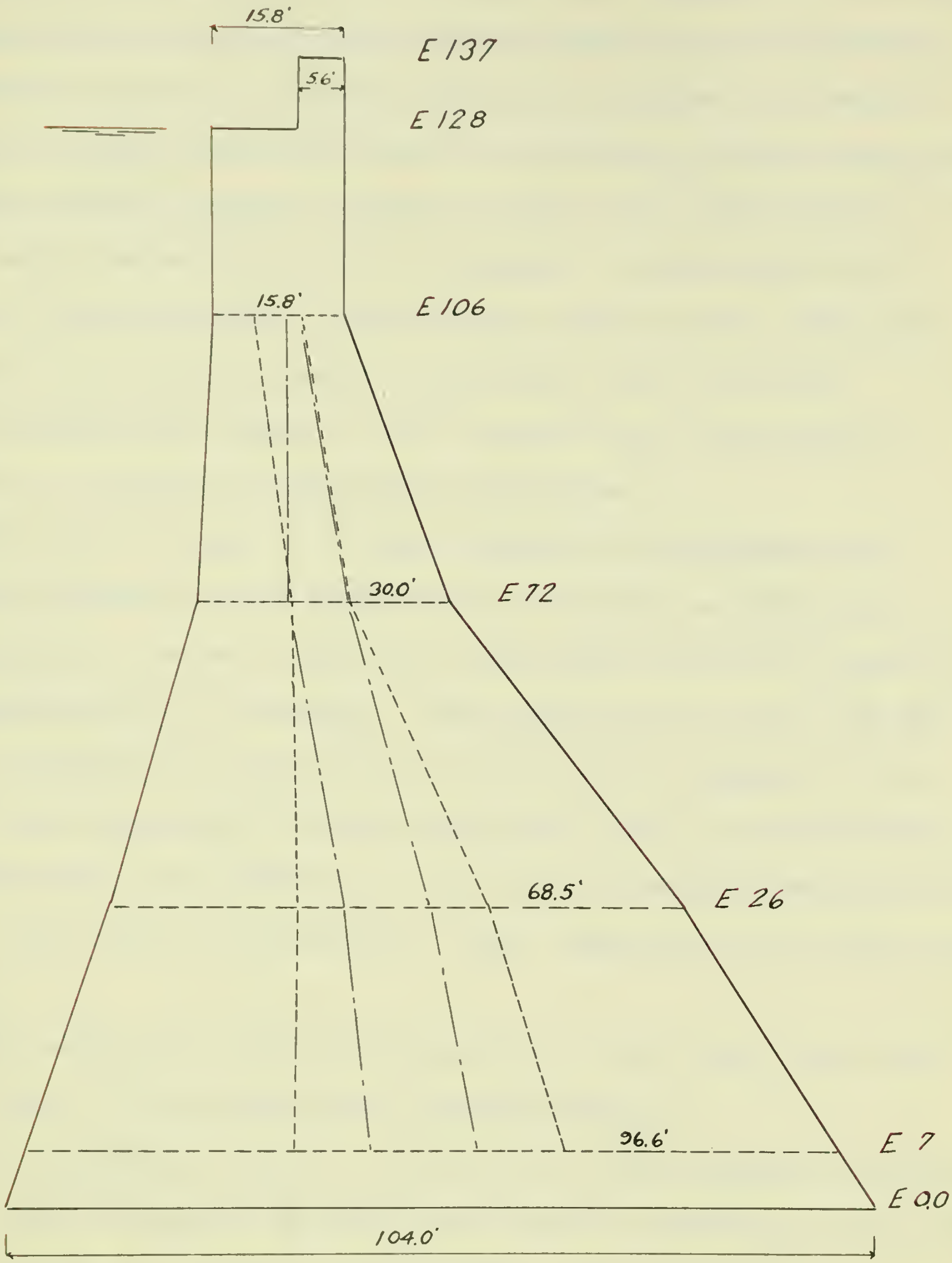
HABRA DAM



Scale- 1" = 20'-0"

The Habra Dam.

The Habra dam was one of the largest dams built in Algeria by the French Government. The construction was commenced in 1865 and was completed in December.1871. After seveal years of successful operation it failed in December.1881. The failure can hardly be bedause of under-design and must be attributed to poor construction.

The dam was founded entirely on rock, the irregularities of which were leveled with a bed of concrete. On this was laid a block of rubble masonry 2 meters high. Upon this the dam proper was built. The dam is staight in plan and 1066 feet long.

In March,1872,a part of the overflow wall failed under a freshet, on account of defective foundations.

The dam was built of a rock composed of calcareous grit . The sand employed in the mortar was not entirly satisfactory and the hydraulic lime was not good. A red earth was used in part of the mortar for the structure. The permeability of the dam was so great that when the water rose to the crest the seepage was of such a large quantity that the dam looked like a gigantic filter. In a few months the downstream face of the structure became coated with a thin layer of carbonate of lime, deposited by the filtering waters.

The rupture was 328 feet long and 115 feet deep, going down to the base. It occurred after a heavy rainfall.

It is probable that the percolating waters gradually disintegrated the lime mortar and when the reservoir suddenly filled the water entered the joints under pressure and overturned the upper part of the structure. In the upper section the line of pressure lies near the edge of the middle third. When the rupture reached the central portion of the section, where the "red earth" mortar

had been used. the highly concentrated load due to the overturning of the section, caused a vertical shear failure which extended to the foundation.

The Habra dam failure is a lesson on the results of the use of inferior materials in construction. The section of the dam was designed more than ample if reasonably good materials had been used. Reference-Wegmann; "Design and Construction of Masonry Dams."

PUENTES DAM, GUADATANTES RIVER, SPAIN

The Puentes dam is a straight gravity rubble masonry structure, built in 1785-91. It is polygonal in plan, being convex upstream. The maximum height of the dam is 164 feet:its length on crest is 925 feet.

