H. Bloompot

MARKED TREE SIPHON

A STUDY OF WELL POINTS AND GROUND-WATER CONDITIONS AT THE MARKED TREE SIPHONS

ST. FRANCIS BASIN PROJECT

WAR DEPARTMENT - CORPS OF ENGINEERS, U. S. ARMY

U. S. ENGINEER OFFICE, MEMPHIS, TENN.

1940

A STUDY
OF WELL POINTS AND GROUND-WATER CONDITIONS
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by
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A STUDY OF WELL POINTS AND GROUND-WATER CONDITIONS AT THE MARKED TREE SIPHONS.

Ground-water elevation during floods on rivers is built up by percolation and seepage only. There can be no direct flow into the underground reservoir from the rising water under any condition because of clogging of the filter. The experiment that made possible the above deductions was conducted by the Memphis Engineer District during the winter and spring of 1939 at the Marked Tree Siphons. It related primarily to the determination of hydrostatic uplift but also touched upon the effect of well points in lowering ground water and the effect of sheet piling walls on the hydrostatic head within the enclosure formed by the sheet piling.

To give the reader a clearer picture of just what was done, it might be well to elaborate to some extent on the history of the project that made the experiment possible. In the year 1923, as a method of flood control, local interests decided to divert the flood flow of the St. Francis River from the main channel to a constructed floodway some two miles in width just west of Marked Tree, Ark. However, the St. Francis at this point is a navigable stream, and so to comply with legal statutes it was necessary to pass the low-water flow across the levee and into the old channel. To accomplish this a lock and a sluiceway were built by the local interests where the new levee crossed the original channel. Difficulties in maintaining the sluiceway, which was a floating structure, were encountered shortly after its construction and, during the 1938 high water, failure occurred.

By Congressional action, the Memphis District was authorized to replace the destroyed sluiceway. Studies were inaugurated with a view to determining a safer and more suitable structure for passing the water. After considerable original research and preliminary design, it was decided that a siphon or battery of siphons would be the least expensive and, in view of the poor foundation conditions, the safest type.

While it is generally recognized that the siphon is one of the oldest of engineering structures, yet, strangely enough, practically no data as to its hydraulic characteristics can be found. Many and varied were the problems that faced the designing section, but a discussion of these is beyond the scope of this paper. The final approved design called for an excavated inlet channel, a reinforced concrete inlet basin, 3 electrically welded steel tubes, 9 feet in diameter and 228 feet in length, a reinforced concrete outlet basin and an excavated outlet channel. (See Fig. 1.)

The construction of the inlet-outlet basins was let to the List & Weatherly Construction Co., while the fabrication and installation of the 9-foot steel tubes were done by the Memphis Engineer District. Work began Dec. 1, 1938, and the siphon was officially put into operation and turned over to the Drainage District on June 7, 1939.

With that very sketchy history of the project in mind, we may now turn our attention to a discussion of the experiment itself. The object of this experiment as has been stated was, in reality, three-fold.

PRIMARILY, we hoped to determine what uplift pressure under the outlet basin would be built up by any given raise in the lake level;

SECONDLY, to ascertain the effect of a well-point system in lowering the ground water level; and

FINALLY, to determine the influence of a sheet piling wall on the hydrostatic head within the enclosure.

To gather the necessary data unconnected well points were driven along a general line extending from the inlet basin to the lower end of the outlet basin. These pipes were placed at varying intervals and all driven well into the water-bearing strata. (See Fig. 2.) The elevations of the tops of the pipes were determined by a careful line of levels, and with these known it was a simple matter, with the use of a calibrated chain, to calculate the elevation of the water standing in the pipes.

Initially, readings were taken every hour. As conditions became relatively stable, the periods between readings were lengthened to eight hours and then to twenty-four. If at any time, however, due to the contractor changing his system of pumping, a variation in the ground-water level was likely to take place, our schedule of readings was altered to meet the situation.

It was believed the placing of 25 test points, as shown in Fig. 2, would cover all critical areas in the layout and give us ample data. As construction proceeded, extreme difficulty was experienced in maintaining the original number of points, and in many cases they were lost entirely. Nevertheless, a sufficient number was retained throughout the experiment to give adequate and reasonably correct results, and it is believed that the subsequent deductions made are based on sound experimental data.

In this paper each of the three objectives of the experiment will be treated separately. While such procedure may necessitate some repetition, in all probability it will minimize confusion in the reader's mind.

A. Hydrostatic Uplift. - In the original design, it was assumed that the hydrostatic uplift under the outlet basin would be equal to the differential in elevation between the upper and lower pool; or in effect saying that the hydrostatic head of the general ground water would be equal to the elevation of water standing against the levee. A study of hydrology indicated that the maximum differential on record that existed between the two pools was 12 feet during the years the sluiceway was in operation. Accordingly, the outlet basin was designed for a hydrostatic uplift equivalent to 12 feet of water. Since the inlet basin is under water during all operating conditions, there can be no uplift under it and there need be no discussion as to its design. While we made our design for a 12-foot uplift, we realized that this was based largely on assumption and followed standard practice with little regard to its exact correctness. Accordingly, to obtain sufficient data for more precise design in the future was the primary object of these investigations. With the points as shown in Fig. 2, and following a schedule of readings as outlined above, a considerable quantity of data was gathered relative to the uplift under the basins. It must be remembered that during a portion of this time the contractor was using a well-point system to dry the site. Naturally, this had a material effect on our readings, and the actual relation existing between the river stage and hydrostatic head could not be properly determined until such time as his operations ceased. They continued from December until middle April and considerable data taken during this period must be viewed in the light of that condition. On April 28, he pulled all the points and, for practical purposes, ceased all but minor pumping operations. Consequently, such data as were obtained after that date can be considered as unaffected by outside influence.

