THE DESIGN AND CONSTRUCTION OF THE CONASAUGA RESERVOIR

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INTRODUCTION

Known as the headquarters of the nations carpet industry, the city of Dalton, in Northwest Georgia, has enjoyed steady growth over the past few years due in large part to the continuing expansion of this dynamic business. This growth has also been paralleled by an expanding population base and an increase in water demand. In prolonged periods of drought, existing water supplies are inadequate to meet present needs let alone future requirements. Furthermore, even in years of normal precipitation, the demand for water can often exceed the available supply, especially during the drier summer months.

Following a characterization of potential groundwater and surface water sources over an area of approximately 500 square miles, an off-stream storage site on the flood plain of the Conasauga river was selected for further investigation and subsequent construction. The project's main structure is a ring-dike impoundment with a storage capacity of 500 million gallons. The impoundment is contained within an earth embankment which has a crest length of approximately 5200 feet and a maximum height above ground level of 39 feet. The project also includes a 2,000 feet long, 3.5 feet diameter inlet/outlet pipeline and an emergency spillway structure (Figure 1).

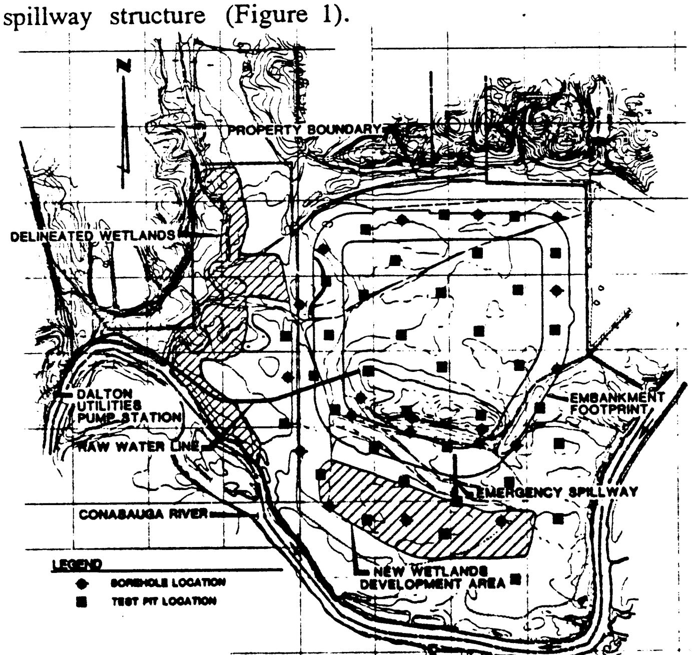

Figure 1: Conasauga Reservoir Project

In the current climate of environmental regulations, off-river storage allows the owner to harness and manage an existing water source to the maximum benefit of the general public and local industry with minimum adverse affect on the local environment. During the winter months while river levels are high, water is pumped into the impoundment rather than allowed to pass downstream unused. During the drier months of the year, when river levels are low and water supply cannot meet demand, the impounded water is released into the existing distribution network thus reducing the possibility of water rationing throughout the service area.

SITE DESCRIPTION

Physical Setting

The site is approximately 6 miles east of the city of Dalton in Whitfield County, Georgia. As shown in Figure 1, the southern, southeastern and southwestern boundaries of the property are defined by the Conasauga river. The site, which is wholly within the flood plain of the river, occupies an area of approximately 160 acres. Prior to construction, the area consisted mostly of open fallow fields, with the terrain sloping gently towards the southwest. Although most of the site was nearly level a small hill, approximately 10 feet high, was located at the center of the property.

Wetlands

A complete delineation of the property identified only a relatively small area of wetlands in the northwest corner of the site where a canopy had regrown after historical farming practices had stopped (Figure 1). The original alignment of the embankment was adjusted to completely avoid incursion into this wetland area. This realignment negated the lengthy Corps of Engineers (C.O.E.) 404 permitting process.

Due to land constraints, the pipeline for the intake and outlet of water from the impoundment had to be aligned across the wetland area. Since the pipeline is defined as a utility line under nationwide permit 12, as stated in Federal rules and regulations 33CFR part 330, no special permitting was required prior to construction.