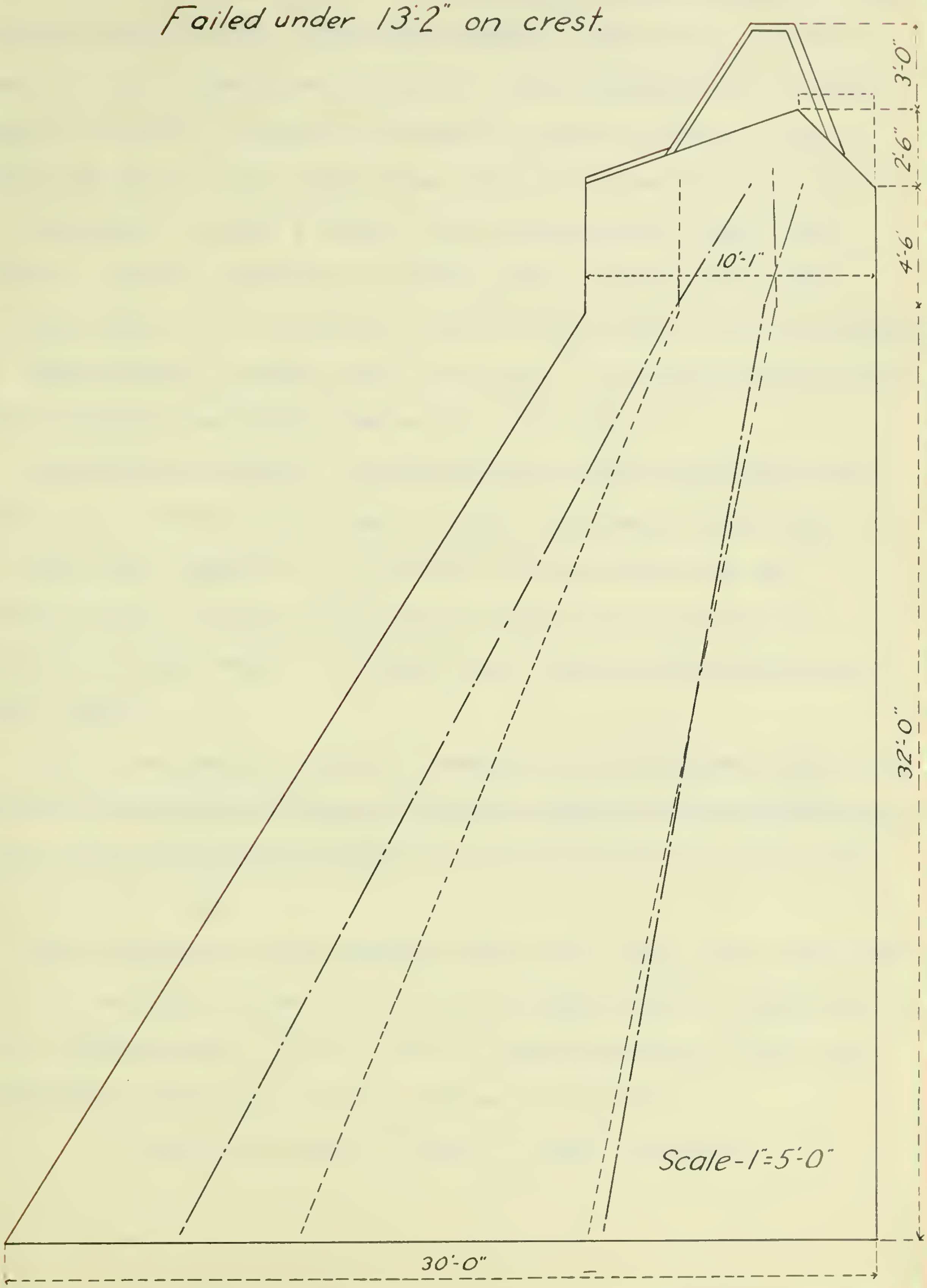
The original plan of construction was to found the structure entirely upon rock, but a large pocket of earth was encountered near the center of the valley and it was decided to use a pile foundation at this point. A timber grillage covered with masonry 7 feet thick and carried on piles was extended 131 feet downstream to protect.

On April 30,1802,this grillage was burst by the water pressure and an opening 56 feet broad and 108 feet high was eroded,leaving the masonry structure intact.

This failure shows that the safety of a dam is as much dependent upon the formation of an impervious and water-tight wall as upon the formation of a structure which will resist the pressure from the water.

COLUMBUS GEORGIA DAM

Failed under 13'-2" on crest.



Failure of the Columbus, Georgia, Dam.

In the winter of 1901 an extraordinary flood occurred in the Southern States during which four masonry dams failed. One of these was the Columbus, Georgia, dam. This ~~structure~~ was a rubble masonry structure, 39 feet in height to which 3 feet of flash-boards had been added. During the flood the crest of the masonry was overtopped 13 feet 2 inches. The dam failed at this flood height by the top rupturing from the lower sedtion. The break occurred about 5 feet below the crest at the back of the structure and dipped toabout 20 feet below the crest at the down-stream face. Longitudinally the rupture are nearly horizontal.

The failure occurred simultaneously at two points where the current was the strongest. Many of the flash-boards were torn off and those that remained in place were badly twisted. On the diagram of the dam section, p. 61, it will be seen that the line of pressure for this height of flood falls considerably outside the middle third.

The dam evidently failed by tension in the upstream face. The tension failure was followed by vertical shear failure under the highly concentrated load caused by the over-turning of the upper portion of the dam.

