DEPARTMENT OF THE ARMY

CORPS OF ENGINEERS

MISSISSIPPI RIVER COMMISSION

STUDY OF RELIEF WELL INSTALLATIONS AT WOLF RIVER AND NONCONNAH CREEK PROJECTS MEMPHIS, TENNESSEE

TECHNICAL MEMORANDUM NO. 3-247

WATERWAYS EXPERIMENT STATION

VICKSBURG, MISSISSIPPI

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STUDY OF RELIEF WELL INSTALLATIONS AT WOLF RIVER AND NONCONNAH CREEK PROJECTS, MEMPHIS, TENNESSEE

Introduction

Background

1. Wolf River. The investigation of underseepage along the south levee of Wolf River at the Nonconnah Creek Pumping Station was begun in October 1942. An analysis based on the borings indicated that some provision should be made for the control of underseepage along this levee. The method of control selected was a system of relief wells. This well system was designed by the Experiment Station and installed along the levee in the most critical reach during July and August of 1943. In compliance with letter from the Office, Chief of Engineers, to the Mississippi River Commission, dated 19 March 1946, subject "Study of Relief Well Installation at Wolf River and Nonconnah Creek Projects", a piezometer system was installed at this site in March 1947. The piezometers were located in general to measure the hydrostatic pressure in the pervious substratum midway between the wells along the levee and the hydrostatic pressure gradient along lines perpendicular to the levee.

2. Nonconnah Creek. During the period September to December 1939, a relief well system was installed along the north levee of Nonconnah Creek at the Nonconnah Creek Pumping Station. These wells were 15 in. in diameter and were filled with sand and gravel. In June 1942 the Experiment Station made a review of the design of this well system. Based on laboratory and field tests, the review showed that additional measures were necessary to prevent the development of excessive substratum pressure

beneath the relatively impervious top stratum. Therefore, a new well system was designed and installed by the Experiment Station in 1942. Twelve piezometers were also installed at the same time to measure substratum pressure along and perpendicular to the levee. Six additional piezometers were installed in March 1947 so as to give a more complete coverage of the hydrostatic pressures. Four additional borings were also made at this time.

Scope

3. To augment the description of each project, drawings were prepared showing the location of the wells and piezometers in relation to the levee and drainage ditches, the elevation of well screens and discharge pipes, the elevations of the center of the piezometer screens, the soil profiles along the line of wells parallel to the levee and along the piezometer lines perpendicular to the levee, and the permeability of the pervious stratum.

4. Other plates present piezometer and well flow data in graphical and tabular form. The scales for these plates were chosen so that the entire range of data could be plotted. These plates are designed so that the piezometer readings and well discharges for each high-water period can be added, using a new set of plates for each year. These plates fall into four categories for each project:

a. A sheet on which may be plotted the hydrostatic pressures midway between wells and the discharge of each well. (Plate 5 for Wolf River and 12 for Nonconnah Creek.)

b. Several sheets on which the hydrostatic pressure gradients in the pervious stratum perpendicular to the levee may be

plotted. From these sheets the effective point of entry of the seepage may be determined. In addition, the discharge of the well nearest the piezometer line and the average discharge of the wells along the portion of the levee crossed by that piezometer line may be plotted on a daily graph. (Plates 6 and 7 for Wolf River and 13, 14, 15, and 16 for Nonconnah Creek.)

c. A sheet on which daily graphs of selected piezometer readings may be plotted together with the stage of the river and reservoir. (Plate 8 for Wolf River and 17 for Nonconnah Creek.)

d. A sheet on which all piezometer and well discharge readings may be entered in tabular form. (Plate 18 for both projects.)

From the data on these plates, the effectiveness and efficiency of the well system may be determined. As data become available and are plotted on the plates, analyses will be made and appended to this report. Appendix A contains an analysis of the data for the years 1943, 1944, 1945, and 1947.

Wolf River Relief Well Installation

Soil conditions

5. In general, the foundation under the levee at this location consists of a surface stratum of 8 to 15 ft of relatively impervious silts and clays underlain by approximately 40 ft of sand and gravel which, in turn, is underlain by impervious Tertiary clay. Soil profiles of the foundation along the toe of the levee and perpendicular to it are shown on plates 2, 3, and 4. The locations of these profiles are shown on the plan of the relief well system, plate 1. The upper 20 ft of the pervious

stratum consists, in general, of fine to medium sand with a trace of gravel. The lower 20 ft consists of coarser sand which contains a considerable amount of gravel. Based on laboratory tests the over-all permeability of the pervious stratum was estimated as about 750 x 10^{-4} cm per sec for the design of the wells. The laboratory permeability test data are shown on the soil profiles, plates 2, 3, 4, and in table 1.

6. From Sta 4/47 to Sta 5/13 (west of Bellevue Boulevard) seepage can probably enter the pervious stratum at Wolf River 800 to 1800 ft distant. Landside of the levee Leath Bayou extends along the full length of this section at a distance of 100 to 300 ft from the levee toe. This ditch cuts 4 to 8 ft into the top stratum, leaving only about 3 to 5 ft of the top stratum in the bottom of the ditch over the pervious sand stratum. Farther landside of the ditch, the natural ground surface rises 10 to 20 ft in elevation, thereby confining the exit for any underseepage to the strip 200 to 600 ft wide between the levee and the higher ground landward.

7. From Sta 5/13 to Sta 5/46 (Bellevue Boulevard to Payne Avenue), seepage can enter the pervious stratum in a large gravel pit adjacent to the levee. Landside of the levee, Cypress Creek has been relocated and straightened by excavating a channel to approximately elev 204. The boring logs indicate that the bottom of this new channel either cuts into or closely approaches the pervious substratum. The distance from the gravel pit to the relocated Cypress Creek is approximately 800 ft.

Operation of storage reservoir

8. The area between the levee and the high bank approximately 600 ft landside of the levee is used during high water as a storage reservoir

for surface water and industrial wastes which drain into Cypress Creek Pumping Station. As long as Wolf River is below elev 216, the gates of the pumping station are kept open, allowing the drainage in Leath Bayou and Cypress Creek to pass directly into Wolf River. When the river rises higher than elev 216, the drainage water must be pumped out to the river. The pumping station is designed so that the maximum pool in the reservoir is at elev 226. With water at this elevation in the storage reservoir, the wells are submerged 11 to 14 ft. The reservoir must be pumped to less than elev 212 to obtain discharge readings from all the wells. The net levee elevation at this site is 241.6 mGl.

Relief wells

9. Description. The relief wells at Wolf River consist of a screen section in the pervious substratum, a riser pipe through the impervious top stratum, and an outfall pipe which carries the well flow out into either Leath Bayou or Cypress Creek. The screen sections of the wells, composed of 2-1/2 in. I.D. perforated clay pipe (48 - 3/16 in. holes per ft of pipe) surrounded by 4-in. I.D. Walker porous concrete pipe, vary in length from 13 to 25 ft. The clay pipe was made by the Bowerston Shale Company in 1-ft lengths, with 1/2 in. wall thickness, and with flush shiplap type joints. The porous concrete pipe was made by Walker Cement Products Company in 2-ft lengths with flush shiplap joints. The gradation of aggregate used for the porous concrete pipe is shown on figure 1. Photographs of the different types of pipe used in the wells are shown on figure 2. The buttering of Asgum on the ends of all pipe sections can be noted in the lower picture.

10. The riser pipe is 3-in. I.D. clay pipe, also manufactured by the

Bowerston Shale Company, and joins the perforated clay liner of the screen section with a 3-in. to 2-1/2-in. reducer. It is connected to the 3-in. I.D. clay horizontal discharge pipe with a 3-in. clay pipe tee. The tee is set with the stem horizontal, and the top capped so as to make the well accessible for cleaning. The 3-in. I.D. clay pipe was made in 1-ft lengths with 1/2-in. wall thickness and flush shiplap joints. In March 1944, a vertical tie rod, anchored in a foot of concrete at the bottom of the well, was installed in each well to maintain a state of compression in the screen and riser pipe sections and thus prevent the joints from possibly opening and allowing sand to enter the well.

