Hydraulic Design of the Fort Campbell Storm Drainage System

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The Corps of Engineers, U. S. Army, awarded a contract in June 1963 for construction of a storm drainage system to serve a major portion of the cantonment area of Fort Campbell, Kentucky. A total of 31,000 lineal feet of pipe, ranging in size from 12- to 84-in. diameter, was required to drain 2,000 acres of land in the main post area. The cost of this drainage system was approximately $2,000,000.

While the project is unusual, based on pipe quantities and construction costs involved, it is also somewhat unique in that the original estimated cost of $5,500,000 was drastically lowered by the use of temporary ponding to reduce peak discharges in the main trunk sewer. The hydraulic design of the system permits a limited amount of ponding (at the majority of storm drainage inlets) as a result of a 10-yr frequency design storm. This relatively small amount of temporary storage capacity reduced required pipe sizes considerably, but the largest percentage of cost savings was effected by enlarging two major ponding areas in the upstream portion of the project. These two excavated temporary retention basins permitted large reductions in pipe diameters for nearly three miles of trunk sewer.

•FORT CAMPBELL is located on the Kentucky-Tennessee state line about 50 miles northwest of Nashville, Tennessee. The region is characterised by gently rolling terrain having a thick clay overburden underlain by a cavernous limestone formation. The development of solution channels in the underlying limestone, with accompanying erosion of the overburden due to circulating groundwater, has resulted in the formation of a typical Karst topography with saucer-shaped depressions on the ground surface and, in some instances, open sinkholes. The cantonment area comprises about 6,000 acres located in the eastern part of the base adjacent to US 41A. Figure 1 is a map of the main post area of Fort Campbell.

The central and western portions of the cantonment area occupy higher ground, containing fewer sinkholes, than the eastern section. The central portion of the built-up area was constructed on a low ridge which runs generally north and south. Because this higher ground is relatively well drained, with few sinkholes, it was developed before the lower land to the east. The later development of Fort Campbell into a permanent army facility, however, necessitated the expansion of the cantonment to the east. The eastern section had very few drainage lines and contained numerous large sinks. The natural drainage provided by the sinkholes was not satisfactory. Following a heavy rainstorm, water would stand for days or weeks in some sinks, would be readily drained from others, and would remain ponded in others almost indefinitely. Dating from World War II, attempts were made to drain the sinks by constructing vertical drainage wells through the clay overburden into the underlying weathered limestone. These wells were only moderately successful, since only a few would handle the runoff from a storm of normal intensity without ponding, and all presented a continuing maintenance problem to keep them functioning. When postwar expansion of the main

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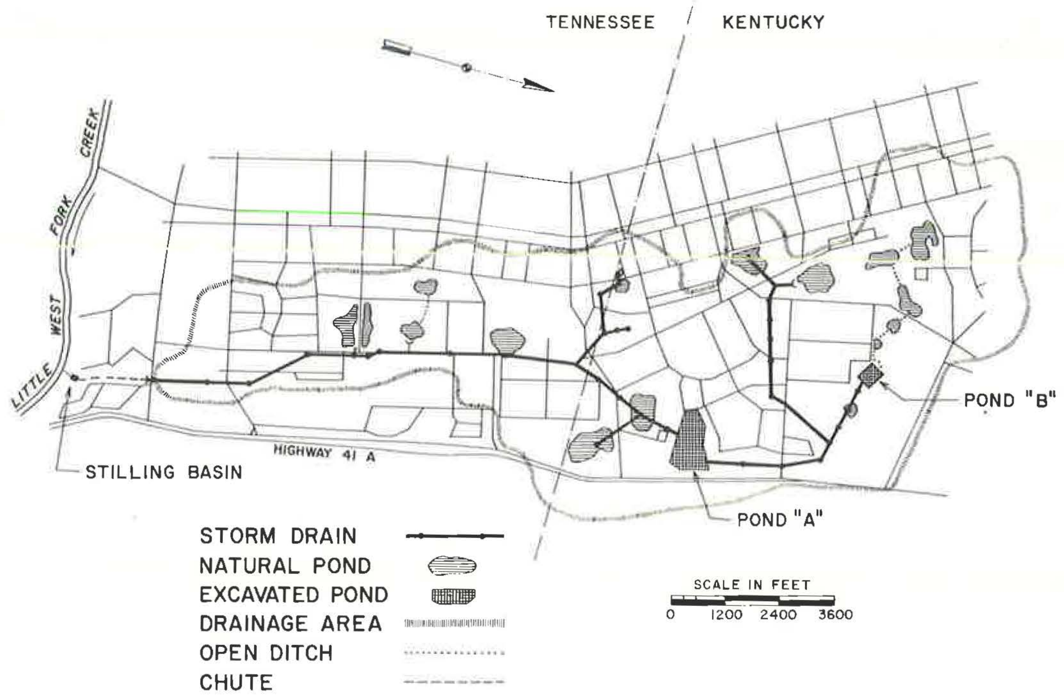