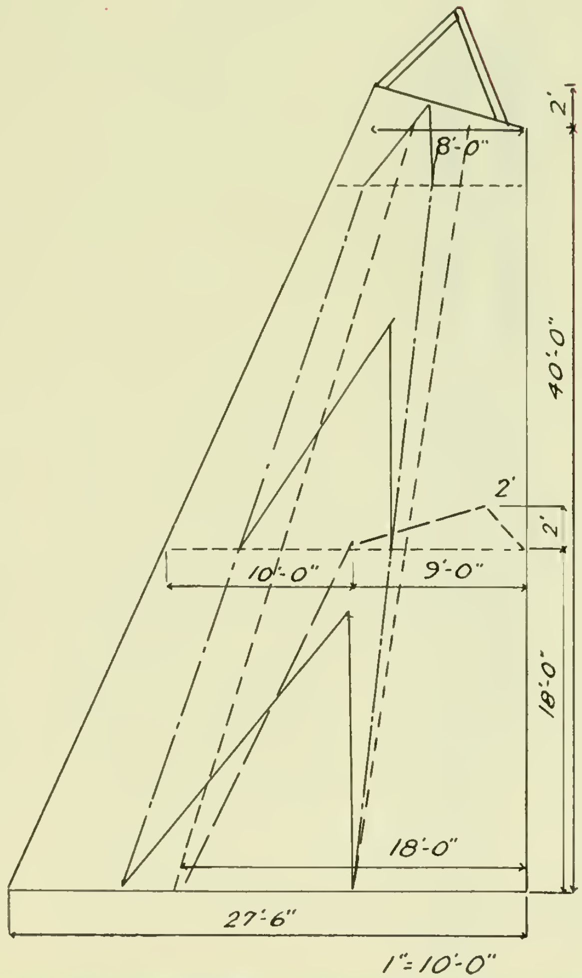
The failure was due primarily from the design not having been made for so high a flood, but this was aggrevated by the addition of the flash-boards. It is a case of under-estimated flood flow.

References- Engineering Record. Volume 45, page 30.

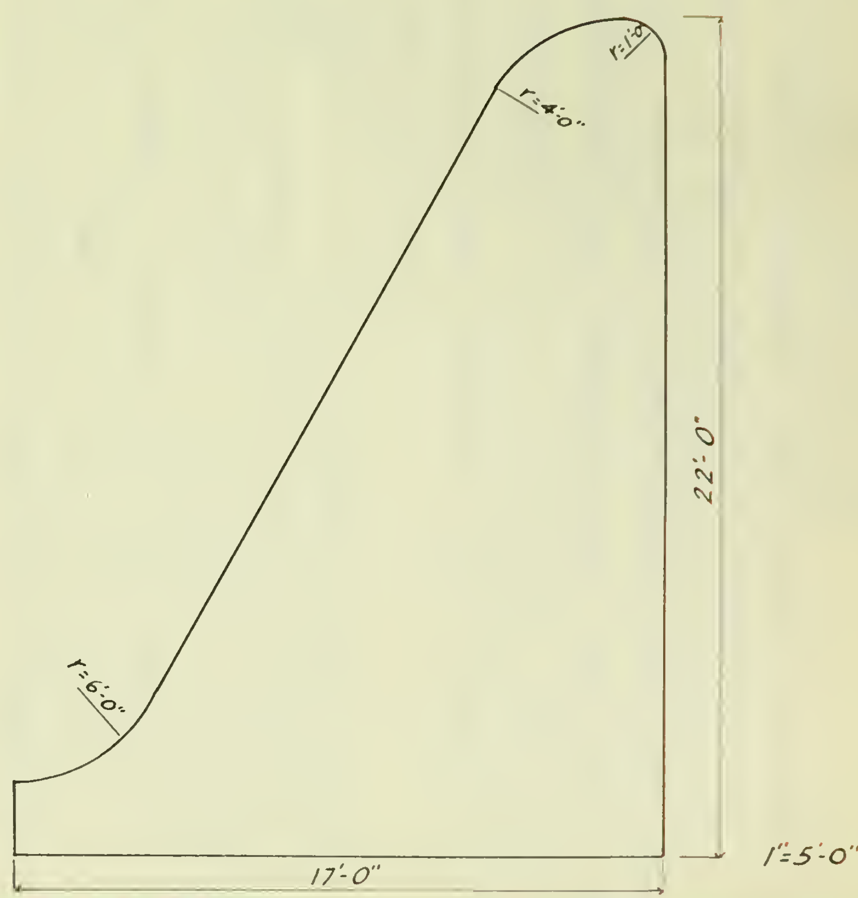
Engineering News, Volume 47, page 34 and 60.

ANDERSEN, S.C., DAM.

Failed under 10 feet on crest.



HOT SPRINGS S.D. DAM.



Failure of The Andersen, S.C., Dam.

The Andersen dam was originally built 20 feet in height and was later raised to 42 feet in height by an addition to the original structure. It was one of the Southern dams which failed under the extraordinary flood mentioned above. The flood at this structure was 6 feet above 4 feet of flash-boards. The rupture occurred at the top of the old structure, which withstood the flood after the upper part had failed.

From the position of the line of pressure in the diagram of the dam section, p 63, it is evident that the dam was not designed for so great a flood. If the new portion is considered to act independent of the old part, the line of pressure will fall considerably nearer the down-stream face. It seems as if the bond between the old and new structures was not suffieient to resist the percolation of water into the crack.

This failure seems to illustrate that additons to old structures should be designed to be stable without the assistance of the old portion.

References- Engineering Record. Volume 45, page 30.

Engineering News. Volume 47, pages 34,62,70,107,177.

Hot Springs, S.D.

This structure is a concrete spillway dam, founded on sandstone. The central portion failed by sliding when overtopped by a flood of unexpected magnitude, leaving a section at each end in place and sliding the central part in two sections several feet downstream.

The failure seems to have been due to uplift of percolating water under the dam thru the fissured sandstone, and the ruptured portions seem to have been simply floated from position. This failure shows clearly the necessity of designing a dam to resist upward pressure, especially if the foundation rocks are fissured. Reference- Engineering Rec., Volume 57, page 795.

Port Angeles, Washington, Dam.

The Port Angeles dam is a concrete wedge built in a narrow canyon on gravel foundation. Its height is 130 feet and length is 40 feet. It was proposed to prevent seepage under the dam by a cut-off wall. This wall was not made sufficiently impervious to prevent the percolation of water and the seepage eroded the gravel, leaving the structure intact above the stream.

This dam is a failure of an experiment which seems quite feasible. The compacted gravel should be firm enough to carry the weight of the dam and to support the pressure transferred thru the cut-off wall. The cut-off wall is supposed to prevent seepage and the erosion of the gravel.