11. The discharge pipe is about 10 ft long, discharging onto a small concrete apron on the bank slope of the drainage ditch. (The lower photographs in figure 3 show a typical installation of a well discharging into a shallow collection ditch.) The upper portion of the riser pipe and the discharge pipe are encased in concrete. Provision was made for inserting a screw plug into the discharge pipe to prevent the wells from becoming silted up when submerged by the reservoir. These plugs have since been replaced with simple brass flap-type check valves which prevent the water in the storage reservoir from entering the well but allow free discharge. As the reservoir receives large amounts of industrial wastes, it is felt that if water were allowed to enter the top of the wells, the wells would soon become clogged and inoperative. A typical well detail is shown on plate 1.

12. Design of well system. The design of the well systems was based on the assumption of effective tailwater at the well outlets and a completely impervious landside top stratum. Other assumptions made in

the design of the wells west of Bellevue Boulevard are as follows:

Thickness of sand stratum, D	40 ft
Distance to source of seepage, d	1200 ft
Net head for computing discharge, h	30 ft
Radius of drainage well, r_w	0.25 ft
Permeability of foundation, K	750×10^{-4} cm/sec = 0.15 ft/min
Well penetration into pervious stratum	50%
Maximum allowable net head midway between wells, P	5 ft

The Muskat-Jervis method of analysis was used for the design. (See Part V, WES Technical Memorandum 195-1, "Field and Laboratory Investigation of Design Criteria for Drainage Wells", 1 October 1942.)

Using a 75-ft well spacing:

$a = 75$ ft $\qquad r_w = 0.25$ ft $\qquad D = 40$ ft

$$\frac{a}{r_w} = \frac{75}{0.25} = 300 \qquad \frac{D}{a} = \frac{40}{75} = 0.53$$

$$\frac{\text{Extra length}}{a} = 1.1 \qquad P \cdot \frac{K\,D}{Q} = 1.4$$

Extra length = 1.1 x 75 = 82 ft

$$Q_w = \frac{K \cdot h \cdot a \cdot D}{d + \text{extra length}} = \frac{0.15 \times 30 \times 75 \times 40}{1200 + 82} = 10.5 \text{ cfm} = 79 \text{ gpm}$$

$$P = \frac{1.4\,Q}{K\,D} = \frac{1.4 \times 10.5}{0.15 \times 40} = 2.5$$

Friction losses within well:

Length 2-1/2-in. pipe	= 20 ft
Equivalent length 2-1/2-in. perforated pipe (assumed)	= 15 ft
Length riser pipe	= 13 ft
Length discharge pipe	= 7 ft
Total length 3-in. pipe	20 ft

Assuming Hazen-Williams C = 100 and a flow of 79 gpm, head loss in well and discharge pipe = 2.8 ft.

Due to these friction losses within the well, this flow will be reduced a certain amount which in turn will lower the friction losses. By a

process of adjustment (see WES T.M. 195-1) the resulting flow has been computed as 72 gpm with a friction loss in the well of 2.2 ft. The final design is as follows:

Well spacing	75 ft
Penetration	50%
Estimated Q	70 gpm per well
Maximum P = 2.5 + 2.2	4.7 ft
Top of screen, elev	200.0 ft
Bottom of screen, elev	180.0 ft
Center of discharge pipes, elev	213.0 ft

13. These wells (1 to 25) are located on a line paralleling Leath Bayou and 10 ft from the top of the north bank. The location of the wells was determined from levee stationing starting at Sta 4/47+20 and extending to Sta 5/13+00. These wells discharge directly into Leath Bayou.

14. The design of the well system east of Bellevue Boulevard was made in a manner similar to that described in paragraph 12. The resulting design is as follows:

Number of wells	41
Spacing of wells(26 - 57)	75 ft
The spacing of wells from 57 to 66 increased 5 ft for each well	
Penetration	50%
Estimated Q	80 gpm per well
Maximum P	5.0 ft
Top screen, elev	200.0 ft
Bottom screen, elev	180.0 ft
Center of discharge pipe, elev	213.0 ft

15. These wells (26 to 66) are located on a line paralleling and at the top of the bank of Cypress Creek nearest the levee, and discharge directly into Cypress Creek. Wells 58, 61, 63, and 64 were destroyed since installation and have been abandoned. The well layout and elevations are shown on plate 1 and are given in table 3.

16. Method of installation. The wells were installed by jetting and

fishtailing a 10-in. casing to the required depth in the pervious stratum, lowering the perforated clay, porous concrete, and riser pipe on a drill rod into the casing, pulling the casing, and properly backfilling. All joints were buttered with an asphaltic material, "Asgum". All joints of both the riser pipe and porous concrete pipe were wrapped with a 5-in.-wide strip of cloth wrapping material about 36 in. long, thoroughly permeated with liquid Asgum. At the junction of the riser pipe with the perforated clay and porous concrete pipes, a waste packing filler and heavy coating of Asgum were first applied, followed by the above-mentioned wrapping. Lowering of a made-up well into a casing is shown in the photographs of figure 3. The space around the porous concrete pipe through the pervious stratum was backfilled with clean sand to the bottom of the impervious layer. The zone between the bottom of the top stratum and a depth 2 to 3 ft below the T-section was filled with a tamped sand-bentonite mixture. The remainder of the backfill was concrete. After installation, each well was pumped or surged for approximately 20 minutes to insure that the well was operating properly.

17. Operation of wells. Because of the large amount of silt and industrial waste carried in Cypress Creek and Leath Bayou and the low elevation of the well outlets, it was believed necessary to prevent the reservoir water from entering and fouling the wells. The first method of accomplishing this was with screw plugs in the discharge pipes which were removed during high-water season. Brass flap valves, designed by the Experiment Station, were used to replace the plugs to insure that the wells would operate as required. Regular inspection and maintenance are necessary to keep the flap valves in operating condition. Details of

the flap valve are shown on plate 1. Well discharge measurements are made with a calibrated container and stop watch; the vertical tie rod precludes the use of a velocity meter in the riser pipe.

Piezometer system

18. Location of piezometers. The 18 piezometers are located on three principal ranges. One is along the line of wells, with the piezometers midway between the wells. The other two ranges, A and B, are perpendicular to the center line of the levee and are used to measure the pressure gradient in the pervious stratum landward from the levee. These lines cross the levee at about the center of the line of wells on each side of the pumping station. The letter A or B in the piezometer number designates the piezometers on line A or B. Multiple letters, AA or BB, indicate that the piezometer is beneath another piezometer in the pervious stratum. The locations of the piezometers are shown on plate 1 and are given in table 4. The soil profiles along the piezometer lines and along other sections are shown on plates 2, 3, and 4.

19. Two sets of piezometers were installed to measure the hydrostatic pressure at various depths in the pervious stratum. These are on lines A and B at the line of wells. Piezometers 4A and 5AA are near the top and bottom of the pervious stratum, respectively, between wells 13 and 14. Piezometers 13B, 14BB, and 15BBB are between wells 39 and 40. Piezometers 14BB and 15BBB are below a clay lens in the pervious stratum.

20. Description of piezometers. The piezometers consist of a 3.2-ft length of 1-1/4-in. metal well point connected to a vertical riser pipe. The center of the screen is indicated on all sheets. In order that the tops of all piezometers will be above the maximum operating pool of

the reservoir and, in addition, that readings may be made when the reservoir is pumped down, the riser pipes were brought up 5 to 7 ft above the ground surface and offsets made. An additional piece of vertical riser pipe then extends up to above elev 226. When the reservoir is down, readings may be made at the offset in the main riser pipe. When the reservoir is full, readings may be taken in the offset riser pipe from a boat. Piezometer 16B has two offset riser pipes. Readings are made with a float attached to a tape. A sand-bentonite seal was placed around the riser pipe through the impervious top layer.

Nonconnah Creek Relief Well Installation

Soil conditions

21. The locations of the relief wells and piezometers at Nonconnah Creek are shown on plate 9. The locations of the soil profiles shown on plates 10 and 11 are also indicated. The soil profile shown on plate 10 indicates that west of Florida Street the foundation consists of a surface stratum 2 to 15 ft thick of relatively impervious clay silt underlain by a pervious stratum 30 to 50 ft thick on top of Tertiary clay. Mallory drainage ditch, running parallel to the landside toe of the levee at a distance of roughly 200 ft, is 7 to 10 ft deep, cutting completely through the impervious layer near wells 20, 21, 22, and 23, and reducing it to less than 7 ft in thickness for the remainder of the section. The upper 2 to 10 ft of the pervious stratum consists, in general, of sandy silts and silty sands. The lower 20 to 45 ft is medium to coarse sands with some pockets of gravel and of clay. The over-all permeability of the pervious stratum was estimated to be 500×10^{-4} cm per sec for the design

of the relief wells. Permeabilities as computed by the approximate formula, $k = (D_{10})^2$, k in cm per sec, D_{10} the effective size of soil sample in mm, are shown on the soil profiles, plates 10 and 11, and in table 2. This method of obtaining the permeability was used as there were in general no laboratory permeability data available at this site.

