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AN INVESTIGATION OF
HARDY DAM

Thesis for the Degree of B. S.
MICHIGAN STATE COLLEGE

Leo G. Stearns
1948



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An Investigation of Hardy Dan

A Thesis Submitted to

The Faculty of

MICHIGAN STATE COLLEGE

of

AGRICULTURE AND APPLIED SCIENCE

by

Leo G. Stearns

Candidate for the Degree of

Bachelor of Science

June 1948

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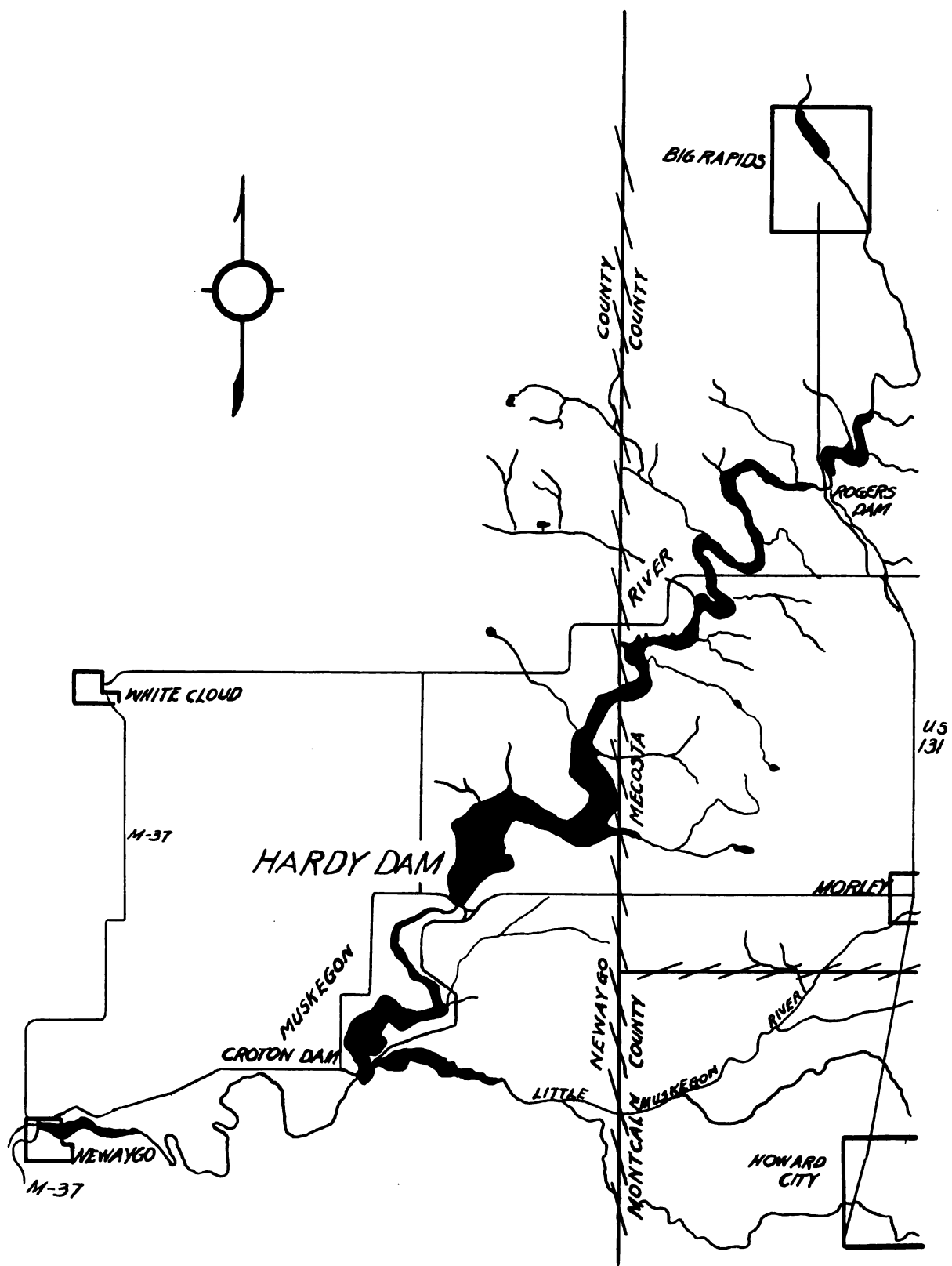
Dams should preferably be built on rock foundations - good rock foundations - as emphasized by past and present experience, but as the demand for electric power increases dams will be built on the best foundations available.