It is proposed to fill the space below the structure with rock and form a rock dam. This will be constructed of material which will not be so easily displaced as the gravel. Eng. Rec., Vol. 66.

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[illegible]

[illegible]

[illegible]

[illegible]

[illegible]

[illegible]

[illegible]

Fergus Falls, Minn. Dam.

This dam was a concrete structure founded on what appeared to be hardpan. In making the borings for the exploration of the foundation materials, the stream bed was found to consist of 4 feet of sand and gravel, 3 feet of sand and gravel mixed, 4 feet of blue clay and gravel known as hardpan. However this latter material was only bored to a depth of 3 feet.

During the excavation of the foundation, numerous small springs were encountered coming up thru crater-like openings and forming small mounds of sand at their mouths. With one exception these had disappeared when the excavation had been completed. After the structure was completed a spring opened up below the power house which discharged about 1 gallon in 5 minutes.

At the time of the failure the first sign of trouble was an increase in the discharge of the tail-race. This was soon running full and the water commenced spouting thru the drainage opening in the floor of the power house. The failure must have been caused by the settling of the foundation in the vicinity of the junction of the wasteway and the powerhouse.

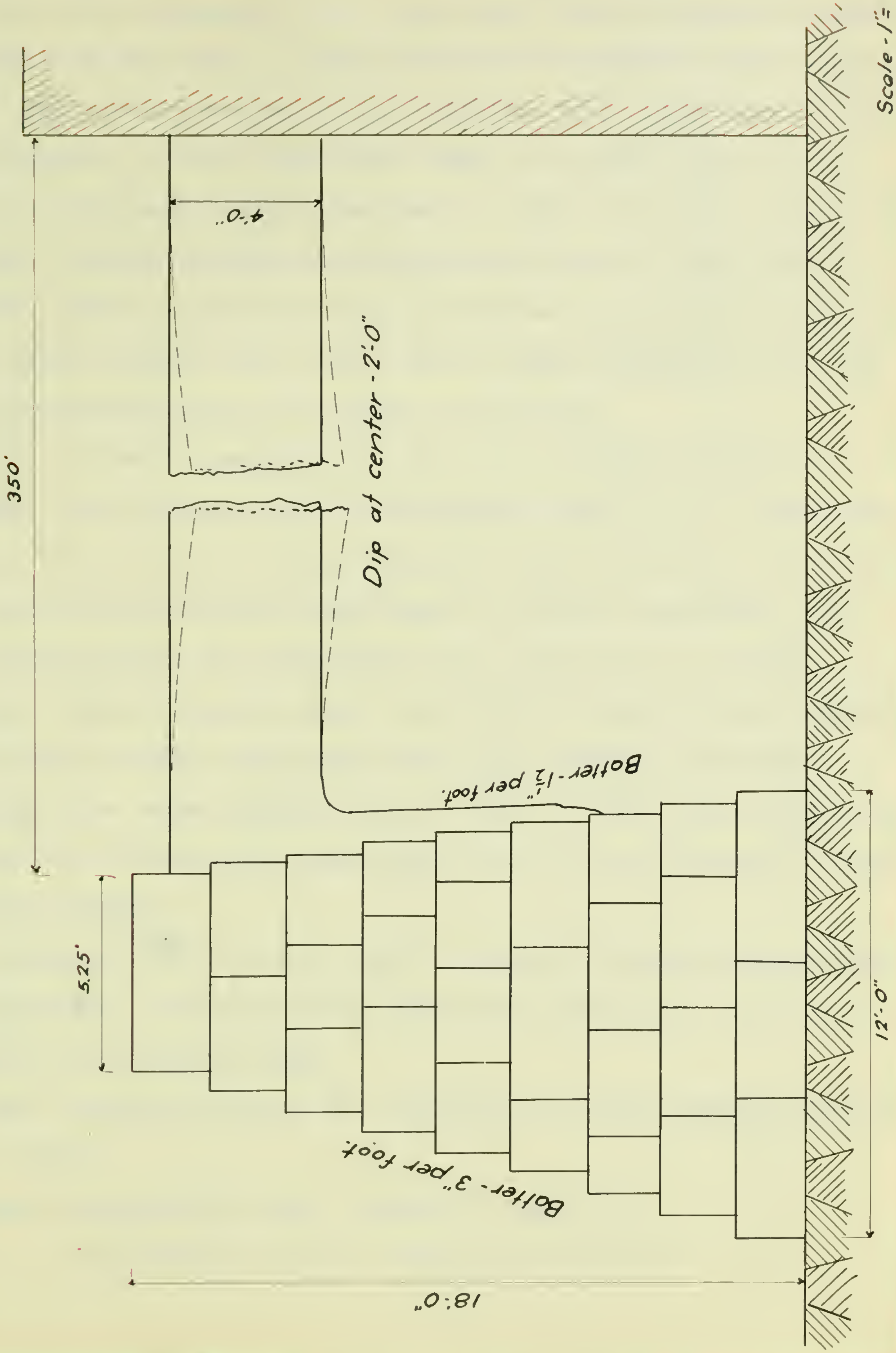
The concrete in the dam sheared along a horizontal plane which seems to have been the boundary of successive days work. This occurred in a number of places. The concrete seems to have been a dense mixture of good quality. The designer claims that the city officals closed a spring which he had instructed to be kept open.

This failure is apparently another victory for disreguarded upward hydrostatic pressure.

Reference- Engineering News. Volume 62 ,pages 391,393,477,497.

[illegible]

MINNEAPOLIS MILLS DAM



Minneapolis Mills Dam.

The first Minneapolis Mills dam was an ashlar masonry structure 535 feet long was built in 1894 and formed a pond for water power supply. The winters are so severe that ice forms about 4feet thick on the surface. In this particular case ice formed from 10 to 12 inches on the water face of the dam to a depth of about 12 feet below the crest. During the week the pond was gradually lowered two or more feet, which again filled up on Sundays. The ice at the center of the pond followed this motion but the edges adhered to the dam and the retaining wall at the back of the pond.