Subsurface Conditions

Geological records indicate that the site is underlain by bedrock of Cambrian Age which is comprised of fissile shale, with local thin interbeds of limestone and sandstone, from the Conasauga and Rome Formations (Cressler, 1974). Results of a field exploration program indicate that the bedrock is overlain by typical alluvial deposits consisting of fine-grained cohesive soil above coarse grained granular materials.

The cohesive soil is a relatively impermeable soft to stiff silty clay of low to medium plasticity. The average thickness of this stratum is 9.5 feet although it varies randomly across the site from 3 feet to 13.5 feet. The silty clay is underlain by a nearly continuous layer of permeable silty sands, gravelly sands and sandy gravels. The average thickness of this granular unit is generally between 3 feet and 6 feet although it also varies randomly across the site from zero to 8.5 feet. Deposited on the floodplain and close to river level, all soils were measured as being close to saturation.

As expected, artesian or confined groundwater conditions were encountered near the clay/sand interface in the majority of exploratory holes located across the site. The piezometric levels were found to fluctuate both with rainfall and the level of water in the adjacent river. In general, the groundwater level was within 0.5 feet above to 3.5 feet below the existing ground surface.

Construction Materials

Laboratory testing for strength, compaction and permeability indicated that the alluvial clay deposits and the rippable weathered shale would be suitable as earthfill for the embankment, although the material would have to be dried significantly to achieve adequate compaction. This was obviously a concern because of the specified short construction period, so a decision was made to construct a test fill. This demonstrated that drying could be achieved by discing, harrowing, plowing or scarifying. However, it was possible that some other means of drying or conditioning the soul would be necessary, especially after rain. The results from the test fill and laboratory testing program (Table 1) indicated that the addition of lime, in quantities of 1% to 3% by dry weight of soil, would be a relatively expedient and effective way to accelerate the conditioning process. During embankment construction, it would be necessary to drain, develop and work the borrow area carefully to obtain sufficient materials at the specified moisture content for placing.

Table 1. Effects of the Addition of Lime on Max. Dry Density/ ( ) and Optimum Moisture Content (OMC).

T.P. NO.	SAMPLE NO.	Wn %	Wl %	Wp %	Ip %	%< #200	MAX lb/ft3 NAT SOIL	MAX lb/ft3 LIME	O.M.C. % NAT SOIL	O.M.C. % LIME	REMARKS
3	7	20.7	31	17	14	85.7	108.8	104.8	16.3	20.3	1% Lime
5	2	22.3	27	18	9	79.7	109.4	102.5	17.0	20.0	3% Lime
12	4	25.1	--	--	--	50.6	109.8	105.2	15.8	22.3	5% Lime

The alluvial granular deposits proved to be unsuitable for use as filter and drainage materials within the embankment. The material for these zones, along with rip rap for erosion control and wave protection, would have to be obtained off-site, from local commercial quarries.

EMBANKMENT DESIGN

Located on the flood plain of the Conasauga river, the property proposed for the construction of the impoundment plays an important role in the control of flooding along this stretch of the river. Hence, an important objective during the earlier stages of design was to maximize the water storage capacity of the reservoir while minimizing the impact of the project on wetlands and flood levels of the Conasauga river. The design also intended to minimize costs by making maximum use of the on-site materials for construction.

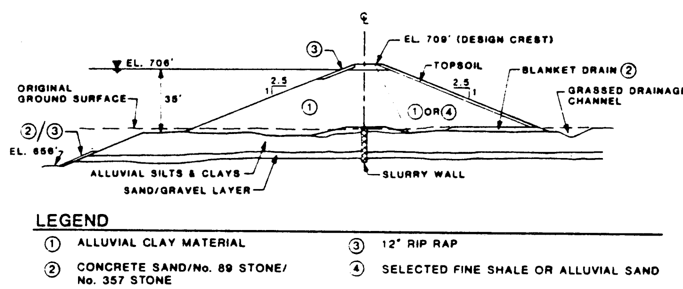

Figure 2: Embankment Cross Section

The major thrust of the design was to optimize land use requirements by minimizing the need to import borrow material and creating embankment sides with relatively steep slopes, commensurate with maintaining adequate stability during construction and in the longer term. To achieve this, maximum use was to be made of the alluvial clay materials within the impoundment for embankment construction. The size of the impoundment was, to a large extent, dictated by the volume of borrow material available on site.