B. Results. - The following table shows the relation of the upper and lower river channel gages and two critical well points numbered "O" and "B" in Fig. 2. Naturally, all the readings taken are not tabulated, as their inclusion would be of little value. Instead, the writer has tabulated only the maximum and minimum readings noted during any period of sustained rise or fall in river elevation. The reader must bear in mind that the experiment covered the so-called "flood season" and there were several substantial rises and falls in the river. These were naturally reflected in the well-point readings, and so a tabulation of maximum and minimum readings for both river stages and well points gives an ideal comparison without the necessity of including numerous unimportant readings.

TABLE I.

Date	Upper River Gage	Outlet Channel Gage	Well Points: O	Well Points: B
1938				
Dec. 9	212.1	-	202.5	-
1939				
Jan. 27	212.95	-	202.5	
31	213.2	-	207.5	
Feb. 3	214.4	-	206.8	
7	216.5	205.8	211.3	
15	219.2	203.0	213.5	
Mar. 1	218.6	205.6	211.7	
4	218.6	205.1	209.9	
9	218.8	205.0	209.4	188.3
17	219.9	204.2	209.9	184.8
31	217.8	201.5	207.3	
Apr. 4	216.9	201.5	211.6	
9	216.5	203.5	208.5	
12	216.7	202.5	205.9	190.3
30	219.1	201.9	211.2	204.8
May 8	217.8	201.4	209.8	204.5
18	215.8	201.1	209.5	204.6
21	215.5	201.5	208.4	204.6
31	215.6	202.5	208.4	204.5
June 5	214.9	-	208.0	204.5

Fig. 3 shows graphically the plotted results tabulated above, and forcefully points out the total lack of one hundred per cent uplift under the outlet basin. With a maximum differential between the upper and lower pools of 17.2 feet, the hydrostatic uplift on the discharge basin was only equal to 2.9 feet while the maximum observed uplift was only 3.5 feet with a head of 14.7 feet. Indeed, even under the inlet basin, which is in the river itself, the hydrostatic head after all pumping ceased was some six to seven feet lower than the river stage - and this, directly under the river. The reader may question the high differential obtained in view of our previous statement that the maximum on record was only 12 feet. However, it must be remembered that during construction there was no flow into the lower channel from the St. Francis itself, and consequently we find very low stages there in comparison to the high stages in the upper pool. Another point worthy of mention is the similarity of pattern followed by all the curves. You will notice that points of maxima and minima readings for both river stages and well points occur very nearly simultaneously. If anything, there is a slight tendency for the well points, that is the hydrostatic uplift, or, in other words, the hydrostatic head of the general ground water, to lag somewhat the river stage.

Fig. 4 shows a profile of all well-point readings taken simultaneously and illustrates how the ground-water level slopes from the upper pool or inlet basin to the outlet basin. Notice here, too, that the maximum stage is considerably below the river stage. Fig. 4 reflects condition when all the contractor's well points had been shut down for over thirty days and should represent the true relationship between the river stage and the ground-water level.

Still another condition not reflected in any tabulation or plotted graphs is one that must be substantiated from the writer's actual observation. During the course of the experiment, it was noticed that in the outlet basin water was flowing from the well points and equalizing pipes built into the basin. This would serve to demonstrate that the actual ground-water level lags the hydrostatic head of the ground water.

C. Conclusions. - Although the majority of our readings were taken prior to the shutting down of the contractor's pumping system, the writer believes such readings as we do have after April 28 are ample to satisfactorily demonstrate that the outlet basin was not subject to uplift pressures of the magnitude basically assumed.

An examination of the data gathered during the last 20 days of the experiment when all pumping had ceased and so outside influence was eliminated shows that the general ground-water elevation was some eleven feet less than the upper lake elevation; whereas, the differential in pool level was thirteen feet. Further examination of Fig. 3 shows that the curves of lake elevation and ground-water elevation closely parallel each other as to rise and fall and indicates conclusively that the rise and fall of the ground-water level is a function of the rise and fall of the river. However, the ground-water level not only lags the movement as indicated by the discharge of the well points, but also there is a material difference in height obtained. This apparent new conception is not difficult of analysis. If the river bottoms were always composed of a permeable sand and gravel, there would be a direct flow to the ground water with the result that the full hydrostatic head of the river would be transmitted to it. In times of flood, the river carries large quantities of silt and impervious material. The material is deposited on the bottom and tends to clog the filter bed and make it relatively impermeable. In this case the ground water is not fed by a direct supply, but receives its supply by percolation through the upper layer to the water-bearing strata. The percolation results in a great loss of head with the result that the actual river stage is never reproduced in the ground-water elevation and consequently the hydrostatic uplift is not the differential between the two pools, but some much lesser figure. In brief summary, our final conclusions on this phase of the experiment are:

1. Rise and fall of the ground-water level is a function of the rise and fall of the river.
2. The ground-water level lags and never equals the level of the upper pool.
3. The ground-water level is built up by percolation and seepage during periods of flood.
4. There can be no direct flow to the underground reservoir during periods of high water.