This dam stood successfully until the spring of 1899. In February it was noticed that the downstream edge of the coping was 3 inches lower than its normal position. This was found to be due to the top of the dam being turned about 10 inches downstream, the dam revolving about the downstream toe. Apparently no sliding occurred. The ice was cut along the top of the dam and the structure settled back to some extent but not to its original position. Four-inch holes were drilled thru the dam, 25 feet apart, and 4 feet into the rock foundation. Three-inch iron rods were placed in these holes and grouted.

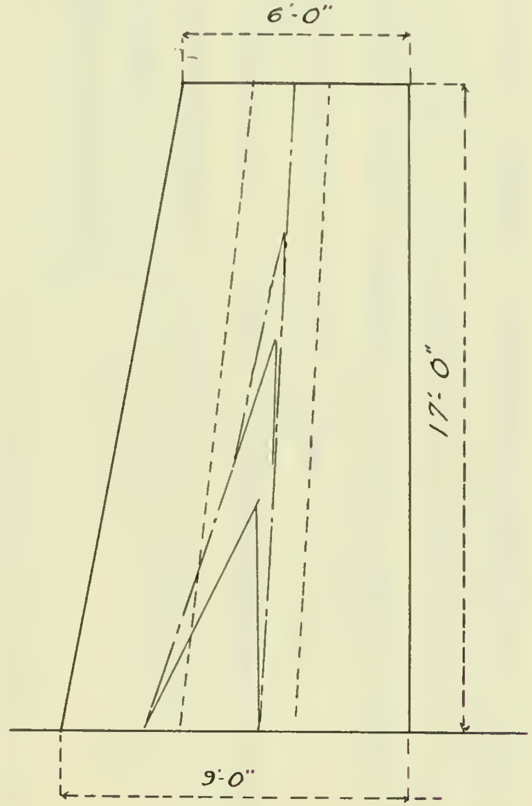
On April 30, 1899 at 7:30 a.m., with 2 inches of water passing over the waste-weir, 170 feet of the dam at the waste-weir slid out., in spite of the anchor bolts.

This failure is one of the few which have been caused directly by ice thrust.

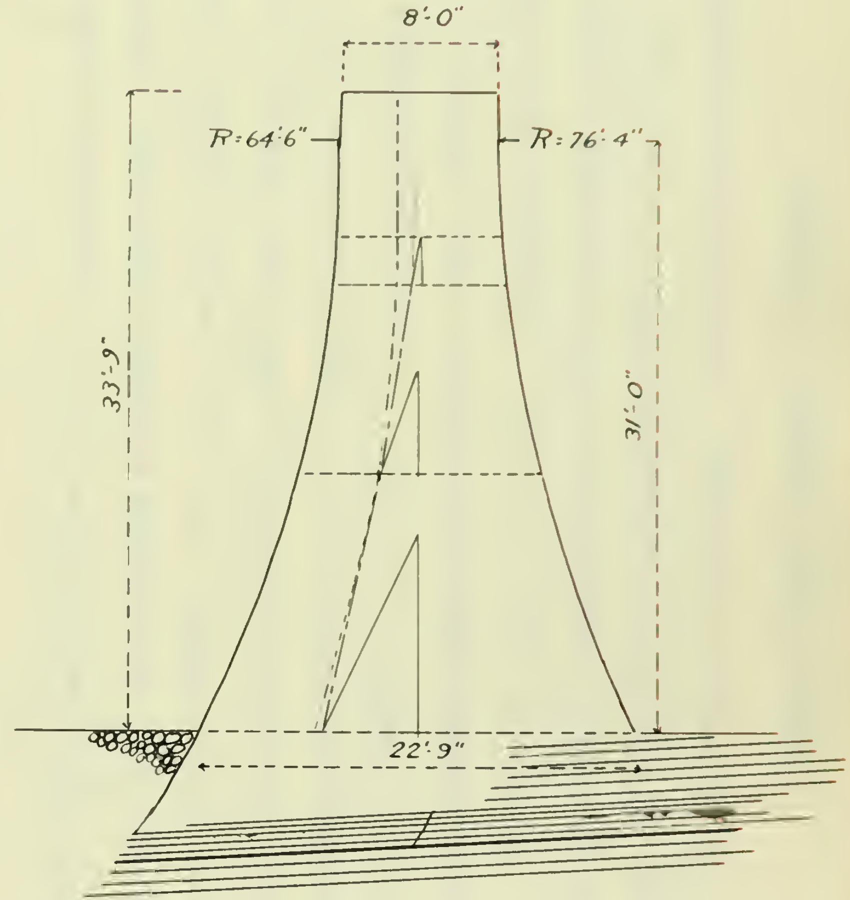
References- Engineering News. Volume 41. page 307.

Engineering Record. Volume 46. page 293.

PORTLAND, Me., DAM.



NASHVILLE RESERVOIR WALL.



Nashville Reservoir Wall Failure.

A portion of the ellipical reservoir at Nashville Tennessee. 150 feet long gave way with the reservoir practically full. The wall is of masonry founded on a hill of striated limestone with layers of clay and shale interposed. The dip of the strata is 3 degrees north and 25 degrees west. At the point where the failure occurred there is a small fold producing a dip of 8 degrees in the opposite direction, away from the center of the reservoir. There was a 10-inch layer of clay 4 feet below the base of the wall at this point. These layers of clay became moist from seepage. Then the beds of stone ruptured from water pressure and the wall slid on one of the clay layers. Four feet of the rock adheres to the base of the wall.

When the water stands at the waterline of the reservoir, the line of pressure falls outside the middle third and it is necessary, if the structure stands, for the wall to carry considerable tension in the wall ring. It is probable that the wall first failed by tension in the elliptical ring, which accounts for the radial breaks in tha wall, and then slid on the clay layers below the base.

This failure is evidently due to lack of investigation of the foundation materials and disreguard of geological conditions.

Reference-Engineering Record, Volume 66, page 539.

Portland Maine Dam.

The Portland Maine dam is a trapizoidal masonry structure, 17 feet high and of considerable length. It was designed for an over-topping flood of 2.5 feet and failed by overturning when overtopped 6 feet. This is another case of underestimated flood flow and too small a factor of safety.