Seepage and pore water pressures were a concern from the beginning of the design. The granular materials above the bedrock have direct hydraulic connection with the Conasauga river. Thus it was necessary to include a slurry wall, keyed into bedrock on the embankment centerline, to prevent heave and piping of the interior alluvial clays during construction and when the reservoir is empty and river levels are high.

A plan of the impoundment and a cross-section of the embankment are shown in Figures 1 and 2, respectively. The key components of the project include:

- a ring-dike embankment approximately 5200 feet in length with an average height of 35 feet, a crest width of 10 feet, and upstream and downstream slopes of 2.5H:1V;
- a continuous 3 feet wide slurry cut-off wall keyed into the underlining bedrock;
- a 2 feet thick, 50 feet long horizontal drainage blanket under the downstream slope to control seepage through and underneath the embankment;
- a uniformly graded 12 inch rip rap on the upstream slope from the crest to 5 feet below maximum pool elevation to provide wave protection;
- a corrugated metal pipe emergency overflow spillway with a 4 feet diameter riser and 3 feet diameter horizontal limb. The emergency spillway, which is encased in concrete through the embankment, is designed to pass ten inches of rainfall from 1/3 of the probable maximum precipitation (PMP) in combination with a maximum over pumping flow should the pumps used to fill the impoundment fail to shut off during the storm event. Alternatively, the emergency spillway will pass 100% of the PMP without over pumping;
- a 2,000 feet long, 3.5 feet diameter ductile iron inlet/outlet pipe encased in concrete through the foundations of the embankment and connected to the existing raw water pumping station which is located adjacent to the river, immediately downstream of the site. A bifurcation in the pipeline close to the pumping station will allow water to be pumped into the reservoir or alternatively, by manipulation of a system of valves, for gravity flow of water from the reservoir back into the wet well of the pumping station;
- piezometers to monitor the performance of the slurry wall and seepage under and through the dam. Settlement of the dam will be monitored by monuments along the crest of the embankment; and
- the construction of a wetland in the borrow area to the south of the impoundment.

EMBANKMENT CONSTRUCTION

Construction of the ring-dike embankment commenced in August 1989 and was completed during May 1990. The principal phases of the work are briefly described below:

After the site had been cleared, a 3 foot high working platform was constructed for the installation of the cut-off slurry wall along the center line of the embankment. The slurry wall trench was excavated to refusal or keyed into the shale bedrock to a minimum depth of 2 feet using a Caterpillar 235C excavator. The average depth of the cut-off wall was approximately 20 feet. The slurry backfill consisted of a soil/bentonite mixture with 20 to 40 percent fines and an initial minimum bentonite content of 2%, by dry weight. Due to failing permeability values the bentonite content of the mix was slightly increased throughout the duration of the work.

After completion of the cut-off wall drainage channels and sumps were excavated to the top of bedrock, within the area bounded by the wall, in an attempt to drain the sand and gravel layers and lower the groundwater level so that the internal borrow area did not flood during excavation and development. To facilitate construction activities, these channels and sumps had to be continuously realigned and deepened as the work proceeded.

The embankment fill consisting of the alluvial clay deposits, was placed, after proof rolling of the subgrade soils, in maximum loose lift thicknesses of 8 inches and compacted to a minimum of 95% of the standard Proctor density at a moisture content of -1% to +3% of the optimum moisture content. Since the moisture content of the insitu soils was +6% to +12% of the optimum moisture content, the borrow materials were dried to achieve the specified compaction by discing and scarifying. Approximately 5,300 tons of lime were used, mainly after periods of rain, to accelerate the drying and conditioning process. Approximately 700,000 cubic yards of borrow material were excavated, placed and compacted in the construction of the embankment.