22. East of Florida Street, the soil profile of the upper portion of the foundation is very erratic. In some places the impervious stratum is non-existent, but at other points it is 15 ft thick. The sandy silt and silty sand portion of the pervious stratum varies similarly up to 12 ft in thickness. At well 9, the entire foundation to Tertiary clay consists of medium coarse sand. In general, the pervious stratum is 40 to 50 ft thick.

23. Between Florida Street and Highway 61, a storage reservoir for the runoff water behind the levee has been constructed. The excavation for this reservoir removed the impervious top stratum from most of this area and left only a very thin layer in the remainder. A paved ditch crossing the reservoir bottom along the edge adjacent to the levee has a gravel blanket beneath the concrete. This gravel blanket rests on the sand stratum. Weep holes on 10-ft centers penetrate the concrete paving to relieve uplift pressures beneath the paving.

24. With the wide variation in the thickness of the impervious top stratum, the entrance point of the underseepage is difficult to estimate. It may be in the borrow pits, the edge of which is about 200 ft from the levee, or at Nonconnah Creek, 500 to 1200 ft distant.

Operation of storage reservoir

25. The reservoir serves the Nonconnah Creek Pumping Station by

storing the runoff water behind the levee. The pumping station is designed for a maximum pool elevation of 213, corresponding to a river stage of 29 ft on the Memphis gage, and a minimum pool of 203. At the maximum pool stage the depth of water in the reservoir is 10 ft, and the wells west of Florida Street are submerged as much as 3 ft. The reservoir must be pumped down to elev 210 before all well discharge measurements can be made. The design net grade of the levee at this site is elev 239.

Relief wells

26. Description. The wells at Nonconnah Creek are of the same type as at Wolf River (see paragraphs 9 and 10). However, some of the wells at Nonconnah Creek have an outfall discharge, and others discharge from a concrete dome. In the outfall type, the T-section is set 18 to 30 in. below the ground surface, and an outfall pipe 5 to 10 ft long carries the water to a collection ditch. (A typical well installation discharging into a shallow collection ditch is shown in the lower photographs of figure 3.) In the dome type, the T-section is near the ground surface, and a small dome of concrete is poured around it. The water is discharged onto the ground surface through a short outfall pipe about 18 in. long. Details of the wells are shown on plate 9.

27. Design of well system. The design of the wells between Highway 61 and Florida Street was made in a manner similar to that followed in paragraph 12. The assumptions and final design are as follows:

Assumptions:

Completely impervious landside top stratum	
Tailwater at well outlets	
Thickness of sand-gravel stratum, D	30 ft
Distance to source of seepage, d	1000 ft
Net head for computing discharge, h	23 ft

Radius of drainage well, r_w	0.25 ft
Permeability of foundation, K	500×10^{-4} cm/sec = 0.10 ft/min
Well penetration into pervious stratum	50%
Maximum allowable net head between wells, P	5 ft

Final design:

Well spacing	100 ft
Estimated quantity	45 gpm per well
Maximum head between wells	4.0 ft
Top of screen, elev	190 ft
Bottom of screen, elev	175 ft
Number of wells	14

28. These wells are 5 ft from a concrete collection ditch running along the top of the bank of the storage reservoir. The invert of the discharge pipe rests on the edge of the half-circle concrete ditch. Local considerations at the time of installation required the placing of the outlets at this elevation. Well 6 has been grout-filled and abandoned.

29. The final design of the 14 wells west of Florida Street and the 4 wells east of Highway 61 is as follows:

Well spacing	200 ft
Penetration (approximate)	100%
Estimated quantity	55 gpm per well
Maximum head between wells	6.5 ft
Top of screen, elev	185 ft
Bottom of screen, elev	170 ft
Total number of wells	14 + 4 = 18

30. The 14 wells west of Florida Street discharge directly into the Mallory drainage ditch, and are located on a line approximately 10 ft from the top of the ditch bank toward the levee. Wells 1 to 4 east of Highway 61 are 3 ft from the landside levee toe and discharge onto the ground. Wells 1 and 2 have been grout-filled and abandoned. Locations and elevations of all wells are shown on plate 9 and are given in table 5.

31. Method of installation. The method of installation was the same as outlined in paragraph 16.

32. Operation of wells. The remarks in paragraph 17 under Wolf River are applicable also to the Nonconnah Creek well system.

33. Gravel-filled wells. The original relief wells at Nonconnah Creek were installed September to December 1939. These wells consisted of a 12-gauge steel welded pipe, 15 in. in diameter, filled with sand and gravel. They are on 50-ft centers 3 ft from the landside toe of the levee. The pipes, coated with an asphalt varnish, penetrate 20 ft into the sand substratum. The bottom 20 ft of pipe is perforated with 36 - 3/16-in. holes per foot of pipe. In the Experiment Station report "Review of Design of Drainage Well System, Nonconnah Creek Levees, Memphis Flood Control System", dated 25 June 1942, the average permeability of the sand and gravel in the wells was taken as 5 cm per sec, based on design values, but doubt was expressed that it would be greater than 1 cm per sec. The review showed that the hydraulic gradient through the top stratum would be 0.6 to 2.5 with the design permeability and would be higher with the permeability of the gravel filling reduced to 1 cm per sec. The review further showed that a dangerous condition could result at some locations when the river stage was above 230 ft. In addition, experimental tests showed that this type of well is subject to clogging, which would also increase the hydraulic gradient through the top stratum. The review recommended the installation of the new well system.

Piezometer system

34. Location of piezometers. The 19 piezometers are located on five ranges, one along the line of wells, one along Florida Street, and one along Highway 61, one perpendicular to the center line of the levee crossing the levee near the center of the line of wells west of Florida

Street, and one perpendicular to the center line of levee midway between Florida Street and Highway 61. The locations of these piezometer lines are shown on plate 9 and are given in table 6. The soil profiles along these lines are shown on plates 10 and 11.

35. Description of piezometers. The same type of piezometer as used at Wolf River was also used here. No offsets were necessary in the riser pipe in order to have the top of the piezometer above the maximum reservoir pool and still have the top of the piezometer accessible from the ground.

TABLE 1

PERMEABILITY DATA -- WOLF RIVER

Boring Number	Elevation Ft - mGl	Permeability 10^{-4} cm/sec	Avg Permeability of Boring 10^{-4} cm/sec
1	195	360	
	186	1060	1410
	173	2800	
2	204	140	
	194	2050	1070
	172	1030	
3	190	1470	
	170	920	1190
4	193	820	
	186	560	890
	173	1300	
12	188	320	
	179	840	940
	163	1670	
16	192	280	
	179	810	
	174	45	590
	168	1200	
18	185	660	
	175	830	600
	168	410	
19	183	610	610
20	197	275	
	185	750	510
P-17	201	300	
	201	440	450
	178	860*	
P-18	202	400	
	202	530*	
	189	840*	910
	174	2900*	
	174	2080	
	Average	850	830

Results from permeability tests adjusted to void ratio of 0.50.

* Permeability calculated from approximate formula, $k = (D_{10})^2$, k in cm/sec, D_{10} in mm.

TABLE 2

PERMEABILITY DATA -- NONCONNAH CREEK

Boring Number	Elevation Ft - mGl	D_{10} in mm	Permeability 10^{-4} cm/sec	Avg Permeability of Boring 10^{-4} cm/sec
N-14	187	0.45	2020	
	165	0.29	840	1040
	162	0.16	260	
N-16	194	0.37	1370	
	184	0.36	1300	
	184	-	790*	870
	170	-	10*	
N-17	187	0.26	680	680
		Average	910	860

Approximate formula $k = (D_{10})^2$, k in cm/sec, D_{10} in mm, used to calculate k.

* Permeability test results adjusted to void ratio of 0.50.