Before the Hardy Dam was designed several problems were encountered. Previously water power plants were built on the cheapest sites first, leaving each successive site less attractive as an investment unless new and more economical ideas could be devised. Another problem encountered was the fact that steam and diesel generators had reduced the hydro-electric field to that of supplying peak power - which requires large ponds, high heads, large machinery capacity, and points toward back pumping during slack periods to supply water power for the next peak.

The Hardy Dam, which creates a 100 foot head, was placed in operation at full head on April 30, 1931. It is composed essentially of a porous sand embankment, 130 feet high with a flexible concrete core-wall. The core-wall is set on steel sheet-piling driven into underlying pervious, and heavy water bearing sand and glacial drift several hundred feet thick over bed-rock. Three penstocks 14 feet in diameter, which are used both for power and spilled water, extend through the base of the deep section.

Before anything else is adduced about the dam let us investigate its location and design.

The Muskegon River falls some 225 feet in 40 miles of river



LOCATION OF HARDY DAM

between Big Rapids and Croton. Upstream from Big Rapids it flows more slowly, through a wider, flatter valley, with few if any economical power sites. Below Croton is a fall of 70 feet through Newaygo to Bridgeton, then less than 30 feet through some 80 miles of sluggish channel to Muskegon Lake. Surveys started in 1903 showed this situation and indicated 40 foot power sites at Croton and Rogers, which were built in 1906-07, leaving a 100 foot fall undeveloped between them.

Surveys and studies at that time located two intermediate sites for the ultimate development of this 100 foot fall, the lower one being Oxbow at the head of Croton Pond, the upper being Erwin at the Mecosta-Newaygo County Line.

Investigation of the 100 foot project was begun in 1926. Progress was retarded somewhat by the inability to find good foundation conditions. The heavy covering of glacial drift put looking for rock foundation in this area out of the question, but the action of the glaciers, however, did leave a deposit of finely ground rock flour which now shows up in what are commonly called clay beds, or technically called mudstone.