Reference-Engineering News, Volume 25, page 279.

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FAILURE OF THE DANVILLE, NEW YORK, DAM.

The Danville, N.Y., dam is a combined earth and reinforced concrete structure, 133 feet of which is of concrete and the remainder of 368 feet is earth embankment. The spillway is located on the earth embankment.

The concrete was made of materials found in the creek bed near the dam site.

The earth embankment failed upon the first filling of the reservoir. This break was repaired and the reservoir again began to fill in December.

When the reservoir was about half full, those who examined the structure say, water and sand came up thru some of the pipe weep holes in the footing and shot 6 to 8 inches into the air. On the night of December 4th the water stood 14 inches on the flash-boards and on the 5th the structure gave way. The rush of water carried 36 feet of the dam 60 feet downstream. The gravel foundation was entirely eroded from the underlying rock. The deck wall snapped off at a vertical joint made by the horizontal reinforcing bars.

The failure was due to the erosion of the gravel foundation by the seeping waters. This failure is typical of reinforced concrete dam failures. Nearly every failure has been caused by seepage and erosion of the gravel foundation.

LAKE LEHIGH, PENNSYLVANIA, DAM.

The Lake Lehigh dam is a hollow reinforced concrete structure, 32 feet high and 320 feet long. It is founded on a sandstone or conglomerate, with a thin overburden of soil. The buttress footing and cut-off walls were reported to have been carried down to rock. There was no apron on the downstream side of the spillway.

In 1909 a leak appeared under the dam. Repairs were made but the leak did not dissappear. In 1911 a leak of 5 cubic feet per second flow appeared. The seepage finally resulted in the destruction of the structure.

Janesville Wisconsin Dam.

This dam is a reinforced concrete structure. 9 feet high and 100 feet long. The foundation was washed out at the junction of the dam and gates, without injury to the structure.

Pittsfield, Massachusetts, Dam Failure.

A reinforced concrete dam some 400 feet long and 40 feet high, located at Pittsfield, Massachusetts, failed by an opening being eroded under the structure.

The dam is triangular in form, resting on an 18 inch horizontal floor. The maximun width of the floor is 50 feet. The upper cut-off wall is 3 feet wide and 8 feet deep and is situated at the up-stream edge of the floor. The down-stream cut-off wall is 3 feet wide and 5 feet deep. In construction the top soil was stripped off to a depth of about 4 feet and the horizontal floor built on this surface.

The water in the reservoir stood within 10 feet of the top for a considerable period prior to the rainstorm at the time of the failure. The reservoir was nearly filled when the water was seen to issue as a slight percolation thru the natural ground below the dam. The flow increased steadily for nearly two hours before a clear opening was made. A small quantity of water found its way thru the weep holes in the floor.

It appears that the water found its way thru a layer of gravel below the impervious layer upon which the cut-off wall rested.

FAILURES IN WHICH THE MINOR DETAILS ARE MISSING.

DYER DAM, Danielson, Conn.

This dam was built in 1880 of stone filled cribs between massive masonry abutments. In 1899 the timbers were removed and the stone covered with concrete. The height of the dam after repairing was 12 feet. In 1901 one of the abutments failed by the foundation sinking.

Reference- Eng. News. Volume 45, page 231.

HASFELD DAM, Shippensburg, Pa.

This dam was built of concrete in 1908-11 over a timber flume. It failed in 1911 by seepage of water along the base of the dam.

Reference- Penn. Water Supply Report, 1912.

HOUSATONIC DAM, Burmingham, Conn.

A curved masonry dam built in 1869, 636 feet long, 40 feet high, 26 feet wide at the base and 8 feet wide at top, failed in 1891. While under construction in 1869, a flood carried away about 160 feet of the structure and scoured a hole 20 feet deep in the river bed. This cavity was filled with rock and timber crib, with a timber and concrete apron to protect it from seepage. This section was undermined by seepage 22 years later.

Reference- Eng. Record. Volume 46. page 292.

MACDONALDSON, Pa. Dam.

A gravity masonry dam 419 feet long and 16 feet high, founded on clay and sand, failed upon the first filling, by cracking. Later the water bubbled up along the downstream face and finally eroded an opening under the structure, leaving it in position.

The dam moved about 8 inches downstream and deflected about 2 inches. The construction was supposed to have been faulty.

References- Engr. Record. Volume 64·page 581.

Penn. Water Supply Report.1912.

NECEDAH DAM. Wis.

This dam was a gravity concrete structure, founded on sand at one end. A sheet piling cut-off wall was supposed to have been carried down to hardpan, but this was evidently not accomplished as the sand was eroded at this point.

Reference- Eng. Record·Volume 52·page 533.

OSWEGODAM. NEW YORK.

This was a gravity dam built in 1870 on a timber crib foundation. It failed in 1912 from the timber crib breaking up by undermining, The dam was 360 feet long and 14 feet ih height.

Reference- Eng. Record,Volume 65,page 401

Eng. Record·Volume 67·page 426.

OWASCOLAKE DAM. Auburn. N.Y.

A gravity masonry spillway dam, 857 feet longand 10 feet high, built in 1860, failed in 1912. Heavy water pressure caused by a fl flood started the rupture at the south end. After this both ends were undermined.

Reference. Eng. Record·Volume 65,page 476

Eng. Record,Volume 67,page 426.

ANGEL'S DAM. Calcaras County. California.

This dam is a curved gravity structure, 400 feet long, 52 feet high, and 35 feet wide at the base. It was founded on fissured rock. On April 10,1895, a section failed by being undermined. A hole was eroded in the bedrock at the point where the break occurred.

References- Engineering Record. Volume 46, page 292.
Engineering Record. Volume 23, page 307.

CHAMBY DAM, Montreal. Canada.

The Chamby dam is a gravity concrete structure, 1600 feet long, 29 feet high, 27 feet wide at the base, and 8 feet wide at the top. It had been overtopped by 29 feet or more three times within 20 months of the time of the failure. The failure occurred on October 17, 1900, when 28 feet of water was passing over the spillway. The dam was built in a trench excavated three feet de ep in shale.

