THE WHEELER PROJECT

UNITED STATES
TENNESSEE VALLEY AUTHORITY

THE WHEELER PROJECT

A Comprehensive Report on the Planning, Design, Construction, and Initial Operations of the Wheeler Project

TECHNICAL REPORT No. 2

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TENNESSEE VALLEY AUTHORITY,
Knoxville, Tenn., August 15, 1939.

Mr. JOHN B. BLANDFORD, Jr., *General Manager,*
Tennessee Valley Authority, Knoxville, Tenn.

DEAR MR. BLANDFORD: The attached report on the planning, design, construction, and initial operations of the Wheeler project has been prepared[1] by my own staff with contributions from a large number of persons in other departments of the Authority.

Because of the general interest in the water-control projects of the Tennessee Valley Authority by engineers and others, I recommend that this report be printed as a public document.

Very truly yours,

T. B. PARKER, *Chief Engineer.*

Approved by Board of Directors, October 6, 1939.

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[1] For detailed acknowledgments, see page 303.

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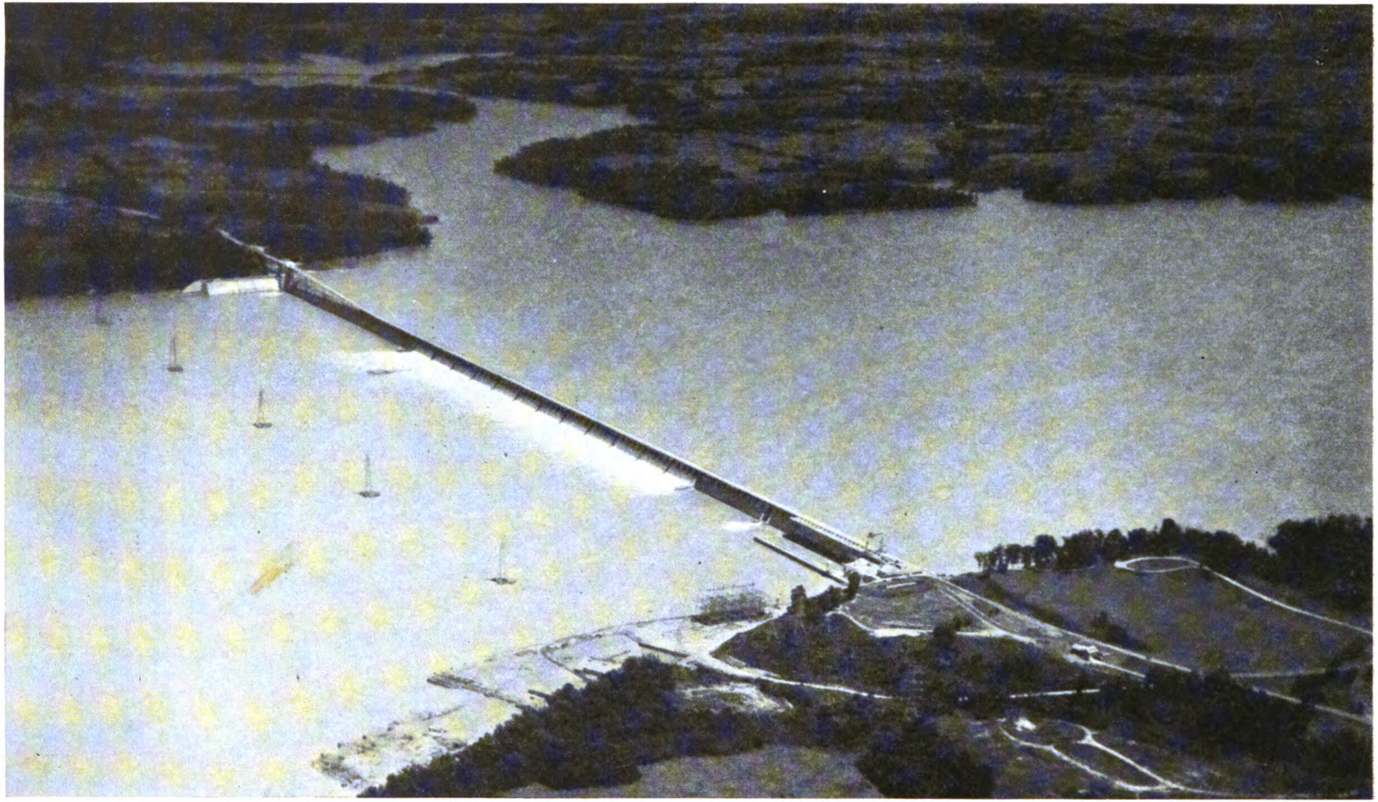

Wheeler Dam

THE WHEELER PROJECT

CHAPTER 1

INTRODUCTION

This report is published for the purpose of giving to the engineering profession and to others interested in river-control projects the important and useful facts about the Wheeler Dam and Reservoir located on the Tennessee River in northern Alabama and constructed by the Tennessee Valley Authority, an agency of the United States Government. The material presented herein has been selected and condensed from the large mass of data, many times greater in volume, contained in the Authority's files. To make this report of greatest use to those engaged on other similar projects, relatively little space is given to those parts of the work which followed well-established engineering practice; while, on the other hand, novel or unprecedented features are described and explained in much more detail. It is hoped that many parts of this exposition will be found useful and interesting also to the general public.

The Tennessee Valley Authority, established by the Congress on May 18, 1933, was assigned a broad program of regional, social, and economic planning with specific instructions to develop the Tennessee River and its tributaries to provide a channel for 9-foot navigation from its mouth to Knoxville, Tenn.; to provide for the control of floods; and to generate the power made available by the water-control facilities.

The system adopted for coordinated water control comprises main-river locks and dams to create pools for slack-water navigation through the 650 miles of river from the mouth to Knoxville, with flood-control space reserved; and tributary storage dams with large reservoirs providing a high degree of control of tributary flows for flood storage and main-river flow regulation. The main-river projects are, in the order named from the mouth of the river upstream: Kentucky (originally named Gilbertsville), Pickwick Landing, Wilson, Wheeler, Guntersville, Hales Bar, Chickamauga, Watts Bar, and Coulter Shoals, of which all but Coulter Shoals are now (1939) completed or under construction. Norris Dam on the Clinch River, completed in 1937, and Hiwassee Dam, now under construction on the Hiwassee River, are tributary dams. Additional tributary control is required for a well-balanced system, and investigations of its feasibility are being made.

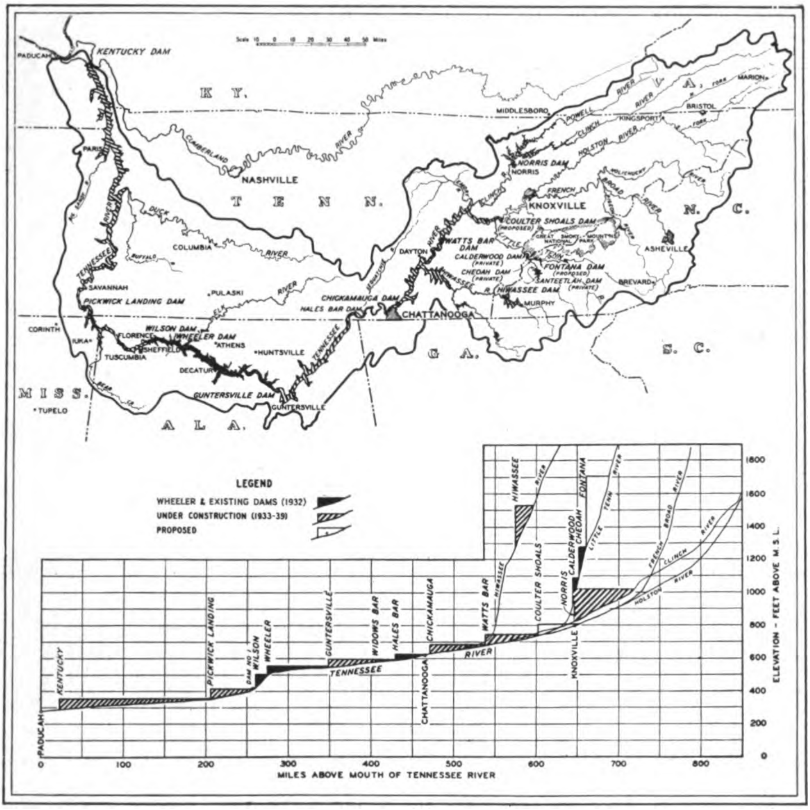

Figure 1.—*Map and profile of the Tennessee River System.*

LOCATION AND CHARACTER OF THE TENNESSEE VALLEY

The headwaters of the Tennessee River are in the Great Smoky and Blue Ridge Mountains—the highest ranges in eastern United States—which are located in eastern Tennessee, in western Virginia and North Carolina, and in northern Georgia. The main river begins immediately above Knoxville, Tennessee, at the confluence of the Holston and French Broad Rivers. It flows in a southwesterly direction through Tennessee, crosses northern Alabama, forms a small portion of the northeastern boundary of Mississippi, and then flows north through western Tennessee and western Kentucky to discharge into the Ohio River at Paducah, Kentucky, a river distance of 650 miles.

The Tennessee River drains an area of 40,910 square miles, equal to four-fifths the area of England. The mountain region in the upper part of the valley—a region of forests, pasture lands, orchards, and small farms—is in striking contrast to the relatively flat lands of northern Alabama, occupied by large cotton plantations, and to the rolling land of western Kentucky. The Tennessee Valley area is populated by two and a half million people, one-half of whom live on farms, while the remainder are divided about equally between cities and towns.

The Tennessee River drainage basin has a wide variety of natural resources. Half of its area is covered by forests from which most of the merchantable timber has been cut. The region has a large supply of coal, iron, and other minerals. The fall of the streams from elevations of more than 3,000 feet in the upper valleys to about 300 feet at the outlet of the Tennessee River into the Ohio, combined with the large annual rainfall, which ranges from 40 to 80 inches, offers great possibilities of water-power development. The distribution of this rainfall results in definite wet and dry seasons, and without water control works the Tennessee River creates alternately a flood hazard and a barrier to open channel navigation.

The Tennessee River between Chattanooga and Wilson Dam flows generally southwest to Guntersville, Ala., and then northwest to Wilson Dam. After leaving the gorge below Hales Bar Dam, it enters a wide valley and flows on a low gradient to the upper end of Muscle Shoals a few miles west of Decatur, Ala., where it breaks into the shoals and drops rapidly to the pool of Wilson Dam. Existing projects in this section of the river at the time of the establishment of the Tennessee Valley Authority, were Hales Bar Dam, a private development[1] just below Chattanooga (at mile 431.1 above the mouth), Widows Bar Dam, a low navigation dam (at mile 407.6), and Muscle Shoals and Elk River Shoals Canals just below the mouth of Elk River. The Elk River and Muscle Shoals Canals are flooded by Wheeler Reservoir, Widows Bar Dam has been inundated by the recently completed Guntersville Reservoir, and only the Hales Bar Dam will be utilized as a permanent part of the system development between Wilson Dam and Chattanooga.

HISTORY OF THE WHEELER PROJECT

The series of rapids between the cities of Decatur and Florence, Ala., through which the river falls 134 feet in 37 miles, created a formidable obstacle to transportation during the early period of settlement when water routes constituted almost the only avenue of penetration into the wilderness west of the Alleghenies. The 16-mile reach at the downstream end of these rapids, where the river falls 100 feet, is the famous Muscle Shoals. The obvious desirability of continuous navigation along the river resulted in many plans and suggestions for improvement, and with the advent of organized government in the territory, two attempts were made to construct canals and locks to bypass the shoals. These canals, although eventually constructed and used, were inadequate, expensive to maintain and operate, and of little lasting benefit.

The growing utilization of hydroelectric power introduced a possible solution to the navigation problem by making possible a multi-purpose, income-producing development. Starting in 1898 and continuing until September 1917, when Muscle Shoals was selected as the site for the power dam and nitrate fixation plant authorized in the National Defense Act of June 3, 1916, various bills were introduced and reports issued concerning the disposition and development of the site. Most of these proposals involved a Government subsidy to cover cost of navi-

[1] Acquired by the Tennessee Valley Authority on August 16, 1939.

gation facilities; the utilization and the installation of power facilities were planned to be available for private development.

Construction of Wilson Dam was started in 1918, suspended owing to lack of congressional appropriations in 1920, resumed in 1922, and completed in 1925. From 1921 to May 1933, negotiations with private companies, first for the site, the nitrate plant, and the partially completed dam, and later for the dam and nitrate plant, made the disposition of the property a national issue but did not culminate in any definite or permanent working agreements. Negotiations ended with the assignment of the property to the Tennessee Valley Authority on May 18, 1933.

Navigation facilities.

Projects constructed and channel improvements made prior to 1932 provided a 6-foot depth in the canals and canalized sections of the river between Chattanooga and the mouth of the river and slack-water navigation in the Wilson Dam and Hales Bar Dam pools, but in the open river sections a 6-foot channel was available only during high river flow. With the canals then in use, the limiting depth was approximately 3 feet under minimum flow conditions. The lateral canals above Wilson Dam were inadequate to carry much traffic through the upper shoals, and the Elk River Shoals was impassable for large craft. This reach of river limited the draft and volume of river traffic more than any other.

The River and Harbor Act, approved by the Congress on July 3, 1930, states in part:

> The project for the permanent improvement of the main stream of the Tennessee River for a navigable depth of nine feet in accordance with recommendations of the Chief of Engineers in House Document No. 328 of the Seventy-first Congress, second session, is hereby authorized: *Provided*, That an expenditure of $5,000,000 shall be authorized to be appropriated for the prosecution of work under this project. *Provided further*, That the Chief of Engineers is hereby directed to ascertain and report to Congress on the first day of the first regular session of the Seventy-second Congress, advising the prospective cooperation offered by responsible interests under the Federal Water Power Act, in the program of construction recommended by the Chief of Engineers, providing for the nine-foot project by means of high dams.

As instructed by this act, the Chief of Engineers reported in House Document No. 131, Seventy-second Congress, first session, dated December 7, 1931, that no definite assurance of immediate development of power sites on the river was made by the responsible interests contacted and that there was at that time little or no interest evidenced in the proposed sites between the head of Colbert Shoals (mile 234.9) and Hales Bar Dam. He recommended, therefore, immediate construction of navigation dams between the head of Colbert Shoals and Hales Bar at an estimated cost of $22,500,000.

Wheeler navigation lock.

The Emergency Relief and Construction Act of July 21, 1932, appropriated $30,000,000 for previously authorized river and harbor projects. A contract for the construction of the lock at the Wheeler site was let in November 1932, under the provisions of this act. Initially the lock was to be a single, 37-foot lift structure. This lock, with the construction of a dam of corresponding height, would have established the reservoir level at elevation 542 and would have inundated the locks in the Muscle Shoals and Elk River Shoals Canals and eliminated the proposed Milton Bluff low dam project, and would have extended 9-foot navigation approximately 45 miles upstream without additional channel improvement.

Soon after the creation of the Tennessee Valley Authority the Board of Directors arranged with the United States Army Engineers for changing the Wheeler lock then under construction to allow more adequate development of this section of the river. The changes made initially in the lock included a redesign of walls and gate blocks to provide sufficient strength and stability to permit raising the walls to elevation 566 for an equivalent pool level elevation of 561. The change made at this time contemplated topping the walls at elevation 549, providing sufficient height for a pool elevation of 544. This revision in plan permitted the Authority to proceed with planning studies to fix the upper pool elevation of Wheeler Dam at a level best adapted to the program; and before construction was completed to elevation 549 on the base section of the lock, the top of the gates was fixed at elevation 556.3 [2] and the lock was finished to accommodate the lift required.

Wheeler Dam.

The Federal relief agencies urged immediate activity in starting construction of the dam as a reemployment measure, and in October 1933 the President requested that construction be started as soon as practicable, taking the necessary funds for 1 year's work from the initial Congressional appropriation for the Authority. The exigencies of the unemployment situation greatly accelerated the normal progress of design and construction of this project, and to save the time which would be required to develop the necessary engineering design organization, arrangements were made for the United States Bureau of Reclamation to perform this work at the expense of the Authority. Practically all phases of the planning and design of this project were somewhat complicated by the participation of the three agencies concerned in those activities—the Tennessee Valley Authority, the United States Bureau of Reclamation, and the United States Army Engineers. However, the close cooperation of both of these agencies enabled the Authority to proceed with a minimum of confusion.

The studies made by the Authority indicated the advisability of raising the dam above the height planned by the United States Army Engineers to provide a longer navigation pool, increased flood storage space in the reservoir, and additional power facilities; and the final design of the dam provided for a maximum controlled pool level at elevation 556.3,[2] approximately 12 feet higher than the elevation planned by the Army Engineers.

Wheeler Dam was named for General Joe Wheeler whose home, at Wheeler, Ala., about 17 miles from the dam site, for years has been one of the principal points of interest in that part of the State. General Joe Wheeler, as a member of the United States Congress, from

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[2] See page 221.

Alabama, in 1898 introduced a bill providing for the development of Muscle Shoals as a power-navigation project by the Muscle Shoals Power Co. The proposed project contemplated the construction of canals and power plants in the shoals. Although no construction was undertaken, this bill was the first of the series which ultimately led to the passage of the Tennessee Valley Authority Act in 1933.

A bronze tablet on the Wheeler project bears the following inscription:

WHEELER DAM

NAMED IN HONOR OF

JOSEPH WHEELER

GENERAL IN THE ARMY OF THE CONFEDERACY
AND A LEADER OF THE U.S. VOLUNTEERS
IN THE SPANISH AMERICAN WAR

BUILT FOR THE PEOPLE
OF THE UNITED STATES OF AMERICA

BY THE TENNESSEE VALLEY AUTHORITY
UNDER DIRECTION OF THE CONGRESS
AND THE PRESIDENT

1933-1936

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Chapter 2

PRELIMINARY INVESTIGATIONS

SELECTION OF DAM SITE

The construction of the low navigation lock and dam at site No. 1, which maintained the pool level of the Florence Canal, and the completion of Wilson Dam at site No. 2 provided a fair degree of improvement for slack-water navigation through all but the upper 20 miles of the Muscle Shoals section of the river. The limiting navigation depth in this upper 20 miles above the head of Wilson pool was about 1 foot during low water, and the lateral canals constructed at upper Muscle Shoals and Elk River Shoals were inadequate for large craft. It was essential that Wheeler Dam be located to extend the required navigable depth from the head of Wilson pool to a site to be selected in the general vicinity of the town of Guntersville, Ala.

This requirement dictated that the general location of Wheeler Dam should be at the head of Wilson Lake. The United States Army Engineers had studied a site at mile 275.1, but if the dam had been placed there, considerable rock excavation would have been required to provide a channel for 9-foot navigation below the Wheeler lock when the Wilson pool was at its lower levels. Since estimates indicated that the location of the dam at mile 274.9 would eliminate some of the dredging requirements and would thus be more economical, the latter site was eventually selected.

DESCRIPTION OF SITE

Between the Guntersville and Pickwick Landing Dam sites, the Tennessee River flows generally west-northwest, although near Wheeler Dam the flow is almost due west. At the axis of the dam the river is about 6,200 feet wide with rock bluffs on each side rising steeply from elevation 498 in the river to slightly over elevation 600 where they flatten to meet the plateau.

The river gradient at the site is steep, causing high velocities and shallow depths in the channel. The underlying Fort Payne formation with its hard and almost horizontal beds of limestone resisted erosion and caused the development of steep slopes and successive steps along the bed of the stream. High velocities in the river have washed practically all overburden from the channel; only a slight covering exists at the base of the bluffs forming the riverbanks. The top of sound rock in the north and south bluffs is at about elevations 550 and 540, respectively, and is covered with clay and gravel overburden to eleva-

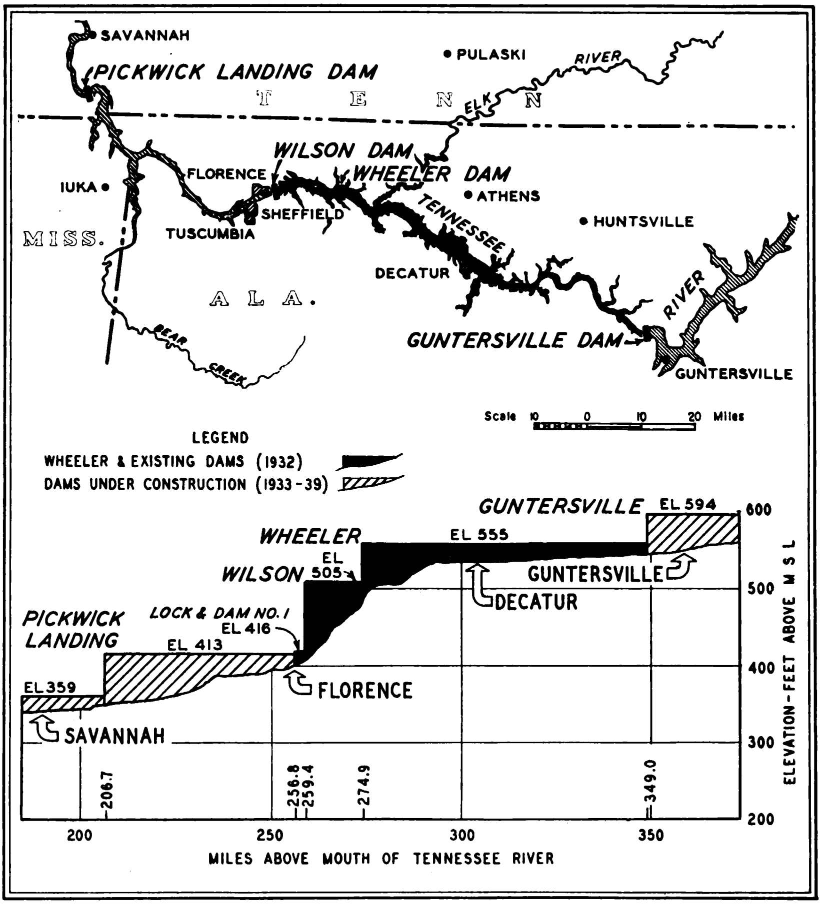


FIGURE 2.—*Map and profile, Tennessee River near Wheeler Dam.*

tion 608 on both sides of the river. On the south bank the top of the rock extends to elevation 560 but is a soft, broken chert of poor quality with numerous solution cavities and caves.

GEOLOGY OF SITE

The site lies in the Tennessee Valley division of the Highland Rim Plateau. In the vicinity of the dam the topography consists of a flat to semirolling plateau at an average elevation of 600 feet. Back from the river and on interstream divides, the chief interruptions to the normally flat plain are sinkholes. Near the river small streams have cut sharp gullies at their mouths in attempting to keep pace with the downcutting action of the river. One of these occurs a short distance downstream from the left abutment.

Near the bottom of the powerhouse area, New Providence limestone and shale were encountered. About 10 feet of the top of this formation was exposed, but as no deep holes were drilled, the total thickness was not determined. This formation consists chiefly of a very coarse-grained, greenish crystalline limestone with some shaly layers. Although this formation, like most limestones, is somewhat soluble in ground water, no cavities were found, since this horizon is well below the ground-water table where no active solution has recently taken place. The formation provides a good sound foundation rock.

The Fort Payne formation at this point is approximately 100 feet thick. With the exception of the lower portion of the powerhouse, it forms the foundation of the entire dam, as well as both abutments. In general, it consists of a silico-argillaceous limestone, although it includes some pure shale layers, some black flint, and a few lenses of pure crystalline limestone. One of the dense siliceous layers forms the river bed at this point and is responsible for the formation of the shoals because of its resistance to erosion. There is a wide variation of rock within the formation although it is mostly of the lime shale type.

The dark-colored shaly rock, which forms most of the foundation, splits up easily under weathering. This characteristic slabbing led to difficulties at Wilson Dam, where falling waters tore out many tons of rock at the toe of the dam. To protect against such erosion at Wheeler, the spillway was provided with a deep, concrete-lined stilling pool and energy dissipators. Although shaly and easily split, the Fort Payne forms an excellent foundation for the dam and abutments. It is dense, compact, and relatively insoluble in ground water, qualities which make for a good foundation.

The rocks at the dam site lie almost flat. No disturbance has occurred to fold and fault the rock to any significant degree. Gentle folds were observed in the excavation as it progressed across the river, and the river at this point appears to flow along a broad, low anticline, the crest of which is in the middle of the river. Extensive jointing has

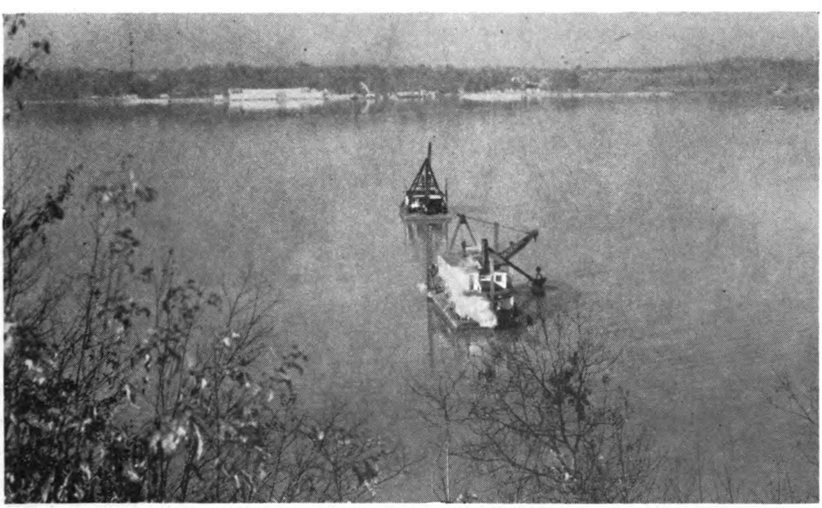


FIGURE 3.—*Wheeler site before the start of TVA construction.*
(Note lock under construction in left background)

occurred in the foundation rock. These joints in the river bed have been eroded to a width of a foot or more, but they generally do not extend more than a few feet below the surface. At some points, particularly in the south abutment, the joints had been enlarged to small caves.

During a comprehensive exploratory drilling program carried out by the United States Army Engineers, no extensive solution channels normally to be expected in a limestone of this kind were revealed. It was concluded that the extent and details of any needed foundation treatment could best be determined after the actual condition of the rock could be observed in the powerhouse excavation.

Diamond drill cores selected from borings made by the United States Army Engineers in the dam foundation were tested for shearing and compressive strength in the TVA laboratory. The values of the compressive and shearing strength were sufficiently high to indicate that the foundation rock for the dam is adequate to withstand all normal pressures to which it may be subjected. Nine specimens showed an average compressive strength of 23,660 pounds per square inch, and 12 specimens tested in shear developed an average shearing stress of 2,112 pounds per square inch.

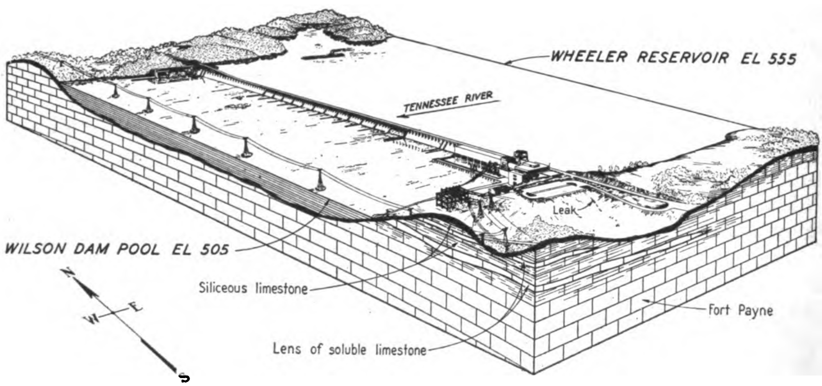


FIGURE 4.—*Geologic relationships at the dam site.*

RAINFALL

The Tennessee River Basin has the appearance of two distinct basins placed end to end. The upper and lower basins have areas of 21,400 square miles and 19,510 square miles, respectively, or a total of 40,910 square miles, of which 29,590 square miles are above Wheeler Dam. The eastern or upper basin is surrounded by mountain ridges. Much of the area within these ridges is mountainous, and, except for the central portion, the area has rugged topography—some of the mountain peaks in the eastern parts of the basin rise to an elevation approaching 7,000 feet above sea level. All of the main tributaries drain mountain lands.