Figure 1. Layout of storm drainage system.

post began to infringe upon the sink areas, it became apparent that a positive drainage system would have to be provided for Fort Campbell.

COMPARISON OF DRAINAGE SCHEMES CONSIDERED

Early in 1955 a drainage study was prepared for the Nashville District (of the Corps of Engineers) by a private consulting engineer firm. This study proposed a system of underground conduits to remove the storm runoff as fast as it was collected. The system involved reinforced concrete box conduits having cross-sectional dimensions as large as 17 by 14 feet and as much as 45 feet below the ground surface at the downstream end of the project. The enormity of such a project is reflected in its estimated cost of $5,500,000.

The Nashville District, in an attempt to devise a satisfactory drainage system that was economically feasible, prepared and submitted a drainage report in 1957 to the Chief of Engineers. Five possible plans were considered in this study. Plan A was essentially a refinement of the original consulting engineer report and provided for immediate removal of storm runoff. The estimated cost of this plan was $4,600,000. Plan B permitted a minor amount of ponding in the existing sink areas which reduced the cost estimate considerably. Plans C and D were alternates to Plans A and B and involved disposing of a major portion of the runoff into an existing large open sinkhole. While both of these plans were probably feasible, the uncertainties involved with underground disposal of storm runoff ruled out their use. Plan E was essentially the same as Plan B except for the use of certain open ditches instead of pipe to reduce costs. The estimated cost of this plan was $3,370,000.

In addition to these five plans, consideration was also given to the use of a storm water pumping station at a strategic location to reduce pipe sizes and to the construction of an unlined drainage tunnel driven through rock in the lower portion of the system. Neither of these schemes proved to be economically feasible.

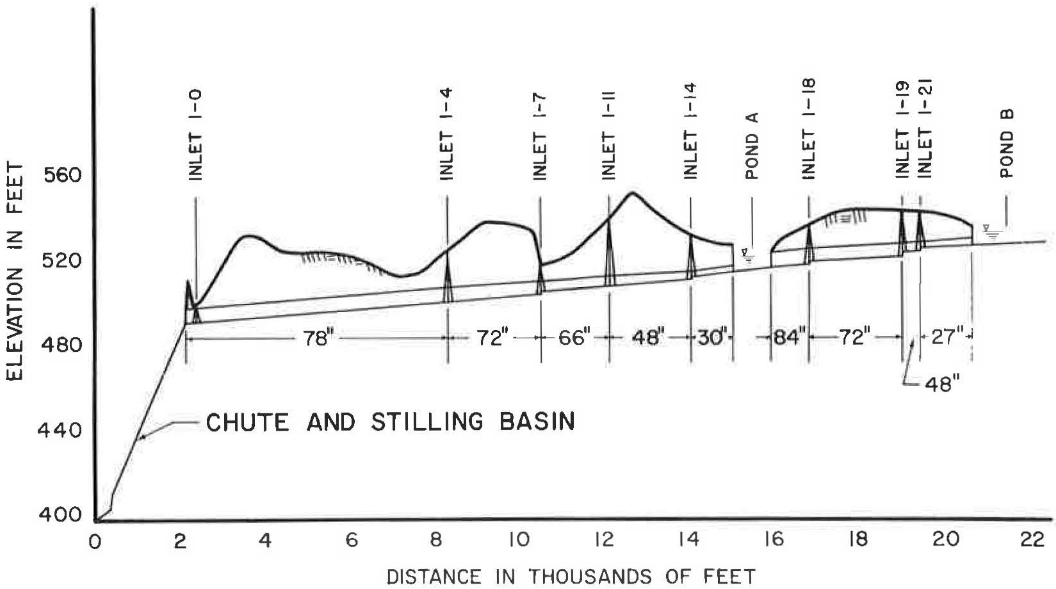

Figure 2. Profile of main trunk sewer.