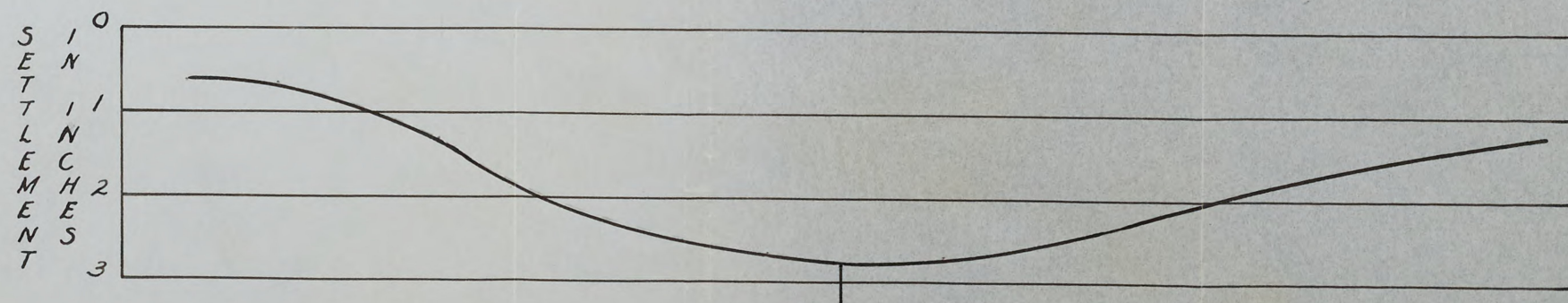
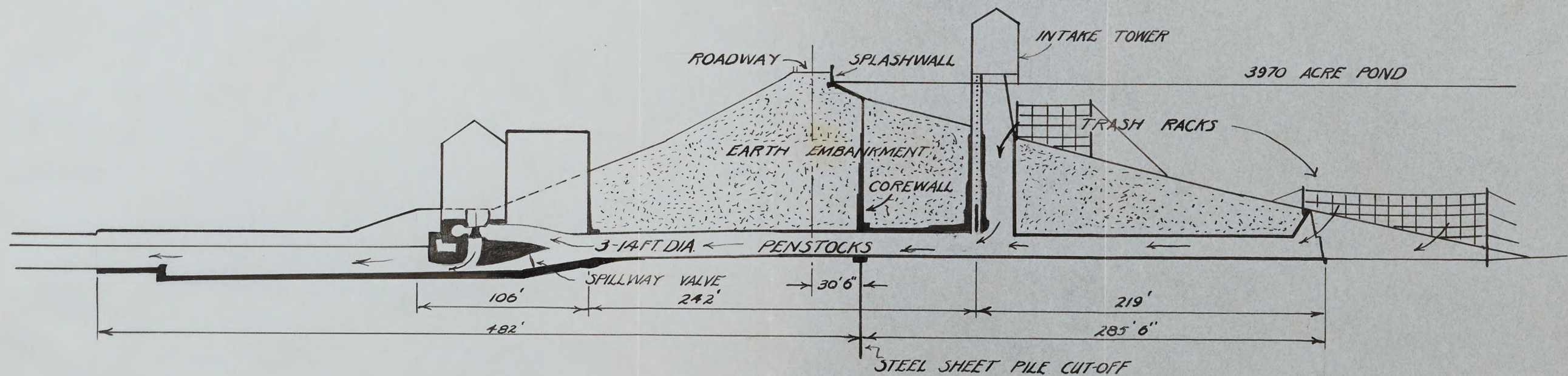
The location problem was to find the best mudstone deposit at foundation elevation. Test holes were drilled across the valley all the way from the upper side of the Big Oxbow to a half mile below the site finally selected. Heavy mudstone deposits were evident at the cut banks above river level but only thin and scattering ones at foundation depth. This prospecting extended over several years and finally located a 20 foot thickness on which the intake tower and power house could be arranged to best advantage. The site having



Valley of the Muskegon River.

been selected, the power house, spillway and embankment had to be arranged to best suit the foundation and topography. The final arrangement was on the seventh base line.⁴ The separation of intake and power house, differing from all other hydro-electric projects, was partly a matter of economy and partly a matter of foundation. The separation of intake and power house by buried penstocks lessens and spreads out the loading and simplifies both design and construction. Settlement is the major item to be guarded against, but past experience in building dams on Michigans glacial drift has shown that all dams settle about two inches during construction and in proportion as the load is added.

Hardy settlement, based upon this experience, was estimated at three to four inches, with a possible maximum of six inches. Penstocks, however, were designed with flexible joints at 30 foot spacing, both in the riveted steel liner and encased reinforced concrete shell which transmits the embankment load to the penstock foundation. Joints between intake and penstocks, and where penstocks join the power house substructure were also made flexible, all to permit a possible movement of several times the expected six inch maximum settlement, just as a matter of assurance. Except for these joints the intake tower and the entire power house block were each designed as separate units. Actual settlement records were kept throughout construction at twenty points on the core-wall, twenty-five points on the embankment, and twenty points on the penstocks and power house structures. Core-wall points were carried up from