The lower basin is much lower in average elevation and is comparatively flat with gently rolling slopes. Some tributaries of the lower basin, however, have their headwaters in the highlands of the Cumberland Plateau. There is little variation in the average annual rainfall throughout the lower basin, the mean rate being about 52 inches with an average run-off of about 24 inches.

There is a generous amount of rainfall, varying from about 55 inches to over 80 inches in the mountain area and from about 40 to 45 inches in the drier sections, with run-offs from 40 to 44 inches in the mountain section dropping to 14 to 18 inches in the sections of low rainfall. Most of the rainfall is supplied by storms traveling from the southwest to the northeast, but limited areas along the eastern border of the basin are occasionally subject to storms from the east and southeast. The most intense recorded rainfall in the basin resulted from a West Indian hurricane which penetrated the basin from the east.

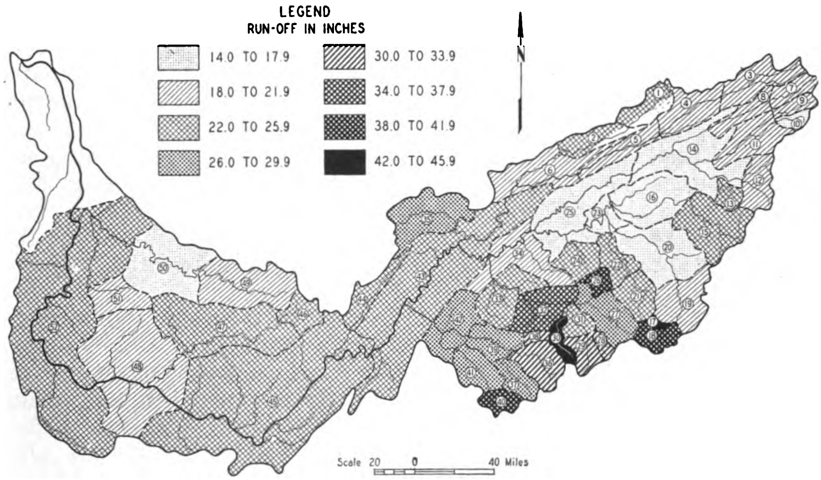


Figure 5.—*Distribution of run-off for 12-year period, 1920–32.*[1]

Large general floods in both the upper and lower basins are caused by storms crossing the basins from the southwest, usually during the season from December to the first half of April. In storms of limited duration a large proportion of the rainfall in the lower basin is discharged before the arrival of the flood wave from the upper basin. In storms of long duration, however, floodwater from all parts of the basin may be contributed to the lower river at the same time. Flood waves from the upper basin, however, are generally lengthened with only slight increase in height as they pass through the lower basin.

Cloudbursts, which are generally confined to small areas, may occur anywhere in the valley. There have been notable examples of these on the Cumberland Plateau and also in the mountain lands of the easterly part of the upper basin. Such cloudbursts have caused high local floods, the damage from which has been small because of the limited development in the path of the flood flow.

In 1933, rainfall records were available from about 128 stations[2] in and near the Tennessee Valley, and stream flow records were available from 52 stations in the valley. Both the rainfall and the stream-flow records differ widely as to the length of time covered. Systematic

[1] See Tennessee Valley Authority, Technical Report No. 1, *The Norris Project*, p. 29, for details of rainfall and run-off.
[2] In July 1939 there were 403 rain gages in and adjacent to the Tennessee Valley.

records of rainfall and stream flow were started about 1875, although rainfall records were commenced earlier in a few scattered places.

For determining the power possibilities, the 12-year period, October 1920 to September 1932, inclusive, was used for precipitation, average flow, and the average duration of flow. This period includes the driest year of record, 1925; a fair representation of an average single year, 1927; a representative wet year, 1929; and a severe flood, December 1926 and January 1927. The variation in rainfall in the driest year, 1925, and the wettest year, 1929, caused a variation in run-off from 12.64 to 30.96 inches. In the assumed typical year, 1927, the run-off was 27.39 inches.

STREAM FLOW

Gage-heights have been recorded at Florence, Ala., almost continuously from 1871 to date. Prior to 1926 a staff gage was maintained, and the records were fairly accurate except during the period of filling Wilson Reservoir. In 1926 a water-stage recorder was installed. Repeated checks on the ratings over a period of 30 years indicate that the control is practically permanent. The drainage area above the gage is 30,810 square miles and requires about a 4-percent reduction to be applicable at Wheeler Dam. Stream-flow records at Decatur, Ala., with a drainage area of 26,900 square miles, are also available, starting in 1924.

For the 12-year period from 1920 to 1932 the average annual rainfall above the Florence gage from a drainage area of 30,810 square miles, was 51.4 inches; and the average run-off was 1.69 cubic feet per second per square mile or about 22.9 inches in depth, equivalent to about 45 percent of the precipitation.

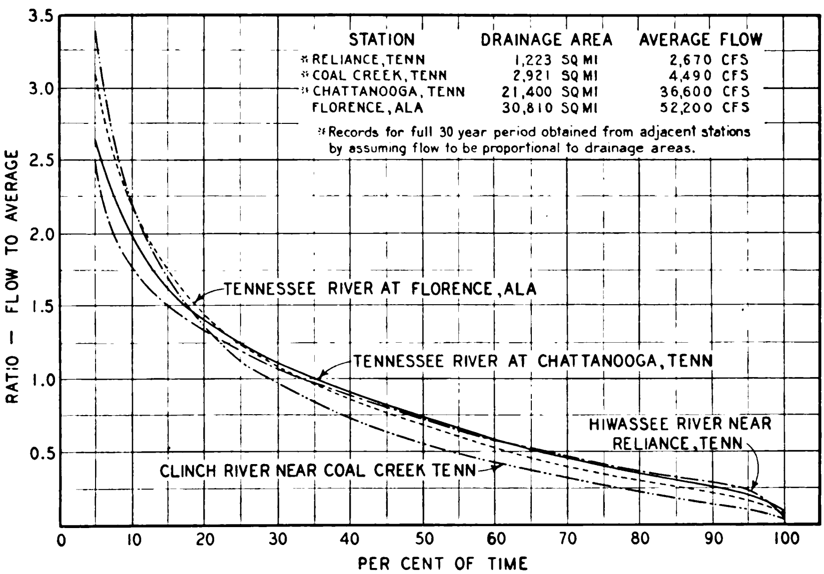


FIGURE 6.—*Relative duration of flow curves for four points in the Tennessee valley for 30-year period—October 1, 1903, to September 30, 1933.*

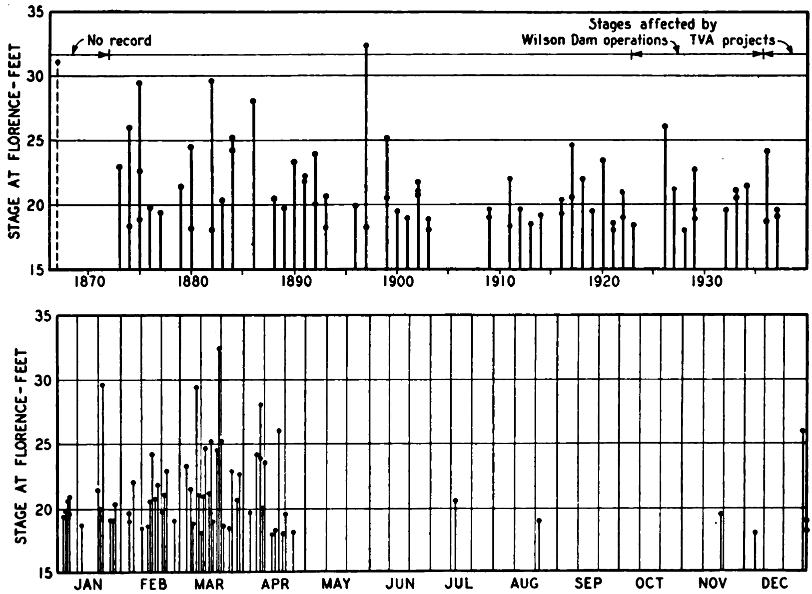


FIGURE 7.—*Distribution of flood flows, Tennessee River, at Florence, Ala.*

The duration of flow curves for four points in the basin for the 30-year period between October 1, 1903, and September 30, 1933, are shown in figure 6. To check the dependability of the available gaging records and to show the flow characteristics of different gaging stations, a method was developed involving cumulative variations from the average flow. This method permits a direct comparison of the flashiness of the streams at the points considered.

The mean annual flow at Florence for the 39-year period from 1894 to 1933 was 52,300 cubic feet per second, while the minimum flow, which occurred in 1925, was estimated at 4,070 cubic feet per second. With normal regulation by Norris and Hiwassee Reservoirs, minimum flow into Wheeler pool will probably be not less than 13,000 cubic feet per second. Chickamauga, Guntersville, and other reservoirs in the integrated plan will also contribute storage releases for increasing the low flow.

The maximum flood in the upper basin which has been recorded at Chattanooga, Tenn., occurred in March 1867. This flood recorded maximum stages downstream from Chattanooga to some point below Decatur, Ala. As a rule, flood crests have not risen greatly between Chattanooga and Decatur, but the Elk, Paint Rock, Flint, and Sequatchie Rivers drain considerable areas which under some conditions of rainfall and run-off might increase the flood flows substantially between Chattanooga and Wheeler Dam. The drainage area between Wheeler Dam and Florence is comparatively small, so that floods recorded at Florence are more indicative of the maximum to be expected at Wheeler Dam than the floods recorded at Chattanooga or Decatur. The maximum recorded flood at Florence in March 1897 was estimated to have been about 470,000 cubic feet per second.

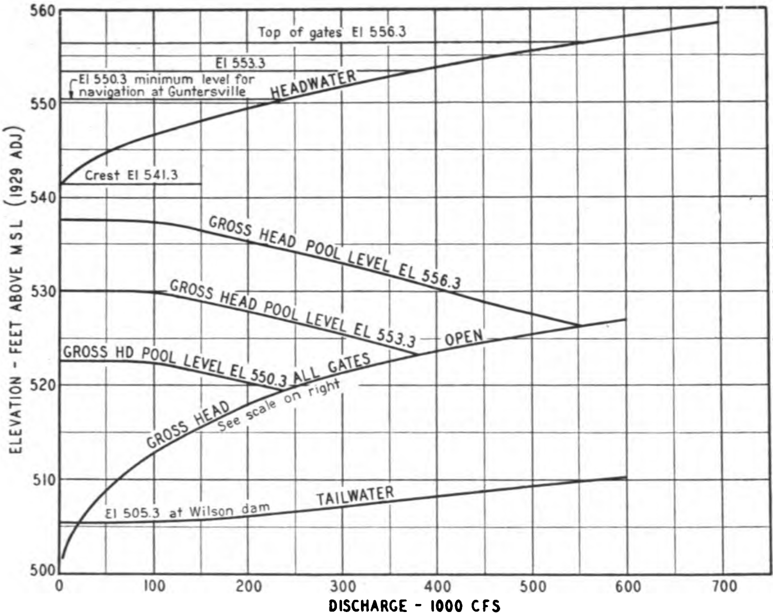


FIGURE 8.—*Wheeler headwater, tailwater, and gross head curves.*

The spillway at Wheeler Dam was designed for a flow of 687,000 cubic feet per second with a depth of 17 feet above the crest. This flow is approximately 50 percent more than the maximum recorded flood in the lower basin.

TAILWATER ELEVATIONS

The tailwater elevation fluctuates with the Wilson headwater level and with varying flows into Wilson pool, because Wheeler Dam is located at the upper end of Wilson pool. The difference in water surface elevation between the two ends of Wilson pool is the small slope required to carry the flow through the pool. At present, the maximum regulation range at Wilson is from design flood at elevation 506 to the minimum pool elevation 503, although it will be drawn down to the latter elevation only when the flow is large enough to develop a backwater depth at the Wheeler lock sufficient for navigation requirements. About 5 feet is the maximum variation in tailwater elevation. Wilson pool may be raised permanently about 2.5 feet in the future, and if this is done, it also will affect the prevailing tailwater elevation at Wheeler.

The tailwater rating curve is based on proportional gage readings made by the United States Army Engineers at Florence and lock No. 2 (Wilson) for intermittent periods from 1926 to 1932 and flows calculated by the Authority for Wheeler and Wilson in 1934 and 1935. Beyond 340,000 cubic feet per second the extension was based on cross-sectional areas and proportional velocities.

A backwater effect of about five feet is developed for a flow of 600,000 cubic feet per second through Wilson pool. This will make the tailwater at the Wheeler site during extreme flood stages about elevation 510. The capacity of the Wilson spillway with the pool at normal elevation is approximately 630,000 cubic feet per second. The low tailwater stages at Wheeler during high flows are due to the unusually large cross-sectional areas prevailing through most of Wilson pool.

HEADWATER ELEVATIONS

The lowest water surface above a dam must be adequate to provide navigation to a favorable site for the next upstream development. The highest water surface should be that which would provide the maximum amount of storage for flood control and low-water releases consistent with the expense of obtaining flowage rights and providing protection from inundation to existing developments along the river. In the Wheeler Basin the most important limiting factor was the city of Decatur at which not only property within the city itself but highway and railroad lines approaching and passing through the city had to be considered. Pool levels from elevation 544 to elevation 565 were investigated. After a study of all factors involved in the multiple use of the reservoir, the top of the spillway control gates was set at elevation 556 and the dam was designed to provide a spillway capacity which would limit the headwater to elevation 558 with the spillway gates open and discharging 687,000 cubic feet per second. A minimum reservoir level at elevation 550 will provide full depth navigation to the site of the Guntersville Dam when the river flow is at a minimum following a nominal amount of dredging immediately below Guntersville Dam.

Flowage costs increase rapidly with increasing pool levels, both in actual acreage required and in direct property damage at Decatur. For various pool elevations, the flowage costs for Wheeler Dam were estimated to be:

Pool level, Wheeler Reservoir	Flowage (acres)	Cost at $80 per acre	Estimated cost of protection, city of Decatur	Total
544	25, 000	$2, 000, 000	$59, 700	$2, 059, 700
551	42, 000	3, 360, 000	200, 450	3, 560, 450
555	60, 000	4, 800, 000	423, 100	5, 223, 100
561	69, 000	5, 520, 000	2, 206, 600	7, 726, 600
565	84, 000	6, 720, 000	3, 404, 500	10, 124, 500

The investigation of damages at Decatur arising from the higher water levels, and the fact that the Guntersville site (mile 349) appeared to be the better upstream dam site, showed definitely that a maximum controlled surcharge level of approximately 556 was as high as was feasible. This fixed the reasonable upper limit of pool elevation.

A further study was undertaken to determine the most favorable balance between costs and benefits provided by the Wheeler project. These costs and benefits were estimated for different levels of headwater and draw-down. It was assumed in all cases that the Wheeler project must provide a 9-foot navigation channel to the Guntersville site. With the lower levels for Wheeler, the cost of a low navigation dam needed to supply this navigation requirement was added to the project cost. Studies of this reach of river indicated that the navigation channel could be provided most economically by a dam at the Wheeler site built to elevation 550. The estimated cost of this single-purpose dam was taken as a measure of the value to navigation of the multipurpose development. The value of controlled flood storage was assumed at $20 per acre-foot. The value of the project for power was determined by capitalizing the estimated annual net revenue from power generation. In the estimate of power from Wheeler at pool elevations from 562 to 556, Wheeler was given full credit for the increase in head, although that increase would reduce the effective head at Guntersville Dam but would make very little difference in the system's power output after construction of Guntersville Dam had been completed. This study indicated that a controlled surcharge elevation of 556 with draw-down to elevation 550 was most desirable. Table 1 gives a summary of this study.

It was also deemed advisable to provide for lowering the headwater elevation to 548 in advance of floods. This would add to the value of the project for flood control and would not reduce the value of the project to navigation since, during the time when the pool was lowered, the river flow upstream would be ample to develop adequate navigable depths in the upper pool.

Backwater from Wheeler Dam will be effective in raising the tailwater elevation (and reducing the head) at Guntersville Dam for all discharges below about 250,000 cubic feet per second. Above that flow, Wheeler Dam will have no influence on the effective head at Guntersville. Figure 9 shows backwater curves for Wheeler pool at elevations 558, 556, 550, and under natural conditions.

TABLE 1.—*Estimated costs and benefits for various pool elevations*

	Assumed pool elevations—From surcharge to minimum for low-water releases					
	562–556	556–550	[1]550–550	550–544	[1]544–544	544–538
	Estimated costs in thousands of dollars					
A. Navigation lock	$1,921	$1,734	$1,547	$1,547	$1,370	$1,370
B. Dam, spillway, powerhouse, and reservoir	37,989	30,128	24,967	25,253	21,968	22,267
C. Navigation dams where required [2]	None	None	None	[3] 5,519	[3] 5,519	[4] 10,431
Total estimated cost	39,910	31,862	26,514	32,319	28,857	34,068
	Estimated benefits in thousands of dollars					
A. Navigation—Cost of the most economical provision for a 9-foot channel to Guntersville Dam	$21,541	$21,541	$21,541	$21,541	$21,541	$21,541
B. Flood control—At assumed value of $20 per acre-foot between levels indicated	13,700	7,100	None	5,000	None	3,500
C. Power—Equivalent capital value of plant	41,724	37,647	32,503	36,189	31,893	34,006
Total estimated benefits	76,965	66,288	54,044	62,730	53,434	59,047
	Value ratio					
Ratio of total benefits to total costs	1.93	2.08	2.04	1.94	1.85	1.73

[1] Constant pool level.
[2] The cost of additional navigation dams when required to provide navigation to the Guntersville Dam site is assumed as a project cost.
[3] Indian Creek Bar project.
[4] Milton Bluff project.

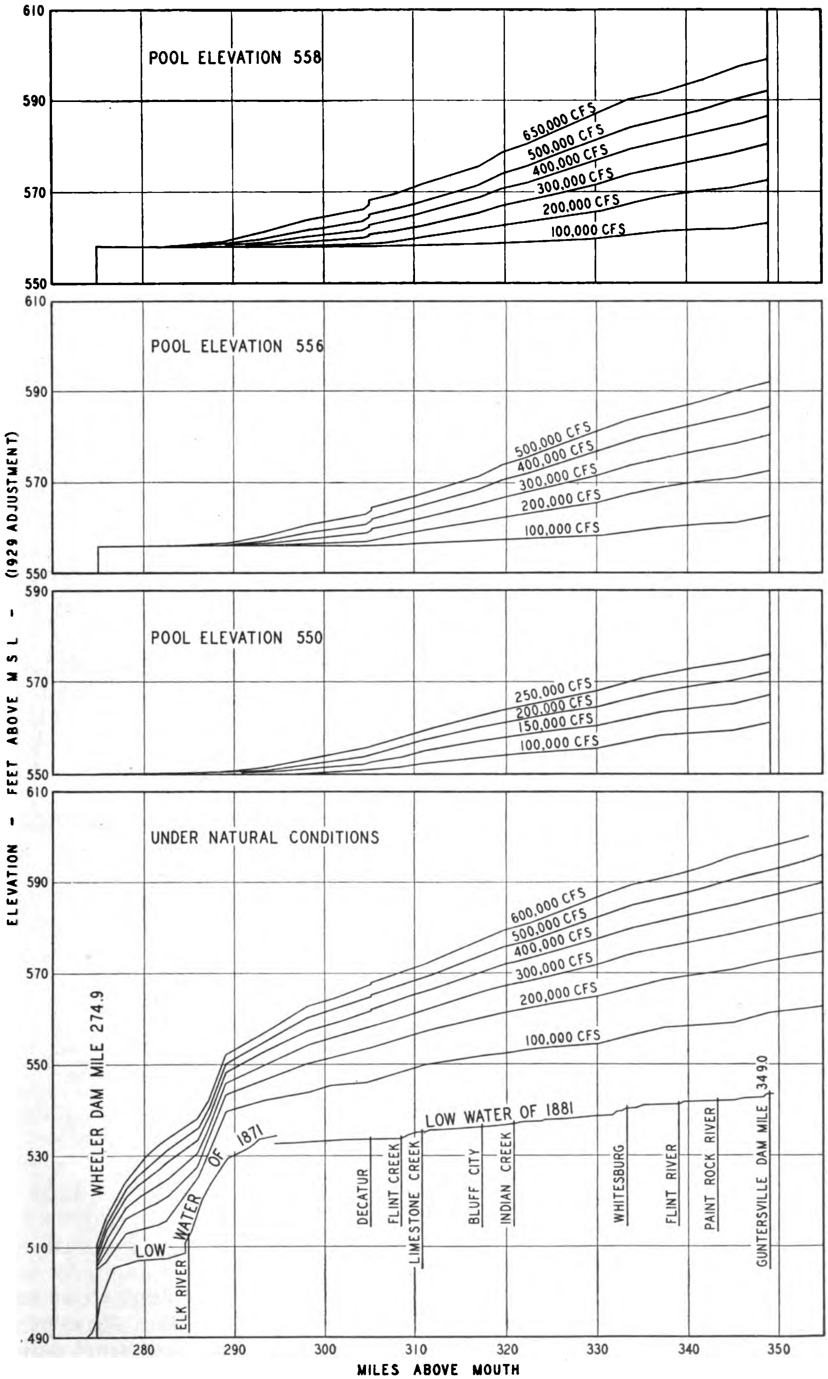


FIGURE 9.—*Tennessee River backwater curves above Wheeler Dam site.*

FLOOD CONTROL

Flood-control operations at Wheeler Dam are of particular value to supplement the Pickwick Landing and Kentucky Reservoirs in their operation for the reduction of Ohio and Mississippi River floods. Each of the Tennessee River projects will be able to retain a portion

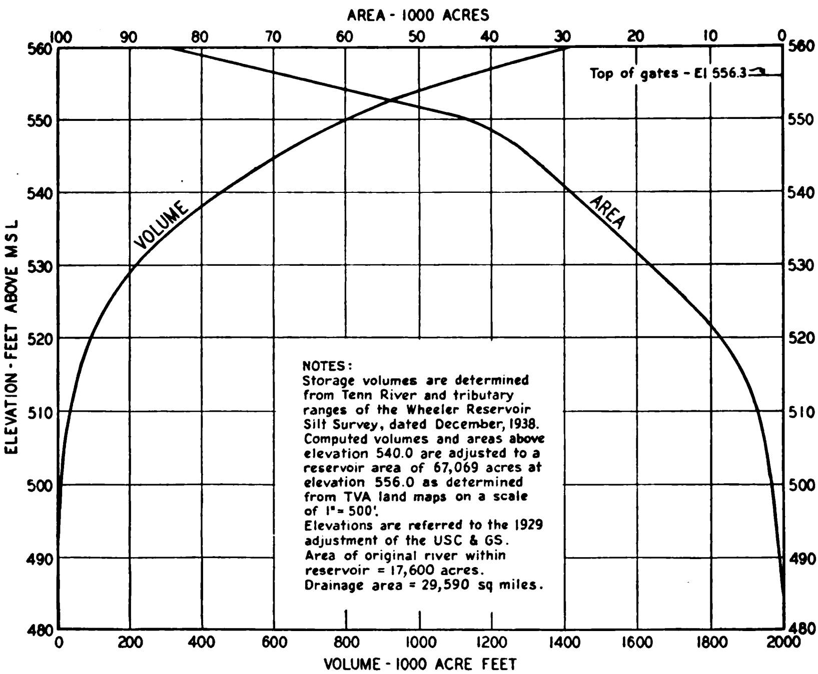


FIGURE 10.—*Wheeler Reservoir areas and volumes* (*see table* 2).

TABLE 2.—*Wheeler Reservoir areas and volumes* (*see fig. 10*)

Elevation	Area in 1,000 acres	Volume in 1,000 acre-feet	Elevation	Area in 1,000 acres	Volume in 1,000 acre-feet
560.0	84.2	1,433.3	550.0	43.0	802.9
559.0	79.8	1,351.3	549.0	40.6	761.3
558.0	75.6	1,273.6	548.0	38.7	721.8
557.0	71.3	1,200.2	547.0	37.2	683.9
556.3	68.3	1,150.4	546.0	35.9	647.4
556.0	67.1	1,131.0	545.0	34.8	612.1
555.0	62.9	1,066.0	540.0	29.0	452.8
554.0	58.8	1,005.2	530.0	18.0	217.3
553.0	54.6	948.5	520.0	8.4	86.9
552.0	50.6	895.9	510.0	3.6	31.1
551.0	46.5	847.4	500.0	1.8	5.3

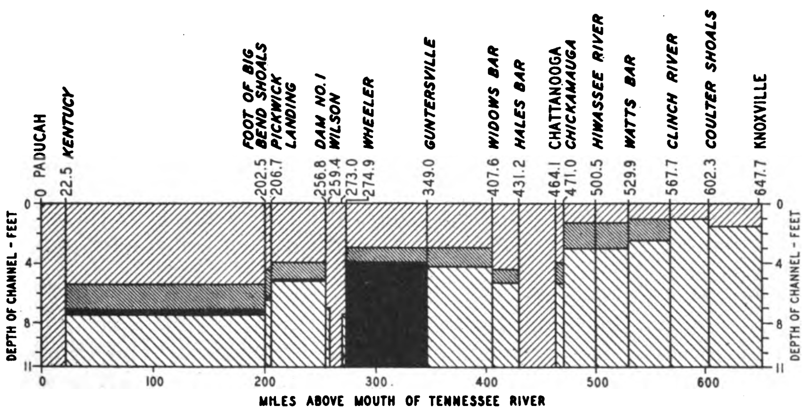
of the discharge during the crest of a flood; and the retention of such water, which under natural conditions might have been superposed on the peaks of Mississippi River floods, will be of distinct value.

The reservoir volume from the assumed draw-down level, elevation 548, to the elevation of the top of the gates, elevation 556.3, is about 429,000 acre-feet, determined on the basis of flat pool levels. Wheeler

Reservoir must be operated with proper regard to needs at Florence and points below as well as Decatur and points above. The clearing of trees and other obstructions in the flood plain in the lower part of Wheeler Reservoir tends to flatten the flood profiles. This, together with the retention of floodwater by the reservoirs above, reduces flood stages at Decatur in very large floods below those which have occurred under natural conditions.

NAVIGATION IMPROVEMENTS

After the completion of Wilson Reservoir, the reach in the middle section of the river that was most critical for navigation lay between the head of Wilson pool and Widows Bar Dam and was logically



LEGEND

- CONTROLLING CHANNEL DEPTHS PRIOR TO TVA
- INCREASE IN CONTROLLING DEPTHS DUE TO NORRIS RELEASES BEFORE CANALIZATION
- INCREASE IN CONTROLLING DEPTHS DUE TO WHEELER RESERVOIR AND WHEELER RELEASES BEFORE CANALIZATION
- DEFICIENCY IN CONTROLLING DEPTHS IMMEDIATELY AFTER COMPLETION OF NORRIS AND WHEELER

Controlling depths greater than those shown were secured in some reaches for brief periods to meet special navigation requirements during the low water season by additional releases from Norris and Wheeler.

NOTE:
Dimensions indicate minimum depths below water surface.
11' depths provide for 9' draft

REACH OF RIVER IN MILES	CONTROLLING DEPTH - FEET		
	1933	AFTER NORRIS	AFTER NORRIS AND WHEELER
0 TO 22.5	11.0	11.0	11.0
22.5 TO 202.5	5.5	7.2	7.5
202.5 TO 206.7	4.5	6.2	6.5
206.7 TO 256.8	4.0	5.1	5.3
256.8 TO 259.4	7.0	7.0	7.0
259.4 TO 273.0	11.0	11.0	11.0
273.0 TO 274.9	7.5	7.5	7.5
274.9 TO 349.0	3.0	4.0	11.0
349.0 TO 407.6	3.0	4.2	4.2
407.6 TO 431.2	4.5	5.3	5.3
431.2 TO 464.1	11.0	11.0	11.0
464.1 TO 471.0	4.0	5.3	5.3
471.0 TO 529.9	1.3	3.0	3.0
529.9 TO 567.7	1.0	2.5	2.5

FIGURE 11.—*Navigable depths, Tennessee River before and immediately after construction of Wheeler project.*

the next step in the improvement of navigation on the river. Within this stretch, the lower end between Wilson pool and the head of the Elk River shoals, mile 287.1, provided only 1 foot of depth in the river channel. The old Muscle Shoals and Elk River shoals canals bypassed that critical reach but were entirely inadequate for modern barge traffic. In the remainder of the river between the mouth and Chattanooga, the limiting low-water depth was 3 feet and occurred in the 50-mile reach above Guntersville, Ala. Under these conditions it was obvious that the Wheeler Dam and lock would provide the most effective improvement by permitting slack water navigation over the upper shoals.