TABLE 3

RELIEF WELL INSTALLATION — WOLF RIVER

Well	Levee Station	Distance from Center Line Levee (ft)	Elevation Center of Outlet mGl
1	4/47+20	200	214.0
2	4/47+85	212	214.0
3	4/48+65	220	213.4
4	4/49+38	238	213.6
5	4/50+15	248	213.8
6	4/50+85	272	213.7
7	4/51+62	280	213.2
8	5/0 +39	295	213.3
9	5/1 +12	305	213.1
10	5/1 +88	312	213.4
11	5/2 +62	315	213.5
12	5/3 +40	322	214.0
13	5/4 +10	338	213.9
14	5/4 +80	348	214.2
15	5/5 +60	370	213.6
16	5/6 +30	375	213.4
17	5/7 +10	385	213.5
18	5/7 +80	387	213.3
19	5/8 +58	390	213.7
20	5/9 +42	390	214.0
21	5/10+10	390	214.2
22	5/10+90	380	213.6
23	5/11+60	368	213.0
24	5/12+38	335	212.7
25	5/12+90	290	213.2
26	5/19+40	200	212.8
27	5/20+10	232	212.0
28	5/20+80	275	212.1
29	5/21+30	310	212.7
30	5/21+95	350	213.2
31	5/22+60	393	213.0
32	5/23+20	438	213.2
33	5/23+48	470*	212.4
34	5/23+48	500*	213.1
35	5/23+48	510*	213.4
36	5/23+85	522	213.4
37	5/24+60	548	213.0
38	5/25+25	550	213.0
39	5/26+00	582	213.2
40	5/26+62	600	212.4
41	5/30+40	528	212.8

(Concluded on next page)

* These distances are measured radially from the station given which is a P. I. of the center line of levee.

TABLE 3 (Cont'd)

Well	Levee Station	Distance from Center Line Levee (ft)	Elevation Center of Outlet mGl
42	5/31+08	500	213.2
43	5/31+88	490	212.9
44	5/32+58	475	213.3
45	5/33+32	455	213.3
46	5/34+08	438	213.4
47	5/34+78	418	214.0
48	5/35+18	405*	212.9
49	5/35+25	405	214.0
50	5/36+00	410	213.6
51	5/36+72	420	213.5
52	5/37+50	428	214.1
53	5/38+22	438	214.1
54	5/38+98	448	214.4
55	5/39+60	470	214.5
56	5/40+08*	488	214.4
57	5/40+00*	520	214.1
58	5/40+70	565	Well abandoned
59	5/41+42	600	213.6
60	5/42+22	660	213.7
61	5/43+08	700	Well abandoned
62	5/43+90	740	213.0
63	5/44+80	790	Well abandoned
64	5/45+85	835	Well abandoned
65	5/46+69	900*	213.6
66	5/46+69	985*	212.8

* These distances are measured radially from the station given which is a P.I. of the center line of levee.

TABLE 4

PIEZOMETER INSTALLATION -- WOLF RIVER

[redacted] Line	Piez	Levee Station	Distance from Center Line Levee (ft)	Elev Center of Ocreen mGl	Elev of Offset mGl	Elev of Top of Piez mGl
	1	4/50+55	260	187.90	220.50	226.60
A	2A	5/4 +50	15*	200.32	None	247.66
	3A	"	200	199.75	220.88	226.60
	4A	"	350	199.83	221.10	226.85
	5AA	"	350	172.35	221.11	226.87
	6A	"	670	200.00	225.22	226.99
	7	5/10+50	390	200.07	222.05	227.14
	8	5/14+80	260	197.06	221.40	227.04
	9	5/19+75	220	197.03	220.96	226.75
	10	5/23+48	500	194.61	220.94	226.40
B	11B	5/29+42	15*	192.99	None	246.99
	12B	"	270	199.75	222.22	226.68
	13B	"	550	201.36	222.64	226.75
	14BB	"	550	184.77	222.66	226.76
	15BBB	"	550	164.79	222.67	226.71
	16B	"	750	200.84	216.64 222.85**	227.22
	17	5/36+90	420 L.S.	199.70	223.08	226.78
	18	5/42+65	680 L.S.	187.64	224.48	226.82

* Riverside of levee center line. Other distances are measured landside.

** Elevation of second offset.

TABLE 5

RELIEF WELL INSTALLATION -- NONCONNAH CREEK

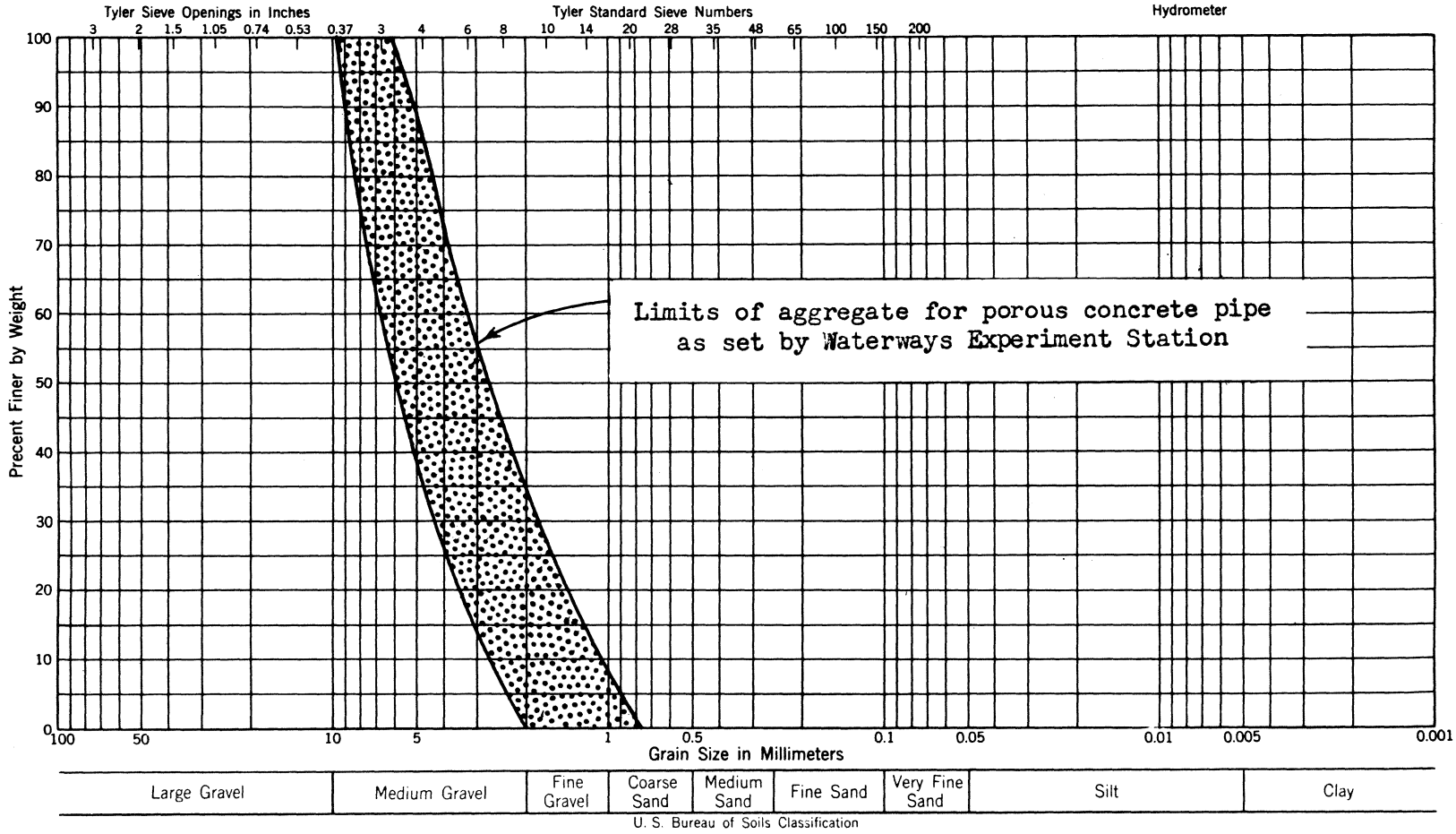
Well	Levee Station	Distance from Center Line Levee (ft)	Elev Center of Outlet mGl
1	2/15+35	140	Well abandoned
2	2/13+35	140	Well abandoned
3	2/11+36	140	219.6
4	2/9 +37	140	218.4
5	2/6 +37	326	215.6
6	2/5 +37	326	Well abandoned
7	2/4 +07	312	216.3
8	2/3 +07	310	216.1
9	2/2 +07	310	216.5
10	2/1 +07	308	216.5
11	2/0 +07	306	216.6
12	1/52+11	306	216.7
13	1/50+75	310	216.8
14	1/49+75	305	216.2
15	1/48+75	300	215.9
16	1/47+75	295	215.7
17	1/46+75	272	215.6
18	1/46+14	240	216.8
19	1/44+60	238	213.8
20	1/42+60	238	211.0
21	1/40+60	238	211.6
22	1/38+60	238	210.2
23	1/36+60	238	210.6
24	1/34+60	238	210.6
25	1/32+60	238	211.4
26	1/30+60	238	212.0
27	1/28+60	238	213.2
28	1/26+60	238	214.0
29	1/24+60	238	212.3
30	1/22+60	238	212.0
31	1/20+60	248	212.2
32	1/17+95	270	211.6