Based on a preliminary design prepared by the South Atlantic Division, the Nashville District submitted a revised drainage report in 1960 which provided additional storage capacity by enlarging two key ponding areas by excavation. This was essentially Plan E with the addition of two major ponding areas. The estimated construction cost of this plan was $2,000,000. This was the scheme that was later adopted for construction.

DESCRIPTION OF ADOPTED SCHEME

Superimposed on the street layout, shown in Figure 1, is the storm drainage system including drainage area limits, open ditches, ponding areas, pipe, discharge chute, and stilling basin. The cross hatching indicates the approximate limits of ponding for a 10-yr frequency storm. All ponding occurs in natural sink areas except for excavated Ponds A and B. Beginning at the upstream end of the system, the pipe sizes increase progressively until an 84-in. diameter pipe is required to handle the flow entering Pond A. Sufficient storage capacity is provided in Pond A to limit the outlet pipe size to 30-in. diameter. Progressing downstream, pipe diameters increase again until the required outfall pipe diameter is 78 inches. The flow discharging from the outlet pipe enters a 6.5-ft wide concrete chute, approximately 1,800 feet long, which terminates in a stilling basin at the elevation of the creek. Figure 2 is a profile of the main trunk sewer.

OUTLINE OF THE HYDRAULIC DESIGN

The design rainfall used for the Fort Campbell drainage project has an expected frequency of recurrence of once in ten years and a maximum hourly intensity of 1.95 inches. Rainfall intensity-duration data for the Fort Campbell area were obtained from U. S. Weather Bureau publications. Accumulative volumes of rainfall were computed by use of the developed intensity-duration curve. Rainfall excess values were then obtained by applying estimated infiltration rates. The relationships between duration and rainfall intensity, volume, loss, and excess are shown in Figure 3.

Drainage areas to be served by the system were delineated on topographical maps and the times of concentration computed based on length of overland flow, slope of terrain, and type of vegetive cover. Peak inflow rates were determined by use of the Rational Formula using a runoff coefficient of 0.90 for impervious areas and 0.3 for

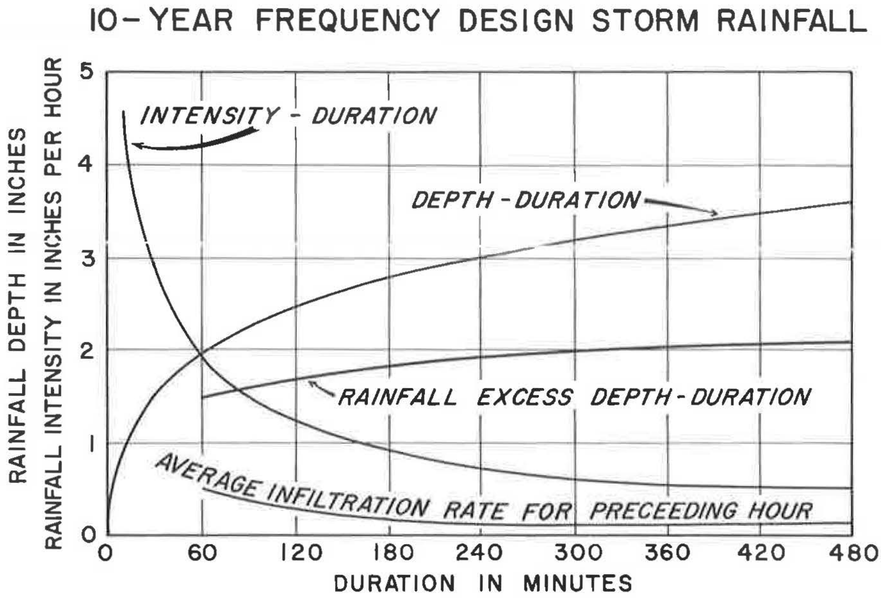

Figure 3. Rainfall relationships for design storm.