The proposed scheme of development of navigation in the main river included the construction of Wheeler, Pickwick Landing, and Guntersville Dams; the execution of minor improvements at the Florence Canal; and some channel improvements to provide a 9-foot navigable depth from Pickwick Landing Dam to the upper end of Hales Bar pool. Below Pickwick Landing the normal low-water depth was 4.5 feet, but regulation of low-water flows by Norris and other upstream projects will increase this depth to 7 feet during most of the year. Canalization of the river between the upper end of the pool of Dam 52 on the Ohio River and Pickwick Landing will be provided by the Kentucky project. Chickamauga, Watts Bar, and Coulter Shoals projects, with some type of improvement for raising Hales Bar pool, will extend the 9-foot channel to Knoxville.

In addition to its function as an essential link in the chain of dams providing navigation on the main river, Wheeler Dam backs water up the Elk River, providing a 6-foot channel to mile 28, with the reservoir at elevation 550, and to mile 30 with the reservoir at elevation 555. This facility is of little commercial value at present, but the construction of a dam with pool elevation 568 at mile 28 on the Elk River would give 6-foot navigation on that river to the mouth of Richland Creek to serve operations in the rich phosphate deposits in that area.

Barge terminal facilities located in the Wheeler pool are in initial stages of development at Decatur, Ala., which is a central and convenient location for a river-highway-rail terminal. The comparatively stable water elevation in Wheeler pool is of great advantage in the practical development and operation of this terminal.

POWER

The Wheeler Reservoir contains a large volume for flood and power regulation. The gross head varies slightly through the entire range of stream flows, owing to the small backwater effect of Wilson Dam. The head at normal reservoir level is 50 feet, but may vary from 40 to 53 feet. This condition makes Wheeler particularly useful during periods of high flow, when some of the other plants lose their generating ability, because of the serious reduction of head by rising tailwater.

The plant is more valuable as a unit in an integrated system than it would be if operated independently. The increase in minimum flow from low-water regulation by upstream plants permits greater power production at Wheeler, whereas the use of Wheeler storage through downstream plants adds to their production. Installed capacity for production of power at main-river plants during periods of high flows permits the storing of the high tributary flows in the tributary reservoirs. This stored water represents potential system energy to be generated in the dry season. The power potentiality of the Tennessee Valley Authority system is limited by the requirement to operate all reservoirs for maximum navigation and flood control benefits. Fluctuating the water level in reservoirs for malaria control constitutes another limiting factor in the operation of the power system under certain conditions of stream flow.

Peak capacity should not be distributed evenly over the system, but should be concentrated at those plants where incremental cost of generating capacity is low, where reservoir volumes are adequate to meet peak demands for power, and where transmission lines to principal load centers would be short. In this system, however, the tributary storage plants which are most useful for peaking because of their high heads and ample storage are a long distance from the down-river plants and the down-river communities. Therefore, some peak capacity is desirable in the lower river plants.

Wilson Dam, with a normal operating head of about 92 feet, is adapted for use as a peak plant, but lacks sufficient pondage to permit the required water regulation. Wheeler Dam, just upstream, has ample pondage for peak operation. The small pondage at Wilson dictates tandem operation of the Wheeler and Wilson plants for full and economical water use and thus makes it necessary to provide generating equipment at Wheeler to help generate the peak requirements of both plants. Provision for an ultimate installation of 444,000 kilowatts had already been made at Wilson, and it was decided to provide for approximately 256,000 kilowatts as the ultimate Wheeler capacity. This capacity bears approximately the same ratio to the ultimate capacity of Wilson as the average head at Wheeler does to the head at Wilson. Other considerations governing the amount of installed capacity at Wheeler were its low cost compared with other main-river plants, possibility of load growth in the western portion of the TVA area, and freedom from capacity reduction during floods. The provisions for generating capacity at the Wheeler plant are ample to utilize at a low capacity factor the greatest regulated flow which could be expected after a fairly complete development of the Tennessee River and tributaries.

GROUND-WATER CHANGES

Changes in ground-water levels because of the filling of Wheeler Reservoir are more significant than similar changes in any other Tennessee Basin reservoir. This is the result of the presence of innumerable sinkholes and depressions developed on the surface of the very pure and soluble Warsaw limestone underlying the reservoir; the presence of major joint systems in the rock which allow comparatively free circulation of underground water; and the low elevation of the land bordering the river upon which the sinks occur. The lowness of the land permits the water in the sinks to fluctuate more or less in harmony with the rise and fall of the water table. Water surfaces in the sinks never fall lower than the upper surface of the water table.

This situation was observed during the early period of construction. In sinks which contained water permanently and which were situated near the reservoir edge, the water surface would rise proportionately with the filling of the reservoir and would thereafter maintain a level slightly higher than the reservoir due to the ground-water gradient. Moreover, new permanent ponds appeared in some of the existing depressions which tended to cause increased damage to property and to complicate the malaria problem throughout the reservoir area.

The rock formations in the reservoir itself and immediately adjacent to it are the Fort Payne and overlying Warsaw limestone.

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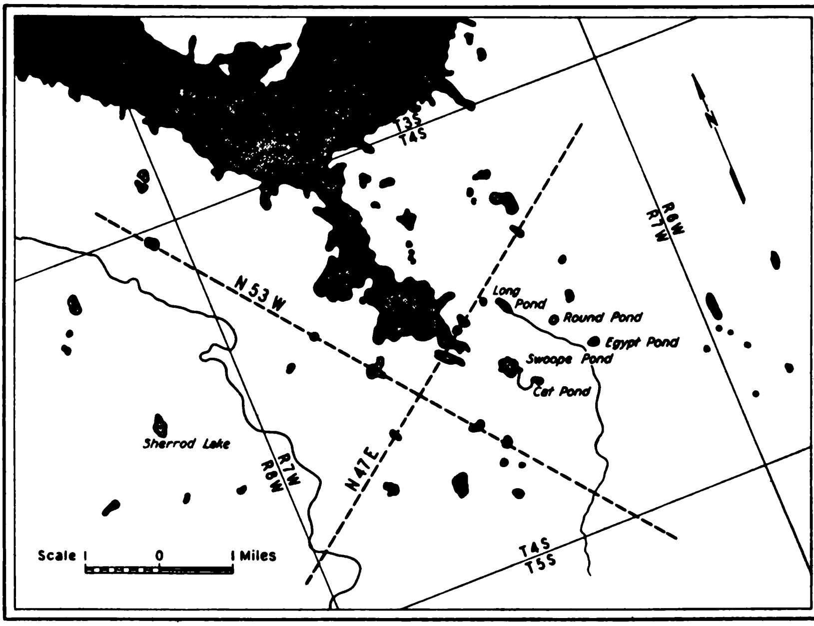

FIGURE 12.—*Joint and sinkhole distribution in Spring Creek area.*

Long exposure to weathering, particularly by underground water containing dilute amounts of carbonic and humic acids, has produced caves, open joints, and seams. Water percolates with relative ease and rapidity through these open channels. Due to such high permeability, the ground-water gradient from points a short distance from the reservoir to the river is very flat; hence fluctuation of the sink ponds is synchronous with that of the river. This situation prompted a thorough investigation of the effects of impounding water in the reservoir.

Sinkholes are the most prominent feature of ground-water changes in the reservoir area. Many of these are formed by the collapse of a cave roof which has become too weak to support the overlying rock and residual material. Another type, however, is represented by those depressions which are formed by the gradual underground erosion of soil overburden into underground streams flowing along joint cavities until the earth itself falls in and is washed away. These are by far the most numerous. Since the most prominent sink development occurs in the vicinity of Spring Creek near Courtland on the south side of the river, most of the observations were made in that area.

Joints in this region are more or less vertical cracks in the rock. They trend in very definite directions and usually consist of several sets. In the Spring Creek area there are two major sets, one striking N. 53° W., and the other N. 47° E., almost at right angles to one another. These openings determine in general the direction of the underground flow, and the shape and distribution of sinks follow the major sets. They are oriented in lines along these joint sets with remarkable consistency, forming a crisscross pattern upon the surface of the limestone bordering the reservoir. The larger sinks appear to occur at an intersection of two major joint sets.

It should not be construed, however, that each prominent line of sinks represents a single joint, for joints may be spaced from a few feet to as much as several hundred feet apart. It is probable that the most prominent lines of solution occur where joints are spaced closest together. The courses of larger streams of the area are also influenced by the joint systems.

PRELIMINARY DESIGN AND PROJECT LAYOUTS

The United States Army Engineers recommended the construction of a dam at mile 274.9 on the Tennessee River—the location of

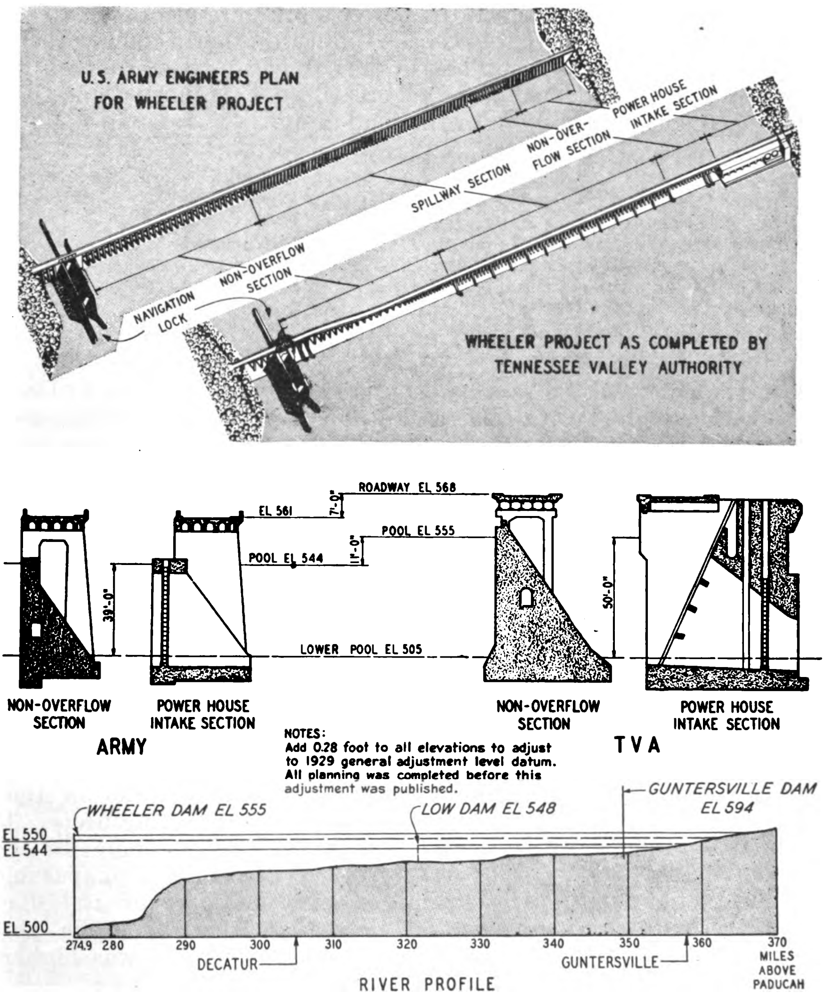

Figure 13.—*Comparison of U. S. Army Engineers and TVA designs.*

the present Wheeler Dam—following their study[3] of the Tennessee River and tributaries completed in 1930. The preliminary design submitted in that report was changed somewhat upon further study and the latest plan adopted by the United States Army Engineers was very similar to the one adopted as the final design of the Authority.

A comparison of the project proposed by the United States Army Engineers and the project actually constructed by the Tennessee Valley Authority is shown in figure 13. The United States Army Engineers' plan contemplated a short bulkhead section between the right bank and the lock, a lock 60 by 360 feet with a lift of 39 feet from the lower pool at elevation 505 to upper pool at elevation 544, a short nonoverflow section, a spillway section, a second nonoverflow section, a trash chute, a powerhouse section with the powerhouse intakes provided with stop logs to close the openings until the power units were installed.

The studies made by the Authority indicated that a dam with the following requirements would best fit the navigation, flood control, and power program as outlined in the act creating the Authority, and these requirements[4] were adopted for the final design:

Location—Miles above mouth of Tennessee River	274. 9.
Type	Straight concrete gravity.
Maximum high water (for structure design)	Elevation 558. 3.[5]
Spillway crest	Elevation 541. 3.[5]
Top of spillway gate	Elevation 556.3.[5]
Design flood over spillway	687,000 cubic feet per second.
Number of power units (ultimate)	8.
Number of power units (initial)	2.
Total capacity (ultimate)	256,000 kilowatts.
Assumed power factor	0. 9.

These requirements necessitated substantial protective work at Decatur as well as highway, railroad, and utility adjustments. Other clearing and draining was necessary on low lands adjacent to the reservoir.

After considerable study, it was decided to construct a highway across the top of the dam to link the existing highways on both sides of the river and make the railroad on the south side more available to shippers on the north side. This crossing will also be advantageous if the area around Wheeler develops into an industrial center.

PRELIMINARY COST ESTIMATES

The United States Army Engineers published, in their 1930 report[6] on the Tennessee River and tributaries, a preliminary cost estimate of a project to be placed at the present location of Wheeler Dam. This project, shown in figure 14, was for a development somewhat different from that actually constructed by the Authority or that finally recommended by the United States Army Engineers. It served in a broad sense, however, as a preliminary estimate for the project constructed by the Authority if certain noncomparable items are eliminated. The actual construction cost of the dam and powerhouse was, in general, less than that given in this preliminary cost estimate, but the cost of the reservoir was higher.

[3] H. Doc. No. 328, 71st Cong., 2d sess.
[4] See appendix A for details of features actually constructed.
[5] See page 221.
[6] H. Doc. No. 328, 71st Cong., 2d sess.

The cost of the navigation lock slightly exceeded the original preliminary estimate.

The chief difference in the two plans is that the Army Engineers' earlier design provided for the installation of 18 units of 19,600 kilowatts each while the dam built by the Authority contains an initial installation of two 32,400-kilowatt units with provision for the future installation of six additional similar units. The dam proposed by the Army Engineers, although for the same spillway crest

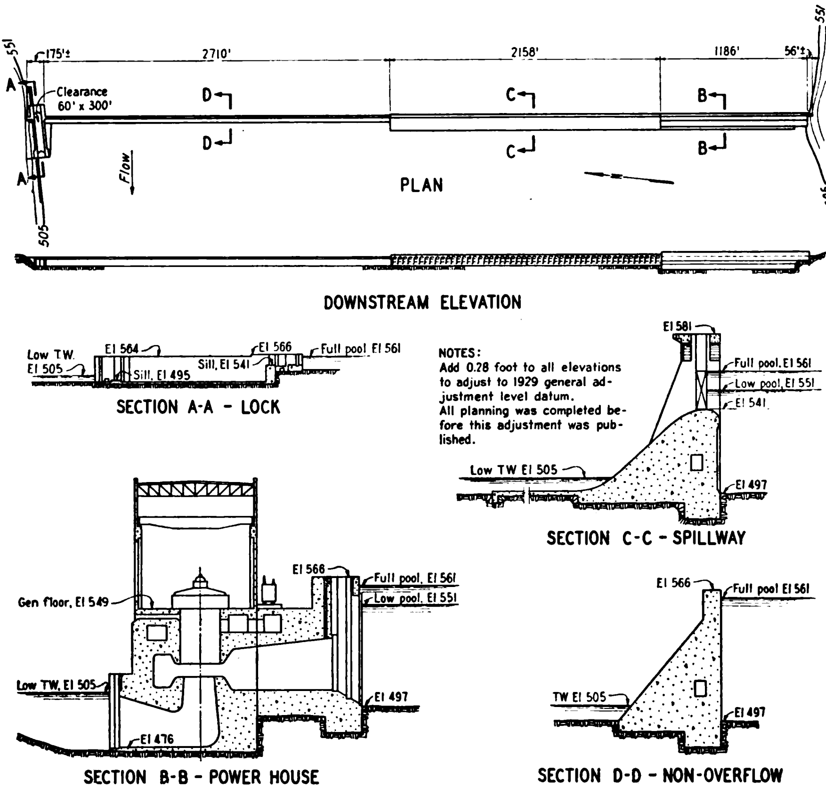

FIGURE 14.—*U. S. Army Engineers' design[7] on which the preliminary cost estimates were based.*

(elevation 541), was to have its gates and pool elevation only 5 feet higher. The Army estimate was based on an enclosed powerhouse, whereas an outdoor one was constructed. The Army estimate did not include a roadway across the dam; one was included in the TVA design. A summary of United States Army Engineers' preliminary cost estimate is given in table 3, the details of which are shown in their report.[8]

[7] *Ibid.*, pl. 101, pt. 2.
[8] H. Doc. No. 328, 71st Cong., 2d sess.

TABLE 3.—*Preliminary cost estimate*[1] *for the Wheeler project as proposed by the United States Army Engineers*

Dam and spillway (Including cofferdams; excavation; foundation preparation; concrete; gates, machinery, and equipment; and power and lighting.)	$7,859,900
Powerhouse (Including cofferdams; excavation; foundation preparation; concrete; building; turbines and governors; generators and exciters; switchboard and wiring; transformers and high-tension switching; auxiliaries; and gates, racks, and equipment.)	17,526,460
Reservoir (Including land; clearing; buildings; and highway, railroad, and cemetery relocation.)	3,851,850
Lock (Including cofferdams; earth and rock excavation; foundation preparation; concrete; backfilling; earth and rock dredging; valves and machinery; gates and machinery; houses and land; and miscellaneous.)	1,314,800
General (Including construction camp, plant, and railroad; permanent railroad and highway; grading and planting; garage; office and clerical; superintendence and layout; engineering; and legal.)	5,555,000
Net total cost	36,108,010
Contingencies, taxes, insurance, and interest during construction	8,742,090
Grand total (installation based on 50 percent load factor)	44,850,100
Less cost of lock (including overhead)	1,709,240
Grand total (excluding navigation)	43,140,860
Grand total (installation based on 75 percent load factor)	37,099,250
Grand total (installation based on 100 percent load factor)	33,679,260

[1] Condensed from the preliminary cost estimate made by the U. S. Army Engineers in their report on the Tennessee River and Tributaries, pp. 256–257, H. Doc. No. 328, 71st Cong., 2d sess.

SOCIAL AND ECONOMIC STUDIES

Preliminary investigations of the engineering and geologic aspects of the project had been made several years prior to the passage of the TVA Act because of the prominence of the site. Beyond ascertaining the probable value of the land to be acquired for the completed Wheeler project, no social or economic studies had been made relative to family removal, access to isolated areas, or the effect of governmental purchase of real property on local government finance. Although these studies are normally a part of preliminary investigations, time was not available for such investigations prior to construction, but these were started as soon as possible and served as the basis for treatment[9] of these problems.

Effect of governmental purchase of real property on local government finance.

A study was made to determine the direct and immediate effect which the acquisition of the reservoir property and its removal from the tax base might be expected to have on county finances. This study was restricted to county finances because there are no other local taxing jurisdictions within the area considered, and the amount of State revenues involved was relatively small.

[9] See ch. 6 for the treatment of these problems.

Approximately 110,000 acres of land, situated in Lauderdale, Limestone, Madison, Lawrence, Morgan, and Marshall Counties of northern Alabama, were purchased for the project. A detailed study of individual tracts indicates that, for 1935, the land which had been purchased had an assessed value of $1,438,485, or 2.1 percent of the six-county tax base. The exemption of this property from the tax base in 1934, the last year this property was taxed, would have reduced the property tax of the six counties and their school districts by $20,160 and State taxes by $9,351.

The other immediate effects on the tax base are less significant. The small amount of public utility property in the flooded area has been relocated, but taxable values have not been reduced. Any change in personalty assessments following the removal of approximately 90 families from the 6 counties has been offset by increases in personal property in the hands of county residents who sold most of the 1,200 real estate tracts to the Authority. The population shift among the 6 counties, and especially among local taxing districts, has undoubtedly made some important modifications in the relative personalty tax basis.

On the basis of the 1935 assessed value, the computed local revenue loss in 1934 for the six counties would have amounted to 0.8 percent. This reduction is less than the percentage loss in tax base, because property tax revenues provided considerably less than half of the local revenues in 1934. The other important sources were State aid, Federal aid, and local license taxes. Federal ownership of reservoir land will have no appreciable effect on the amount of such revenues. State aid is apportioned according to area, population, or source of collection, or is divided equally among counties. Licenses and county gasoline taxes, which are the other important local taxes, are derived largely from the cities and towns and will not be adversely affected by the project.

An examination by counties shows that the revenues of Limestone County will be reduced by a greater percentage than those of any of the other five. In this country, the real-estate purchase comprises 11 percent of the county area, represents 6 percent of the 1935 property assessment, and yielded 2 percent of the total county revenues for 1934. Morgan and Lawrence Counties each lost an amount of its property tax base sufficient to yield approximately 1 percent of the 1934 revenue. In the other counties, the losses are less than 1 percent of the 1935 property tax basis and would have caused revenue reductions in 1934 of only 0.3 percent.

Relocation of people living in flooded area.

The original act creating the Authority did not specifically cover population adjustment activities. As reservoir construction progressed and the complexity of the family relocation problem became manifest, it became apparent that a procedure would have to be adopted utilizing, as far as possible, the services of agencies already in the field. Therefore, when the original act was amended in 1935, provision[10] was made for the cooperation of the Authority with

[10] See sec. 4, par. 1, Tennessee Valley Authority Act, Public, No. 17, 73d Cong., and amended by Public, No. 412, 74th Cong.

Federal, State, and local agencies in assisting the readjustment of the population displaced by the construction activities. The Authority was not empowered to give direct benefits to reservoir families but was allowed, as a strictly service agency, to act as a liaison between the families and the various outside agencies interested in the problem.

Early in 1935, representatives of the Authority interviewed the 835 families affected by the flooding of the Wheeler reservoir land. The interviews were made:

1. To secure information that would be the basis for the assistance to be given to individual families in solving their relocation problems;
2. To provide the basis for future studies which would determine the relative conditions of the same families after resettlement; and
3. To gather basic data of a definite area which would be available for use in the planning activities of the Authority.

These interviews revealed that about 87 percent of the 835 families in the reservoir were engaged in farming. Those not engaged in farming ranged from comparatively well-to-do retired farmers to squatters on the relief rolls. One of the most striking facts brought out by the survey was the small number of owner-operators living in the area. Of those engaged in farming, 93 percent were either tenants or farm laborers. The lowest level in the economic scale of all the farm families was represented by the farm laborers who made up 14 percent of the total farm families. The tenant group was divided into sharecroppers, share tenants, and cash tenants. Their comparative economic status is in that order, with the cash tenants highest. Nearly half of the families in the area were Negroes. The median size of the white farm families was 5.6 members as compared to 4.5 for Negro families. The use of child labor in the growing of cotton is generally considered to make a large family an economic asset.

In general, the educational attainments of the reservoir population were relatively low. More than 16 percent of all male heads of farm households never attended school, and an additional 28 percent did not complete the fourth grade. Seventy-eight percent of the children attending school during 1934–35 were below the normal grade for children of their ages.

The families moved often, but usually not very far. The median length of residence was 2.9 years for Negro families and 1.6 years for white families. Of the husbands of white farm families 68 percent were living in the county of birth as compared to 84 percent for Negroes.

The average farmhouse in this area contained three rooms. When the size of families is recalled, these data revealed an acutely crowded conditions in the homes surveyed. Most homes were frame construction and in poor repair. Only 3.3 percent of the families had their water supply in the house or on the porch. No farm homes had inside baths or sanitary facilities. Only one farm had electricity, and only one had telephone service. Fireplaces or stoves were used for heating. The median total value of household furniture amounted to $89 per white family and $59 per Negro family. Less than one-sixth of the farm families owned automobiles or trucks.

In general, cotton was the only market crop raised by farmers. Food and feed were raised only incidentally, although there was a sizeable acreage in corn, raised primarily as feed for work stock. Most farms were small, the median size being about 35 acres. The smallest farms of all the tenure classes were operated by the sharecroppers. Crops were harvested from 27 acres of land on the average farm in 1934. Here again the sharecroppers averaged less acreage than any other group. Over 86 percent of the farm families relied on cotton as their principal crop; a number of others raised it as a second crop. Seven acres of cotton were handled by the average farm family in 1934.

The value of farm machinery presents striking evidence of the extent to which cotton growing has resisted mechanization. A few sharecroppers owned no machinery of any kind. The median value of machinery owned by share tenants was $65; by cash tenants, $72; and by owner-operators, $96.

The median value of the livestock on all farms was only $171 as compared to the figure for the average farm in the United States, which in 1925 was about four times this amount. The ability of the region to live on its cash crop, cotton, had made livestock relatively scarce, although some cattle and hogs were owned by the majority of the farmers in each tenure class.

Principal farm expenditures were for seed, fertilizer, cotton ginning, feed, labor, taxes, and sometimes rent or interest on mortgages. Expenditures of sharecroppers were small because all their seed and half of their fertilizer, as well as their tools and work stock, were furnished. Only seven in this group had farm expenses exceeding $150 in 1934. The median total annual per capita expenditure for the sharecroppers was $18. On the other hand, expenditures by cash tenants were considerably higher than those of any other group. Whereas 17 percent of the share tenants and 33 percent of the owners spent more than $150 on the farm, 45 percent of the cash tenants exceeded this figure. The cash payment of rent was probably the chief factor in this difference.

The chief sources of farm cash income were from lint cotton, cottonseed, and rental and parity benefits from the Agricultural Adjustment Administration. In addition, some income was often obtained from the sale of corn, hay, hogs, poultry, dairy products, or truck products. The median total cash farm receipts in 1934 varied from $156 for sharecroppers to $465 for owner-operators.

In measuring the real income of families in a rural area, it is essential to determine the value of goods produced on the farm and consumed by the family. More than half of all the families raised and used products valued at between $50 and $200, whereas products valued at $300 or more were used by 17 percent. The median total value of products furnished by the farm and consumed by the family was $168 for all families.

An estimate was made of the total net real income for each farm family in 1934. This figure was obtained by adding the total cash income, the value of relief receipts in kind, and the total value of products furnished by the farm and consumed by the family, and subtracting from this the total farm expenditure. On this basis, about 49 percent of all the farm families had net real incomes between

$200 and $500. The median net real income for farm laborers was $258; for sharecroppers, $347; for share tenants, $556; for cash tenants, $531; for owner-operators, $655. The median net real income for all farm families in the area was $416.

Debts were owed by only 37 percent of the farm families, and the average amount owed by each family was about $143. In view of the low income, however, this indebtedness was burdensome to most of the families. Only 18 percent of the farm laborers had incurred indebtedness, the reason probably being that their economic condition was such as to give them little credit standing. The sharecropper owed two types of debts: (1) long-term debts, or those which run from year to year; and (2) short-term debts, or those yearly debts which form the foundation of the furnishing system. Since the latter are incidental to the rental arrangement, they were not technically considered as debts in this study. Almost 25 percent of the sharecroppers, however, had debts which continued from year to year. Debts were found to be most numerous among the share and cash tenants. Of the owner-operators, 38 percent had debts, which were in most cases secured by mortgages on their real or personal property.

Federal unemployment relief benefits were received by only 78 farm families during 1934, in spite of their low average income. The median total amount of the aid received per family, including relief wages, direct relief, and cash value of payments in kind, was $48.

Access to isolated areas.

The acquisition of reservoir land up to the normal taking line left several areas of various sizes which would be severed from the existing road system and to which access would have to be provided. Studies were made of each isolated area, and where the cost of access facilities exceeded the purchase price of the land in the area, it was recommended that such areas be purchased. Some isolated areas which were subject to severe soil erosion or were submarginal were recommended for purchase, even though not economically justified on the basis of cost of access versus cost of land. Many inaccessible areas were studied originally to determine the advisability of purchase. Those areas actually acquired are considered in chapter 6 of this report. Families in these isolated areas whose land was purchased were given the same consideration in regard to assistance and advice for relocating as the families in the flowage area.