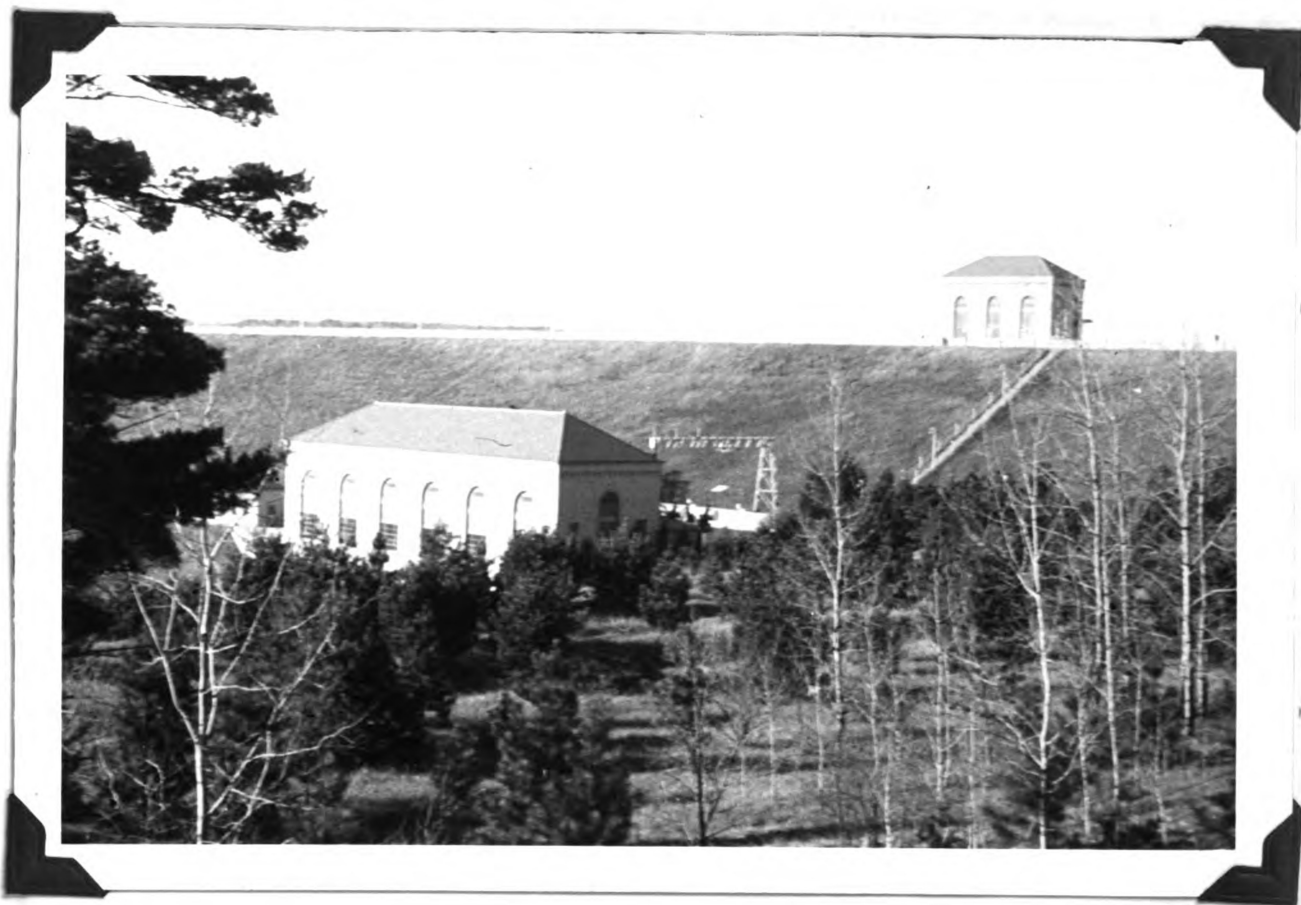
CROSS SECTION AND SETTLEMENT OF HARDY DAM

the base to make certain that true settlement at the base of the dam from the beginning was obtained not including consolidation of the embankment itself. Results have shown that the core-wall base settled $3 \frac{1}{8}$ inches the highest part of the dam, with a maximum of 4 inches, and the settlement has decreased proportionally to load toward each end of the dam. The consolidation of embankment was $1 \frac{1}{2}$ inches at the deepest section, this small amount being due to complete and effective settlement during placement by sluicing all embankment fill into place with water. The penstocks settled about three inches under the core-wall, decreasing gradually to 1 inch at the power house, which likewise settled only 1 inch. The additional load due to filling the pond increased the settlement only a small fraction of an inch, less than expected.

Before inspecting the design of the Hardy Dam let us consider some earth dam failures that might have been avoided by proper design.

By far the commonest cause of earth dam failures is lack of sufficient spillway capacity to take care of floods. An analysis of the records of 102 earth dam failures revealed the following:

<u>Cause of failure</u>	<u>Percentage of total</u>
Overtopping - - - - -	39%
Leakage along conduit - - - - -	19%
Other leakages - - - - -	39%
Slides - - - - -	5%
Miscellaneous - - - - -	4%
Not stated - - - - -	4%



View of Hardy Dam Showing Earth Embankment, Power House and Intake Tower.