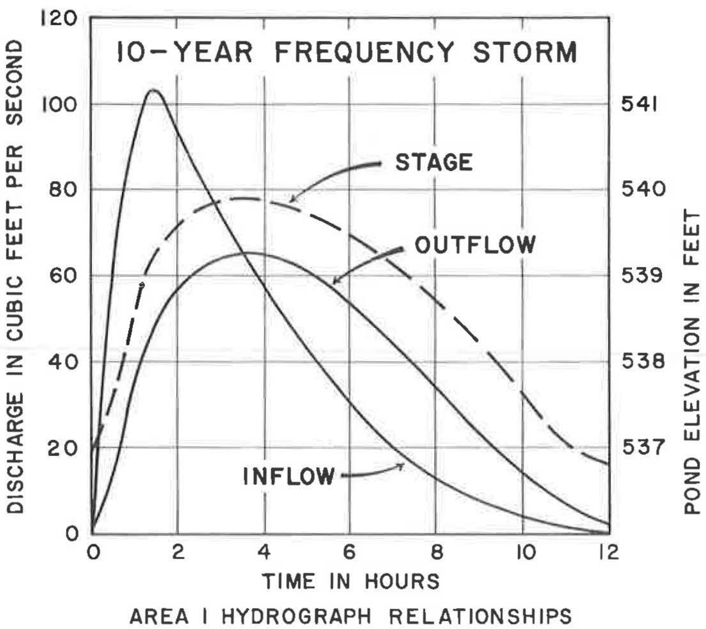

Figure 4. Inflow-outflow hydrograph for area 1.

pervious areas. Inflow hydrographs were developed for each drainage area by using the peak rate of runoff and the time of concentration to define the crest and the rising portion of the hydrograph and then drawing the recession side in such a manner as to balance the total runoff (2.1 inches in 8 hours). Figure 4 shows the inflow hydrograph for drainage area No. 1, which is the area tributary to the northernmost natural pond

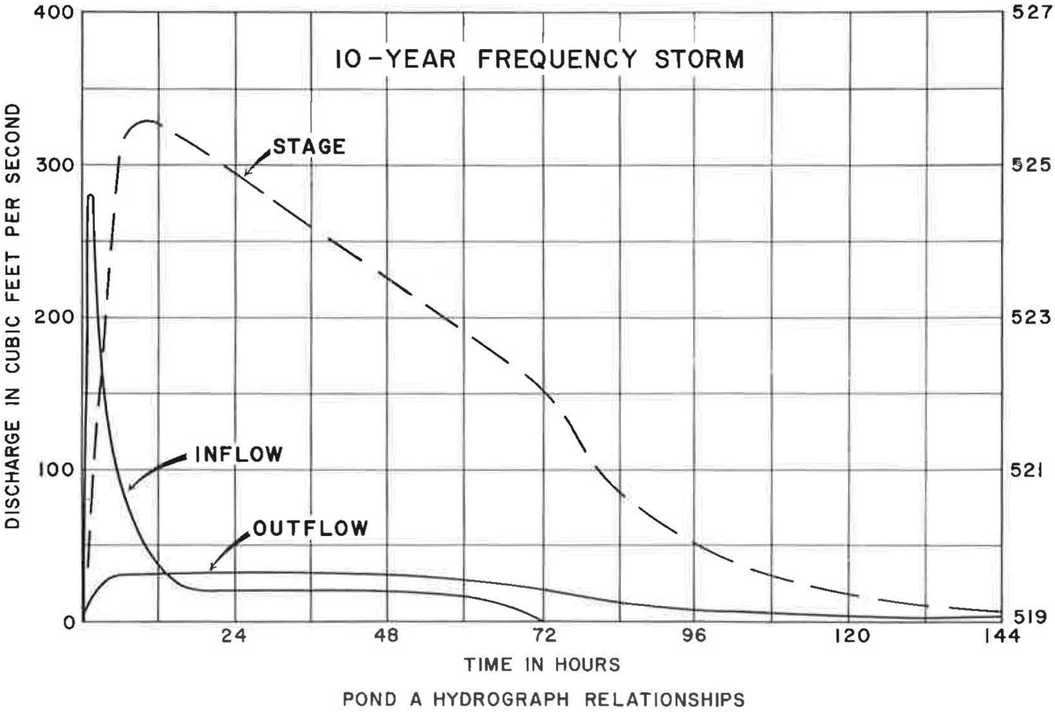

Figure 5. Inflow-outflow hydrograph for Pond A.

shown in Figure 1. For clarity, the limits of the individual drainage areas have not been shown in Figure 1.