TABLE 6

PIEZOMETER INSTALLATION -- NONCONNAH CREEK

Line	Piez	Levee Station	Distance from Center Line Levee (ft)	Elev Center of Screen mGl	Elev of Top of Piez mGl
A	1	2/5 +40	60*	189.8	229.6
	2	2/5 +40	160	187.5	219.3
	3	2/5 +84	316	188.2	220.0
	4	2/6 +70	676	188.9	221.4
	5	2/5 +96	1215	194.5	223.0
B	7	1/46+85	105*	185.0	227.8
	8	1/46+14	205	186.8	220.6
	9	1/46+14	508	187.0	220.0
C	10	1/33+78	68*	184.2	227.0
	11	1/33+25	228	184.4	217.5
	12	1/34+35	450	184.2	216.2
	17	1/33+78	240	154.86	214.36
	17X	1/33+78	240	186.70	214.40
D	6	1/52+85	295	188.8	220.1
	13	1/52+75	15*	188.97	242.57
	14	1/52+75	305	169.95	221.05
E	16	1/43+80	238	189.99	216.49
	18	1/23+80	245	185.09	216.39

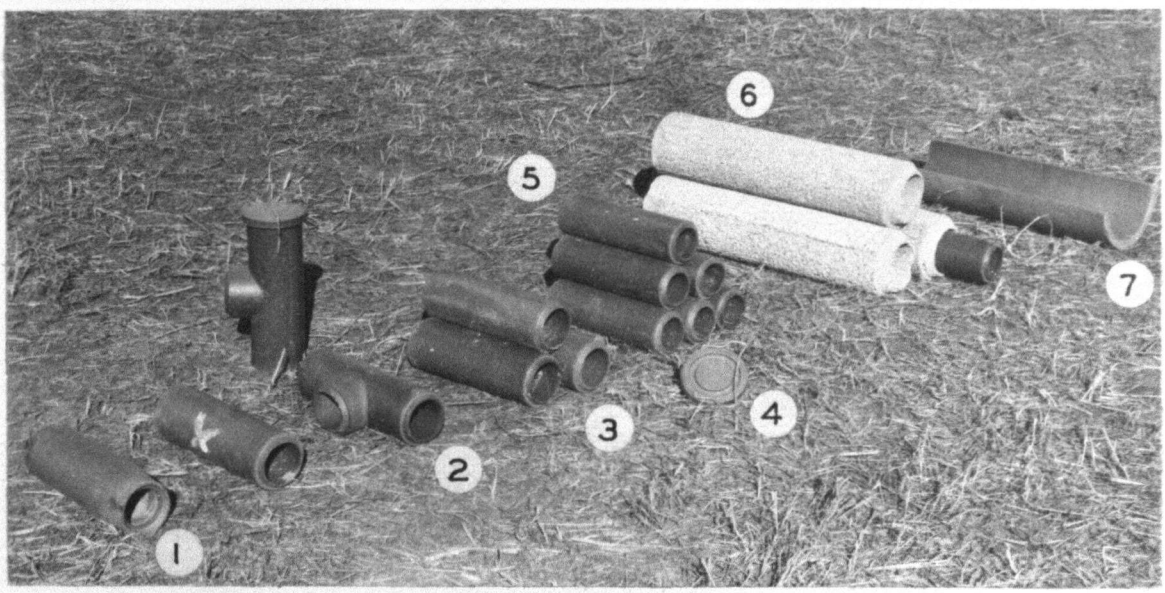
* Riverside of levee center line. Other distances measured landside.



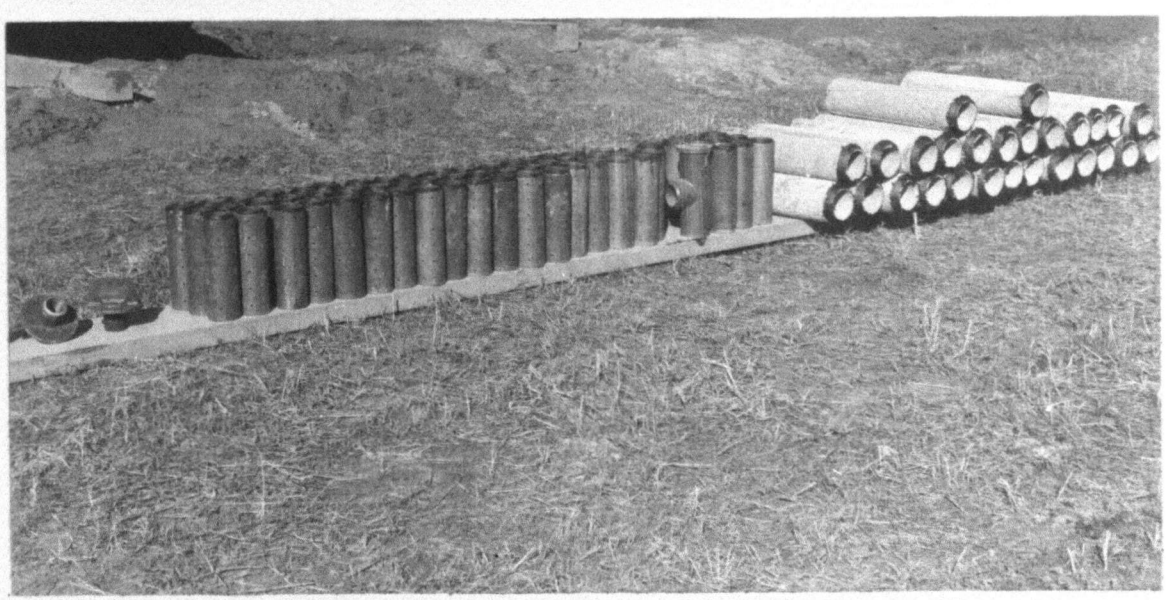
Wolf River and Nonconnah Creek Relief Well Installation

Aggregate for Porous Concrete Pipe Manufactured by
Walker Cement Products Company