The practical criteria for the design of the Hardy Dam or any earth dam may be stated briefly as follows: An Earth Dam should be designed so that;

1. The line of saturation is well within the downstream toe.
2. There will be no opportunity for the free passage of water from the upstream to the downstream face.
3. Water which passes through and under the dam must when it rises to the surface, have a velocity so small that it is incapable of moving any of the material of which the dam or its foundation is composed.
4. The freeboard must be such that there is no danger of overtopping by wave action.
5. The upstream and downstream slopes must be such that with the materials used in construction, they will be stable under all conditions.
6. The spillway is so great that there is no danger of overtopping.

An earth dam designed and built to meet these criteria will prove as permanent as any of the works of man if proper attention is given to important details during construction.

CRITERION 1

Line of saturation within downstream toe. The hydraulic gradient, or plane of saturation, through sand embankments is relatively high, and is marked by the separation of dry and saturated material that is

quite definitely defined. It varies over quite a range in any vertical plane parallel with the face of the dam. If the fill material were truly graded and of known size this seepage surface would presumably be regular, uniform, and determinate by analytical means. Any such graduation of the wide range of random occurrence in economically available borrow pits has seemed impossible although it is desired theoretically. Consequently, the saturation line represents an average of many experiments with a Pitot Tube in a number of embankments. Experience has indicated a safe generalization for this form and composition of embankment, namely, this plane remains below a 1 on 5 slope from head water to tail water. Embankments thus designed without drainage, occasionally have a few damp or possibly wet spots at the toe. The drains at Hardy Dam reduce this percolation factor to 4.25, but the filter bed arrangement around the drain heads restrains the material effectively, as compared with a free surface.

The embankment is underdrained by means of a downstream trench drain, constructed by filling the lower part of a dirt trestle, properly lagged with poles, with cobble stone, out of which 12 inch tile headers project at 100 foot spacing. All drain outlets are preferably placed above tail water and are arranged separately and visibly for observation and measurement.

CRITERION 2 and 3

Should be no opportunity for the free passage of water. Velocity of water through dams must be too low to cause piping. No such thing

as an absolutely impervious earth dam exists. Even if a dam is entirely composed of the tightest, finest clay, some water will get through. Indeed, water is constantly flowing through and under all earth dams, in a never ending stream. The earth dam simply interposes resistance to the flow of water and slackens its velocity, so that the water descends to join the ground water before it reaches the downstream toe of the dam.

Seepage from headwater through, around and under the embankment is another major design consideration peculiar to Michigan conditions. With a thickness of miscellaneous and unknown water saturated drift hundreds of feet in thickness even under river beds, and underground waters moving through deep sand and gravel with considerable pressure and volume, there is always a possibility of underground loss of considerable magnitude out of a pond, increasing of course with head. Embankments, even when built with sand, are made as tight as possible with a core-wall.

The answer has been to drive a continuous steel sheet pile cut-off beneath the core-wall, full length of the dam, and to a depth depending upon the head and the underground formation. At the Hardy Dam, beneath the deep section of the embankment, sixty foot heavy arch section piling was driven under considerable difficulties. This cut-off, intercepting several layers of mudstone in its full length, has effectively stopped underground seepage. The east end of the embankment is joined into an almost solid clay bank and tightly sealed. The west end tails out across a wide sand and

gravel flat where the head is only 25 feet but where seepage is relatively greater than elsewhere at the dam.

CRITERION 4

Earth dams must be safe against overtopping. Freeboard is usually defined as the difference in elevation between the top of the spillway and the top of the dam, but as the spillway of the Hardy is covered by almost 100 feet of water, in this case, the freeboard is the difference in elevation between the full pond and the top of the splashwall.

The core-wall of the Hardy Dam was moved 30 feet 6 inches upstream from the usual location and was "topped out" with slope paving to take wave action for any probable drawdown. The splash-wall, slope paving, and core-wall are all jointed and water stopped. The water elevation of the full pond is 822 feet and the splashwall elevation is 832 feet. The downstream slope of the dam was sodded to prevent any erosion that might be caused by rainfall.