D. Effect of well points in lowering ground water. - It was originally anticipated that considerable data might be obtained showing the quantity of water actually pumped out by the well-point system. However, it was not feasible to gather such data in quantity, and all that this writer can do is briefly explain the contractor's well-point system and, by means of plotting show the ground-water level that existed at the same river stage, both during pumping and again several weeks after all operations ceased. Since work was almost finished on the inlet basin when this experiment began, we will ignore the contractor's plan of operation there and study only the construction of the outlet basin.

The elevation of the bottom of this basin was to be 187, with a 2-foot blanket of gravel, making the extreme low elevation 185. Natural ground water was considerably above this level, and for the contractor to work in the dry, it was necessary for him to install a rather elaborate system of well points and pumps. As you see from Fig. 1, around the basin and down to elevation 165 was to be driven a rectangular steel sheet pile wall. The contractor originally contemplated using 150 1-1/4-inch well points outside the sheet piling and 30 inside, all working off three 6-inch pumps. On March 1, he began pumping with 111 well points around the outside, working off three 6-in. pumps. With this layout, the header pumps were at elevation 205, and the points approximately 180. Three days later he installed 48 points inside the basin, working off one 6-inch pipe. These points were driven to elevation 173. After pumping some 12 hours, it was possible to shut down one pump in the exterior system, and no change was needed until the excavation reached elevation 185. At this point the installed system was not capable of further lowering the water. The contractor then completely revised his interior system, added additional points (see Fig. 5). With this layout he was able to satisfactorily do the construction work in the basin with little trouble. In summary, his final system was as follows:

Exterior System - 111 points on 2 1/2-foot centers down to elevation 180, working off three 6-inch pumps.

Interior System - 100 points on 2 1/2-foot centers down to elevation 177, working off three 6-inch pumps.

Results.

The graphs, Fig. 5, show the results of these systems in lowering the ground water. Any attempt to discuss the merits of the installations must be overlooked, because we have no data which will substantiate any deductions. Only this do we know from observation - that in every case the contractor achieved better results by driving his points deep. That is, a well point at elevation 173 discharged 50% more water than did one at elevation 180 with both points working under the same vacuum. This would indicate that the most economical results can be obtained by driving the points well into the water-bearing strata, thus taking advantage of the freer access of water to the point and the full hydrostatic head available, thereby, in effect, decreasing the lift of the pump.

As was expected, the shape of the ground-water curves during pumping was conical. The rather surprising thing was the comparative length of path affected by the well points. Fig. 6 illustrates a splendid example of this. It shows the ground-water curves for the same river stage when pumping was going on and again when practically all pumping had been shut down for ten days. The comparison of these two curves shows a lowering of the level from 20 feet at the points to 3 feet at 186 feet out. This lowering was accomplished with the final pumping system as mentioned above.

Although not directly related to the experiment, the reader's attention is invited to this fact. During construction, the bottom of the discharge basin was 35 feet below the level of the water against the levees only 112 feet away.

E. Effect of the sheet piling on the ground-water level. - During actual construction of the outlet basin no satisfactory data could be determined as to the relative effect of the sheet piling on the ground-water elevation; for during this period, the elevations of the well points in the interior and exterior systems were quite different, the exterior having been jetted to 180, and the interior to 173. Naturally, under such a setup the interior basin always showed a lower water level than immediately outside. However, with the completion of the pumping, we were able to determine some rather interesting data. Three points, No. 11, just outside the sheet pile, No. B, inside the basin, and No. 18, on the downstream side of the basin, were used.

The observations covered a period of forty days, and a sample of the readings is tabulated below:

Date	:	Point 11	:	Point B	:	Point 18
Apr. 28	:	208.3	:	205.9	:	206.9
29	:	205.9	:	204.9	:	206.9
30	:	205.8	:	204.8	:	206.7
May 1	:	205.9	:	204.9	:	206.8
8	:	207.3	:	204.5	:	204.5
13	:	205.7	:	204.8	:	204.4
17	:	207.4	:	204.6	:	205.6
21	:	207.3	:	204.6	:	205.5
31	:	207.2	:	204.5	:	205.6
June 7	:	208.2	:	204.5	:	205.6

Between the dates of April 28 and May 13, some pumping took place, probably sufficient to disturb to a certain degree the above readings. After May 13, all operations ceased, and we see the observations became relatively stationary. Figs. 4 and 6 illustrate graphically the relation existing, and even a casual glance at this will show the definite drop in the hydrostatic head inside the basin and the apparent resumption of the original water-line profile immediately below the sheet pile.

Conclusions.

From these data the logical conclusion is that the sheet piling acts as a cutoff and reduces the hydrostatic head, in this case about 2.5 feet. Such a reduction in head can only be caused by a loss due to friction. In other words, the length of path of the ground water under the outlet basin has been increased by the length of the sheet piling. Under ordinary circumstances, such an increase would have no effect, because, with no flow, the water would build up to its static head in spite of the extra resistance entailed by the sheet piling. In this case, however, pressure relief wells were left in the outlet basin and, as a result, the water constantly flows up along the sheet piling and out the wells. This flowing water must overcome the additional friction, and in so doing loses a portion of its head, thus making the hydrostatic uplift within the basin somewhat less than that immediately adjacent to the exterior. It would seem from this, then, that sheet piling walls may be expected to cause a lowering of the uplift, providing some suitable relief wells are left within the enclosure. With no such device, the sheet piling will have little, if any, effect on the ground water.

The above conclusions are substantiated in a large measure by observations taken within the inlet basin. The readings there show no definite drop inside the sheet piling. Rather, the typical curve shows a gradual decrease until the outlet basin is reached (see Figs. 4 and 6). It must be remembered that in the inlet basin there are no relief wells, and so from the above deductions we would expect to find no sharp reduction of uplift or hydrostatic head.