Figure 1

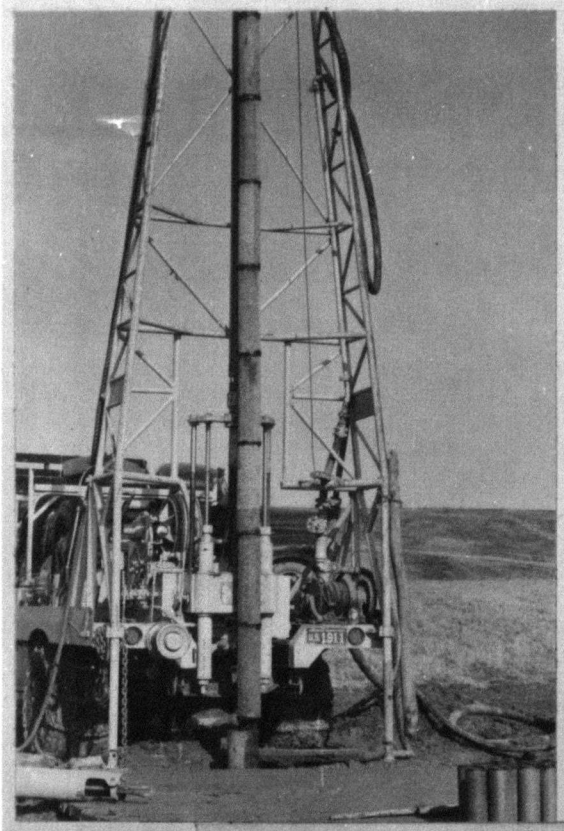


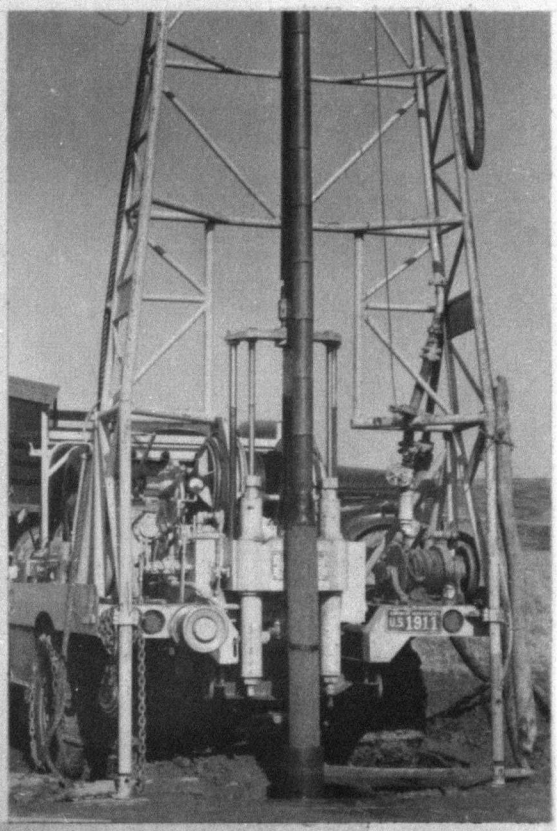
1 Reducer
2 Tee
3 Riser Pipe
4 Cap
5 Perforated Clay Pipe
6 Porous Walker Concrete Pipe
7 Discharge Apron



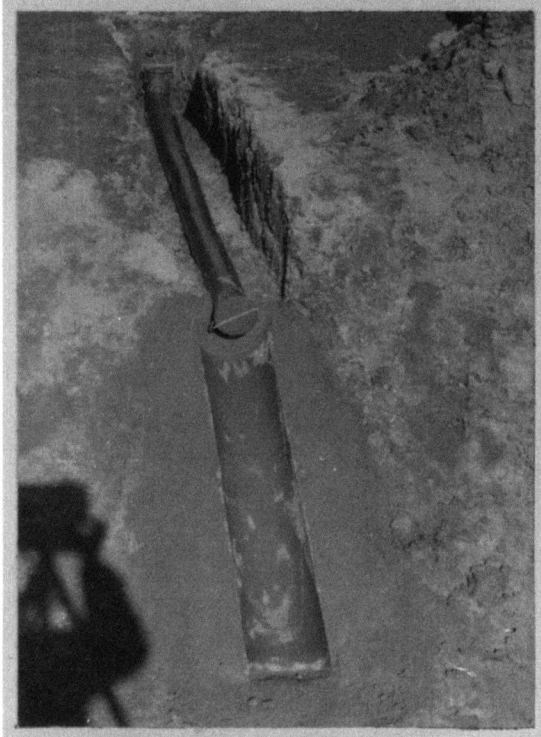
WELL CONSTRUCTION MATERIALS

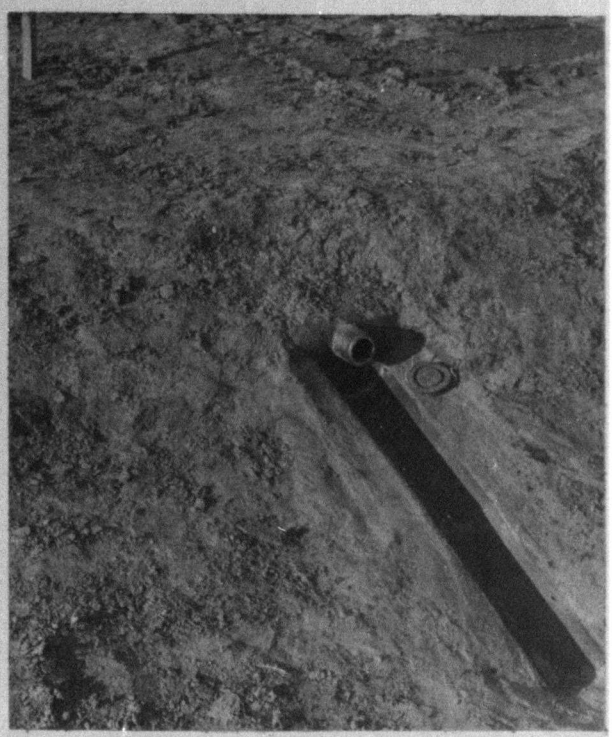
Figure 2





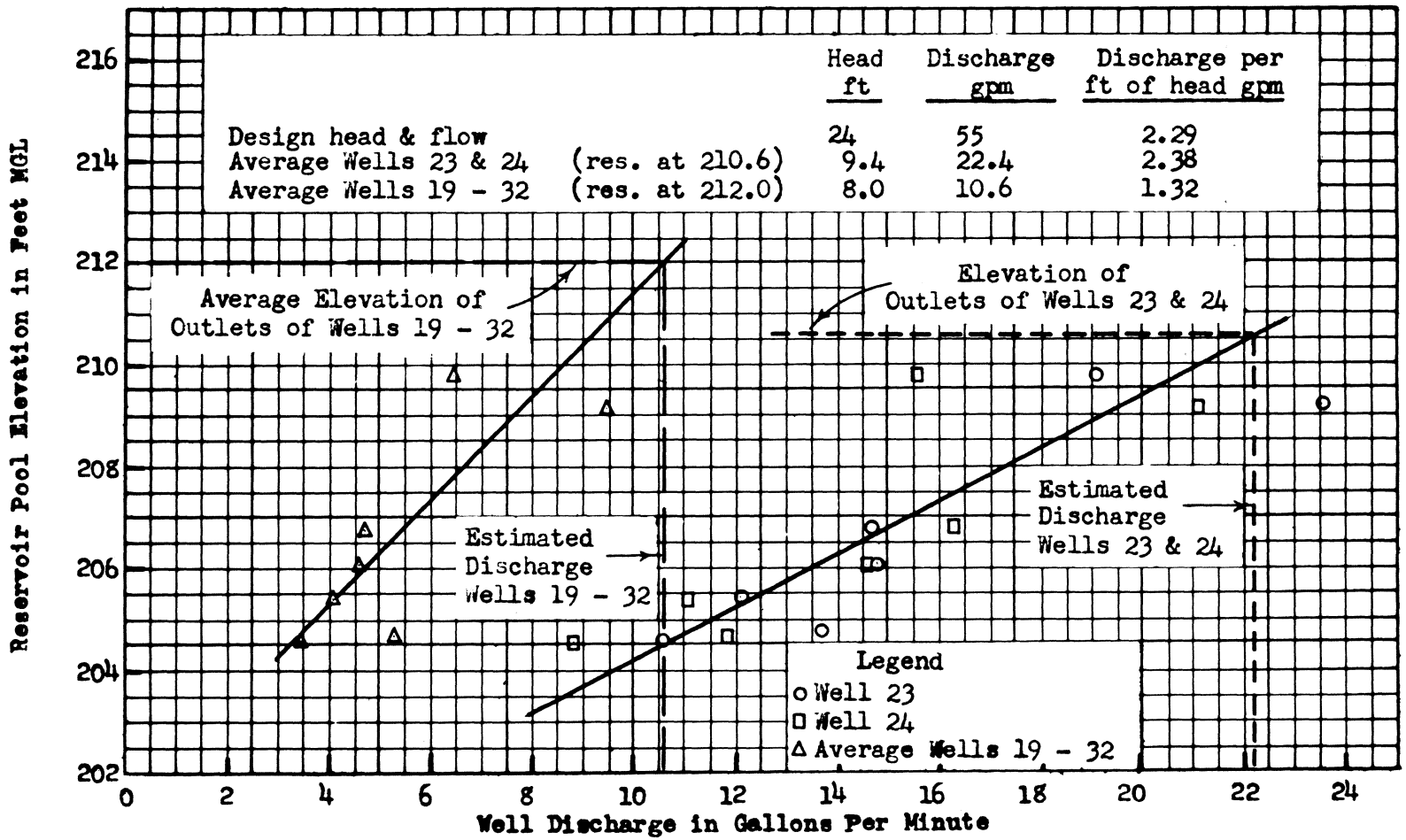
Lowering Screen and Riser Section Into Casing