CRITERION 5

Stable sloped required. Downstream slope. The slope of the downstream face will often be determined by criterion 1, which requires the line of saturation to intersect the base at a point well within the downstream toe. The slope of the downstream face, however, should be flatter than the angle of repose of the material of that part of the dam even if the position of the line of saturation would permit the use of a steeper slope. The downstream slope of the Hardy Dam starts at the top of the dam with a 1 on 2 slope and midway down the bank



Emergency Spillway.

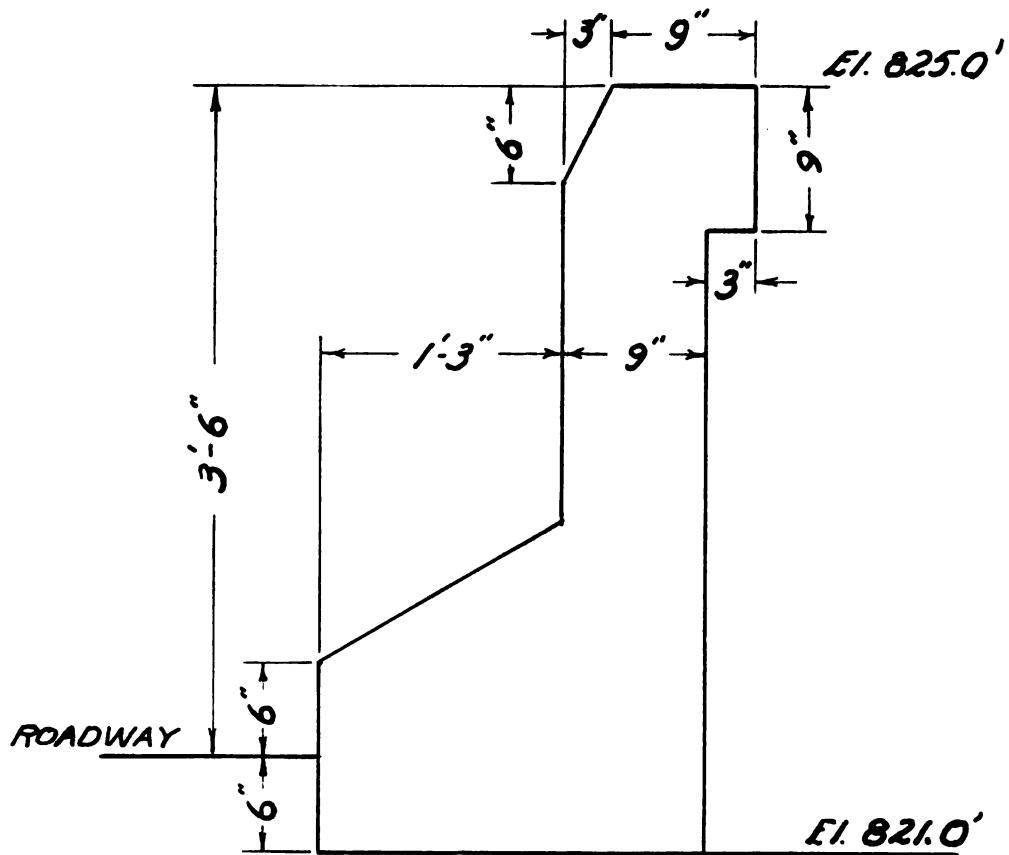
the slope becomes a 1 on $2\frac{1}{2}$.

The slope of the upstream face of the dam should generally be determined by the underwater angle of repose of the material. Other things being equal, a material having a high unit weight will stand on a steeper slope under water than one with a low unit weight. The slope of the upstream face should be flatter than that indicated by the experiments as safe. From the results of the experiments taken on the borrow pit soil, a 1 on 4 slope was finally decided on as a safe slope.

CRITERION 6

Ample spillway provision. The effect of soil cover and slope on flood discharges is very marked. Thus the watershed of the Muskegon River, which is covered with sand of indefinite depth, has a ground water storage which is tremendous. The result is that for this river the recorded maximum discharge is only two or three times the minimum.