Summary.

In summarizing this paper, I should like to emphasize again that while a considerable portion of the data gathered could not be used, nevertheless, a sufficient quantity was available to warrant the deductions made. Furthermore, no attempt was made to sort out the data and use only that which readily bore out the conclusions. Rather, all data used are typical.

The conclusions drawn from the experiment in summary are:

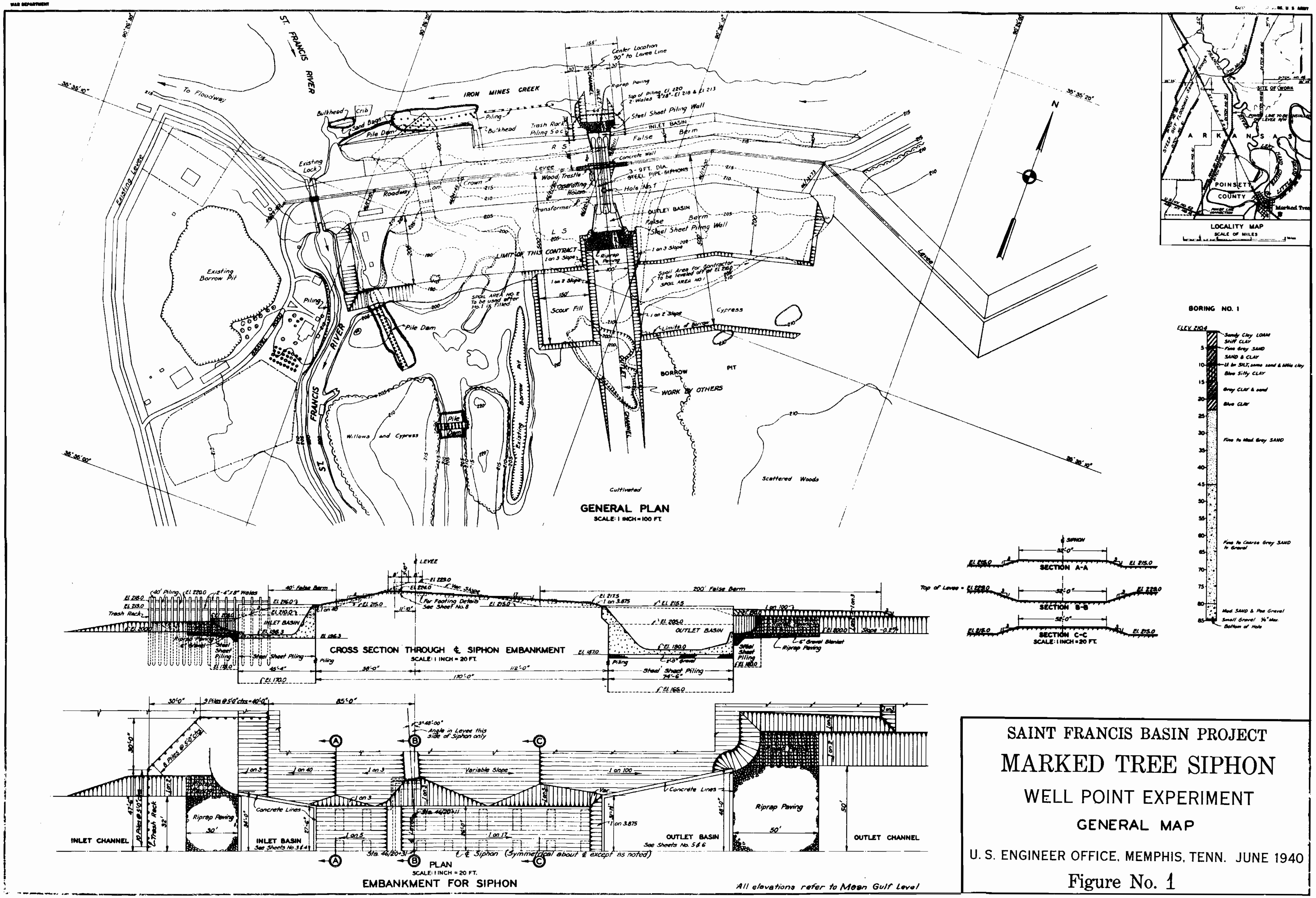
1. The uplift under the outlet basin was considerably less than that designed for. In no case did it approach the magnitude of differential between the upper and lower pools. The reasons for this are:

a. During periods of high water the filter bed of the river tends to become clogged by deposits of mud and silt. Consequently, there is no direct feeding of the ground water from the river. Rather, it receives its supply by percolation and seepage with resulting large losses in head; and so the river stage is never reproduced in the ground-water elevation.

b. Sheet piling in outlet basin gave minor reduction in uplift.

2. Pumping from a well-point system tends to lower the ground-water level in a somewhat conical curve. The maximum lowering occurs at the points and drops off sharply within 50 feet, though the effect is felt for some distance beyond the line of points.

3. Sheet piling tends to reduce hydrostatic uplift to some extent only when pressure relief wells are incorporated in the structure, thus permitting a flow of ground water up and along the piling and preventing the building up of hydrostatic head.