Reference:- Engineering Record, Volume 43, page 149.

HANNAWA FALLS. N.Y.

The Potsdam sandstone facing on the downstream face of a gravity concrete dam caved off to a depth of 7 to 9 feet below the crest for almost the entire length of the dam. The dam was built in 1900 and failed on March 1,1914. At the time of the failure the reservoir was covered with ice.

Reference:- Engineering News, Volume 72, page 4.

LINCOLN POND DAM, Westport, N.Y.

The Lincoln Pond dam is a gravity concrete structure, 100 feet long and 25 feet high. It failed under a heavy flood in the early part of 1912. The foundation material is rock. The failure probably occurred by sliding on the foundation.

References- Engineering News, Volume 67 page 1099.

Engineering Record, Volume 67, page 426.

LOUISVILLE CANAL WALL, Louisville, Ky.

A section of the Louisville canal was being widened and the wall of cut-stone masonry built in 1854 was used as a coffer-dam. The height of the wall was 6 feet above rock and 26 feet above the canal bottom. During the excavation 500 feet of the wall and rock slid off at a section 6 feet below the base of the wall.

During the excavation a few leaks had developed, but they were stopped by bags of manure placed on the water side of the wall.

Reference- Engineering News, Volume 74 page 764.

MORRISONVILLE DAM, N.Y.

The Morrisonville dam was constructed by building two masonry walls and then filling the space between with grouted rock. It was 45 feet high and 152 feet long. The failure occurred on January 16, 1912. The first leak appeared at the center of the face. Later as the water rose a crack appeared and it was found that the crest had moved downstream. It is probable that this was done by ice pressure .

Reference:- Engineering Record, Volume 65 page 94.

RIVERSIDE. INDIANOPOLIS DAM. Indiana.

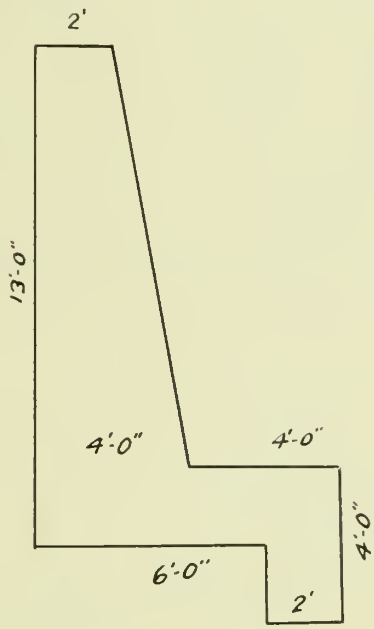
A gravity dam built in 1899 failed in 1906 by undermining due to seepage.

Reference- Eng. News.Volume 68.p

SHIPPENSBURG DAM. PENN.

An earth fill concrete dam built in 1908-11 failed on January 17.1912. Shortly after completion the dam failed by the erosion of the gravel and shale foundation by percolating water. This was repaired and the dam again failed at the date above mentioned, by overturning on the gravel foundation due to ice pressure.

References- Eng. Record.Volume 65.page 179.
Eng. Record.Volume 67.page 426
Penn. Water Supply Report.1912



SHIPPENSBURG DAM.

SHELDON DAM, Conn.

A gravity masonry dam 60 feet long and 30 feet high failed under a flood from an upper dam. The entire crest was carried away to a depth of 10 feet.

Reference- Eng. Record.Volume 47.page 224.

NAMAKA DAM, Bow River, Alberta, Canada.

The sluice gate section of a gravity concrete dan failed by under-cutting on July 15, 1912. The water found its way thru cracks caused by ice pressure during the winter. Ice 4 feet thick buckled above the spillway and cracked the footing floor. The water forced its way under the footing and broke thru by upward pressure. This failure is typical of gates and other regulating structures located on pervious materials.

References:- Engineering Record, Volume 66, page 377.

Engineering Record, Volume 64, page

DAM	TYPE	FOUNDATION	MODE OF FAILURE
AUSTIN. Pa.	Straight gravity	Striated shale.	Sliding due to upward water pressure.
AUSTIN. Tex.	Straight gravity. spillway	Faulted rock.	Sliding due to upward water pressure.
BOZEY.FRANCE	Straight gravity.	Fissured red-sandstone.	Sliding on base and tension in face.
HABRA.ALGERIA	Straight gravity.	Calcareous rock.	Tension in upstream face; vertical and horizontal shear ;poor construction.
PUENTES.SPAIN.	Polygonal plan-straight gravity.	Piling on earth	Undermined by seepage.
COLUMBUS. GA.	Straight gravity	Rock	Tension in upstream face under flood.
ANDERSEN. S.C.	Straight gravity	Rock	Tension in upstream face at junction with old dam.
HOT SPRINGS S.DAKOTA.	Straight gravity. spillway.	Fissured sand-stone.	Sliding to upward water pressure.
PORT ANGELS. WASHINGTON.	Wedge in canyon	Gravel	Gravel eroded.
FERGUS FALLS MINN.	Straight gravity	Blue clay and gravel.	Settling of foundation due to seepage.
MINNEAPOLIS MILLS	Straight gravity	Rock	Overturned by water pressure after being lifted by ice thrust.
NASHVILLE RESERVOIR	Curved gravity	Striated lime-stone with clay	Sliding of section on layer of clay.