Spillway capacity is provided by the three 12 foot diameter butterfly valves controlling bypasses from each penstock to each draft tube. Additional emergency capacity is provided by a concrete overflow weir and paved channel at the west end of the embankment, which is a added safeguard in case of a phenomenal flood. The overflow weir at the spillway intake is built of short concrete sections of varying thickness and stability, designed to overturn and wash away progressively so that the spillway overflow will increase as the pond rises, but will never exceed or even equal the inflow into the pond.



AVERAGE SECTION OF
TIPPING WEIR



View of Tipping Weir?

An interesting item would be to find at what elevation the weir would wash away.

RESISTING MOMENT:

$$(1.25) (.5) (150) = (93.75) (.834) = 78.1$$

$$(1) (1.25) (150) = (187.5) (.625) = 117.$$

$$(.75) (3.25) (150) = (366.) (1.625) = 595.$$

$$(.5) (.25) (150) = (18.75) (1.416) = 26.5$$

$$(.5) (.5) (150) = (37.5) (1.75) = 65.6$$

$$(.25) (.75) (150) = (28.1) (2.125) = \underline{59.7}$$

$$931.9' \#$$

When the overturning moment is slightly larger than the resisting moment the weir will overturn.

$$(x) \left(\frac{x}{2}\right) \left(\frac{x}{3}\right) (62.4) = 931.9 + (.75) (62.4) (x - 4) (1.875)$$

$$10.4x^3 = 87.6x + 581.9$$

$$x = 4.55 \text{ feet}$$

Therefore when the pond level rises to 829.55 feet, the weir washes away and the spillway entrance elevation drops from 825 feet to 821 feet. Other systems could have been used to judge the above but one method is enough to show how the weir was supposed to have worked. Since the time this thesis was started, certain changes have taken place in the design of the weir. This can be shown by looking at the photograph of the weir which reveals the fact that a concrete section has been placed in such a position that the weir is prevented from ever washing away. The regular spillway under the dam is capable of handling 30,000 cfs; this includes the passage of water through the generator turbines. If the emergency spillway has to be used this

capacity will be doubled.

Embankment totaling some 1,400,000 cubic yards was placed at the rate of 6,000 yards each day. A total of 55,000 cubic yards of concrete was poured, including some 8,000 yards of corewall beneath which about 1200 tons of steel sheet piling were driven for cut-off. Total cement used was 80,000 barrels and total reinforcing steel 1600 tons.

The fullpond creates a lake with a length of 20 miles and a maximum width of $1\frac{1}{2}$ miles backing onto Rogers Dam a few feet to permit drawdown without loss of head. Its area is about 4000 acres, its volume is 160,000 acre feet, equivalent to 250 feet deep on a square mile.

Higher earth embankments alone have been built, but either on rock foundation or separate from power house and penstock structures. As far as known no embankment of this height has been built on earth foundation and containing through penstocks and power house in the deep section. Because of this certain theories have been developed that can be used in designing future structures. These are:

- (1) Coarse sand, gravel, or glacial mudstone river bottom foundations support loads satisfactorily if such loading is graduated gently to a maximum of 13,000 pounds per square foot, with an average allowable settlement of 1 inch per 5000 pounds per square foot of loading.
- (2) The performance of this type of foundation indicates the probability that tests may develop still greater load carrying ability without excessive settlement or other difficulties.
- (3) Settlement is essentially proportional to load and takes place as the load is applied; it practically ceases thereafter.

(4) Sand embankments, composed of river valley and glacial outwash material, settle in permanent position and form when washed to place with water. Any subsequent consolidation is too slight to effect upstream paving that has been properly detailed and laid on a 1 on 2½ slope.

(5) This available embankment material is deficient in fines for the usual impervious earth core designs, and must be supplemented by some impervious element forming a part of the embankment. Alkali treatment should be investigated.

(6) Embankments of this material, on good sand and gravel foundation, can probably be constructed satisfactorily for heads in excess of 100 feet, by re-arranging the foundation cut-off and impervious members to better structural advantage, and with improved economy.