SAINT FRANCIS BASIN PROJECT

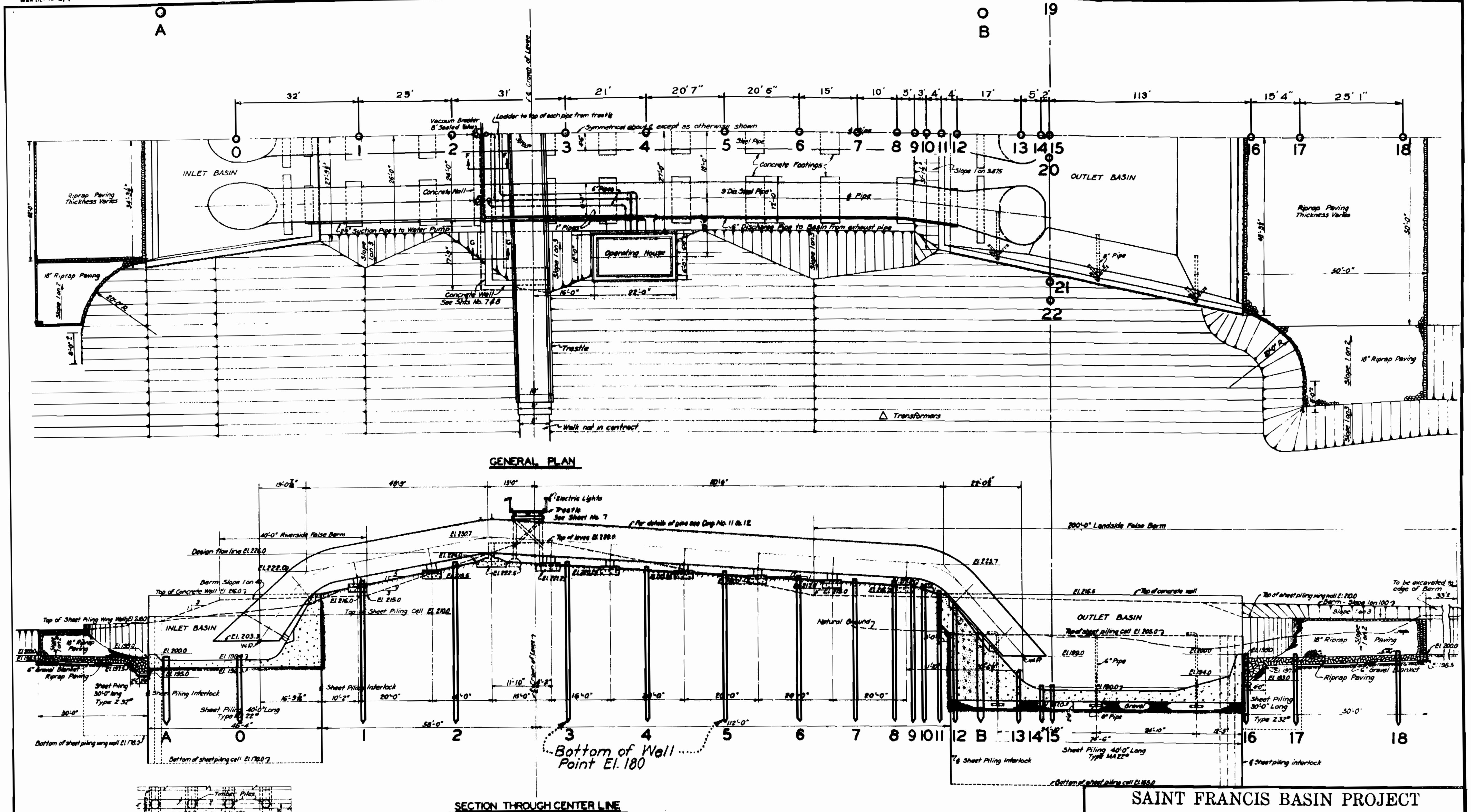
MARKED TREE SIPHON

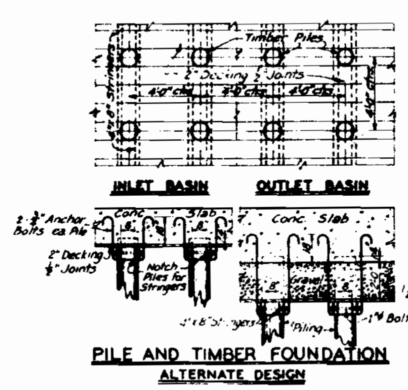
WELL POINT EXPERIMENT

GENERAL MAP

U. S. ENGINEER OFFICE, MEMPHIS, TENN. JUNE 1940

Figure No. 1





Pile and Timber Foundation will not be installed unless directed by contracting officer after excavation is carried to indicated level of basins.

Gravel blanket for Outlet Basin may be omitted if directed by the contracting officer and foundation for the Outlet Basin made the same as Inlet Basin

GRAPHIC SCALE

10 0 10 30 50 FEET

GENERAL NOTES

Gravel blanket shall be the same gravel as used for coarse aggregate of concrete.