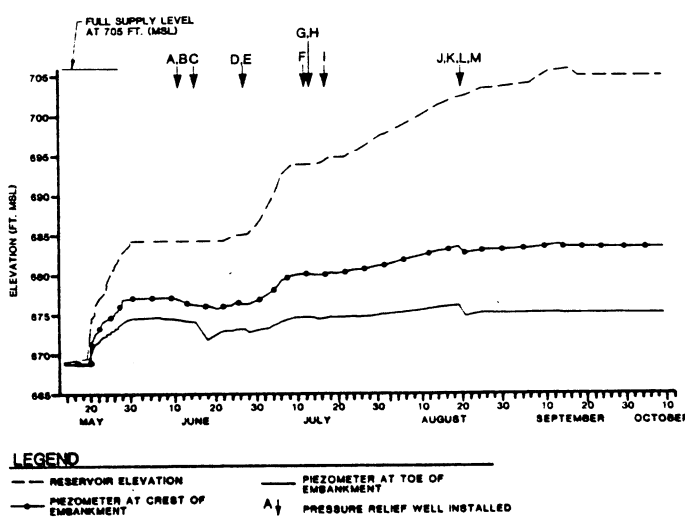

Figure 3: Reservoir & Piezometer Elevations

RESERVOIR IMPOUNDING

Due to the nature of the geology and the specific ground water conditions on site it was anticipated, during the early stages of the design, that there could be a problem with excessive uplift pressures during impounding of the reservoir. A ground water monitoring plan which included the installation of piezometers and a contingency for foundation grouting were included in the construction documents to counteract such problems should they arise during impoundment.

The 30 piezometers, installed in "nests" of 3 at 10 locations around the embankment, were monitored daily to determine the increases in ground water pressure during impounding of the reservoir. During the early stages of impoundment, water levels in the piezometers rose at an alarming rate. Figure 3 shows a typical plot of the reservoir and piezometer levels with time. At the start of filling, the rise in reservoir level triggered a corresponding and almost parallel rise in piezometric levels at the downstream toe of the embankment. Although increases in the piezometric levels were anticipated, the rate of increase was a cause of concern because of the potential for "blow out" or heave at the toe of the embankment. After careful deliberation and discussion about potential seepage pathways, potential effects and possible remedies, it was agreed that seepage was probably occurring through discrete joints in the underlying bedrock. However, a decision was made to install pressure relief wells rather than perform foundation grouting to reduce the uplift pressures at the downstream shoulder and toe of the embankment. As shown in Figure 3, the installation of these wells, which consisted of 4 inch diameter schedule 40 PVC pipe with 0.02 inch slots surrounded by 6 inches of fine-grained aggregate, lowered the water levels in adjacent piezometers almost instantaneously and reduced uplift pressures along the toe of this section of the embankment. Through careful monitoring of the piezometers and the strategic installation of additional pressure relief wells, impounding of the reservoir was able to continue virtually uninterrupted up to full supply level. A total of 50 pressure relief wells were eventually installed at various locations around the perimeter of the embankment at a fraction of the cost of the grouting alternative.

CONCLUSIONS

Site specific engineering geology, delineated wetland areas and hydrologic characteristics played a major role in the planning, design, construction and performance of the Conasauga reservoir.

By avoiding the wetland areas, the project was able to proceed expeditiously without going through the lengthy C.O.E. 404 permitting process.

The necessity of locating the impoundment near the existing raw water pumping station and adjacent to the Conasauga river, could have resulted in a considerable reduction of the flood plain area. However, the detailed hydrologic analysis resulted in the selection of an embankment alignment which utilized the available land for maximum storage capacity while having minimal impact on flood levels along this stretch of the river.

The alluvial clay deposits and the rippable weathered shale present on site were suitable as earthfill for the embankment, although the material had to be dried significantly to achieve adequate compaction. The construction of deep borrow area dewatering trenches and the discing, horrowing, plowing and scarifying of these highly saturated soils helped to dry these materials to the specified moisture content for placement. The addition of lime, in quantities of approximately 2% by dry weight of soil, was often necessary, especially after rain, to accelerate the drying and conditioning process.

Daily monitoring of piezometric levels indicated that the effects of reservoir impounding were influenced by foundation geology and the design of seepage control measures. The slurry cut-off wall significantly reduced seepage into the impoundment, through the alluvial soils, during construction. However, during impoundment, seepage through discrete joints in the underlying bedrock created excessive uplift pressures along the toe of some sections of the embankment. The installation of pressure relief wells successfully reduced these pressures and enabled impounding of the reservoir to continue virtually uninterrupted.

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