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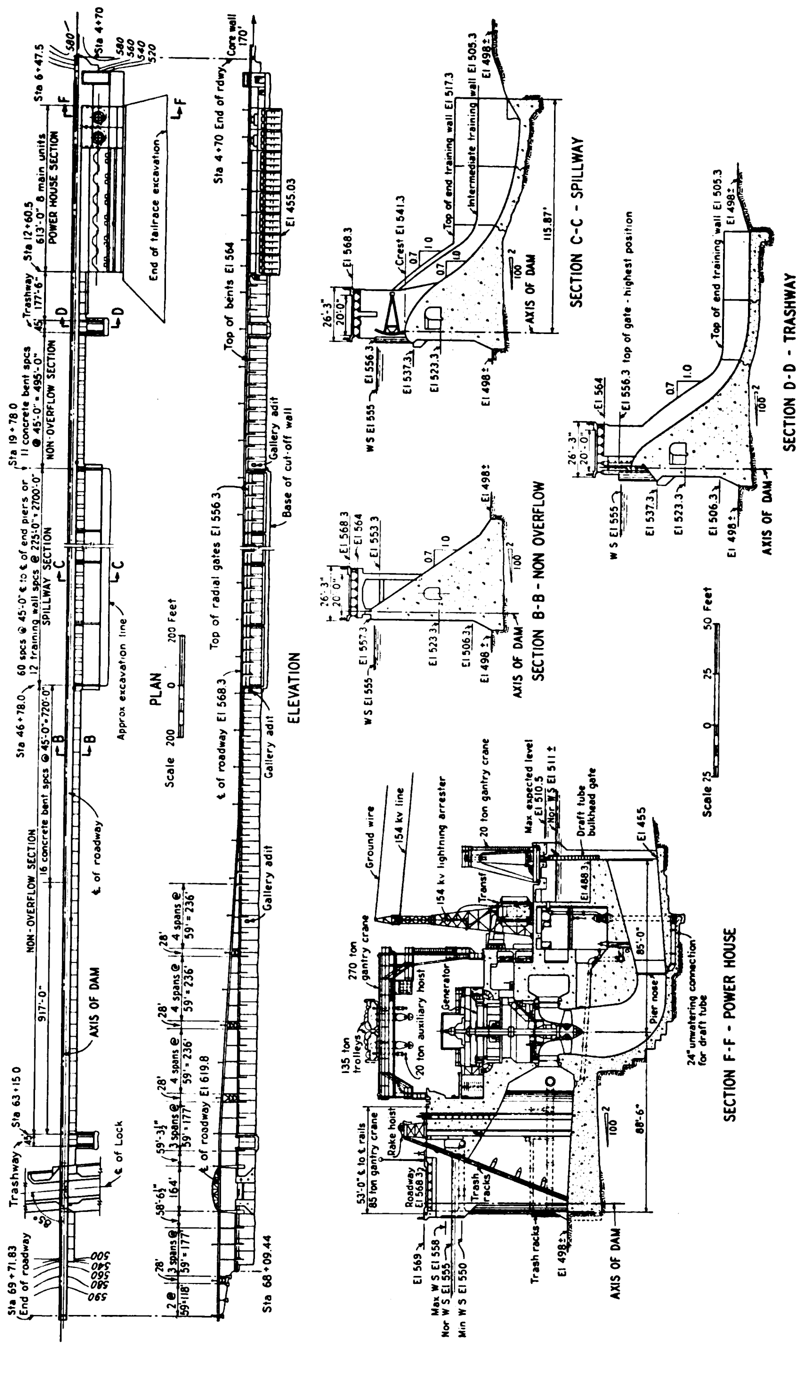

Figure 15.—*Plan, elevation, and sections.*

CHAPTER 3

DESIGN

The principal structures of this project consist of a dam, a navigation lock, a power plant, and a switchyard. The dam extends across the river in a straight north and south line, with the lock at the north end and the power plant at the south end. The overall length, including lock and power plant, measured from end to end of the roadway pavement over the top of the dam, is 6,502 feet. The dam proper, occupying 5,174 feet between the lock and powerhouse, consists of a series of nonoverflow, spillway, and trashway sections. Figure 15 shows the succession of these several parts of the dam.

The spillway section, 2,700 feet long, is approximately in the center of the river to secure the most satisfactory conditions for discharge of floodwater. This section is flanked by two nonoverflow sections, 1,756.2 and 717.5 feet long, each containing a 45-foot trashway section. The spillway is capable of discharging a flood of 687,000 cubic feet per second with the reservoir at elevation 558.3. In order that the energy from full gate discharge at low tailwater stages may be dissipated without damage to the foundation, an apron equipped with a series of toothlike diffuser sills along the downstream edge was installed below the spillway.

The navigation lock was placed near the north or right bank of the river because of better approach conditions and because the old Muscle Shoals Canal, which was a part of the navigation channel, is on that side of the river. The lock chamber is 60 feet wide and 360 feet long and provides a normal lift of 49 feet, or a maximum lift of 53 feet. The lock wall on the land side was located 200 feet from the north bank of the river in order to leave space for a future larger lock, with a 110- by 600-foot chamber. A concrete bulkhead section fills this 200-foot gap.

The power plant is near the south or left bank of the river and extends in a line parallel to the axis of the dam. It is composed of an intake section for a total ultimate installation of eight generating units, a complete powerhouse substructure for two initial units, a skeleton substructure for six future units at the north or river end, a service bay south of unit No. 1, and a control building nearest the south bank. The power station is the outdoor type. Two identical vertical-shaft generating units have been installed initially, each consisting of a propeller-type runner rated at 45,000 horsepower under a net head of 48 feet and a speed of 85.7 revolutions per minute, and a 36,000-kilovolt-ampere generator with all the necessary auxiliaries. Two additional units have recently been authorized, and should be completed during 1941.

Power from the generators at 13,800 volts is stepped up by the main transformers to 154,000 volts. The switchyard is on the shore, 500 feet downstream. Four outgoing transmission lines tie this station to the Wilson, Guntersville, and Norris projects, and to Columbia, Tenn. Provisions have been made for future extensions to the switchyard.

Since the power station is the outdoor type, the cranes are exposed, the generators are protected by removable metal housings, and the sundry auxiliaries that would ordinarily be distributed over a generator room floor are on the turbine operating floor and in the service bay. The service bay is between the powerhouse proper and the control building. It contains a workshop equipped with machine tools for the repair of equipment. The floor of the service bay is on the same level as the lower yard and is connected with the unloading deck by a standard gage railway track.

The control building is a five-story structure immediately south of the service bay. The top floor opens onto the roadway that crosses the dam and is largely devoted to public use. The control room containing the principal control boards is in the east end and is visible both from the lobby and the roadway. Operating offices are on the fourth floor, and the remainder of the building is used for the electrical and mechanical needs of the station.

A 20-foot roadway crosses over the top of the dam. Near the lock it rises and passes over the lock chamber with a clearance of 57 feet above maximum high water to permit the uninterrupted passage of river and highway traffic.

DAM

The type of dam best suited to the topography, the available materials, and the foundation conditions at this site was a straight, concrete, gravity section with an overflow spillway. All parts of the dam, being gravity section type, necessarily have many features in common. The nonoverflow, spillway, and trashway cross sections differ in so far as modifications had to be made to adapt these sections to the particular functions which they serve. An inspection gallery, with most of the floor at elevation 523.3, extends through the full length of the dam, from the north end of the powerhouse, under the lock, and to the extreme end of the north abutment wall. The entrances are at the extreme ends. Hoisting machinery for the spillway and trashway gates are housed in bays or niches in the inspection gallery.

STRUCTURAL FEATURES

To test the adequacy and safety of the final dam design, studies were made of stresses and stress distribution, loadings and other factors, such as earthquake effect, which might affect the stability of the structure. Although stress and stability studies[1] had been made by the United States Bureau of Reclamation, an independent check for some of the conditions of loading was made by the Authority. All data on stress and stability studies presented here apply to the finally adopted design.

[1] Houk, Ivan E., *Stress and Stability Studies for Wheeler Dam,* U. S. Bureau of Reclamation, Technical Memorandum No. 365.

For the design of the cross section the maximum levels of headwater and tailwater were taken at elevations 558.3 and 510.8 respectively, and the minimum tailwater was assumed at elevation 503.

The following constants were used in the stress and stability analyses for all sections of the dam [1]

1. Unit weight of water—62.5 pounds per cubic foot.
2. Unit weight of concrete—150 pounds per cubic foot.
3. Coefficient of friction of concrete on rock or concrete on concrete—0.65.
4. Ultimate strength of concrete in shear—400 pounds per square inch.

The following assumptions were made in the stress and stability analyses for all sections of the dam: [1]

1. Vertical stresses vary uniformly from upstream to downstream face of dam at all elevations.
2. Stresses at the faces of the dam are acting in directions parallel to the slopes of the faces and are equal to the calculated vertical stresses divided by the squares of the cosines of the angles between the faces and the vertical direction.
3. Uplift pressures vary uniformly from full reservoir pressure at the upstream face of the dam to zero pressure at the downstream face, or to full tailwater pressure at levels where the plane being analyzed is below the tailwater surface.
4. Uplift pressures, according to the above distribution, act over two-thirds of the horizontal area of the base and over two-thirds of the horizontal area of the concrete sections analyzed at elevations above the base, uplift pressures being assumed to act in the pores of the concrete, as well as along the plane of contact between the concrete and the foundation rock.
5. A maximum earthquake is assumed to have an acceleration equal to 0.1 of gravity, a period of vibration equal to one second, and a direction of vibration at right angles to the axis of the dam.
6. The maximum effect of the earthquake shock for empty reservoir occurs when the inertia force of the dam caused by the earthquake is acting in an upstream direction.
7. The maximum effect of the earthquake shock for full reservoir occurs when the inertia force of the dam and water caused by the earthquake is acting in a downstream direction.
8. All loads are assumed to be carried by gravity action.
9. Resistance to failure by shear at any elevation is assumed to be increased by the coefficient of friction times the summation of the vertical forces acting at the elevation.
10. The lowest foundation level assumed was elevation 485.3 in spillway, abutment, and trashway sections, and elevation 498.3 in power plant section.

Analyses of stress distribution, sliding factors, and factors of safety against failure in shear for both spillway and nonoverflow sections were made for the following assumed loads; [1]

1. Reservoir empty, with and without earthquake load.
2. Maximum flood conditions without earthquake, reservoir water surface at elevation 558; radial gates open; tailwater surface at elevation 512. [2]

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[1] Houk, Ivan E., *Stress and Stability Studies for Wheeler Dam*, U. S. Bureau of Reclamation, Technical Memorandum No. 365.

[2] 1912 datum; see p. 221 for explanation.

3. Normal full load operation with and without earthquake shock; reservoir water surface at top of radial gates in closed position elevation 555; tailwater surface at elevation 505.[2]

No analyses were made of the effect of sand and gravel deposits on internal stress conditions, inasmuch as such loads at either face would simply mean a slightly increased vertical load on the inclined portions of the face with no change in unit horizontal pressure or total horizontal force.

Nonoverflow section.—This is essentially a triangular-shaped gravity section as shown in figure 15. Its width at elevation 485.3, which level is considered the base for the purpose of stability analysis, is 59.60 feet. However, the foundation was actually excavated only to approximately elevation 492, about 6 feet below the original rock surface, with about a 2 percent upward slope from the upstream heel to the downstream toe. No heel or toe trenches were excavated.

Analyses for the nonoverflow sections of the dam indicate that the maximum stresses are relatively low in all cases. The maximum compressive stress, including that caused by the assumed maximum earthquake shock, when the reservoir is full, occurs at the downstream edge of the base and amounts only to 100 pounds per square inch. The maximum compressive stress, including that caused by the assumed maximum earthquake shock, when the reservoir is empty, occurs at the upstream edge of the base and amounts to only 82 pounds per square inch. With the reservoir empty, a negligible amount of tension, only 3 pounds per square inch, would occur at the downstream edge of the base in case of maximum earthquake load. Shear stresses are relatively low, amounting to only 41 pounds per square inch during earthquake shock. Sliding factors do not exceed 0.60 except when earthquake load is considered. For maximum earthquake load the sliding factor at the base is 0.75, but the corresponding shear-friction factor of safety is 18.6. Shear-friction factors of safety are somewhat higher at elevations above the base of the dam.

The principal structural data which controlled the final design of the nonoverflow section are (see fig. 15.):

1. Location of nonoverflow sections.
2. Top of the parapet wall in the nonoverflow section—elevation 560.3.
3. Top of the dam in the nonoverflow section—elevation 557.3.
4. Width of the top of the dam in the nonoverflow section—4.71 feet.
5. Alignment of the vertical portion of the upstream face of the dam is straight, and the plane is continuous through the spillway, trashway, and non-overflow sections.
6. Weight of the bridge—4,500 pounds per linear foot of dam.

Spillway section.—The general requirement for the design of the spillway was the passage of a maximum flood of 687,000 cubic feet per second. The maximum flood of record was 470,000 cubic feet per second at Florence, Ala., on March 19, 1897. To pass this design flood 60 spillway openings 40 feet wide were included with the concrete crest of the spillway at elevation 541.3. If this design

[2] 1912 datum; see p. 221 for explanation.

flood actually occurred, the headwater would rise to a maximum level of 558.3 with all 60 tainter gates wide open. The top of these gates was set at elevation 556.3.

The spillway section is the standard ogee crest type with a combination diffuser-sill apron. Figure 15 shows the details. There are 59 piers each 5 feet wide between the spillway openings and two half-pier end sections. The spillway piers are therefore spaced on 45-foot centers with a 40-foot opening between. The top of the piers is at elevation 564.0. The upstream face is vertical to elevation 556.3. Below elevation 556.3 each pier has a semicircular nose extending 4 feet 11 inches upstream from the crest of the dam. A corbel 5 feet wide at elevation 537.3 forms a support for a floating bulkhead gate. The downstream face of each pier is vertical above elevation 545.3 and below this elevation the downstream portion of the pier is streamlined to a shape indicated by model studies.[3]

End training walls, formed by extending the piers, are provided at each end of the spillway to minimize the formation of eddies. These walls are reinforced concrete, 5 feet thick, and extend to the end of the apron, or 115.87 feet from the axis of the dam.

Eleven intermediate training walls divide the stilling pool into 12 sections, each of which includes 5 spillway gates. This results in a more effective hydraulic jump when a limited number of gates are discharging. These walls are similar in design to the end training walls except that their tops are 12 feet lower. Differential water loadings for the design of the training walls were determined from hydraulic model tests.[3]

The hydraulic jump pool, designed to dissipate the energy in the water falling over the spillway, is formed by the end and the intermediate training walls and a diffuser sill at the downstream end of the apron. Model tests indicated that the energy dissipation of this arrangement would be sufficient to prevent excessive erosion of the river bed for all reasonable conditions of operation.

The spillway apron slab has a minimum thickness of five feet and is constructed monolithically with the diffuser sill. A toe wall extending into rock to approximately elevation 474 is located underneath the sill. The aprons opposite the trashway sections are similar to that for the spillway except they have no diffuser sills. The elevation of the trashway aprons, however, is eight feet higher than that for the spillway, and a 3 to 1 slope is cut in the rock until the river bed meets the concrete surface of the apron.

The analyses for the spillway section of the dam indicate that the maximum stresses are relatively low in all cases. The maximum compressive stress for the condition with maximum earthquake shock with full reservoir occurs at the downstream edge of the base and amounts only to 99 pounds per square inch. The maximum compressive stress, including maximum earthquake, with the reservoir empty occurs at the upstream edge of the base and amounts only to 70 pounds per square inch. No tension occurred at any location for any condition of loading. Shear stresses are relatively low, amounting only to 41 pounds per square inch with earthquake load.

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[3] Ball, James W., *Hydraulic Model Experiments for the Design of Wheeler Dam*, U. S. Bureau of Reclamation, Technical Memorandum No. 407.

The sliding factors do not exceed 0.65 except when earthquake load is considered. For maximum earthquake load the sliding factor at the base of the dam is 0.764, but the corresponding shear-friction factor of safety is 18.9. Shear-friction factors of safety are somewhat higher at elevations above the base of the dam.

The principal structural data which controlled the final design of the spillway section are:

1. Location of spillway section. (See figure 15.)
2. Crest of the permanent spillway—elevation 541.3.
3. Spillway control—60 tainter gates 15 feet high.
4. Top of the tainter gates in closed position—elevation 556.3.
5. Weight of the tainter gate per linear foot of spillway crest—1,075 pounds.
6. Gate hoist chambers in inspection gallery of dam, floor at elevation 523.3.
7. Alignment of vertical portions of upstream face of dam—straight and continuous through spillway, trashway, and nonoverflow sections.
8. Weight of bridge per linear foot of dam—4,500 pounds.

Trashway sections.[4]—There are two 40-foot trashway openings (37.5-foot clear opening) in the dam and one 6-foot trash chute opening in the north abutment wall. (Not shown in Figure 15.) A section through the trashways is shown in figure 15. The analyses for the trashway sections show that the maximum stresses, sliding factors, and shear-friction factors of safety are about the same as in the nonoverflow section.

Structural data which controlled the final design of the trashway section are:

1. Location of the trashway sections at the ends of the spillway section.
2. Permanent crest in trashway section—elevation 550.3.
3. Trash gate clear opening—37.5 feet.
4. Top of the trash gates in the closed position—elevation 556.3.
5. Weight of the trash gates per linear foot of crest—2,500 pounds.
6. Distance from the center of gravity of the trash gates downstream to the axis of dam—0.23 foot.
7. Requirements and positions of the gate hoist chamber and the operating gallery—same as for the spillway.
8. Alignment of the vertical portions of the upstream face of dam—straight and continuous through spillway, trashway, and nonoverflow sections.
9. Weight of bridge per linear foot of dam—4,500 pounds.

Loading used in the analyses, together with constants and assumptions, was the same as used in the analysis of the spillway section.

The crest of the 6-foot trash chute between the lock and the north abutment is at elevation 550.3. A removable bar grating covers the stop-log grooves in the side walk of the trashways. Creosoted longleaf yellow pine stop logs are provided for closing the 6-foot trash chute, each stop log having a channel iron bolted on the top.

Foundation exploration and treatment.—No extensive stability studies were made of the foundation because the early investigations indicated that it was sound. The dam site foundation is composed of

[4] See figure 25 for intake trash sluice.

a series of sedimentary beds lying nearly horizontal. They consist chiefly of cherty and siliceous rock of a more massive type than the usual shale, alternating with layers of a very fossiliferous and almost pure limestone. The shale beds are much thicker than the limestone beds and alternate with them. On the north side of the river the dip is slightly downstream, but on the south side it is slightly upstream. The changes in dip indicate slight deformation or warping, in the course of which joints have been developed across the strata, some confined to individual beds that are comparatively brittle, but others which might be called master joints passing through the whole succession of beds from top to bottom. Although solution channels were found at numerous places, all the cavities were very small with the exception of two in the foundation for the draft tubes. There the foundation proper was satisfactory as the small solution channels which were found were bridged over by sound rock and the load placed on the foundation was not excessive.

The shearing strength of the foundation rock was tested, using 14 pieces of 1.36-inch diameter core. The results showed:

1. Three specimens with no end load failed at shearing loads averaging 1,375 pounds per square inch;
2. Four specimens with a 713-pound-per-square-inch end load failed at shearing loads, averaging 2,142 pounds per square inch; and
3. Five specimens with a 1,456-pound-per-square-inch end load failed at shearing loads averaging 2,530 pounds per square inch.

Later nine specimens from the same cores were tested for compressive strength and failed at an average value of 11.83 tons per square in.

When the powerhouse excavation had been completed to elevation 471.3, two caverns were intersected. In order to locate other possible caverns, 6-inch shot drill holes were drilled at 20- to 25-foot intervals across the entire length of the dam, except in cofferdams 4 and 5, where the spacing was about 40 feet. In addition to these holes, several 36-inch holes were drilled from which visual inspections of the foundation could be made.

It was decided to excavate to a depth which would permit placing the concrete on limestone strata having a thickness of 4 feet or more. This was usually found 10 feet or more below the bed of the river. Because of the extensive solution in the south abutment above elevation 540, a cut-off trench 15 feet wide was extended 170 feet into the bank and filled with concrete to elevation 557.

In the analysis of the spillway section, elevations for the base of the section were chosen somewhat lower than it was believed necessary to excavate into the foundation rock. The maximum base for spillway section was at elevation 485.3, although most of the actual base was founded considerably above this elevation.

The general plan for grouting the foundation rock under the entire dam provided for initial low-pressure grouting followed by high-pressure deep curtain grouting.

Drainage in both the nonoverflow and spillway sections of the dam is accomplished by a 12- by 12-inch drainage gutter located at the upstream side of the inspection gallery. Twelve-inch cast-iron pipes drain the gutter at the end and intermediate training walls.

Foundation drain outlets are provided under the apron to relieve any unbalanced upward thrust when the hydraulic jump occurs.

The system of drainage under the apron is in units, one in each five-gate spillway bay between the intermediate training walls. Each unit consists of ten 12-inch half circle concrete gutter pipes laid upon the rock surface, and extending upstream from the sill to a 24-inch intercepting header of similar construction. The header is located 2½ feet downstream from the joint separating the apron from the dam, and outlets are provided at its ends, one at each face of a training wall. Each outlet consists of 12-inch standard cast-iron pipe with a 90° band turned downstream at the surface of the apron and protected from the spillway overflow by formed concrete.

Contraction joints and water stops.—Contraction joints in the non-overflow sections are spaced approximately at 45-foot intervals, with some exceptions. In the spillway section construction joints are placed 75 feet each side of the pier center lines making blocks alternately 15 and 30 feet long. These joints, vertical and perpendicular to the axis of the dam, were designed as shown in figure 16. Nine inches from each face and the top of the dam a 12-inch 20-gage stainless-steel water stop with a 1½-inch crimp along each edge was inserted at the joints. The inspection gallery is surrounded by a stainless-steel water stop at the construction joints. In addition asphalt water stops are located in the joints 3 feet from the upstream face of the dam. Each consists of a well 9 inches wide at the joint and 4½ inches deep which was filled with hot asphalt[5] after each 5-foot lift of concrete was poured. Contraction joints in the apron are spaced 25 feet apart transversely. There are two longitudinal joints, one at the toe of the spillway and one near the center of the apron. In all of the training walls two vertical contraction joints

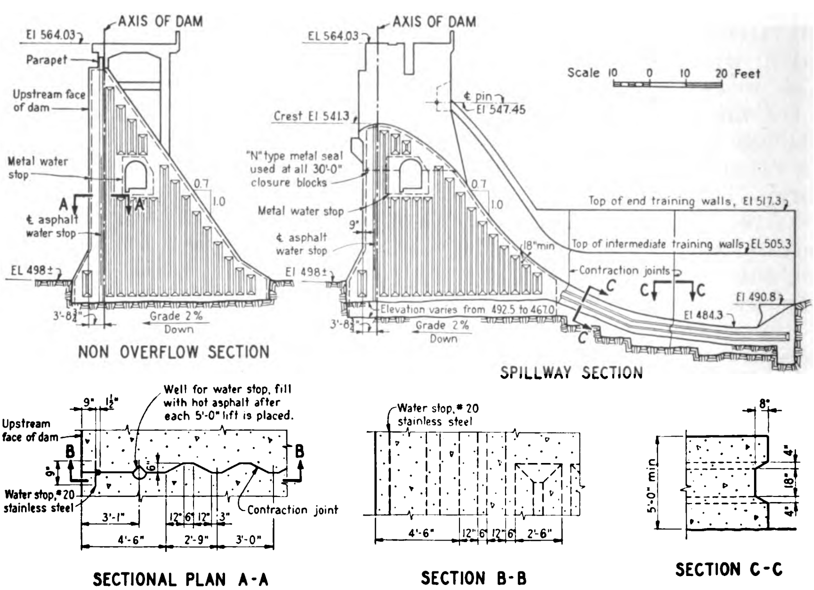


FIGURE 16.—*Contraction joints.*

[5] US No. SS-A-696 Petroleum Asphalt, Type PAF-1-25 Joint Filler.

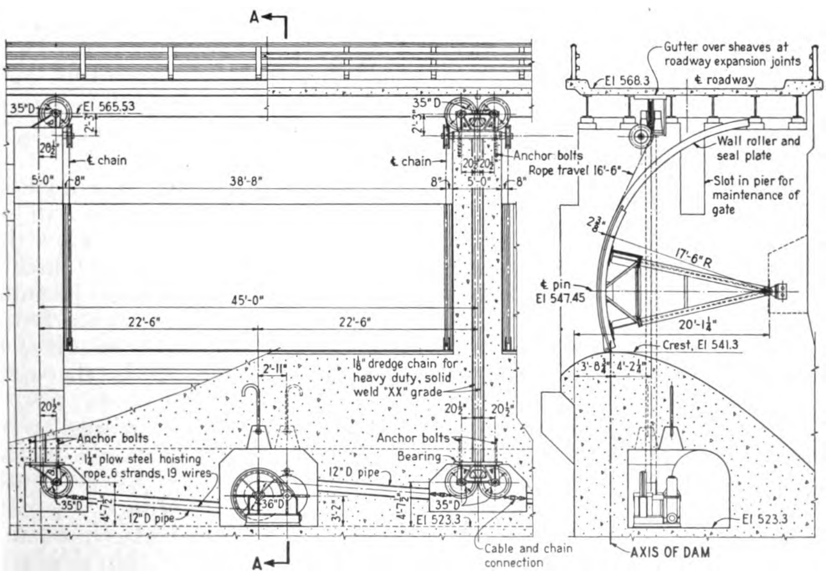
are located immediately above the longitudinal contraction joints in the apron.

Highway bridge.—The highway which connects U. S. Highway No. 72 with Town Creek crosses over the top of the dam at elevation 568.3 and the lock at elevation 619.8 on a bridge whose center line is 7.89 feet downstream from the axis of the dam. This bridge supports a 20-foot roadway with a sidewalk on the downstream side. The roadway is a reinforced concrete slab, varying in thickness from 9 inches at the center to 8 inches at the curb for drainage purposes.

At the south end of the dam the bridge is steel girder and floor beam construction, with a concrete floor slab, supported on concrete bents in the nonoverflow portion and on the spillway piers in the spillway section. The ramp section is similarly designed except that the supports are steel bents. Spanning the lock is a 164-foot through truss supported on concrete piers. Roadway drainage is provided by standard galvanized pipes at the curb lines. Roadway lighting standards are along the downstream curb of the roadway. Each fixture consists of a 300-watt refractor unit in a 20-inch Monax ball, 20 feet 10 inches above the concrete curb surface.

Equipment.

Tainter gates.—The great width of the river and the relatively shallow tailwater depths made a long spillway with rather shallow gates advisable to prevent unreasonable concentration of energy at the toe. The relatively low tailwater for maximum discharges permitted the use of tainter gates. The spillway gates were designed for the condition of normal headwater at elevation 556.3, or a total head of 15 feet, with hoisting mechanism capable of raising the gates



SECTIONAL UPSTREAM ELEVATION SECTION A-A

Scale 5 0 5 10 Feet

FIGURE 17.—*Tainter gate.*

under maximum head of 17 feet. The installation consists of 60 tainter gates 40 feet long and 15 feet high. Each gate is framed on two 36-inch I-beams 39 feet 8 inches long, spaced 7 feet 10 inches apart, and trussed horizontally and vertically by angles. Each end is supported radially by two 8-inch H-beams meeting at the trunnion. Curved 8-inch I-beam and channel purlins in the vertical plane are welded to the 36-inch horizontal beams, and a $\frac{5}{16}$-inch skin plate, curved on a 17-foot 6-inch radius, is welded to the purlins and forms the face of the gate.

The trunnion pins are 5.500 inches in diameter. Embedded grillages and anchorages around the pins relieve the stresses in the concrete. Each gate has two cast steel guide rollers 5 inches in diameter supported on brackets bolted to the curved end channels. These roll on wall plates anchored in the piers and prevent binding of the gate.

Gate seals are provided to minimize leakage of water around the gates. The end seal consists of a continuous piece of specially formed rubber bolted at the end of the skin plate and pressing against the wall plate. The seat seal is specially formed reinforced rubber retained in a slot at the bottom of the gate, provision being made for adjustment to insure uniform contact with the sill.

The gates are operated by individual hoists, the motors being controlled from within the inspection gallery but so connected that gates can be operated singly or in groups of five. The hoisting equipment is placed in chambers midway between the piers and immediately upstream from the inspection gallery. By this arrangement, interference with the roadway on top of the dam was avoided. Each unit consists of an electric motor connected through a flexible coupling to a hoist drum by a series of worm and pinion gears. The motors are 440-volt, 3-phase, 60-cycle, 1,750 revolution-per-minute, totally enclosed, weatherproofed, squirrel-cage induction type. For the radial gates to travel from a closed position to the fully open position requires an average time of 13 minutes and 40 seconds; traveling in the opposite direction requires an average time of 13 minutes and 20 seconds.