Steel sheet piling marked on the plans as MA 22# shall be medium arch type pile weighing 22# per sq. ft., with a section modulus of 5.4 in.³ per foot of wall, and an interlock strength of 8000# per lin. inch.
Steel sheet piling marked on the plans as Z 32# shall be Z type pile weighing 32# per sq. ft., with a section modulus of 38.3 in.³ per foot of wall, a strength of 8000# per lin. inch.

o Well Point

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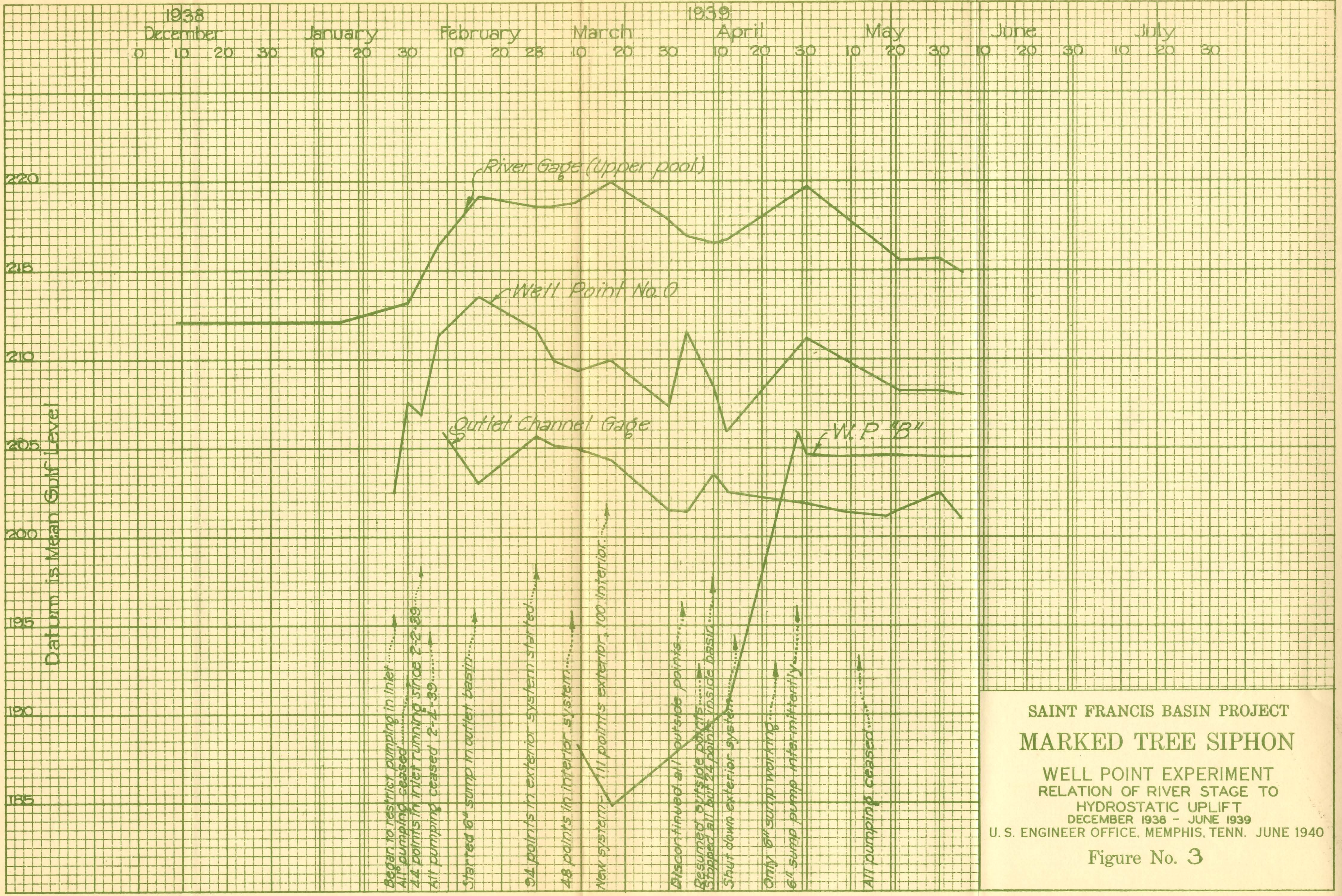
MARKED TREE SIPHON