In addition to the numerous natural ponding areas, two large ponds were excavated at hydraulically critical points. Pond B is approximately 600 feet long and 500 feet wide. Pond A is approximately 1,200 feet long and varies in width from 400 to 800 feet. Each of the two excavated ponds provides approximately 130 acre-feet of temporary storage capacity. To obtain this ponding volume, it was necessary to excavate 250,000 cubic yards of earth (total for both ponds). The surplus material from the excavation was used to fill and grade small sink areas throughout the cantonment area.

The runoff from each drainage area was routed through storage, where available, to determine the outflow hydrograph or rate of contribution to the drainage system. Figure 4 shows the routing for the natural pond serving drainage area No. 1. This routing is typical for all natural ponds in the system. To induce temporary storage and restrict the rate of runoff entering the drainage system, most of the inlets were equipped with a short control pipe. When flowing partially full, critical flow with inlet control was assumed. After the pipe was flowing full, the discharge was computed by the conventional orifice formula. The long pipe lines draining Ponds A and B were rated assuming friction control and an entrance loss of one-tenth of a velocity head.

The outflow hydrographs, separated by the travel time between design points, were added to obtain the maximum rate of contribution to the drainage system. Figure 5 shows the inflow-outflow relationship for Pond A which receives inflow from drainage areas 1 through 13. It will be noted that the peak discharge entering Pond A is 280 cfs while the outflow is limited to 32 cfs. This reduction in peak discharge entering the downstream system, together with a similar reduction upstream at Pond B, reduced the cost of the project approximately $1,000,000. The reason for this large reduction in pipe cost is apparent when it is realized that 13,000 lin ft of trunk sewer lies between Pond A and the outfall. Provision of adequate storage capacity at Ponds A and B permitted the outlet pipe from Pond A to be reduced from 90- to 30- in. diameter.

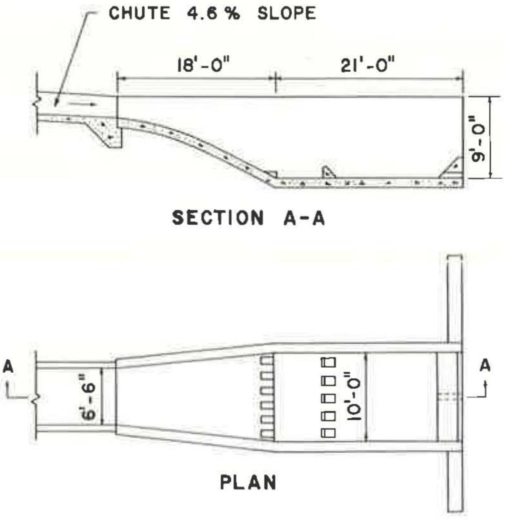

Figure 6. Stilling basin details.

The ground surface, downstream of the pipe outfall, falls rapidly (90 feet) to Little West Fork Creek, the ultimate disposal point for the storm drainage. To prevent erosion of the steep slope, a rectangular concrete chute 6.5 feet in width and 1,800 feet in length was provided. The required height of the side walls was determined by backwater computations beginning at critical depth near the pipe outlet and progressing downstream. Bulking of water due to air entrainment was computed and found to be negligible. Superelevation required at each of the horizontal curves in the chute was computed. It was found that it was significant at only one curve where a superelevation of 0.4 foot was provided. A freeboard of one foot over the computed water surface was provided throughout the length of the chute.

A stilling basin of conventional design was provided at the termination of the chute. As shown in Figure 6, the basin is 39 feet in length, 10 feet wide, and has two rows of baffle blocks and end sill of sufficient height to create the proper depth of tailwater for the formation of a hydraulic jump. Care was taken to locate the basin at the proper elevation to eliminate possible erosion in the ditch section between the stilling basin outlet and Little West Fork Creek. The design discharge for the stilling basin is 240 cfs. The velocity of the flow is reduced from 25 feet per second, at the entrance to the stilling basin, to 3.5 feet per second at the basin end sill.