Trashway gates.—The trashway gates are vertical-lift rolling gates, operated by a lifting chain and hoist. They are designed to be raised to elevation 556.3 when closed or dropped to 1 inch below the crest at elevation 550.3 when open. Each gate is 39 feet 4 inches long and 6 feet 11½ inches high. The gate leaf is made of I–beams and skin plates welded to both flanges of the beams. The downstream face is vertical. The upper portion of the upstream face is shaped to form the lip of the crest when the gate is in the open position.

The bottom seal is formed by a strip of curved spring brass, with rubber strips riveted to the underside, the whole anchored in the concrete and pressing against the downstream face of the gate. Side seals are similar in construction to the bottom seals. The corner seals are blocks of thread rubber, held between steel clamps and angles on the gate guide so that they contact both the side and bottom seals.

Each gate has two steel-flanged wheels at each end. The wheels roll on rails attached by means of rail clamps to the gate guides. The gate guide frames are made of structural steel shapes and embedded in the pier concrete. The trashway hoists are identical with those for the spillway gates. The hoist motors are controlled from push-button stations on the roadway near the trashways.

Floating bulkhead gate.—Instead of using stop logs for blocking the 40-foot spillway and trashway gate openings during maintenance and repairs, a floating bulkhead gate was adopted. This gate is 48 feet 4 inches long and 21 feet 2 inches high. The maximum thickness is 4 feet 2⅛ inches. The gate is made of plates reinforced by structural shapes. It is designed to float as a scow on its upstream face when not in use. The gate is towed or maneuvered into position in front of the permanent gate to be repaired, and water is admitted to the interior of the gate through a 10-inch valve. The water ballast tips the leaf into a vertical position, and it becomes seated on the lip of the concrete structure under the spillway or trashway crest. The seals consist of rubber and four-ply canvas.

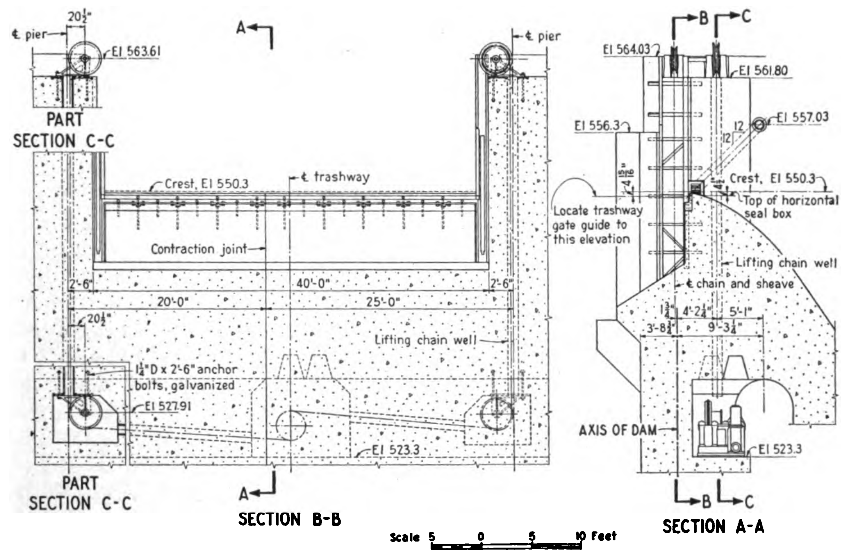


FIGURE 18.—*Trashway gate.*

When the water between the leaf and the permanent gate is released, the water in the leaf is discharged downstream through another 10-inch valve. Pressure of the reservoir water then holds the leaf against the piers. In order to remove the gate, a third 10-inch valve on a pipe passing through the leaf is opened to admit water to the space between the leaf and the permanent gate, and after the pressures are balanced the bulkhead gate can be floated away and turned in a horizontal position. The upper portion (vertical position) is sealed from the lower portion by a watertight bulkhead so that when the lower portion is flooded, the leaf will not sink entirely. When not in use, the bulkhead gate is anchored between the lock and the north bank.

Electric power and lighting.—Two 2,300-volt, 3-phase circuits supply the power and lighting requirements for the spillway and nonoverflow sections of the dam. Both of these circuits, originating at the 2,300-volt metal-clad switchgear in the powerhouse service bay, are tapped for the transformers located in recesses along the upstream side of the gallery and one continues through the gallery to the transformer banks supplying power to the lock.

Transformers in the gallery are used to step the voltage down from 2,300 volts to 440 volts and to 115/230 volts. The 440-volt power is for operation of the spillway gate and trash gate hoisting equipment and for power plugs at each pier along the spillway and at the trash gates. The 115/230-volt, single-phase power is for gallery, roadway, and miscellaneous lighting and small power.

These 115/230-volt, single-phase leads run to lighting cabinets where De-ion breaker protection is provided. The gallery lighting is controlled by three-way switches at the ends of each circuit. On the longer lighting circuits, magnetic contactors are used for control. The gallery lighting units consist of exposed lamps set in recesses. The ordinary lamp size is 50 or 75 watts, but larger lamps are used on stairways, in transformer recesses, and where brighter lighting is desired. The conduit is exposed on the gallery ceiling. The roadway lighting units are supplied from the 115-volt source in the tunnel lighting cabinets. These units are for multiple operation with feeders from each of the five transformer stations in the dam. The principal advantage of the multiple system is that the loss of any transformer will affect only a small number of lights. Each lighting circuit is controlled by a contactor, which in turn is controlled by a telephone relay operated from the power plant.

Telephone stations are mounted on lighting standards along the roadway at three different points. A police recall light is mounted near the top of one of the lighting standards. Code call horns are mounted on lighting standards along the roadway at four points. Bells are used in the gallery for code call purposes. The roadway horns and tunnel bells are operated in parallel with the code call equipment in the power plant.

Hydraulic model tests.

Extensive hydraulic model tests were made by the United States Bureau of Reclamation [6] to check further the adequacy of the design. Tests were made to determine the characteristics of the stilling pool, intermediate training walls, piers, gate operating program, and radial gate discharge.

The problem of dissipation of energy and protection against scour at the toe of the spillway apron was very important and model tests were made for the purpose of determining the most effective and economical form of apron. Numerous types of aprons, sills, and diffusers were tested. In general, tests were made on a level depressed apron, deflecting bucket, and sloping apron and on Rehbock sills, triangular sills, breaker blocks, secondary wiers, sloping sills, and diffuser sills. Results of these tests led to the adoption of the present sloping apron with the diffuser sill. (See appendix C.)

Because of their number, it was not logical to operate the 60 gates individually; hence the spillway was divided into groups of 5 gates each. Hydraulic model tests showed the desirability of placing intermediate training walls between these sections, and accordingly these training walls were included in the design of the dam.

[6] Ball, James W., *Hydraulic Model Experiments for the Design of Wheeler Dam,* U. S. Bureau of Reclamation Technical Memorandum No. 407.

The original design for the piers contemplated a flat face on the downstream end. Tests showed this created undesirable discharge conditions. Four designs for the downstream end of piers were therefore investigated: (1) with a 5.87-foot radius in a plane of the spillway face on each side of the sloping part of the pier nose, (2) with a 36.5-foot radius, (3) with a 64.3-foot radius, and (4) a modification of (3), designed to prevent undercutting of the pier. It was found that type (2) resulted in the best hydraulic condition without requiring an excessive amount of additional concrete.

In order to develop a gate-opening schedule, hydraulic model tests were conducted to determine the limit to which a set of five gates could be raised without destroying the satisfactory hydraulic jump in the stilling pool below, assuming powerhouse in operation, reservoir surface at elevation 555, and tailwater at elevation 503. These tests showed that as a flood approaches, the first set of gates nearest the powerhouse should be raised, but not beyond a maximum of 8.7 feet. With a further increase of flow the adjoining sets of gates should be opened successively, until the entire 60 gates are raised 8.7 feet. This would provide a flood capacity of 360,000 cubic feet per second. With a still further increase of flow, gates at random can be opened completely as needed. A maximum flow of 687,000 cubic feet per second can be discharged with the gates completely open and the reservoir at elevation 558.3. Also, from the model tests, rates of discharge were established for various gate openings and for different headwater and tailwater levels.

NAVIGATION LOCK

When the Tennessee Valley Authority was directed to construct the dam, it was found desirable, in order to obtain proper development of the river, to establish the normal pool elevation at 555 [7] with the top of dam and lock wall at elevation 560.[7]

The United States Army Engineers agreed to complete the construction of the Wheeler lock and, in addition, to perform the following work:

1. Construct the nonoverflow section between the north shore and Wheeler lock.
2. Dismantle the old lock No. 3 of the Muscle Shoals Canal and erect gates on the temporary upper sill of Wheeler lock.
3. Furnish and install the operating mechanism for upper gates and valves including switchboard.
4. Construct the temporary upper miter sill at elevation 498.
5. Dredge a temporary channel upstream from the lock.
6. Protect navigation.
7. Remove the temporary gates prior to the installation of the permanent gates.

Provision for temporary navigation.

The new lock was located in the old Muscle Shoals Canal immediately below old lock No. 2. During construction the outer bank of the canal was breached and navigation was passed directly from the lower pool of lock No. 2 into the upper end of Wilson reservoir.

[7] 1912 datum; see p. 221 for explanation.

During periods of high velocities in the river caused by dam construction activities, navigation was passed through the new lock by the installation of temporary upper miter gates. A temporary upper sill was built to elevation 498.3 and the lower gates from old lock No. 3 installed on this with pintles 26.75 feet upstream from the permanent pintles. This made it possible to pass craft through the lock under any differential heads prevailing with headwater up to elevation 516. The gates were operated by cables from a hoist set on the lock wall about 24 feet upstream from the gate pintles.

Structural features of the lock.

The centerline of the lock makes an angle of 85° with the axis of the dam, placing the downstream approach of the lock away from the north bank. The lock chamber has a 360-foot clear length and 60-foot clear width. The rock foundation is at elevation 490, with the walls founded at elevation 488 except where keys were channeled into the rock to a maximum depth at elevation 484. The walls vary in section, but the principal section of each wall has a top width of 8 feet and a bottom width of 53 feet. The central control station for the lock is on the river wall near the axis of the dam.

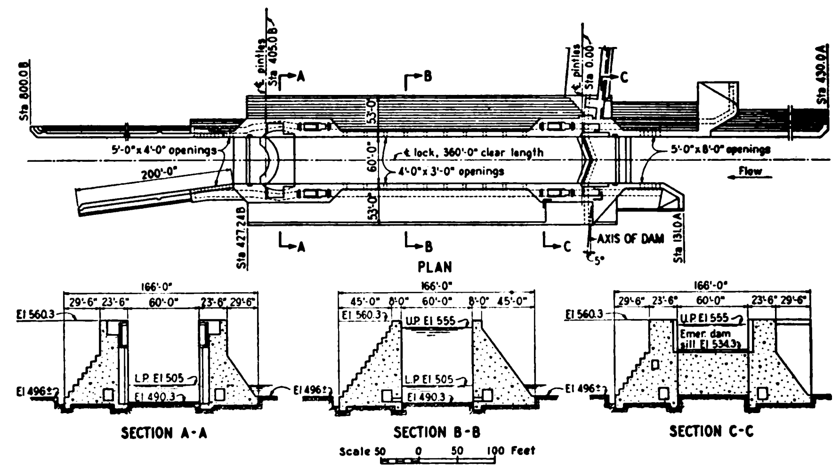


FIGURE 19.—*Navigation lock—Plan and sections.*

Because it was contemplated that a larger lock might be needed in the future, provision was made for a 110- by 600-foot lock between the present land wall of the lock and the riverbank. The present land wall would then become the river wall of the second lock.

Design assumptions for the lock were as follows:

1. Upper pool level—elevation 561.[8]
2. Lower pool level—elevation 505.
3. Earth fill behind the upper guide wall—saturated to elevation 561.
4. Earth fill behind all other land walls—saturated to elevation 505 with dry earth from elevation 505 to the top of the walls.
5. Full hydrostatic water pressure effective over 50 percent of the area of the base.

[8] Upper pool later lowered to elevation 555.

6. Elevation of the top of bedrock—the same as the bottom elevation of the walls.

7. Effects of keys were neglected in calculations.

8. For the design of the upper and lower bay land walls, the lock chamber was assumed empty with the lock gate in mitered position and the culverts filled with water. The maximum tangent of the angle between the resultant forces and the vertical was 0.667 for the river wall between bays when the water was at elevation 561 in the lock chamber with the top of the wall at elevation 566.

9. For the design of the sills, the gates were considered as horizontally framed with no pressure from the gate on the sill. For the upper miter and emergency dam sill, the emergency dam was considered in place and the lock emptied. In the case of the lower miter sill, the tangent of the angle was 0.840, but the sill was keyed into the rock to take the horizontal thrust.

The following assumptions were used in all lock computations:

	Pounds per cubic foot
Weight of water	62.5
Weight of concrete	145
Weight of dry earth	100
Weight of saturated earth	125
Pressure of dry earth on a vertical plane	33.33
Pressure of saturated earth on vertical plane	83.33

Foundation conditions in the lock area were found to be entirely satisfactory. Early investigations of the site had shown that on the lock side of the river the dip is slightly downstream. Although solution existed in the formation at numerous places, all the cavities were very small and were surrounded by sound rock. In order to prevent future action and circulation of water along these channels, they were blocked at the upper end or throughout by concrete cutoff walls or by grouting. No general foundation grouting was required in the lock area.

Lock walls.—The lock walls are the concrete gravity type. A plan and several sections are shown in figure 19. The length of the land wall is 1,230 feet, that of the river wall 776 feet. The tops of both walls are at elevation 560.3 with the exception of the lower guard wall which extends downstream from the end of the chamber. The top width of both main chamber walls is generally 8 feet. The chamber faces of both walls are vertical, forming a 60-foot clear channel.

The land wall may ultimately become the common wall between the two locks. Its landward side was "stepped" in order to provide better bond for the superimposed concrete of the proposed future second lock.

Rectangular filling culverts, 10 feet high by 8 feet wide, with floor at elevation 492.3, are provided in both walls. The centerline of the culverts is 12 feet 6 inches from the chamber face of the wall. Three 5- by 8-foot inlets with floors at elevation 504.3 are provided in each wall. Outlets into the lock chamber consist of ten 4- by 3-foot openings in each wall. These outlets have floors at elevation 492.3. Six 5- by 4-foot openings below the lower gates and at the end of the culverts are provided for discharging the water from the lock chambers.

Two pairs of recesses are provided for the tainter valve operating shaft and machinery. These are above the rectangular filling culverts. The upstream pair—one in each lock wall—is located with center line of the shaft at station 35.33 B (see fig. 19), and the downstream pair is located with center line at station 346.58 B. The shaft is 9 feet 6 inches wide and 16 feet 10 inches long and extends from the top of the filling culvert to the top of the lock walls. The openings are covered with gratings supported on I-beams spanning the openings. Provision is made both upstream and downstream from the operating shaft for emergency gates.

A 7- by 5-foot rectangular inspection tunnel through the upper miter sill connects the tunnel in the abutment section with the tunnel in the dam. An additional inspection tunnel is in the upper land guard wall, and connects with the above tunnel.

Miter sills.—Two miter sills are in the lock. The top of the upper miter sill is at elevation 534.3. The lower sill is in two sections, the upstream section being the sill for the lower miter gate and the downstream section the sill for the emergency gate. The sill for the lock gate is 12 feet wide with top at elevation 491.3. In plan the sill is circular, conforming with the gate, and with a radius of 36 feet 8¾ inches. The emergency gate sill is located with upstream face 20 feet downstream from the center line of the lower gate pintles. This sill is 20 feet wide with the top at elevation 491.3. Drainage is provided under the gate sills.

Wall armor.—The walls of the lock are protected from damage by cast ribbed-steel plates anchored to the chamber face of the walls. The top plate extends 8 inches above the top of the wall. Both the land and river walls are armored from a point 61 feet upstream from the axis of the dam to a point 448 feet downstream from this axis with four rows of plating, and from the point 448 feet downstream from the axis to the lower end with seven rows of plating. The top plate extends along the top of the lock wall and connects with the plating of the upper guard wall.

Central control station.—The central control station for the lock is a 1-story concrete structure, 60 feet by 29 feet 6 inches. The building is on the river wall of the lock adjacent to the axis of the dam. The central control station houses the office, switchboard and control room, air compressor and tank, and other miscellaneous equipment.

North abutment and cut-off wall.—Between the land wall of the lock and the north bluff is the abutment and bulkhead wall section. The section of the bulkhead wall is similar to that of the main nonoverflow part of the dam, and its upstream face is in line with the upstream face of the main dam. The top of this section is at elevation 557.3. The upstream face is vertical down to elevation 506.3 and then batters to the rock line. The back face is vertical to elevation 551.15, from which point the downstream slope is 0.7 to 1 to elevation 502.3, where it flattens out and extends to the rock at elevation 497.3. The inspection tunnel, which is a continuation of the tunnel in the dam and under the lock, continues through this wall.

If the second lock is constructed, this wall will serve as the upstream cofferdam, since the entire lock chamber can be built downstream. In the section which would have to be removed for the lock entrance, the wall is built of blocks which can be removed without blasting.

Lock equipment.

Design data and allowable stresses used in designing the lock gates and design computations of stresses on the filling and emptying valves are in appendix A.

Upper and lower lock gates.—The upstream gates are two straight vertical leaf-type steel gates, 24 feet high and 35 feet ⅜ inch long from the hinge or bearing at the lock wall to the miter point at the center line of the lock. The downstream face of the gates form an angle of 70° with the center line. Each leaf is composed of six horizontal 36-inch I-beams covered on both upstream and downstream faces by a diaphragm of ⅜-inch plates. The top of the gate is at elevation 558.3; the sill elevation is 534.3. A walkway with handrails crosses the top of the gates at elevation 560.3. At the lower hinges, pintle shoes are grouted into the concrete. Rubber seals for the upper gates are fastened to the bottom of the gate leaves, and the seal is made by the contact of the rubber seal with the sill beam, which is anchored to the concrete sill block.

The lower gate consists of two curved leaves, each 37 feet wide and 65 feet 8 inches high. Each leaf has 15 arch ribs. The skin plate is on the upstream face and varies from ½ inch to 13/16 inch, with ⅜-inch butt straps at every second girder. A walkway, similar in design and appearance to that on the upper gates, crosses the

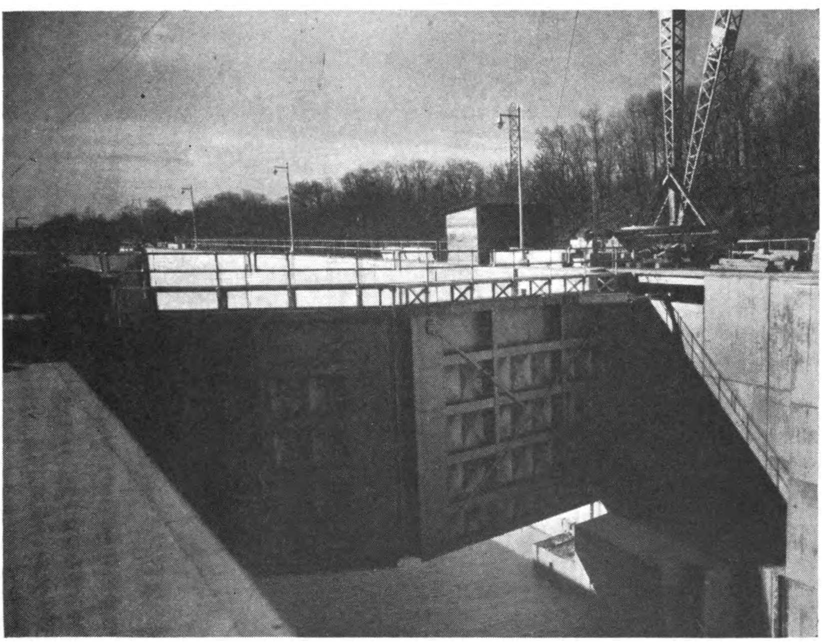


FIGURE 20.—*Upper gate before construction of upper miter sill.*

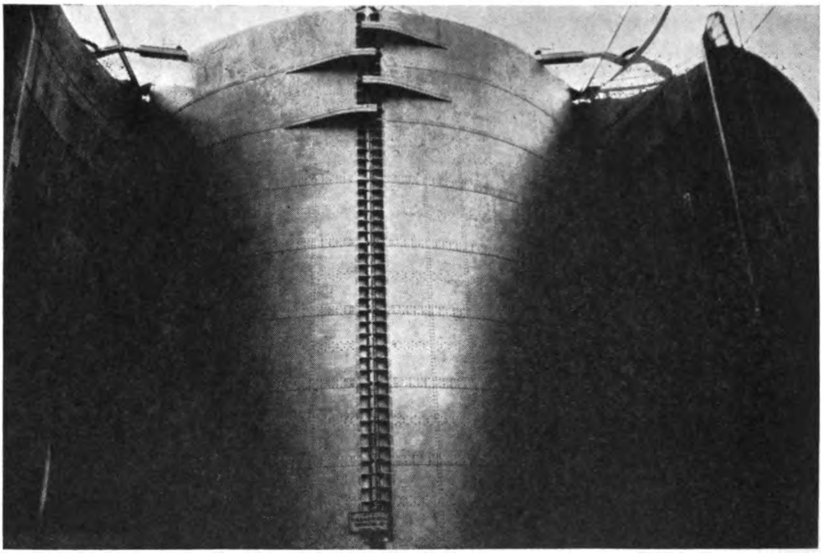


FIGURE 21.—*Lower gate.*

lower gate. The quoin contact blocks, quoin beams, pintle assembly, miter contact blocks below the miter guards, gudgeon pins, aprons, anchorage bars, anchorage pins, collars, and rings are similar in design to the upper gates.

The bottom seal is effected by a 4-inch-diameter rubber seal held under the lip of the sill casting by a seal support. The bottom seal casting riveted to the bottom girder of the gate presses on the rubber seal when the gate is closed.

Each lock gate is operated by a motor driving a sector gear to which is attached a sector arm. The sector arm and the gate leaf are connected with a spring-protected gate strut assembly. The mechanism is designed to open the gates in 1 minute. The total gear reduction from motor to sector gear is 2145 to 1.

The motor used to drive the gate-operating machinery is a 10- to 3⅓-horsepower, 900- to 300-revolution-per-minute, 440-volt, 3-phase, 60-cycle, high-torque, high-slip, squirrel-cage induction motor. A motor-operated brake direct-connected to the driving motor is provided for holding the gate in position.

Provisions are made for the installation of two emergency dams in the lock, one immediately upstream from the upper gates and one immediately downstream from the lower gates. The emergency dams are composed of a steel box girder and 12- by 12-inch by 24-foot oak needle beams. When in use, the girder for the upper emergency dam rests on a bearing plate and grillage inserted in slots in the lock walls, 14 feet above the miter sill. The needle beams bear on the girder and on the miter sill and form the emergency dam. The lower emergency dam is similar in construction to the upper emergency dam, with the exception that the girder when in use is set 12 feet 11 inches above the sill.

Filling and emptying valves.—The filling and emptying valves are structural steel tainter gates inserted in the culverts for controlling the flow of water into or out of the lock chamber. Each valve turns on trunnions at elevation 505.3 and stands in an inclined position when closed, its center line making an angle of 37° with the horizontal. The valve has a radius of 14 feet 2 inches and is 8 feet wide. The top of the gate is sealed by a 3-inch wrought-iron pipe which bears against the seal casting and the lintel casting when closed. The sides are sealed by rubber "music note" seals, bolted between clamping bars and side seal angles, so that the bulb projects and presses against the side seal casting anchored in the wall concrete. The bottom seal consists of a single nickel steel casting, bolted to the bottom flange of the horizontal beam at the bottom of the valves. This casting seals against the sill casting at the bottom of the culverts.

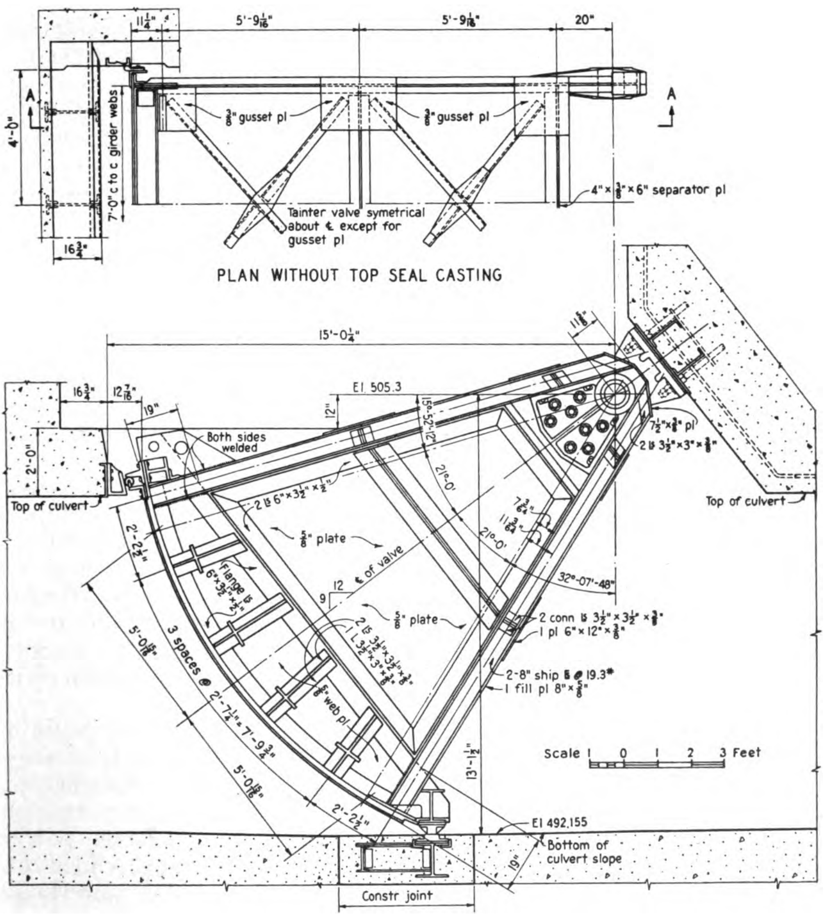


Figure 22.—*Tainter valve.*

The valves are operated by a hoist composed of a train of two reduction gears driven by a 15- to 5-horsepower, 900- to 300-revolution-per-minute, 3-phase, 440-volt, 60-cycle motor. The hoisting speed of the lifting chain attached to the valve can be varied from about 7 to 20 feet per minute. The last gear drives a shaft which carries a sprocket wheel for the valve-lifting chain and on either side of the sprocket are sheaves which carry counterweight cables. The counterweight is reinforced concrete, filled with steel punchings. The counterweight and punchings weigh 20,000 pounds.

Emergency bulkheads or stop logs can be inserted on each side of the tainter valves, when these are to be removed or repaired. The bulkheads are 11 feet high with the bottom at elevation 491.8 [9] and the top, therefore, 6 inches above the top of the culvert.

Auxiliary equipment.—A 25-cubic-foot air compressor with a 40-cubic-foot air storage tank is provided.

Switching arrangements have been installed for two towing engines and an emergency gasoline engine generator set for furnishing an independent source of power for the lock. The engines and generator set will be installed when conditions warrant.

The lighting system is 3-wire 240/120-volt. Two lighting transformers provide all present lighting for the lock. For illuminating the lock, 20-foot steel standards are used, each of which mounts a 300- or 500-watt lighting unit. The fixture units are holophane type supported 3 feet away from the pole. They are so installed that the center line of the lighting fixture is 3 feet 3 inches from the face of the lock chamber.

Navigation lights are vaportight condulet units, each equipped with a 6-inch Fresnel lens. The units are installed on tubular poles similar to the lock service lights. Three single red lens units suitably spaced are used to identify the land wall. The river wall has a group of three green lights spaced one above the other at the upstream end and a group of two green lights also spaced one above the other at the downstream end of the wall.

Electrical system.—The main electrical control equipment for the lock is in a central control building constructed along the top of the river wall between the upstream and downstream gates. A central cubicle-type main switchboard of the metal-enclosed safety type is at one end of the building and provides circuits for feeding four gate-control stations placed close to the upstream and downstream gates. The main control switchboard consists of seven steel cubicles. The panels are the hinged-door-type construction with extended handles.