PORTLAND.MAINE	Straight gravity	Probably rock.	Overturned by flood.
DANVILLE.N.Y.	Reinforced.Con.	Gravel	Erosion of foundation by seepage.
LAKE LEHIGH.PA.	Reinforced con.	Sandstone under layer of soil	Seepage under cutoff wall.
JANESVILLE.WIS.	Reinforced con.	Probably gravel.	Foundation eroded
PITTSFIELD.MASS.	Reinforced con.	Clay on gravel	Seepage thru gravel foundation.
DYER.PA.	Straight gravity. grouted rock		Foundation eroded.
HASFELD.PA!	Straight gravity		Seepage thru foundation.
HOUSATONIS.PA.	Curved gravity	Rock fill & gravel.	Seepage eroded foundation.
MACDONALDSON. PENN.	Straight gravity.	Clay and sand.	Seepage eroded foundation leaving dam intact.
NECEDAL, Wis.	Straight gravity.	Sand	Sand eroded under cutoff wall.
OSWAGO.Wis.	Straight gravity.	Timber crib on gravel	Undermined by seepage.
OSWASCO LAKE. N.Y.	Gravity spillway	Probably gravell	Ruptured by flood and undermined.
ANGEL.Cal.	Curved gravity.	Fissured rock.	Undermined.
CHAMBY. Can.	Straight gravity.	Shale	Overturned by flood.
HANNAWA FALLS. N.Y.	Straight gravity.	Rock.	Facing ruptured by ice pressure.
LINCOLN POND	Straight gravity	Rock	Probably sliding.
LOUISVILLE CANAL	GRAVITY	ROCK	Overturning & sliding.

MORRISONVILLE N.Y.	Straight gravity.	Probably rock	Sliding due to ice pressure.
RIVERSIDE.	Straight gravity		Undermined by seepage?
SHIPPENSBURG.	Earth-concrete.	Gravel & shale.	Foundation eroded by seepage.
SHELDON.Conn.	Straight gravity		Crest carried away by flood.
NAMAKA. Can.	Gravity sluice gate	Gravel	Seepage eroded foundation.

SUMMARY OF FAILURES.

Sliding on base due to upward water pressure.

Austin. Pa.

Austin. Tex.

Hot Springs. S.D.

Tension in upstream face due to upward water pressure.

Habra. Algeria.

Portland. Me.

Tension in upstream face due to poor design.

Bouzey

Columbus

Anderson

Oswasco

Chamby

Sheldon

Ruptured by ice thrust.

Minneapolis Mills

Hannawa Falls

Morrisonville

Sliding on foundation.

Nashville reservoir

Lincoln Pond

Louisville canal.

Undermined by seepage.

Puentes

Port Angels

Fergus Falls

Dyer

Hasfeld

Housatonic

Macdonaldson

Necedal

Angel

Riverside

Shippensburg

Namaka.

Nature of Failures.

					Percent
Due to uplift-	Sliding on base	3			
	Tension in face	2	5		17
Due to poor design-	Tension in face		6		20
Due to seepage-	Undermined	13			43
	Sliding on base	3	16		10
Due to ice thrust			3	30	10
Reinforced concrete dams -by undermining			4	4	
				34	

Character of Failures.

			Percent
Structure			
Tension in face due to uplift	2		
Tension in face due to poor design	6		
Due to ice thrust	3	11	37
Foundation			
Sliding on base due to uplift	3		
Sliding on base due to seepage	3		
Undermined by seepage	13	19	63
		30	

CONCLUSIONS.

The first step in the design of a dam should be to explore the proposed site by making test-pits and borings. A thorough knowledge of the foundation materials is necessary before the type of structure can be selected or properly designed. Sixty-three percent of the gravity dam failures are in the foundation.

Little is known of the value of the coefficient of friction of different foundation materials, especially in a saturated condition. A number of failures have occurred by sliding on the base or in the foundation. However, the base should be so constructed that there will be no possibility of the structure sliding upon it. Horizontal joints and planes should be avoided as these often constitute zones of weakness.

The geological formation of the foundation rocks should be considered. The Austin, Texas, dam and the Nashville reservoir wall failures were the direct results of disregarding this feature. In the Austin dam the folded and faulted foundation rocks permitted the seepage of water so that an upward pressure was exerted on the base of the structure. This reduced the normal pressure between the foundation rocks and the base and cause the dam to slide. The fold in the foundation of the Nashville reservoir wall reversed and increased the dip of the strata so that the slope was away from the reservoir and the wall readily slid outward when a layer of clay in the foundation became moist.

The weakest point in dam construction, as indicated by these failures, is the neglect or lack of success in building a watertight wall which will prevent seepage under the structure. Fifty-three percentof the failures of gravity dams have been caused directly by

[illegible]

seepage thru the foundation. All reinforced concrete dam failures were from undermining by seepage. Also all the failures of curved gravity dams were foundation failures due to seepage. Many of the failures from seepage occurred because the cut-off walls were not carried deep enough. but in the majority of the failures there was no attempt at cut-off walls as the seepage occurred thru fissured rock which was supposed to be sufficiently impervious to retain the water.

Upward hydrostatic pressure should be considered as existing over the entire base of the structure if the foundation is at all pervious. Even on the best of foundations some allowances should be made for it. In concrete dams it is of little importance in the upper sections if proper precautions are taken during construction to avoid planes of separation. Seventeen percent of the failures of gravity dams were due to uplift from hydrostatic pressure.

Excessive head or pressure on the dam should be guarded against by making ample waste-ways to pass floods. Failure to do this has resulted in pressure upon the dams whcih was greater than that forw which they had been designed and was the direct cause of failure in many instances. It is difficult to estimate the probable magnitude of the maximum flood. Even after an estimate has been made it often seems so unreasonable that the designer hesitates to provide so large a waste-way. A factor which should be considered in the determination of the size of waste-way is the probability of some dam failing up the stream and thus aggravating the flood. Many dams have given way under a relatively small flood when a timber or earth dam failed immediately above.

Ice thrust is most destructive on small reservoirs and small structures. Only ten percent of the gravity dam failures have been

attributed directly to this cause. Even these dams were not entirely disrupted by this force. It only commenced the rupture which allowed the other forces to gain a foothold and complete the destruction. Ice thrust should be given more consideration in design than simply making the crest thick enough to resist the shearing force, except possibly in very mild climates.

SUMMARY.

These failures show that the following points of weakness must be guarded against in the design and construction of dams:

1. Rock formation with planes of weakness.
2. Planes of weakness in the structure.
3. Seepage thru permeable or fissured foundations.
4. Uplift from hydrostatic pressure, if the foundation is not impervious.
5. Inadequate waste-ways.
6. Overturning or rupture by ice thrust.