WELL POINT EXPERIMENT

PLAN AND SECTION

U. S. ENGINEER OFFICE. MEMPHIS. TENN. JUNE 1940

Figure No. 2



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MARKED TREE SIPHON

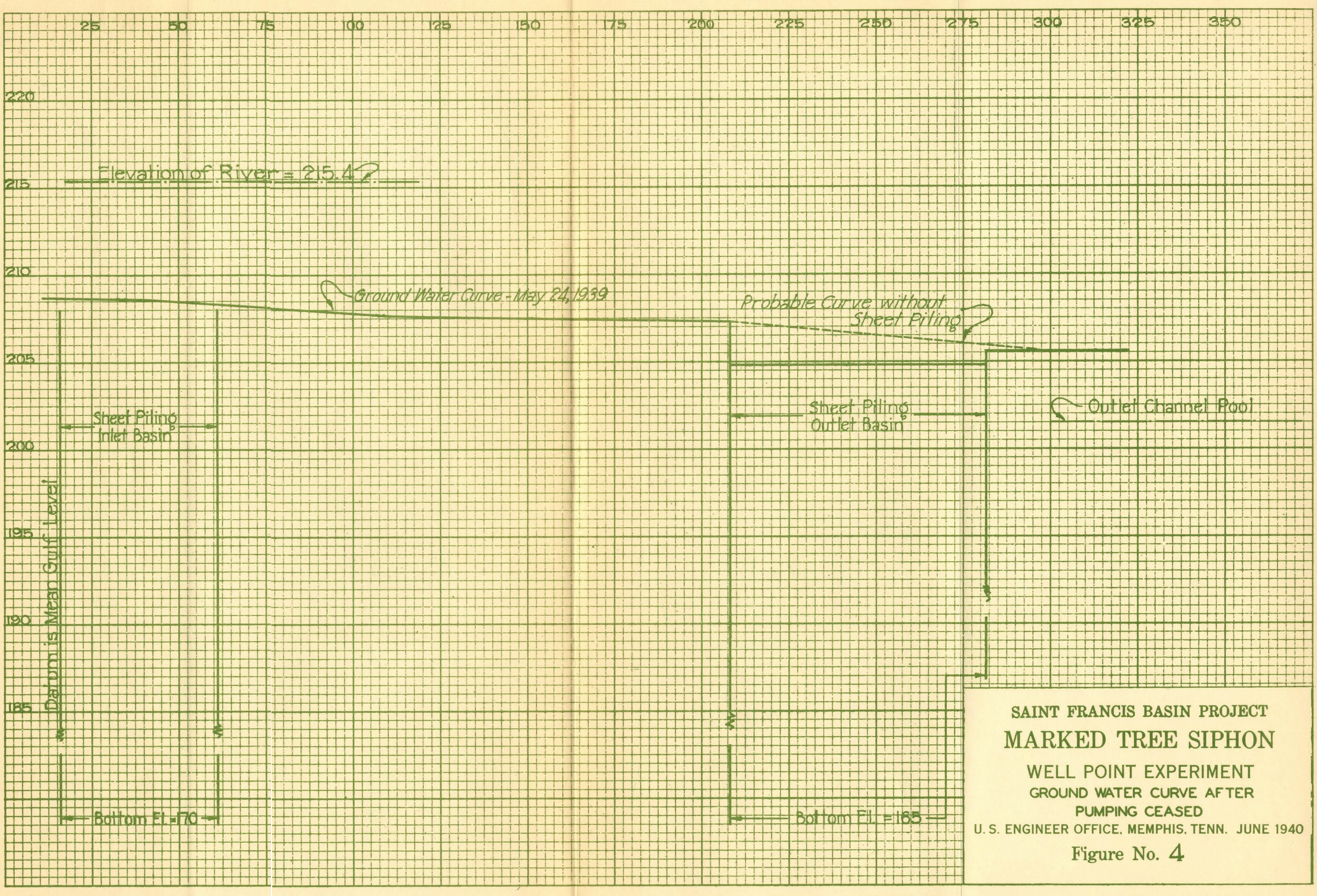
WELL POINT EXPERIMENT

RELATION OF RIVER STAGE TO HYDROSTATIC UPLIFT

DECEMBER 1938 - JUNE 1939

U. S. ENGINEER OFFICE, MEMPHIS, TENN. JUNE 1940

Figure No. 3



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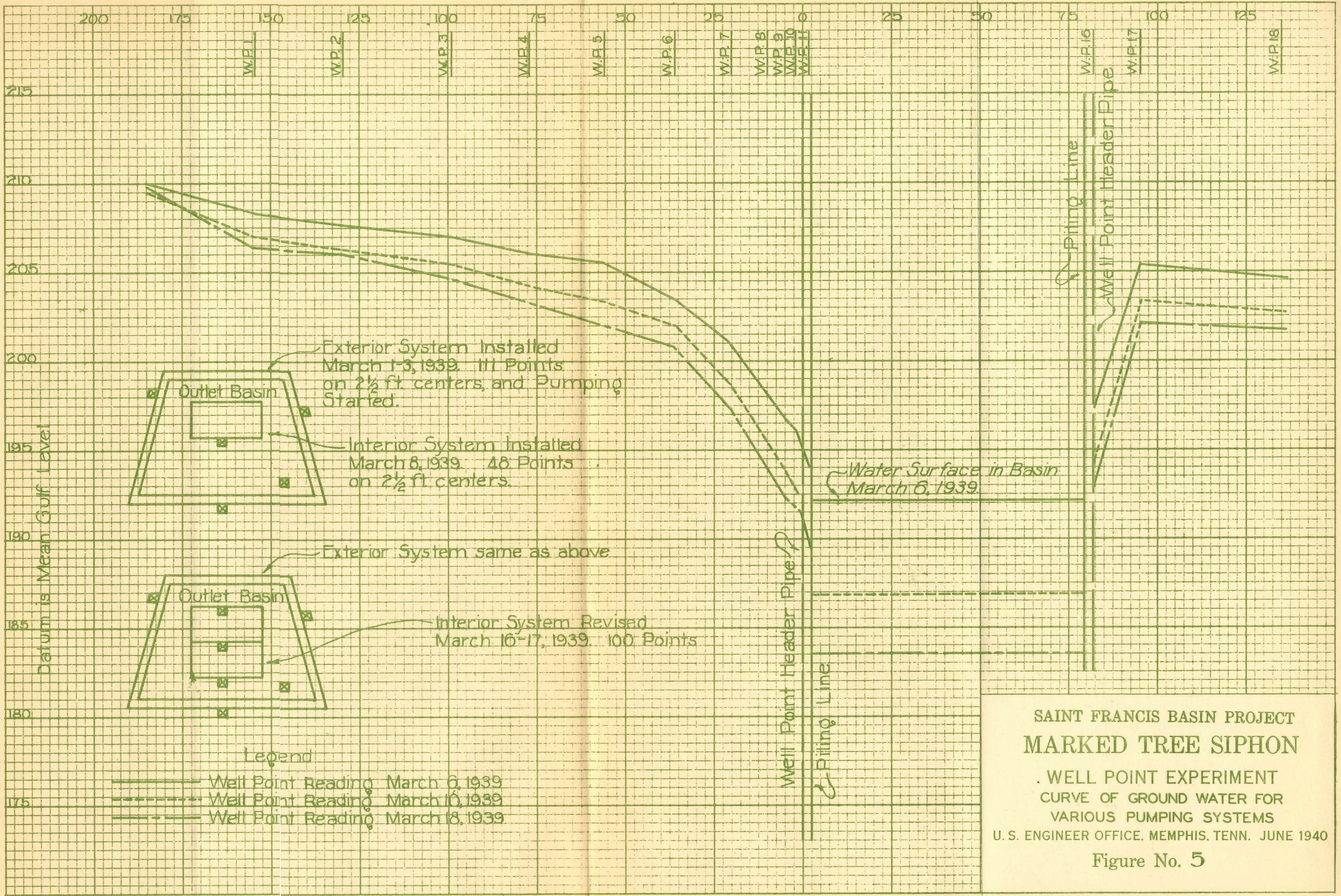
MARKED TREE SIPHON