The cubicle switching equipment in the central control building has a 460-volt bus which receives its power from a bank of three single-phase, 75-kilovolt-ampere, 2,300-volt–460-volt transformers. The 2,300-volt transformer connections are from a cable brought through the dam from the 2,300-volt switch structure in the powerhouse. At the present time only one 2,300-volt cable is provided, but provision has been made for a second cable from the powerhouse to connect with double-throw switches at the transformer bank. Also the low-voltage equipment is arranged to accommodate a second 460-volt, 3-phase circuit from the transformer bank secondary.

[9] 1929 adjustment; see p. 221 for explanation.

Feeder breakers for both circuits are mechanically interlocked to prevent simultaneous feed from both circuits. The bus switching arrangement in the cubicles provides for future installation of a gasoline engine generator set for a source of power in case of interrupted service.

Each gate control is housed in a weatherproof steel enclosure and equipped with complete control not only for its own gate but also for its companion gate across the lock and for operating valves at all four gate control points. Air signals are provided for indicating the various operations.

The main gates can be operated only from their respective gate-control stations. The valve motors may be operated either from the central control board or from the gate-control station. Lights on the central switchboard indicate whether the circuits are energized.

Circuits are interlocked to insure the following operation:

1. Upper gates cannot be operated unless lower gates are closed and lower valves are closed.
2. Lower gates cannot be operated unless upper gates are closed and upper valves are closed.
3. Upper valves cannot be operated unless lower valves are closed and lower gates are opened or closed.
4. Lower valves cannot be operated unless upper valves are closed and upper gates are closed.

The gate stations permit direct supervision by the operator over the operation of the gates. Each of the four stations is a sturdily built steel weatherproof housing enclosing the controls and wiring. A small service door on the front of each housing is opened only when the controls are to be operated.

Lead-covered cables and wires are generally used and are routed through the pits, tunnels, and manholes on asbestos shelving supported by racks anchored to the walls. Three-conductor cables are used for power work and short extraflexible leads are used for connections at the motors. Multiconductor control cables are No. 19/25 wire and range from two-conductor up to and including 20-conductor. Potheads are employed for terminating the control cables at the switchboards. Lead-covered wire serves the lighting system except in the central control building where single-conductor rubber-covered wire is used. Single-conductor 2/0 and 4/0 bare cables provide grounding connection for the equipment.

POWER PLANT

The power plant is the outdoor type, and the initial installation consists of 2 hydroelectric generating units and the necessary auxiliary equipment. Each unit consists of a 6-blade propeller turbine rated at 85.7 revolutions per minute, 45,000 horsepower, under an effective head of 48 feet, directly connected to a vertical shaft generator having a capacity of 36,000 kilovolt-amperes, or 32,400 kilowatts at 0.9 power factor. The turbines are completely housed in a concrete substructure, and the generators, which project above the top deck, are enclosed in individual metal housings. The power plant will ultimately contain 8 generating units.

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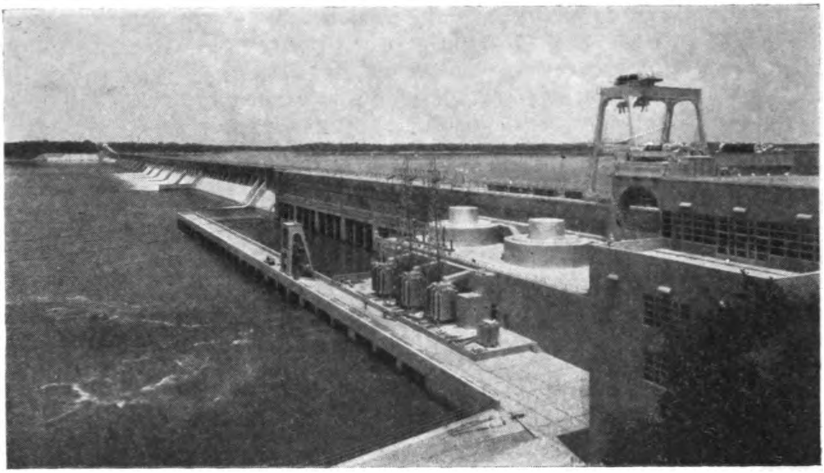

FIGURE 23.—*Power plant.*

Adjacent to the power-generating units is a service bay. Its purpose is to provide space and facilities for assembly and repair of turbines and generators, and storage space for intake gates and oil tanks. It also contains oil purifiers, ventilating machinery, drainage pumps, and other miscellaneous service equipment needed in connection with the powerhouse.

A transverse section of the powerhouse through unit No. 1 is shown in figure 15, and a longitudinal section through the centerline of the units is shown in figure 24.

Structural features.

Intake section.—The intake section controls the passage of water from the reservoir to the scroll cases of the turbines. It is near the south end of the dam and is 613 feet in length, providing for eight units each 76 feet wide from center to center of the 10-foot main piers plus 2 feet 6 inches at each end for pier thickness. Each unit has three intake passages with a clear opening of 18 feet each. The intake section proper, made up of a series of heavily reinforced concrete piers which support the necessary trashracks and emergency and service gates and other equipment, is considered as extending from the upstream face of the dam to a contraction joint 59 feet 6 inches downstream.

The temporary absence of the substructure for the six future units made it necessary that the structure be designed to withstand the water pressure without the support of the powerhouse. Each unit of the structure is stable—independent of either adjacent unit or the draft tube structure. Maximum water surface upstream was assumed at elevation 558.3 and tail-water surface against the service gate at elevation 505.3. The foundation was assumed at elevation 487.3.

The maximum computed stresses were:

Compression on foundation between the concrete and rock	12,950 lb. per sq. ft.
Tension on the same plane	0
Compression in concrete	870 lb. per sq. in.
Tension in reinforcement bars	19,150 lb. per sq. in.

The piers at each unit are 10 feet thick with the upstream ends on a line with the face of the dam. The upstream corners of the pier are rounded on a 3-foot radius. A keyed contraction joint divides the pier on its center line. The two intermediate piers in each intake bay are 6 feet thick and have upstream semicircular faces on a line with the face of the dam. The downstream ends of the piers are also streamlined.

The piers were designed for various loadings resulting from different combinations of emergency and service gates being closed, such as service gates closed in outer bays, and emergency gates closed in the interior.

The bottom slab of the intake section varies in thickness from about 16 feet at the upstream face to 13 feet at the downstream face. The top of the slab is at elevation 498.3 at the upstream face, sloping 17 to 1 to elevation 495.3 at the downstream face. Two horizontal contraction and construction joints are in the slab at elevations 487.3 [10] and 492.3, each joint being provided with five keys parallel to the axis of the dam. The slab above elevation 487.3 is reinforced both at the top and the bottom. Reinforcement is particularly heavy at the top of the slab near the gate slot, where diagonal shear reinforcing is included in addition to the regular reinforcing. The walls and piers are tied to the bottom slab by dowels. The slab was designed to withstand unbalanced loads caused by various gate closures together with the superimposed dead loads and 50 percent uplift. The slab was considered as a continuous beam between contraction joints in the piers.

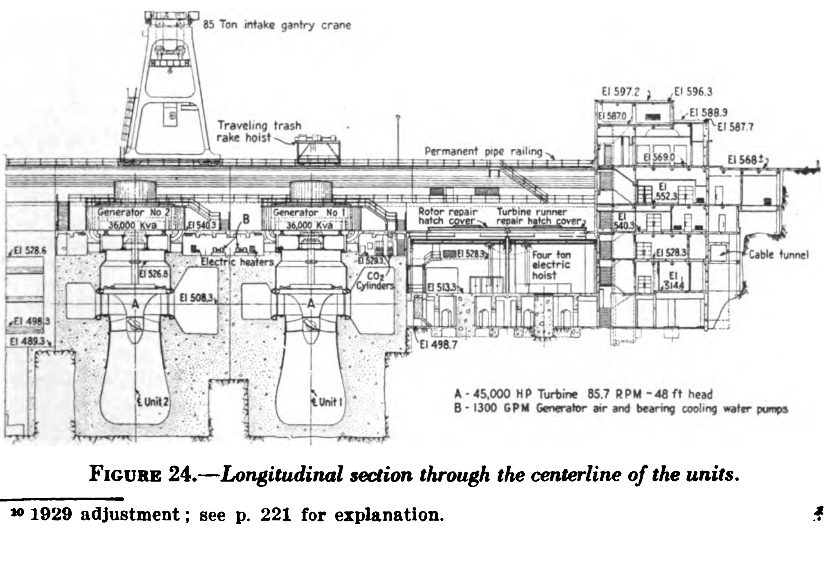

FIGURE 24.—*Longitudinal section through the centerline of the units.*

[10] 1929 adjustment; see p. 221 for explanation.

The top of the intake section is made of four reinforced concrete beams parallel to the axis of the dam, consisting of (1) the crane girder, (2) the upper portal girder, (3) the intermediate portal girder, and (4) the lower portal girder. (See figure 25.)

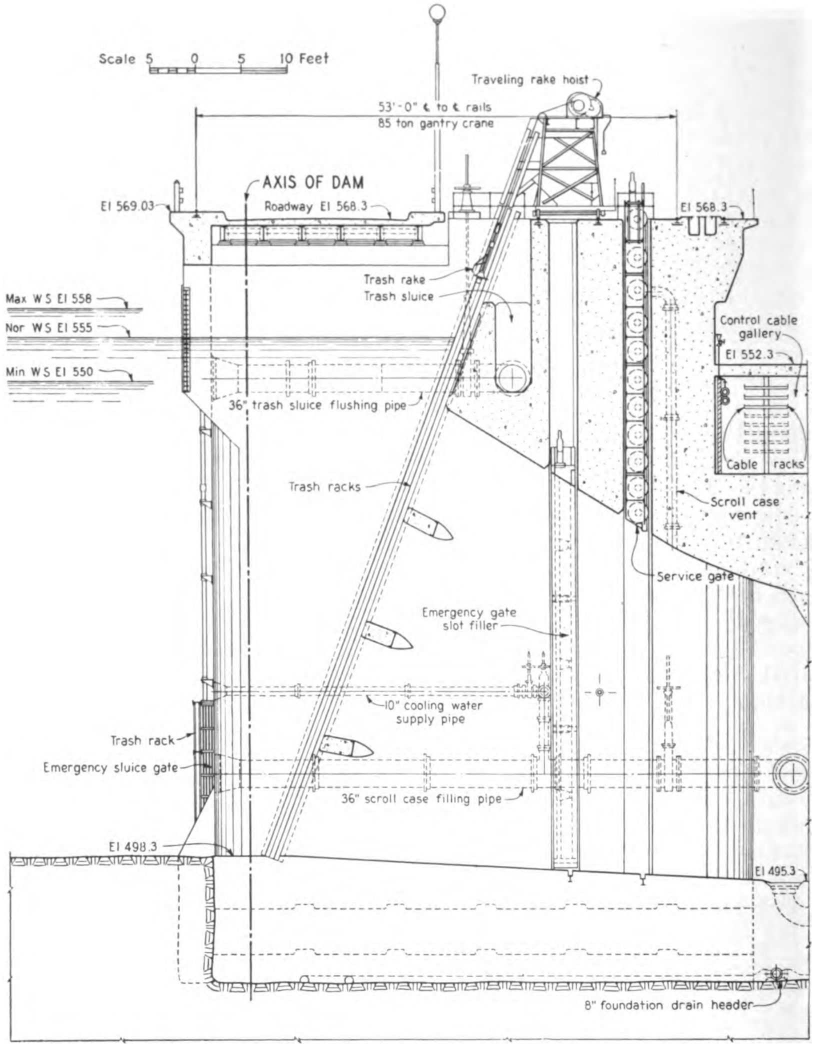

FIGURE 25.—*Intake structure.*

The crane girder (1) was designed to carry the upstream rail of the intake gantry crane. The upper portal girder (2) was designed to carry a portion of the trashrack load, the horizontal and vertical water loads, the trash sluice, 100 percent uplift on the waterway portion, and part of the trash rake. The intermediate portal girder (3) was designed to carry the unbalanced load of the emergency gates, 100 percent uplift on its bottom, and part of the trash rake. The downstream portal girder (4), immediately downstream from the service gates, was designed to carry the unbalanced service gate load, 100 per cent uplift on its bottom, and load from the downstream leg of the intake gantry crane, as well as from the upstream leg of the powerhouse gantry crane.

Foundation grouting in the intake section was similar to the grouting in other sections of the dam. Drainage for relieving the foundation of uplift is laid upon the surface of the rock foundation. Under each group of three bays in each of the eight units are laid two 12-inch gutter pipes, 10 and 15 feet, respectively, downstream from the upstream face line of the dam, each being 72 feet long, and reaching to within 2 feet of the contraction joints in the 10-foot main piers. Other 12-inch gutter pipes join these pipes at

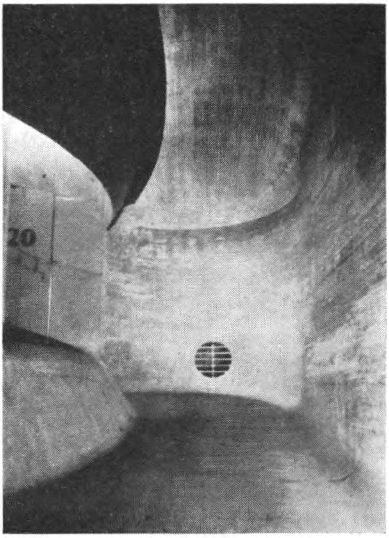

FIGURE 26.—*Inside of scroll case.*

intervals of 24 feet and extend downstream on the center line of each bay carrying the drainage to an 8-inch cast-iron drain pipe which drains into the station drain header in the completed unit and leads to a sump from which it is pumped into the tailrace. Another system of half tile pipe drains the rock area below the intake at the upper level and also connects with the 8-inch cast-iron pipe.

Vertical contraction joints are at the center line of the main piers between each unit. The upstream 25 feet of each joint consists of adjacent vertical keyways. Crimped No. 20 stainless steel water stops are at each joint, one 21 inches from the upstream face of the piers and another around the trash sluice. Additional watertightness is afforded by means of an asphalt-filled hole down the joint 21 inches downstream from the steel water stop.

Initial substructure.—The substructure for the two initial units is concrete, founded on solid rock, in which have been formed the scroll cases and the elbow draft tubes. The walls are all reinforced concrete. Reinforced concrete piers support the draft tube roof slabs and the walls and deck above the draft tubes. Draft tube pier noses for each unit, shaped to avoid undesirable turbulence are made of cast steel for the purpose of supporting the tremendous loads concentrated at this point.

The outline and shape of the scroll case were decided upon from the conclusions of a number of studies and from recommendations of the turbine manufacturers, as well as the results of hydraulic model tests which were given consideration in the designs of the scroll case.

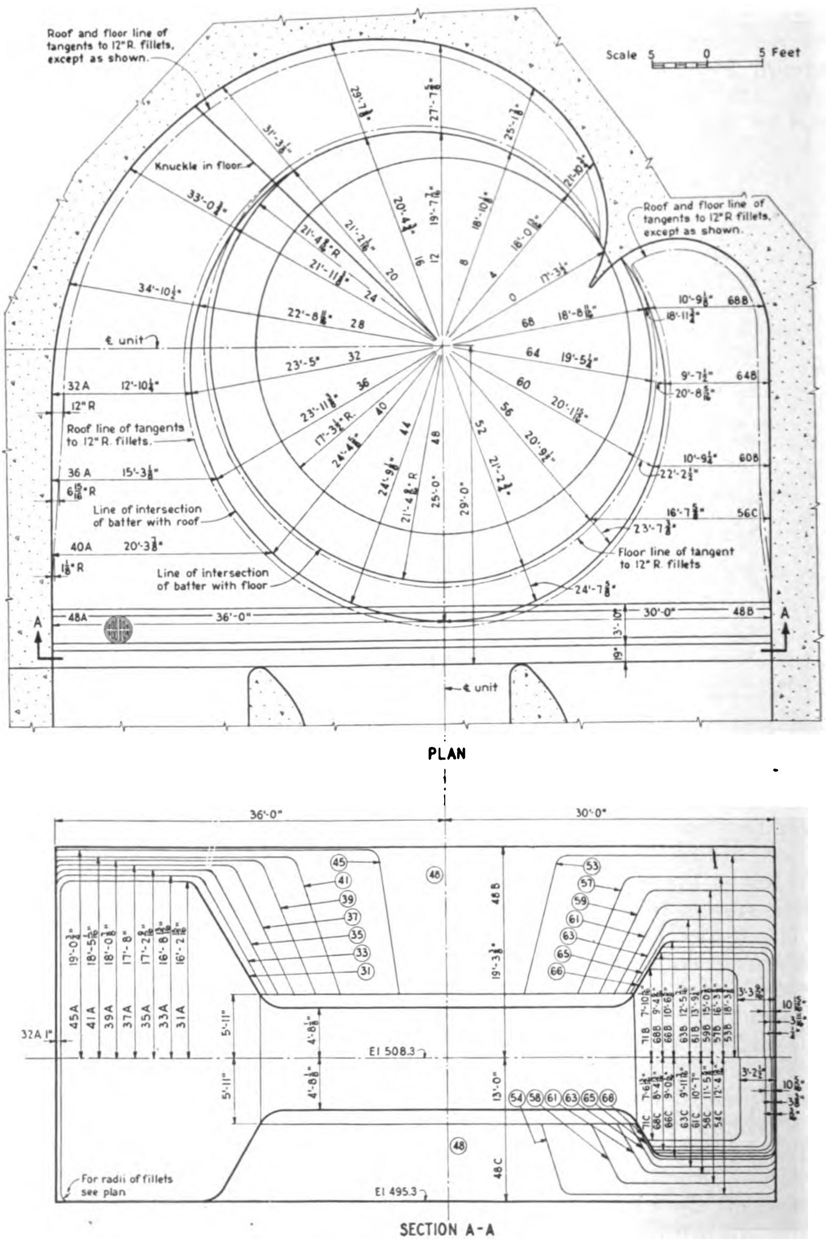

FIGURE 27.—*Details of scroll case.*

The typical cross section of each scroll case is rectangular; the outer face is vertical and the floor and roof are horizontal. The inner face has a sloping, conical surface at the top and bottom of the inlet to the speed ring.

In the main portion of the scroll, water is accelerated from about 6 feet per second to about 9 feet per second at the beginning of the spiral. From this point around the spiral to the splitter vane the velocity remains constant.

The foundation drainage system consists of three separate systems under each unit. The upper system covers the area between the intake section and the scroll case. The second is a grid system of drain tile located on rock under the elbow section of the draft tubes. The third is a grid system of drain tile located on rock under the downstream portion of the draft tubes. Under all of the service bay and the concrete floor portion of the control building, porous concrete drain tile is laid on rock, connecting with vitrified clay drain leading to either the east or west end of the station drainage sump.

The draft tubes are the elbow type 76 feet between centers and each has two 6-foot thick intermediate piers. The end of the draft tube piers is 89.5 feet downstream from the center line of the unit. The draft tube floor at the bulkhead gate is at elevation 455. A plate-steel liner extends from the bottom of the turbine throat ring to elevation 480. From this point the concrete tube is entirely unlined. The type and size of this draft tube were selected after numerous model tests conducted by the United States Bureau of Reclamation and are discussed in some detail in appendix C. The draft tube was designed to decelerate the water velocity from 26 feet per second at the throat to 7.75 feet per second at exit with discharge of 10,200 cubic feet per second.

Admittance to the draft tube for inspection and maintenance is provided through a 24- by 36-inch door below the runner and reached by a passageway and stairs from the elevation 479.7 floor, or through a similar door in the roof of the horizontal leg of the draft tube.

An unwatering gallery extends the length of the two units. The draft tube unwatering pump motor, a 2-ton chain hoist, and miscellaneous valves and piping are in this gallery. The pipe gallery, with floor at elevation 498.8, is immediately above the unwatering gallery. This gallery connects the quadrant rooms of the unit, in which are the valves of the scroll case filling-pipe, scroll case access doors, and other related equipment. Also in this gallery are the oil supply and drain lines for the main transformers.

The turbine gallery, with floor at elevation 525.3, has numerous offsets in the walls for the equipment placed at this floor level. In offsets at the downstream side of the gallery are the generator exciter control equipment and the 440-volt auxiliary switchboard.

In the area of the floor at elevation 527.3, between the units, are the generator and turbine instrument boards, the turbine flow meters, and governor actuator cabinets. At the 529.3 elevation are four governor oil pressure tanks and the generator air and bearing cooling-water supply pump. On the floor at this same elevation, south of unit 1, is the CO_2 fire extinguisher equipment for the generators, together with related controls and valves.

Future substructure.—The incompleted portion of the powerhouse substructure provides for the future installation of units three to eight, inclusive. Construction work completed in this area includes rock excavation, the completion of a portion of the horizontal draft tube leg, and the tailrace bridge and gantry crane tracks. A shaft and sump in the intermediate piers, as well as the draft tube unwatering drain, are provided for unwatering the area. This work was done to assure safety and economy in the installation of the future units. Complete closure of the area may be effected by placing stop logs and gates in the draft tube slots; and after unwatering, work may be started on the installation of any or all future units.

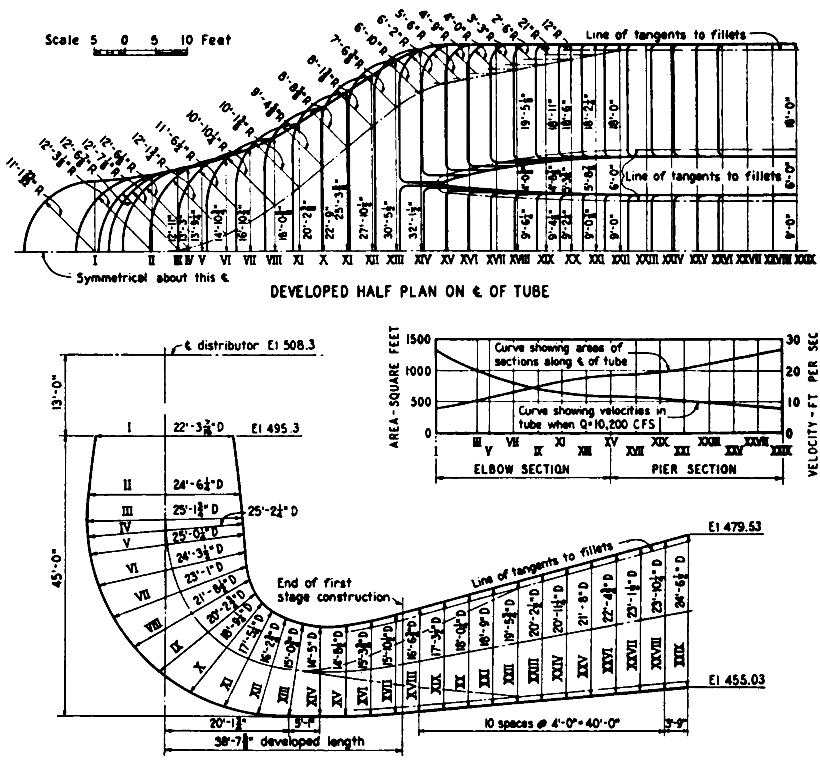

FIGURE 28.—*Details of draft tube.*

Immediately downstream from the intake, the slab was poured to elevation 489.3 over the area which had been excavated to approximately 487. A wall of 2-foot minimum thickness was also poured against the vertical excavation for the draft tube elbow and the section between the units. This construction protected the rock surfaces from erosion or disintegration while water was diverted through the intake.

The buttress section extends the full length of the downstream face of the powerhouse substructure. It includes the discharge openings of all the future draft tubes and is surmounted by the bridge for the draft tube gantry crane. The up- and down-stream width of this block is 51 feet. Footings of the pier nosings, 14 by 14 feet, were also poured to elevation 448.3 and anchor bolts provided for the future pier nose castings. Where needed, keys and metal water stops are provided where future concrete is to be poured against the present concrete.

Service bay.—Adjacent to the power-generating units is a service bay which provides space for erection and repair of turbines and generators and storage space for intake gates and oil tanks. It also contains oil purifiers, ventilating machinery, drainage pumps, and other miscellaneous service equipment needed in connection with the powerhouse.

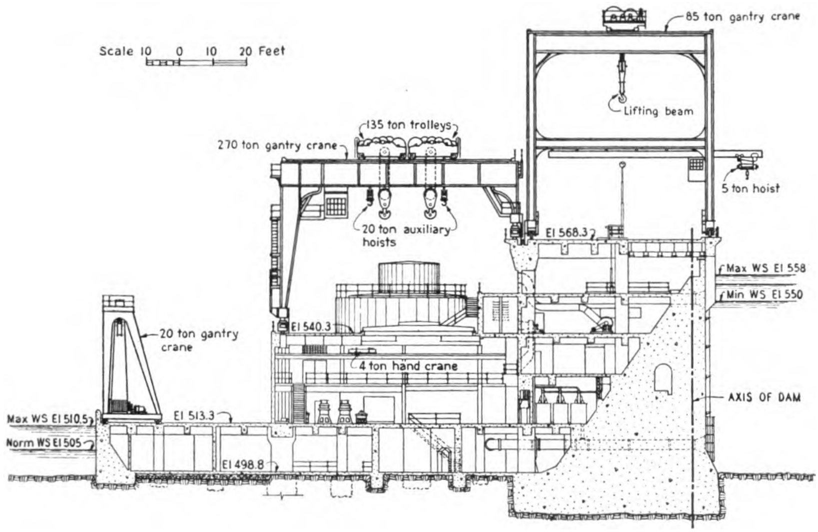

FIGURE 29.—*Section through service bay.*

The service bay floors in general are reinforced beam and girder design, with the girders parallel to the axis of the dam and supported on columns. The main girders are the two which support the 270-ton main powerhouse gantry crane and 85-ton intake gantry crane. The upstream girder extends the full length of the service bay and supports the upstream rail of the 270-ton gantry, the downstream rail of the 85-ton gantry, and a portion of the slab at elevation 568.3. The downstream girder supports the downstream rail of the 270-ton gantry crane.

The first floor of the service bay, at elevation 498.8, covers the entire area and is a 6-inch reinforced slab, except under the oil tank supports where an 8-inch reinforced slab is used. The second floor of the service bay, at elevation 513.3, is in general a 9-inch reinforced concrete slab supported on beams and girders. The third floor of the service bay is an intermediate floor, at elevation 528, extending from the bulkhead section to a line 56 feet downstream from the face of the dam. Twelve-inch concrete walls at the south and west, the intake concrete on the north, and the bulkhead on the east form the room which contains the 2,300-volt switchboard and the 440- and 110/220-volt auxiliary switchboards.

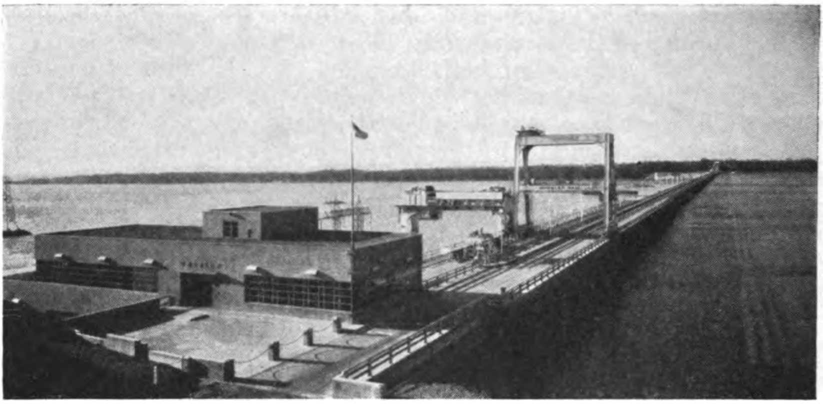

FIGURE 30.—***Architectural treatment—Control building exterior.***

The deck at elevation 540.3 is the roof of the turbine and generator erection floor and also the floor of the control cable gallery and the fan room. The floor slab is 12½ inches thick in the cable gallery and fan room, and 6½ inches thick in the balcony area. Elsewhere the deck consists of a 6-inch slab covered with a five-ply felt and cloth waterproofing membrane and 2 to 4 inches of concrete finish. The deck is supported by the steel frame of the service bay structure. Two 32-foot-diameter hatches covered by removable structural steel covers are provided for lowering the turbine runner and generator rotor to the repair floor at elevation 513.3. A rectangular hatch 12 by 15 feet is immediately over the transformer transfer track.

The fifth floor, at elevation 552.3, is similar in construction to the two floors below. The roof over the fifth floor is formed by the deck slab and the roadway slab at elevation 568.3. The deck slab is 12 inches thick and is waterproofed similarly to the deck slab over the turbine and generator erection room.