Outfall Discharge Pipe

Figure 3



Nonconnah Creek Relief Well Installation

Reservoir Pool Elevation vs. Well Discharge

River Stage 220 ft MGL

Figure 4

APPENDIX A

ANALYSIS OF WELL AND PIEZOMETER DATA -- 1945 TO 1947

WOLF RIVER AND NONCONNAH CREEK

Introduction

1. The purpose of this appendix is to present and analyze the data which have been obtained to date from the piezometers and relief wells installed along the Wolf River and Nonconnah Creek levees. A separate sheet presents the data for each year except as noted in paragraph 16. Thus there are, for example, four sheets of plate 12, one each for the years 1943, 1944, 1945 and 1947. Since the installation of the piezometers and relief wells at these two sites, there has been no major flood. The maximum river stage recorded at Wolf River when the well discharge measurements were made (1944) was 221.5 ft mGl. This is 20 ft below the net levee grade. The highest stage recorded at Nonconnah Creek Pumping Station was about 221 ft mGl in 1943 and 1945; this stage was almost 16 ft below the design flood. Therefore, the performance of the well systems at maximum flood is not known at this time (1947). However, the data available indicate the relative distribution of substratum pressures and flow, and also the approximate distance to the effective entrance point of seepage into the pervious foundation. The data available also show the correlation between piezometer readings, river stage, and the elevation of the water in the landside reservoirs.

2. In analyzing the well systems at these two sites it is necessary to have an understanding of the effect of landside seepage and tailwater elevation on the operation of the wells. Three possible conditions of

landside seepage and tailwater which affect the operation of relief wells are discussed below.

a. The first conditions, the assumption on which the design of these well systems was based, is that the flow from the wells is purely artesian, and that the landside top stratum is completely impervious. For this condition the elevation of the well outlets is the effective tailwater for the wells, and the well flow is directly proportional to the net head.

b. The second condition is one in which the landside top stratum is slightly pervious and the tailwater for the seepage through the landside top stratum is at or above the elevation of the well outlets. As long as the flow through the landside top stratum is laminar, the total flow will be divided in a constant proportion between the wells and the landside blanket. This proportion of flow will be maintained for all heads on the levee; therefore the well flow will be directly proportional to the net head. For the same head the total flow in the second condition will be greater than in the first, but the well flow will be less. The relation between the flow from the wells and flow through the landside blanket will be determined by the characteristics of the landside blanket.

c. The third condition is one where the landside top stratum is also slightly pervious but the tailwater over the landside top stratum is below the outlets of the wells. For this condition the proportion of flow from the wells is reduced below that of the second condition as the pressure in the pervious stratum at the wells is further reduced by the additional seepage through the landside blanket as a result of the lower tailwater. For this case well flow is no longer proportional to the head on the levee, as the elevation of the tailwater in relation to the elevation of the well outlets directly affects the well flows. The effect of the elevation of the tailwater on the well flow was noted in a study* of the relief well system at Sardis Dam.

3. The first condition seldom, if ever, occurs in nature as there will nearly always be some leakage through the landside top stratum. Either the second or third condition, depending on the elevation of the

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* WES Technical Memorandum 195-1, "Field and Laboratory Investigation of Design Criteria for Drainage Wells", 1 October 1942. Part VI.

reservoir pool, can exist at Wolf River or Nonconnah Creek. Varying degrees of imperviousness of the top stratum are found at both sites. At Wolf River east of Bellevue Boulevard and near the pumping station, the excavation for the channel of Cypress Creek removed all or nearly all of the top stratum thus materially reducing the resistance to flow through the top stratum. West of Bellevue Boulevard the top stratum is relatively intact, but some seepage may go through it. At Nonconnah Creek, the excavation for the storage reservoir has provided a relatively free exit for the seepage by removing nearly all of the top stratum over a large area. Although the top stratum west of Florida Street is relatively intact, some seepage may also go through it. In addition, the reservoir excavation may affect the well flows west of Florida Street. Whether or not the second or third condition is applicable at these sites depends on the elevation of the reservoir pools. At maximum pool elevation at Wolf River all well outlets will be submerged, and condition two will prevail. At Nonconnah Creek the wells west of Florida Street will be submerged at the maximum pool, and the elevation of the well outlets between Florida Street and Highway 61 will be approached as the reservoir reaches its maximum elevation. Thus at the maximum pool at Nonconnah Creek condition two will be fulfilled or closely approached. However, at lower reservoir stages at both sites condition three will exist as the reservoirs may be several feet below the well outlets. Therefore, for the same foundation conditions as assumed in design, actual well flows would be expected to be less than the design flows particularly from wells in the reaches where an area of relatively free exit for seepage exists, and when the reservoir pool is below the well outlets.

4. It is pointed out that although the landside top strata along portions of the well systems at Wolf River and Nonconnah Creek are relatively pervious and permit a certain amount of landside seepage with resulting pressure reductions, especially when the landside tailwater is below the well outlets, recent model studies have shown that the wells will intercept a considerable portion of the underseepage, and further, will reduce the substratum pressures when the landside tailwater is at or above the well outlets.

Wolf River

5. Since the installation of the relief wells at Wolf River, the flow from the wells has been measured only once. This was in April 1944 when Wolf River was at an elevation of 221.5 ft mGl, and the reservoir at elev 209.1. The discharge observations made have been plotted on plate 5-1944. The estimated design well flow for wells 1 to 25, reduced to the river stage that occurred, is approximately 18 gpm; but the average measured flow was only 3.6 gpm with a maximum individual well discharge of 15 gpm at well 7. For the wells east of Bellevue Boulevard, wells 26 to 66, the design well flow reduced to the actual river stage would have been about 22 gpm; however, the average measured flow was only 1.0 gpm with a maximum individual well flow of 7.5 gpm at well 31. The reservoir pool at this time was about 4 ft below the well outlets (condition three as outlined in paragraph 2). It may be noted that the average well flow for the wells east of Bellevue Boulevard is lower than that for the wells west of Bellevue Boulevard even though the computed flow on the basis of condition one was higher. This shows that a much easier exit for the seepage is provided by

the excavation for Cypress Creek than exists through the top stratum west of Bellevue Boulevard. The low flow at both reaches may be attributed to a large extent to the low reservoir pool. This low tailwater over the landside seepage exit reduces the hydrostatic pressures in the pervious substratum at the line of wells, thereby reducing the well flow as computed for an impervious top stratum. If the reservoir pool had been at the same elevation as the well outlets, the average well flows would have increased appreciably as the pressures at the wells in the pervious substratum would have been increased. However, the average flow for the wells west of Bellevue Boulevard will probably remain higher than for those east of Bellevue Boulevard due to the large seepage flow into the bottom of Cypress Creek east of Bellevue Boulevard.

6. No piezometers were installed until 1947 at Wolf River, and no readings are available for which a differential in head between the river and reservoir existed. Readings were made in 1947, but at that time the elevation of the reservoir pool coincided with the river stage. Therefore, the readings reflect only the lag of the pressure in the substratum compared to the river and reservoir elevations, rather than hydraulic gradients due to flow in the pervious stratum from the riverside to the landside of the levee. Thus, the effective source of seepage could not be estimated. However, the piezometer data for 1947 were plotted on all plates in order to illustrate the use of the plates (see plates 5 to 8 for 1947). The tabulated data are found on plate 18 along with the data for Nonconnah Creek.

Nonconnah Creek

7. Well discharge measurements were made at Nonconnah Creek in 1943,

1944 and 1945, and the data obtained have been plotted on plates 12 and 13 for each year. Wells 5 to 18, which are along the edge of the reservoir, did not flow at the river stages reached. These wells were designed to reduce the hydrostatic pressures in the substratum at maximum river stages at which time the reservoir pool will probably be operated at or near its upper design elevation so as to reduce the pumping head at the pumping station. The reservoir pool was about 10 ft below the well outlets at the times of observation. There was sufficient pressure reduction by the seepage into the bottom of the reservoir as a result of its perviousness and low tailwater to prevent the wells from flowing. The piezometric head at the line of wells was below the well outlets. Had the water in the reservoir been allowed to rise to near the elevation of the well outlets, the excess pressure in the substratum would have been sufficient to cause flow from the wells.