TABLE 1

COMPARISON OF HYDROGRAPH DATA FOR STORMS OF VARIOUS FREQUENCIES

Area No.	Drainage Area (acres)	1-Year				10-Year				50-Year			
		Max. Inflow (cfs)	Max. Outflow (cfs)	Max. Pond Elev.	Pond. Time (hr)	Max. Inflow (cfs)	Max. Outflow (cfs)	Max. Pond Elev.	Pond. Time (hr)	Max. Inflow (cfs)	Max. Outflow (cfs)	Max. Pond Elev.	Pond. Time (hr)
1	198.6	63.5	40.0	539.0	6.5	103.5	63.6	539.9	12.5	137.5	77.0	540.7	13.0
6[a]	372.0	165.0	18.7	536.4	30.0	270.0	22.3	538.9	72.0	347.0	23.8	540.0	96.0
7	38.4	19.3	16.0	541.8	4.5	31.0	18.4	542.9	8.0	38.9	19.6	543.5	10.5
8	31.9	15.3	6.1	539.2	6.0	25.2	12.1	539.8	10.0	31.6	14.8	540.2	12.0
9	44.3	17.4	7.4	543.2	6.0	28.6	8.2	544.2	12.5	36.4	8.6	544.6	42.0
10	61.5	40.2	26.7	539.2	4.5	64.2	32.9	540.0	9.0	79.8	34.8	540.5	10.0
11	39.9	25.0	7.0	537.9	6.5	40.2	7.8	538.7	12.0	49.8	8.0	539.1	18.5
12	47.9	30.4	12.7	529.8	6.0	49.1	18.3	531.0	10.5	61.3	20.0	532.0	11.5
13[b]	244.7	180.0	19.6	521.8	137.0	270.0	32.0	525.6	192.0	333.0	36.0	528.2	219.0
14	100.0	40.8	12.5	539.3	10.0	66.3	20.6	540.4	17.5	85.0	23.0	540.8	21.0
15	66.2	26.2	7.9	537.9	6.5	42.6	9.0	538.6	18.5	54.6	9.0	539.4	25.0
16[c]	19.3	14.2	14.2	—	—	22.9	22.9	—	—	28.6	28.6	—	—
17	87.2	58.4	50.0	531.2	4.5	93.5	62.0	534.7	10.0	116.2	68.0	535.1	10.0
18[c]	24.9	17.3	17.3	—	—	27.8	27.8	—	—	35.0	35.0	—	—
19	257.0	79.8	26.2	521.0	13.5	132.5	37.0	523.9	21.0	172.9	41.0	525.4	33.0
20	5.4	3.7	1.9	520.8	3.5	5.9	2.6	521.0	5.5	7.2	3.3	521.5	8.5
21[b]	32.0	14.6	11.7	516.9	5.0	22.7	17.4	517.8	9.0	29.8	18.4	519.0	10.0
22[b]	32.7	22.2	9.8	511.1	10.0	37.1	19.9	512.2	13.0	49.7	22.5	513.4	16.5
23	208.0	87.6	26.2	512.1	18.5	144.7	39.0	513.6	19.0	185.0	46.2	514.7	25.0
24	54.6	30.4	7.7	522.6	6.0	50.0	8.8	523.9	15.5	62.3	9.2	524.3	21.0
25[c]	23.3	10.0	10.0	—	—	16.5	16.5	—	—	20.8	20.8	—	—

[a] Includes areas 2 thru 6. [b] Includes discharge from upstream area. [c] No ponding in these areas.

To insure that the drainage system was adequate to handle storms of infrequent occurrence, a 50-yr frequency flood was routed through the system. The computations indicated that the system was adequate to provide for this extreme storm without flooding of any facilities. A 1-yr frequency storm was also routed through the system to determine the depth and duration of ponding that would be expected to occur more frequently. Table 1 shows a comparison of ponding area hydrograph data for a 1-, a 10-, and a 50-yr frequency storm.

CONCLUSION

The Fort Campbell storm drainage system is, in essence, a hydraulically balanced network of temporary ponding areas connected by short control pipes to a main trunk sewer. The major portion of the runoff from the upstream third of the drainage basin is retained for a sufficient length of time, in the two major ponds, to reduce drastically peak downstream discharge rates. The large cost reduction that was accomplished by the judicious use of temporary ponding made the project economically feasible. While drainage projects on the scale of this one are unusual, a comparable percentage of cost savings can be realized on smaller projects by a similar use of temporary ponding. This project emphasizes that in this era of rising costs, the drainage engineer should always be mindful of the potential for drainage cost reductions that are afforded by relatively minor amounts of temporary ponding.

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