The station drainage sump is in the service bay, and has three chambers and a floor at elevation 462.3. The sump chambers are made accessible by means of manholes, a spiral steel stiarway and a steel ladder.

Control building.—Adjacent to the service bay, opposite the power generating units, is a five-story building in which is housed the main control room switchboards, the terminal room for control cables, and space or rooms for telephone equipment, batteries, carrier current equipment, air-conditioning equipment, machine shop equipment, sump pumps, first-aid room, toilet facilities for the employees, offices for the operating staff, and reception room, observation terrace, and other conveniences for the public. This structure consists of a concrete and steel frame superstructure on a concrete foundation. In the basement of this building and adjoining it on the south are filters and pumping equipment for furnishing treated water to the powerhouse and the village.

The downstream wall of the control building is a continuation of the service bay wall up to elevation 513.3. The north and south walls are reinforced concrete supported by the steel framework above elevation 513.3, and the west wall extending to elevation 568.3 is of similar construction. A system of concrete beams and girders supports the floor at elevation 513.3 above which, the steel columns of the superstructure are encased in concrete for fire protection purposes. In general, the exterior walls are 12 inches thick and the interior walls 9 inches. The floors vary from 4½ to 8 inches thick, the latter type having 4 to 5 inches of lightweight concrete for a base.

The roofs of the main building and elevator penthouse are composed of a 4-inch slab and are covered with 2 inches of corkboard and a membrane and gravel roofing.

Floor and roof loadings used in the design of the control building are as follows:

	Pounds per square foot
Penthouse and control building roof	75
Penthouse floor	300
Floor at elevation 569 and observation platform at elevation 568.3	100
Floor at elevation 552.3	100
Floor at elevation 540.3 (cable tray room)	200
(air-conditioning room)	100
(laboratory, etc.)	150
Floor at elevation 528.3	150
Stairs	100

The exterior of the control building is modernistic without cornice or other structural offsets in the wall lines.

Switchyard.—The switchyard is approximately 475 feet downstream from the powerhouse on filled material at the foot of a moderately steep bluff with the switching bays arranged at right angles to the powerhouse It covers an area 192 feet long and 84

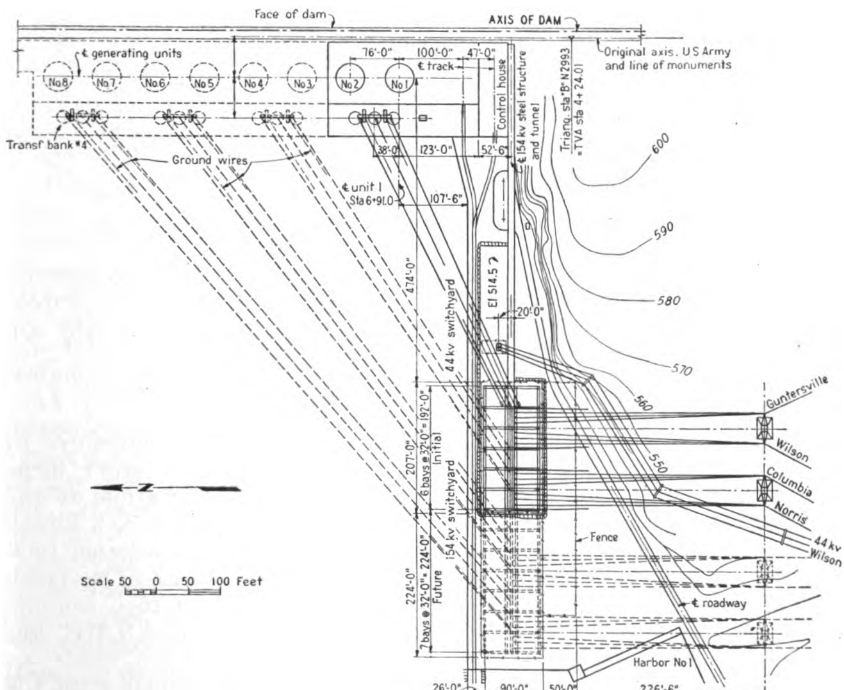

FIGURE 31.—*Switchyard—Location plan.*

feet wide and is so arranged that future extensions can be made both upstream and downstream.

There are six bays in the present 154-kilovolt yard. One bay serves the two present generating units through their common 72,000-kilovolt-ampere transformer bank at the powerhouse; four bays are for outgoing transmission lines, and the sixth bay is temporarily used for a tie between the main and the transfer buses but will be released for the next transformer bank when generator units 3 and 4 are added.

The oil circuit breakers rest on reinforced concrete mats, and the steel structure is supported on concrete footings. The cable tunnnel is a rectangular, reinforced concrete structure with inside dimensions 7 by 7 feet. It is on the longitudinal centerline of the switchyard and extends a total distance of 741 feet. The structure was designed to resist uplift from maximum tailwater at elevation 512. In certain sections of the tunnel provisions were made to care for special fill and foundation conditions.

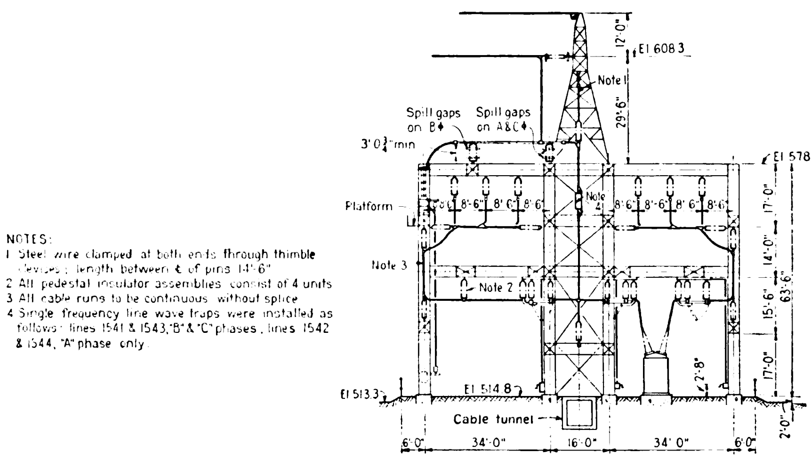

FIGURE 32.—*Section through 154-kilovolt switchyard.*

The bus and switch supporting structure is the conventional latticed galvanized steel column and girder construction with switchyard towers in the form of A-frame extensions attached to the top of the main structure.

Power-plant equipment.

Trashracks.—Trashrack structures protect the intake entrances of the generating units against the entrance of logs and other large trash which might damage or interfere with the operation of the gates or the turbines. Three main trashrack beams span each intake opening horizontally and support the rack panels. Each panel rack is 9 feet 2¾ inches wide and 13 feet 6 inches long. The trash bars are 6 inches deep and ⅞ inch thick, spaced on 6-inch centers. The maximum net velocity through the trashracks is 4.5 feet per second.

Forebay skimming screen.—A forebay skimming screen is in the front of each bay of the installed units, bolted on the outermost face of the piers, with top at elevation 560.5. The purpose of this screen is to keep floating surface trash away from the racks. Each screen is 18 feet 9 inches long, 11 feet 7/8 inch high, composed of 23 horizontal trash bars 6 inches by 7/8 inch and several cross bars added for stiffness.

Trash rake.—The trash rake assembly consists of three major parts: (1) the rake, made up of the rake frame and the rake mechanism and accessories; (2) the hoist, for raising and lowering the rake and accessories; and (3) the hoist carriage, for supporting the motor-operated hoist, the rake, and the traversing gear for propelling the carriage along the operating deck from one intake opening to another. The trash rake will handle and raise a live load of 1,000 pounds at a speed of 25 feet per minute.

When being lowered, the rake mechanism, other than the wheels and their supports, moves outward or upstream to a position 12 inches away from the face of the trash bars.

Upon reaching the bottom of the water passage, the rake moves back against the trashrack bars, enabling it, as it is drawn upward, to collect trash and foreign matter from the face of the trashrack. As the rake is hoisted above the lip of the trash sluice, at elevation 559.3, the collected material is dumped into the trash sluice for disposal.

The rake hoist is the motor-driven, three-drum type, mounted on the hoist carriage. The hoist is designed to raise and lower the trash rake and its live load, control the movement of the rake mechanism, and make any other necessary movements of the trash rake. The trash rake motor is interchangeable with any of the spillway gate motors.

Intake gates.—In addition to two emergency bulkhead gates, 24 service gates are required, 3 for each turbine intake. Of these 24 gates, 7 are active and 17 inactive. This provides active gates for the two units with one spare gate. All are alike except that the wheels have not been installed on the inactive gates. The wheel housings on the inactive gates have been temporarily closed and timbers attached to serve both as bearings and water seals. See figure 33. The gates are raised and lowered by the intake gantry crane utilizing a lifting beam equipped with an automatic grappling device. Normally, both the active and inactive gates will be raised or lowered with no flow through the intake and balanced pressures on both sides of the gate leaves. The active gates equipped with wheels and with spring-loaded bronze seals are designed to be operated with full unbalanced water pressure of 60 feet.

All service gates are equipped with water ballast compartments formed by skin plates on both faces of the gates. These compartments are provided with self-filling and self-draining valves so that as the gate is lowered from the raised position the compartments will fill with water. This water can be drained after the gate is seated.

The gate track seat is of structural steel anchored to the concrete at the downstream side of the gate slots. Cast-steel tracks are bolted to the track seats.

Each service gate is 20 feet 11 1/8 inches wide over all and 38 feet 3 3/4 inches high with lift hooks at the top of the gate. The gate is 24 inches thick with a 3/4-inch skin plate on the upstream side and a 1/2-inch plate on the downstream side. The skin plate is welded to the frame. The seven active gates are provided with a train of eleven 27-inch-diameter wheels on each side.

The bottom gate seat is an I-beam set in the concrete floor. On top of the beam is fastened a 6-inch channel with edges up, forming a trough. This trough is filled with soft babbitt flush with the floor surface. Seats and seals are also provided at the top of the gates. On the active gates the side and top seals, consisting of cast manganese bronze retainers, rolled bronze sealing strips, and phosphor

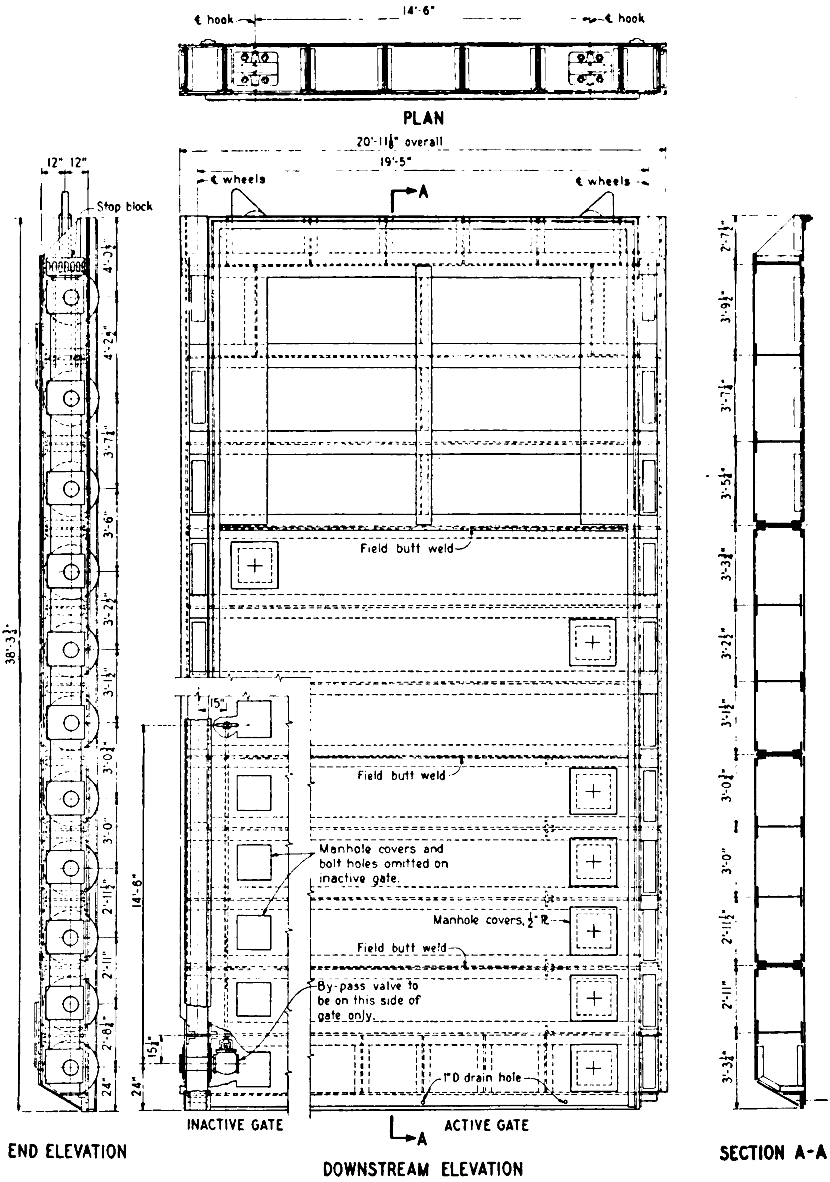

Figure 33.—*Intake service gate.*

bronze wire springs, are bolted to the gates. See figure 34 for these details. For the inactive service gates, steel plates are fastened over the vacant wheel openings, and on top of these are fastened oak seals. At the top a similar oak seal is provided. See figure 34 for these details.

Emergency bulkhead gates.—Two emergency gates 19 feet 2½ inches center to center of end beams and 42 feet 11 inches high are provided for use when service gates are to be repaired. Each gate is equipped with two lifting hooks. The gates are 24 inches thick, and the skin plates are the same as on the service gates. The side seals of the gates, bearing against the track seat in the slot, and the top seal are oak timbers bolted to the gate.

Intake gantry crane.—An 85-ton gantry crane with a 5-ton monorail auxiliary hoist is on top of the intake structure to handle the service and the emergency intake gates. A lifting beam with two lifting links is used. The gantry is the box girder type with four legs of equal length. A closed steel type operator's cage is on the upstream side.

The gantry is driven by a motor geared to one driving wheel on each side. To this is attached an automatic electric brake having a capacity equal to the maximum pull-out torque. Also, a locking device is provided to prevent the crane from drifting along its track

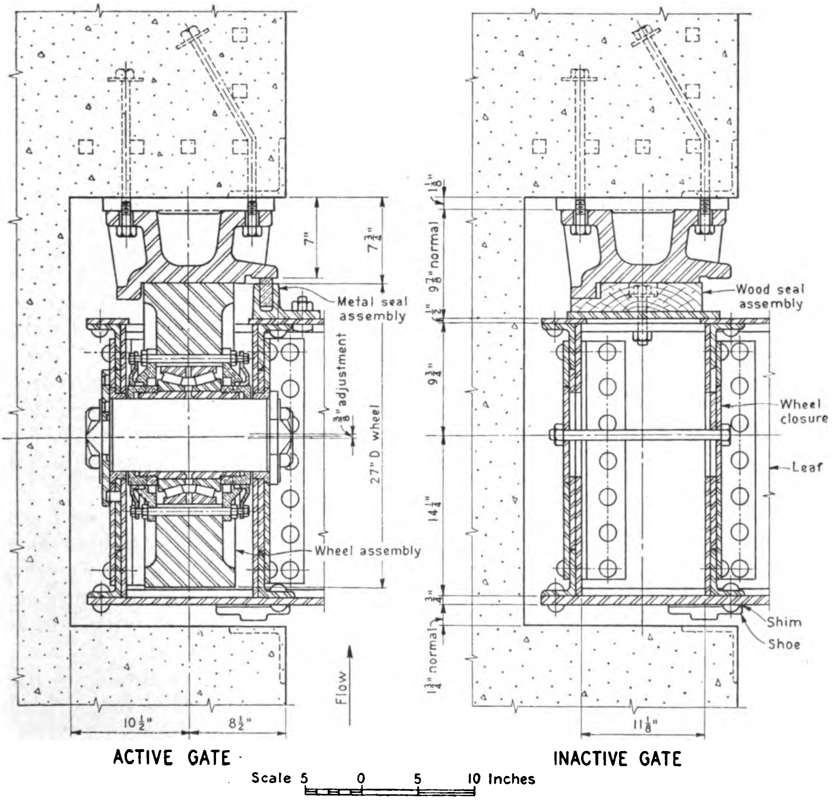

Figure 34.—*Intake gate seals.*

FIGURE 35.—*85-ton gantry crane for intake gates.*

when loaded. Sanders, electrically operated from the operator's cage, are provided for sanding the gantry rails.

The main hoise is motor driven and is mounted on the trolley. An automatic electric brake on the main hoist motor has a capacity equal to its maximum pull-out torque. Both hoists are provided with mechanical load brakes to prevent the loads from lowering unless the hoist motors are revolving under power in the lowering direction. The lifting beam on the main hoist is provided with automatic hooking devices. A lever and pawl mechanism at the center of the beam controls the links which are suspended 14 feet 6 inches apart.

All motors are the outdoor, totally enclosed, wound-rotor type, designed for operation at 440 volts, 3 phase, 60 cycles. Five motors are used on the crane, one 60-horsepower for the main hoist, one 10-horsepower for the monorail auxiliary hoist, one 60-horsepower for the gantry drive, one 5-horsepower for the main trolley drive, and one ¾-horsepower for the monorail or auxiliary trolley drive. The main collector conductors are 20-pound steel rails reinforced with No. 2/0 copper cable and mounted on porcelain insulating blocks. The various speeds in feet per minute of the crane are: main hoist 7, auxiliary hoist 25, gantry travel 100, main trolley 25, and auxiliary trolley 25.

All control equipment is in the operator's cage. Motor speed controllers are the manual reversing drum type providing six speed control points in each direction of operation for the gantry travel, main hoist, and main trolley motors and five points for the auxiliary hoist and the monorail trolley motors. They are fitted with control handles providing automatic return to the "off" position.

Scroll case filling system.—The filling line for unit 1 is a 36-inch cast-iron pipe at elevation 507.3, extending from the face of the dam to the scroll case. A 4-foot square trashrack, together with a 5- by 5-foot emergency gate and hand hoist controlled from the parapet is provided at the face of the dam. The scroll case for unit 2 is filled from scroll case 1 by a 36-inch pipe connecting the two scroll cases and controlled by a low-pressure gate valve. Four piezometer openings for connection with the turbine flow meter are provided.

Scroll case unwatering system.—A cross trench in the floor at each scroll case entrance immediately downstream from the intake piers, intercepts leakage from the head gates. A 20-inch cast-iron drain pipe leads from a sump in this trench to a common 30-inch drainage header located downstream from and parallel to the centerline of units. This header drains into the unwatering sump in the service bay, and the water is pumped from the sump into the tailrace by two 4,500-gpm motor-driven 24-inch deep-well pumps. In the event one pump cannot handle the flow, the second pump goes into service automatically.

Draft tube unwatering system.—The 30-inch common unwatering header is connected to each draft tube through a 20-inch pipe and valve. This permits the draft tubes to be unwatered down to about elevation 470 into the sump in the service bay. For unwatering the remainder of the draft tube a 24-inch drain inlet is set at the lowest point in the floor, from which a 24-inch cast-iron pipe leads to vertical wells in each of the two intermediate piers. Permanent motor-driven pumps may be set over these wells on the floor of the unwatering gallery and connected with piping which discharges above normal tailwater level.

Turbines.—The hydraulic turbines for the two initial power units are the vertical-shaft, single-runner, fixed-blade, propeller type. Gross

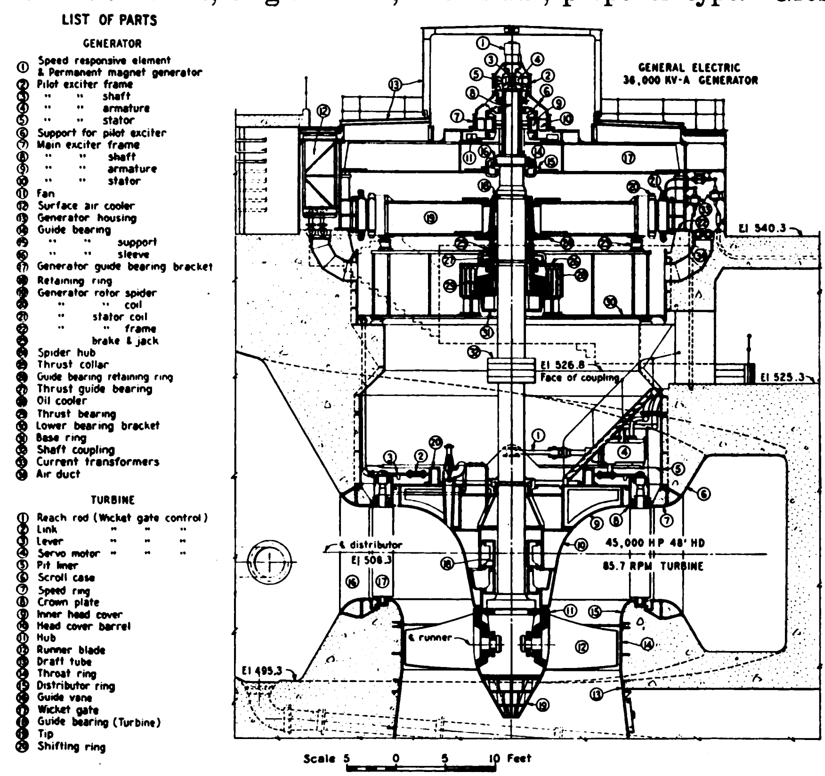

FIGURE 36.—*Section through generator and turbine.*

head on the units varies from about 40 to 53 feet, the normal head being approximately 48 feet.

Each turbine has a rated capacity of 45,000 horsepower, under net head of 48 feet, and an operating speed of 85.7 revolutions per minute. The maximum guaranteed efficiency is 89.5 percent under full load. The turbines have a runaway speed of 155 revolutions per minute under a head of 54 feet with no load on the generator except friction and windage.

To permit the use of generators as synchronous condensers to regulate the power factor and voltage on the transmission lines, provision was partially made for admitting compressed air to the turbine casing to depress the water level in the draft tube.

The turbine runner has a nominal diameter of 264 inches; the diameter of the throat ring, and the actual diameter, is 263.58 inches. The six cast-steel blades are keyed to a 90¾-inch-diameter cast-steel hub at a pitch of about 21 degrees at the tip. Both front and back of the blades are finished smooth by grinding, and any irregularities which might be conducive to pitting were corrected by welding and grinding. A ¾-inch stainless steel wearing surface 36 inches long was welded upon the outer edge of each blade near the discharge end to resist cavitation.

The speed ring is made of cast steel and sectionized into six segments as necessitated by casting and handling. It was designed to support the weight of the superimposed building structure, the weight of the generator stator, and the rotating load carried by the generator thrust bearing.

The top flange of the speed ring is bolted to the lower pit liner and also forms a support for the outer head cover. The bottom flange is bolted to the distributor ring, which in turn is fastened to the throat ring and draft tube liner.

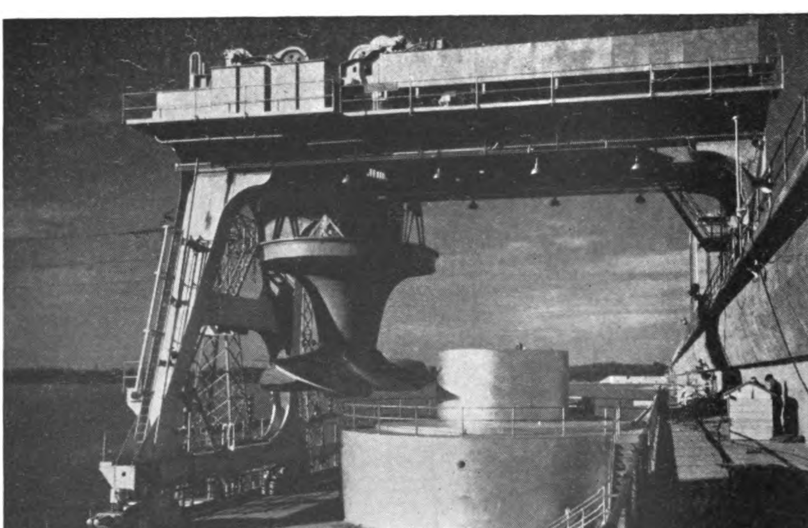

Figure 37.—*Turbine runner.*

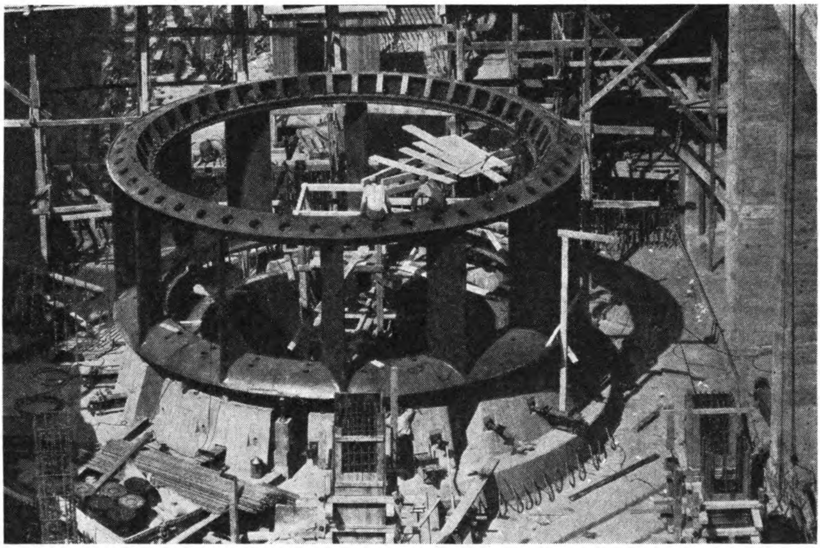

Figure 38.—*Speed ring.*

The shaft guide bearing is the rubber-lined, water-lubricated type located immediately above the runner. Lubricating water is supplied by a 3-inch standard brass pipe line which takes water to the bearing through an opening in the head cover. A flow meter with electrical signal contacts sounds an alarm whenever the water flow becomes inadequate.

The main shaft, made from open-hearth forged steel, is 285½ inches long to the generator coupling, and is 33½ inches in diameter, with an 8-inch center hole for inspection.

The head cover is made in three parts—an outer head cover, an inner head cover, and a head-cover barrel. The outer head cover is made of cast steel and divided into four segments. It supports the interior fixed parts of the turbine and the bearings for the upper guide vane stems. During erection or dismantling, the inner cover provides a temporary support for the runner and turbine shaft. An annular separation between the outer and inner head covers permits the dismantling of the turbine and removal of the runner without disturbing the wicket gates.

The head-cover barrel is bolted to the bottom of the inner head cover and streamlines the path of the water as the direction changes from horizontal radial flow at the wicket gates to axial vertical flow at the runner.

The distributor ring, 33¼ inches high by 332 inches in diameter, is composed of four cast-steel segments bolted together. This ring, supported on a flange of the speed ring, forms the transition between it and the throat ring and supports the bottom bearings for the 24 guide vanes.

The throat ring in which the runner revolves is made of four steel castings and connects the distributor ring with the draft tube liner. It has an internal diameter of 264 inches.

The weight of the rotating turbine parts is 267,000 pounds, and the heaviest complete assembly which is handled by the powerhouse crane weighs 432,000 pounds. The weight of the rotating parts plus the unbalanced hydraulic thrust on the runner is 1,300,000 pounds, and the turbine flywheel effect is 4,500,000 pounds-feet-squared.

The turbine gates, which are the balanced wicket type, control the quantity of water flowing through the turbine runner and hence the load being carried by the generating unit. The position of these 24 gates or guide vanes is in turn controlled through connecting linkage attached to a shifting or operating ring that opens or closes all gates simultaneously and equally, by two oil-pressure cylinders or servomotors. A deviation in the revolving speed of the flyballs of the governor causes the servomotor either to open or to close the wicket gates, thereby bringing the unit to its proper frequency or speed.

The wicket gates and integrally cast stem are made of cast steel. Each gate is 112 inches high, 43 inches wide, and has a maximum thickness of 9.36 inches.