8. The only wells flowing were wells 19 to 32, which are west of Florida Street. An attempt was made to make a more complete comparison here between actual well discharges and the design discharge than at Wolf River, or the other wells at Nonconnah Creek, as more data were available. A study of the data on plate 18 for all years indicated that the flow varied with the elevation of the reservoir pool if the river stage remained constant. Using the flow data from wells 23 and 24, which produced the maximum flows, and the average discharge of wells 19 to 32, a graph of well discharge against reservoir pool was plotted for a river stage of approximately 220 ft (see figure 4). The flows which would have occurred with the reservoir pool at the same elevation as the well outlets (condition two) were extrapolated and then converted to flow per foot of head.

The flows per foot of head are given in the following tabulation:

	Flow per Ft of Head
Design flow	2.29 gpm
Wells 23 and 24	2.38 gpm
Average, wells 19-32	1.32 gpm

If the design assumptions of permeability and distance to source of seepage are about equal to the in situ values, the above table indicates that the seepage through the landside top stratum was sufficient to reduce the average well flow to slightly more than half of the design quantity.

9. Attention is called to the apparent effect of the elevation of the well outlet on the well discharge. This may be seen on plate 12 for 1943, 1944 and 1945, on which the well flows are plotted. The highest discharge rates were from wells 23 and 24, which have the lowest outlet elevations of all wells excepting well 22. No explanation is available for the lower discharge of well 22.

10. The piezometers were installed to measure the hydrostatic pressure in the pervious substratum between wells and pressure gradients in the pervious substratum perpendicular to the levee. From the piezometer readings and river stages plotted on plates 13 to 16, the effective source of seepage at each piezometer line was estimated by extending the piezometric gradient at the levee up to the point of intersection with the river stage (see construction on plate 16, 1943). The results are shown on each plate and in table 7. In addition, the distance to the center of Nonconnah Creek at each line has been included in table 7. It is seen that, with a few exceptions, the results are fairly consistent at each piezometer line and that the average distance to the estimated source of seepage is

approximately equal to the distance to Nonconnah Creek. This indicates that the effective source of seepage is probably the bed of Nonconnah Creek and not in the borrow pits between the river and the levee.

11. The slope of the pressure profiles, shown in plate 12 for each year, toward the reservoir area indicate a definite flow of seepage to the reservoir area. The readings at piezometers 4 and 5 also illustrate this point. These piezometers are well landside of the levee but are offset east of the reservoir from Line A about 130 ft onto the toe of the embankment of Highway 61. The measurements show higher pressures in the pervious substratum at their locations than at piezometer 3, which is at the levee side of the reservoir on Line A. Flow of substratum seepage into the reservoir was expected, as almost all of the relatively impervious top stratum was removed in the excavation for the reservoir.

12. Of interest are the readings of piezometer 14, which are higher than the readings of piezometer 6, as given on plate 18 for 1947. The two piezometers are located only 15 ft apart, but the center of the screen of piezometer 14 is at an elevation 19 ft lower than that of piezometer 6. The data tabulated on plate 18 for 1947 show that the difference ranges up to 1.4 ft. The maximum value occurred after the reservoir had been pumped down. This difference in the readings of piezometers 14 and 6 clearly indicates a vertical flow of seepage from the deep, more pervious soils up into the bottom of the storage reservoir.

13. Similarly, a comparison of the readings of piezometers 17 and 17-X shows a pressure of up to 1.4 ft higher near the bottom of the pervious stratum (piezometer 17) than in the upper third of the pervious stratum (piezometer 17-X); see plate 18 for 1947. This indicates an

upward flow of seepage from the deeper sands to the drainage wells.

14. It is apparent from the daily graphs on plate 17 for all four years that the substratum pressures as recorded by piezometers near the reservoir follow very closely the fluctuations in the elevation of the reservoir pool. The pumping down of the reservoir was reflected immediately in the readings of those piezometers adjacent to the reservoir.

15. The effect of closure of the wells may be determined from a study of the data for 1943, 1944 and 1945, when some readings were made with the wells closed. The effect is reflected in the measurements at piezometers 10, 11 and 12, Line C, as these were the only piezometers adjacent to wells from which there would have been discharge. In 1943 the wells were closed until June 5. The pressure recorded at Line C dropped sharply after the wells were opened and then rose again after the wells were closed on June 6 (see plate 17). Another reduction in pressure occurred when the wells were reopened after the readings on June 7. Similar fluctuations occurred with the opening of the wells in 1944 and 1945.

16. The series of piezometer readings made on March 21, 22 and 23 of 1945 further illustrates the pressure relief obtained by opening the wells (see plate 17 and 18 for 1945). The river and reservoir were almost constant during this period. Opening of well 24 only, which is near piezometer 11, reduced the pressure in the substratum as recorded by piezometer 11 by approximately 1 ft (March 22) from the pressure observed when all wells were closed (March 21). Opening of all wells (March 23) gave a further reduction in pressure of about 0.7 ft, making a total reduction of 1.7 ft of substratum pressure with all wells open compared to that with no wells flowing. The amount of pressure reduction would no

doubt be considerably more at higher river stages.

17. The selection of the days for which data were plotted on the pressure profiles was based on the daily graphs of the river stage, reservoir pool, and piezometer readings (see plate 17). The days selected were, in general, during the rising river stage, at the maximum river stage, and at the minimum reservoir pool.

18. Plate 15 was not used for the years 1943, 1944 and 1945, as during those years only piezometer 6 had been installed on Line D. It was felt that the inclusion of those sheets would not furnish additional information, as no pressure gradients could be plotted. The readings at piezometer 6 are plotted on plate 12 for each year, so no data have been omitted. The additional piezometers at Line D were installed before the measurements were made in 1947, so plate 15 was included in the 1947 series.

19. In plotting plate 17, the data for 1943 and 1944 were put on one sheet. Data for 1945 and 1947 were also placed on one sheet. Similar combinations of years were made for tabulating the data on plate 18.

20. Attention is called to the operation of wells on 17 March 1945 after piezometer readings were made. Discharge measurements were made starting at well 19 and continuing consecutively. As the measurement at each well was completed, the well was closed. The reason for this scheme of well measurement is not known.

Conclusions

21. Due to the small amount of data and comparatively low flood stages, only limited and tentative conclusions may be drawn.

a. Although the data available are rather limited, it appears that the relief well system at Nonconnah Creek is effective in reducing the substratum pressures landward from the levee when the substratum pressures are above the outlets of the wells. The effectiveness of the wells will probably be more pronounced at higher river stages and storage reservoir pool elevations.

b. Where a relatively free exit for underseepage exists besides that afforded by the relief wells, and when the reservoir pool is below the outlets of the wells, the actual flow is appreciably less than the design flow. For the areas where the seepage through the landside top stratum is probably less and when the tailwater is at the well outlets, a reasonable check is obtained between design and actual flows when the many additional unknown variables, such as in-situ permeability and seepage entrance, are considered.

c. At a constant river stage, reduction in the reservoir pool elevation below the elevation of the well outlets causes a reduction in well discharge at the stages measured, and conversely an increase in the reservoir pool causes an increase of well flow until the reservoir reaches the elevation of the well outlets.

d. The elevation of the well outlet is an important consideration in the amount of well discharge at the low differential heads measured.

e. At Nonconnah Creek the source of seepage is apparently in the bed of Nonconnah Creek.

f. Flow from the wells along the storage reservoir at Nonconnah Creek will take place under the conditions for which they are designed; namely, high flood stage and high level reservoir pool operation. The low reservoir pools as have occurred were the primary cause for no discharge from these wells.

g. The higher readings of the deep piezometers at Nonconnah Creek as compared to the shallow piezometers indicate an upward flow from the more pervious deep sands to the drainage wells or landside storage reservoir.

TABLE 7

ESTIMATED EFFECTIVE SOURCE OF SEEPAGE
NONCONNAH CREEK RELIEF WELL SYSTEM

Date	Distance from Center line of Levee in ft			
	Line A	Line B	Line C	Line D
6/1/43	750	1200	2000	----
6/5/43	800	1300	1800	----
6/8/43	700	1250	2000	----
4/27/44	550	1200	2200	----
4/28/44	600	1200	1900	----
3/16/45	530	1050	2000	----
3/21/45	---	----	1050	----
4/25/47	400	500	----	700
4/28/47	450	1150	----	1150
5/7/47	400	1600	1800	1600
Average	580	1120	1850	1150
Nonconnah Creek	500	900	1240	900