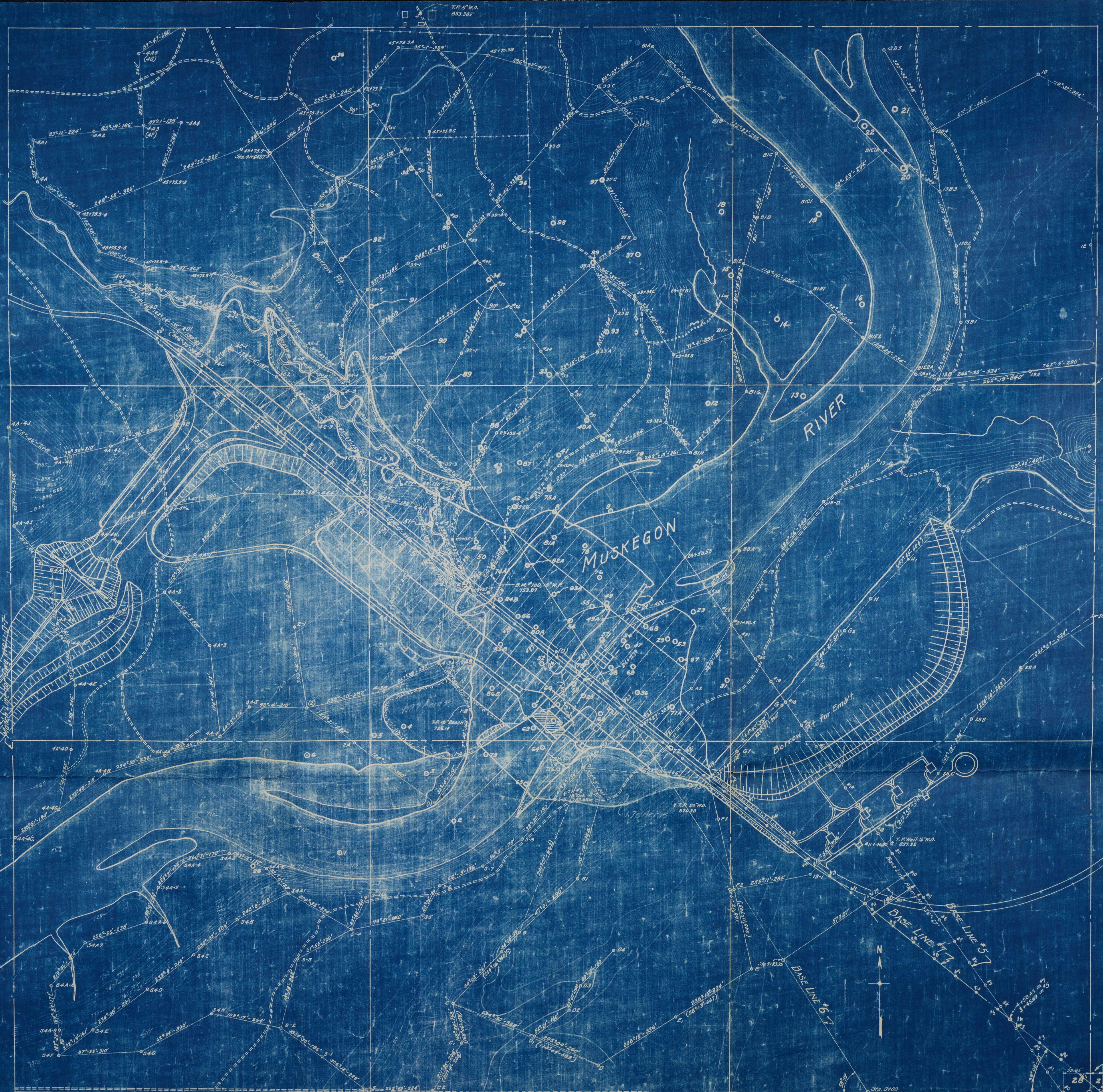
(7) Penstock structures properly designed in articulated sections for settlement may be built safely through embankments and on foundations as described.

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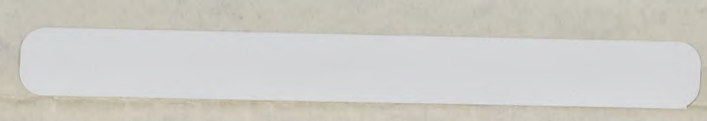
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