WELL POINT EXPERIMENT

GROUND WATER CURVE AFTER PUMPING CEASED

U. S. ENGINEER OFFICE, MEMPHIS, TENN. JUNE 1940

Figure No. 4



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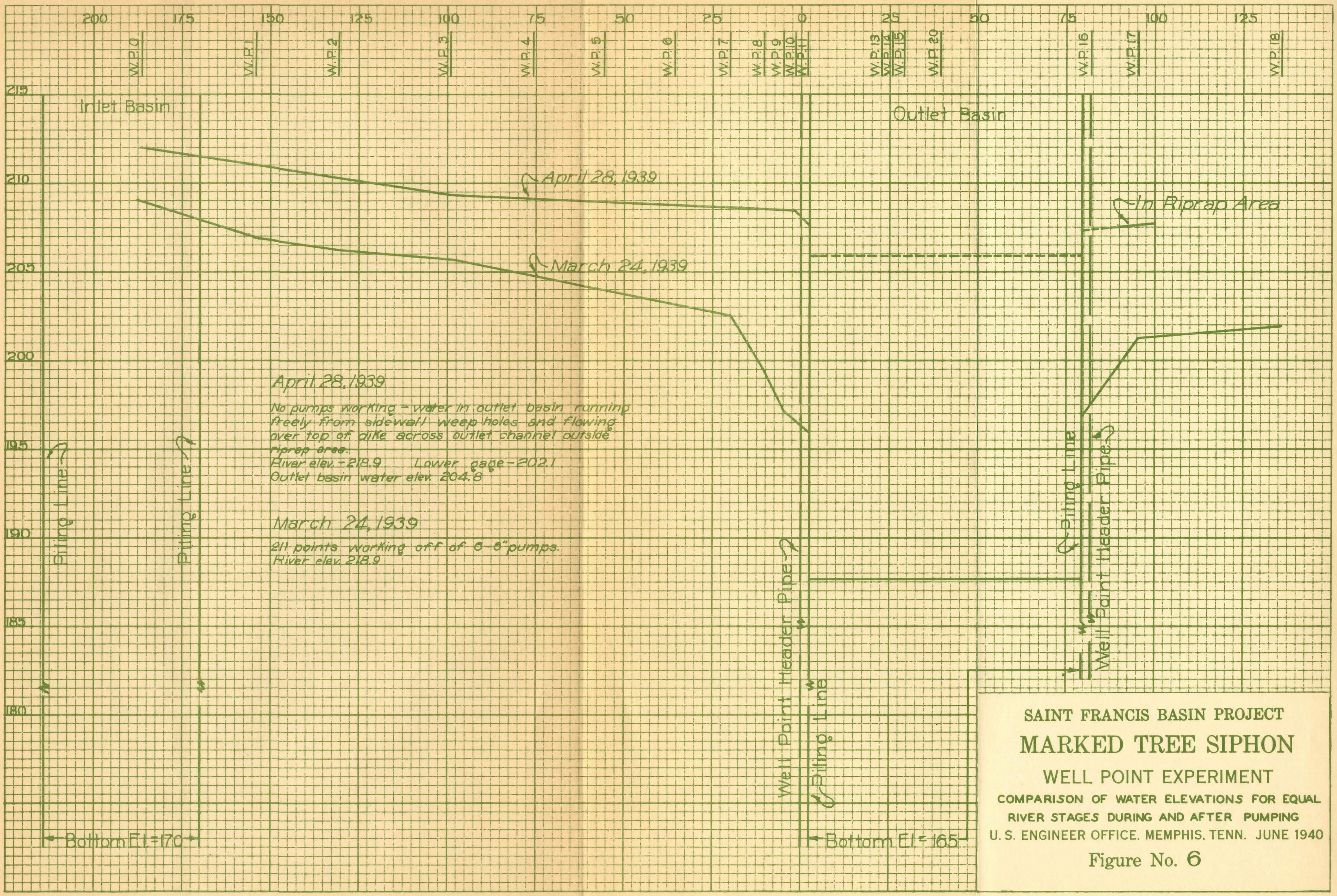
MARKED TREE SIPHON

WELL POINT EXPERIMENT

CURVE OF GROUND WATER FOR VARIOUS PUMPING SYSTEMS

U. S. ENGINEER OFFICE, MEMPHIS, TENN. JUNE 1940

Figure No. 5



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MARKED TREE SIPHON

WELL POINT EXPERIMENT

COMPARISON OF WATER ELEVATIONS FOR EQUAL RIVER STAGES DURING AND AFTER PUMPING

U. S. ENGINEER OFFICE, MEMPHIS, TENN. JUNE 1940

Figure No. 6