The turbine servomotors are double-acting and rated at 600,000 foot-pounds with a net oil pressure of 300 pounds per square inch in the cylinders. This oil pressure may actually vary from 250 to 300 pounds per square inch. The volume of oil per stroke is 24,000 cubic inches; the diameter of the servomotor cylinders is 24 inches, and the stroke 27⅜ inches. The cylinders and governor system are designed for a full opening or closing stroke in a minimum of 4 seconds. The equipment is adjusted to operate in 10 seconds.

The servomotor cylinders are mounted on machined surfaces and rigidly supported by the turbine pit liner. The cylinders are provided with ports to retard the rate of closure from slightly below

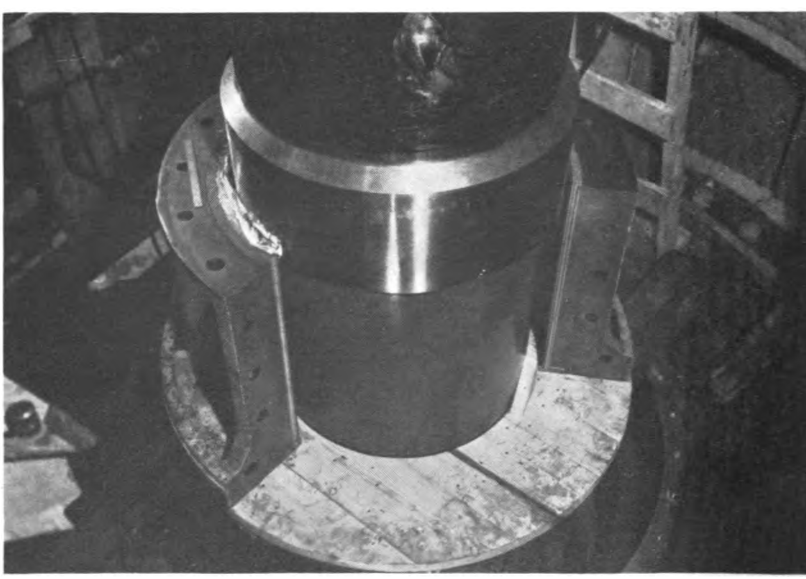

FIGURE 39.—*Guide bearing.*

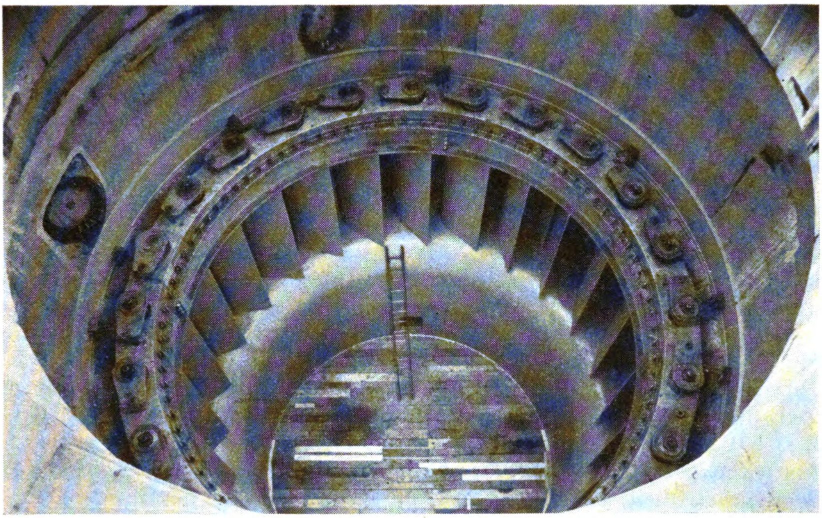

FIGURE 40.—*Wicket gates.*

speed-no-load position to the fully closed position so as to avoid shock when the gates near full closure. Mechanical locking devices hold the guide vanes in full open- or tight-closed position, and also block the gates so that they cannot be opened past any predetermined amount between half gate and full gate.

Each servomotor is connected to the operating ring through a 6.5-inch-diameter forged-steel piston rod and a 6-inch-diameter forged-steel reach rod. The operating ring is composed of four steel castings bolted together and moving circumferentially on hard brass bearing plates with a 95-inch radius at the connecting rod pins. Links connect the operating ring with 36-inch levers keyed to the top of the guide vane stems. A shear pin is in each lever and stops prevent any gate whose shear pin has failed from interfering with adjacent gates.

All of the moving parts of the gate mechanism are provided with standard grease fittings. A grease pump, at elevation 528.3, supplies grease to a pipe ring about the turbine pit. The gate stem bearings are piped for a separate grease supply to three bronze-bushed guide bearings and a thrust bearing.

Air was originally admitted to the turbine at gate openings between 0 and 65 percent through a 10-inch vent pipe entering the turbine through the inner head cover plate. However, the operation of the unit was then objectionably noisy and rough and at the same time a pressure existed immediately under the head cover. This trouble was remedied by welding a ¾-inch plate across the original opening, chipping a 10-inch-diameter hole through the air duct, connecting this opening by means of 10-inch piping to one of the cover plates at the top of the head cover barrel, and drilling twenty-four 2-inch-diameter holes equally spaced about the bottom of the head cover barrel.

Originally the air vent opened into the turbine pit, but to reduce the noise an 8-inch vent pipe was carried to the outside. The vent on unit No. 2 was carried out through the concrete to the downstream side of the building. Construction on unit No. 1, however, was too far along so that this vent was taken up through the opening in the turbine pit where the governor piping is located, thence vertically through the generator floor at elevation 540.

A number of piezometer lines were installed to facilitate turbine testing and to secure a continuous record of flow through each turbine, as well as static water levels and pressure readings. Taps and piezometer lines were installed in each turbine scroll case at locations in accordance with the Winter-Kennedy [11] principle. These piezometer lines were used with the flow meters to indicate the water flow through the turbine.

Acceptance tests were run on both units. The turbine contracts guaranteed that the units would not reach a runaway speed greater than 191 revolutions per minute at a 54-foot head. Field tests were conducted under a head of approximately 50 feet. The maximum speed reached on unit No. 1 was 148.5 revolutions per minute at 72 percent gate opening and 149 revolutions per minute at 76 percent gate opening on unit No. 2.

The official capacity tests on the units were conducted April 15 and 16, 1937. The outputs were as follows:

Unit	Head	Kw.		Hp.	
		Meter	Actual	Tests	At 48-foot head
1	49.90	37,000	37,680	51,600	48,700
2	50.05	37,250	38,100	52,200	49,000

Since the output guaranteed was 45,000 horsepower at 48-foot head, the power exceeded the guarantee by approximately 4,000 horsepower, or slightly over 8 percent.

The official efficiency tests of the units were made on a 16-inch model in the hydraulic laboratory of the manufacturer at Eddystone, Pennsylvania. The model was completely homologous with the prototype and included draft tube, runner, scroll case, intake, and racks. Tests were conducted under actual heads of from 3 to 4 feet but over a wide prototype range of heads that were obtained by varying the test speed of the model.

The model tests indicated the following field performance:

Head (feet)	Efficiency percent	Horsepower
48	92.8	44,000–45,000
43	92.1	38,000–40,000

Tests were conducted in the I. P. Morris Laboratory on a modification of the Wheeler model draft tube to determine the effect on runner

[11] Winter, I. A., *Improved Type of Flow Meter for Hydraulic Turbines.* American Society of Civil Engineers, Transactions, 1934, 99 : 847–872.

performance of various slopes of the downstream leg of the draft tube. From this test it appeared that, for the runner used, different bottom slopes of the draft tube made no difference in the turbine performance. There appeared to be a tendency for the power to increase with greater draft tube slope.

The Simplex flow meters installed at each unit in the powerhouse were calibrated by field tests. The tests consisted of accurate determination of the generator output, headwater, tailwater, gate opening, net head, and difference of pressure existing between pairs of Winter-Kennedy piezometer openings. The data were reviewed by I. A. Winter,[12] who decided upon the flow equation to be used which led to the adoption of 91.4 percent as peak turbine efficiency for unit No. 1 and 91.0 for unit No. 2 at 49-foot net turbine head.

Wicket gate leakage was determined by measuring the drop of water surface in the intake at fixed intervals with the head gates and turbine gates closed. The volume corresponding to each increment of drop was then determined. With 50-foot head the rate of leakage was 37.5 cubic feet per second.

To permit operation of the units as synchronous condensers, it is necessary to depress the water surface in the draft tube by compressed air to a point below the runner; otherwise an input of about 20,000 kw. is required to rotate the runner at normal speed. The water level must be depressed approximately 8.5 feet since normal tailwater at elevation 505.3 is 6 feet above the centerline of the runner. A temporary installation of four pressure tanks with a total capacity of 2,800 cubic feet and an 8-inch supply pipe was made for a test conducted August 3, 1937. This installation operated satisfactorily but was not of greater capacity than needed for proper operation. During these experiments the tailwater was at elevation 504.5. This trial installation was the basis for subsequent partial provisions made at other main-river plants.

Governors.—The governors are the oil-pressure, relay-valve actuator type with synchronous motor-driven, speed-responsive elements. Actuators and oil pumps are mounted on the sump tanks and enclosed in metal cabinets. All gages and indicating devices are mounted on the front panels of the cabinets. The arrangement of pumps, tanks, and controls is such that operation either may be as two independent unit systems or as one twin system.

The restoring mechanism of the governor is operated by ¼-inch-diameter, airplane-type flexible cable connected to one of the piston rods of the servomotors. The cable is supported on grease-packed ball-bearing sheaves. A counterweight at the actuator end forces the cable to work in tension in both directions.

The actuators are provided with an adjustable control which may be set to operate the turbine gates over the full-opening or full-closing stroke at any rate between 4 and 10 seconds. The adjustment positively restricts the oil flow to permit a maximum rate of gate closure of 4 seconds and is so arranged that the operation of the speed-control mechanism, control solenoid, or other devices cannot cause a rate of movement of the turbine gates exceeding the maximum rate for which it is adjusted.

[12] Engineer with the U. S. Bureau of Reclamation.

The speed responsive element of each actuator is driven by an alternating-current synchronous motor which receives its power from a permanent magnet type synchronous generator direct-connected to and mounted above the main generator shaft. The speed of the responsive element varies directly with the speed of the turbine for all rates of acceleration and deceleration.

The governor is sufficiently sensitive to start a corrective movement of the turbine gates upon a speed variation of the turbines of 0.01 percent from normal. The governor control valve is capable of a full opening or full closing stroke in 1/4 second or less, starting from time of speed change.

Each actuator is provided with the following equipment:

1. *Load limit.*—A gate-limit control device which can be operated manually at the actuator and also electrically from the switchboard.

2. *Speed level.*—The speed control can vary the speed from 85 percent of rated speed at no-load and zero speed droop to 105 percent of rated speed at rated load and the maximum speed droop of 6 percent.

3. *Automatic shut-down.*—This is a latching device which shuts down the turbine automatically upon overspeed, failure of governor oil pressure, excessive temperature of main shaft bearing, or upon operation of any of the automatic protective features in connection with the main generator, exciter, pilot exciter, or transformers.

4. *Speed indicators.*—Two electrically operated indicators, one at the governor and one at the switchboard not only indicate the normal speed of the turbine but also show when the turbine begins to rotate and when it comes to a dead stop.

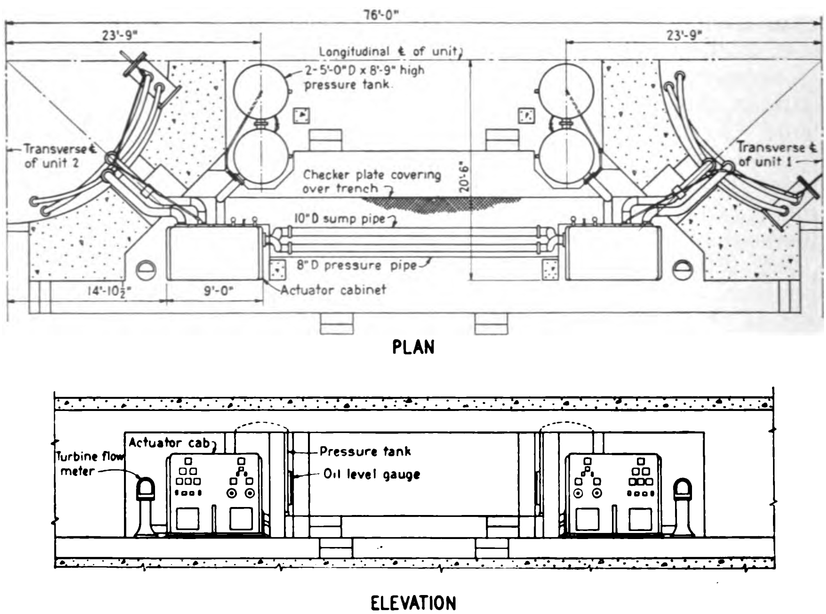

Figure 41.—*Arrangement of governor mechanism and control boards.*

5. *Manual control.*—The turbine gates can be controlled by means of oil pressure from the governor oil-pressure system by manual control at the actuator.

6. *Pressure gage.*—A duplex pressure gage, mounted on the actuator, indicates the air pressure in the air system and also in the generator brake cylinders. Another pressure gage indicates the pressure in the governor oil pressure tank in pounds per square inch.

7. *Limit and position indicators.*—A dual type selsyn gate-limit and turbine gate-position indicator is provided on the switch-board, and on the actuator.

8. *Overspeed switch.*—An overspeed switch mounted in the permanent magnet generator housing shuts down the turbine and sounds an alarm upon overspeed, and is adjustable to shut down at any speed above 125 percent of rated speed.

9. *Speed droop.*—The device for controlling the speed droop of the turbine can be operated manually at the actuator and also electrically from the switchboard. Specifications call for the speed droop to have between 0 and 6 percent adjustment. The test of the first governor indicated that at full gate the adjustment is from 0 to 7.8 percent.

10. *Hand-operated air valve.*—This valve controls the operation of the generator brakes and is provided with interlocks so that the brakes cannot be applied until the turbine gates are fully closed and the generator disconnected from the bus.

11. *Automatic air valve.*—This valve controls the operation of the generator brakes and is provided with interlocks so that the brakes cannot be applied until the turbine gates are fully closed, the generator disconnected from the bus, and the speed reduced to about 50 percent of normal. Brake application is intermittent, and the time periods are adjustable.

12. *Pressure switches.*—One of the switches closes an alarm circuit when the governor oil pressure drops to any predetermined value. Contacts of the other switch close on extremely low oil pressure to cause the unit to shut down if running. If at a standstill, the turbine is prevented from starting.

Each governor has one 300-gallon-per-minute herringbone gear-type oil pump directly connected to a 100-horsepower motor. It has a manually operated discharge valve, a check valve, an oil-pressure-operated unloader valve, a pressure safety valve, and a pressure control switch.

An automatic control is included in each governor system to start the pump when the oil pressure or the oil level in the pressure tank drops to a predetermined amount, and to stop the supply of oil when the oil pressure or oil level rises to a predetermined amount. When operating as a twin system, the interconnection and automatic control of the two pumping units is such that either one may be used for normal operation, the other unit serving as a stand-by arranged to start automatically either on failure of the electric current supply to the operating pump or when the oil pressure or level falls to a predetermined amount. Whenever the stand-by pump starts, an alarm is sounded, and both it and the operating pump continue to operate until manually stopped.

Each governor has two pressure tanks of 196 cubic feet capacity in the turbine gallery. Under normal operation each tank contains one volume of oil to two volumes of air at 250 pounds pressure. Each tank is equipped with sight gages to indicate the oil level and a regular to maintain automatically the proper quantity of oil. A float-operated valve is located within the tank so that the oil cannot fall so low as to admit air from the upper part of the pressure tank into the oil piping system.

One sump tank of 135 cubic feet capacity is provided for each governor. They are interconected to permit either tank to be drained while the other is in service.

All the connecting piping between the oil pumps, sump tanks, pressure tanks, and actuators, and between the actuators and the servomotors is stainless-steel tubing with long radius bends and is welded wherever practicable. The pipe is of such size that the oil velocity for full gate travel in minimum specified time does not exceed 12 feet per second.

Generators.—The initial electrical installation consists of two vertical-shaft generating units coupled to two hydraulic turbines. Each generator is a 3-phase, 13,800-volt unit rated at 36,000 kilovolt amperes, or 32,400 kilowatts at 0.9 power factor, and is driven by a direct-connected turbine at a speed of 85.7 revolutions per minute. The flywheel effect of the generator is 61,260,000 pounds-feet-squared. The main exciter and a pilot exciter are mounted above each main rotor and driven by the main shaft. The generators have enclosed air-circulating systems cooled by water, and carbon dioxide fire-protection equipment.

The stator frame is made of rolled and welded steel plates fabricated in four sections to facilitate shipment. The stator core is made of high-grade silicon-steel laminated punchings.

Twelve temperature detectors are embedded in the slots between the two coil sides and distributed in groups around the periphery of the stator at points subject to the highest temperatures. Leads are brought out to a terminal board mounted on the side of the machine.

The rotor spider was assembled at the powerhouse. The spider consists of fabricated steel radial arms bolted at top and bottom to central circular steel plates bolted to a cast-steel hub. The rim consists of segmental sections punched from thin steel plates built up on long through studs. The lamination plates are punched on their inner edges to form key slots to match equal keyways machined in the ends of the spider arms and also on their outer edges to form dovetail slots to receive the pole pieces. The pole pieces are built up of thin punched steel laminations secured by rivets passing through end plates which clamp the laminations and also support the ends of the rotor coils. The poles are dovetailed to the rotor and held by tapered keys which are locked into place at top and bottom of the rotor by segmental plates.

The field coils are copper strips wound on edge with asbestos insulation between turns. Each coil is insulated from its pole by sheets of mica. Insulating collars of canvas treated with bakelite are placed at the top and bottom of each coil. Under each coil heavy springs exert an outward pressure on the winding and compensate for any shrinkage which may occur.

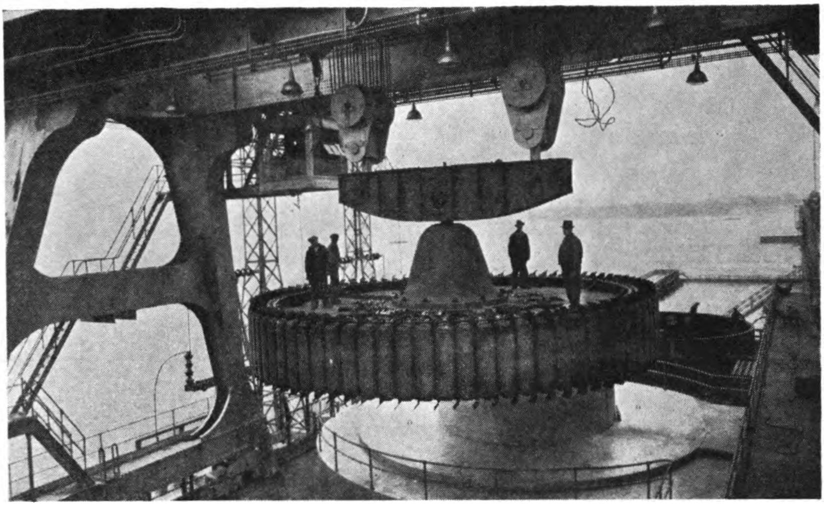

FIGURE 42.—*Generator rotor.*

The generator shaft is made of forged steel, having a flange at the lower end to match the turbine shaft flange. The shaft is 33½ inches in diameter and accurately machined throughout. A hole 8 inches in diameter was bored throughout the entire length of the shaft and machine-finished sufficiently to permit visual inspection of the interior of the shaft.

One guide bearing is above the rotor, and one combination guide and thrust bearing is below. The upper guide bearing and the casing immediately over it are made in sections in order that the bearing may be removed without disturbing the exciter. The bearing is the cylindrical-seat, babbitt-lined sleeve type. The lower guide bearing is in the upper part of the thrust bearing housing. The thrust bearing consists primarily of two circular plates, one a babbitted stationary plate supported by 1,384 precompressed springs resting on a base and the other a special cast-iron rotating plate attached to the lower end of the shaft thrust collar. Both stationary and rotating plates have radial oil grooves chamfered at the edges to insure a generous oil supply to the bearing surfaces. The bearings are self-lubricating by means of oil reservoirs which have piping connections for filling and draining for filtering as may be required. The oil reservoir serving the lower guide and thrust bearing is provided with cooling water coils supplied by cooling water pumped at the rate of 100 gallons per minute.

Air-operated brakes on each generator are of sufficient capacity to bring the rotating parts of generator and turbine to a stop from half-rated speed within 7½ minutes. Eight brakes are mounted on the lower bearing bracket and bearing against the lower rim of the rotor. Brake operation is by means of air at 90 to 100 pounds per square inch pressure. The brake shoes are a metallic asbestos compound designed to reduce burning and thereby to minimize replacement.

The brakes are also designed to serve as hydraulic jacks to lift the generator rotor, shaft, and water wheel runner for removal or ad-

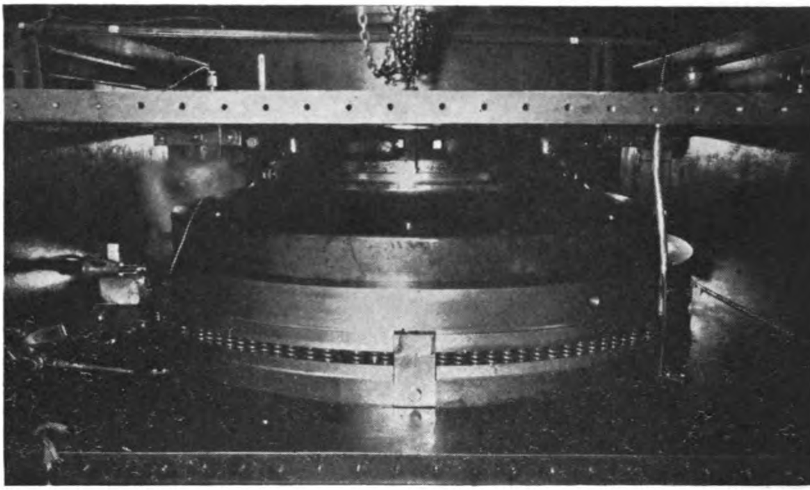

FIGURE 43.—*Guide and thrust bearing.*

justment of the thrust bearing. When thus used as jacks, a small portable oil pump is connected to the header under the brakes. These jacks are capable of lifting the rotating parts one-half inch, and when the rotor is lifted this distance, hinged metal blocks are placed under it to take the weight off the thrust bearing. At an oil pressure of 1,200 pounds per square inch these hydraulic jacks have a lifting capacity of not less than 840,000 pounds.

Each generator has an enclosed ventilating system, self-contained for recirculating air through the machine. Circulation of air is accomplished by fans on the generator rotor. No external blowers are required. The air discharges through stator openings and thence through surface air coolers within the generator housing, then passes back to the top and bottom of the rotor. There are eight surface coolers equally spaced around the periphery of the ventilating housing. Cooling water is supplied by two pumps from separate sources. The cooling water system serves both the generator surface coolers and coils in the thrust bearing oil reservoir. Each pump has a capacity of 1,300 gallons per minutes at 50 pounds per square inch pressure at the coolers.

The exciters of each generator have a separate ventilation system consisting of two small fans mounted on the upper bearing bracket and driven by a 1-horsepower motor. The two fans are capable of delivering 2,000 cubic feet of air per minute against ½-inch water pressure.

Surge protective equipment for each generator consists of lightning arresters and capacitors connected to the main leads. The arresters limit impulse voltages approximately to 38.5 kilovolts, which is safely below the rated impulse strength of the generator windings. The capacitors limit the rate of voltage rise.

Each generator has a neutral oil circuit breaker, but one reactor serves both generators, and the breakers are interlocked so that one circuit breaker trips out when the closing circuit of the other breaker is energized. The neutral breakers and common reactor are enclosed in safety-type metal housings.

A fire protection system of the carbon dioxide gas, solenoid-operated type is provided. When operated, it will maintain a concentration of not less than 25 percent CO_2 gas in the generator housing for a period of not less than 30 minutes. If an internal electrical fault develops in the generator windings, causing differential relays to operate, or if thermal relays installed in the hot-air duct of the ventilating system function because of excess temperature, the generator will automatically be tripped out of service, the turbine gates closed, brakes applied, and the control circuit energized for releasing an initial discharge of carbon dioxide gas. The gas is released from the 50-pound cylinders by mechanically puncturing a thin metal seal in each cylinder by means of falling weights. The gas release can, if necessary, be operated manually from break-glass control stations.

A main exciter of 200 kilowatt and 800 amperes and a pilot exciter of 17 kilowatt and 68 amperes are mounted above the rotor of each generator. Both are operated at 250 volts direct current. The main exciter, which is shunt wound, receives its excitation from the self-excited pilot exciter which is compound wound. The excitation system is designed to give a response ratio of 1.0 with a ceiling voltage of 380 volts.

Main gantry crane.—A 270-ton main gantry crane is provided for assembly and maintenance of the hydroelectric generating equipment. It is the outdoor traveling type with unequal length legs and a horizontal span of 69 feet center to center of the runway rails. The upstream rail is 28 feet above the downstream rail. The crane is electrically operated and equipped with two trolleys, each provided with a main and an auxiliary hoist. Power is supplied from rigid conductors in a trench below and outside the upper gantry rail.

The full-load capacity of the crane is 270 tons or 135 tons for each main hook. Each auxiliary hoist has a capacity of 20 tons. Each

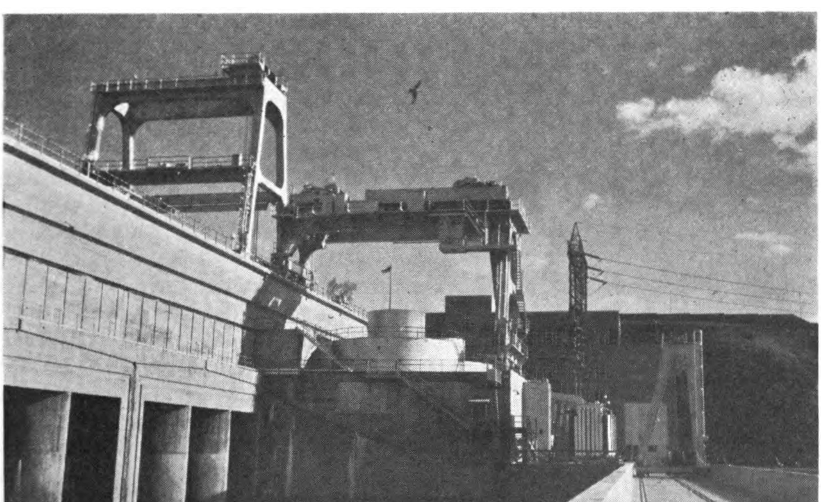

FIGURE 44.—*Cranes.*

trolley is driven by a motor mounted on the trolley frame and connected to one driving wheel on each side of the trolley. Each main hoist and auxiliary hoist is driven by a separate motor. All master control equipment is in the operator's cage where the magnetic control panels and resistors are mounted in weatherproof cabinets. The motor controllers are the reversing magnetic type for plugging service operated by master drum-type switches.

All motors are the splashproof, outdoor, wound-rotor type for operation at 440 volts, 3 phase, 60 cycles. Starting torque is 200 percent of full-load torque with rated voltage and frequency applied. Hoist motors are designed to withstand 50 percent overspeed and all other motors 25 percent overspeed. The motors for the main hoist and auxiliary hoist are rated at 60 horsepower at 600 revolutions per minute, and parts are interchangeable. The gantry travel motor also has the same rating, while each trolley motor has a rating of 15 horsepower at 900 revolutions per minute.

The various speeds of this crane in feet per minute are: main hoist 4, auxiliary hoist 25, main and auxiliary trolley 25, and gantry travel 72.

Draft tube gantry crane.—A 20-ton outdoor traveling gantry crane operates on the elevation 513.3 transformer deck and is used for handling the draft tube gates. It is equipped with a fixed hoist and a lifting beam permanently attached. It is electrically operated, controlled from a platform on the gantry base, and is equipped with cable reel for connection to plug outlets adjacent to the gantry rail. The span is 16 feet. The crane has a net full-load rated capacity of 20 tons on the 2 gate lifting links. The gantry travel is effected by a motor rigidly supported on the gantry base through gears and shafting to 2 wheels. The crane is equipped with two 20-horsepower motors of the splashproof, outdoor, wound-rotor type designed for operation at 790 revolutions per minute, 440 volts, 3 phase, 60 cycles. The starting torque for these motors is not less than 200 percent of the full-load torque with rated voltage and frequency applied.

Main generator leads and switching.—The main leads between the generator terminals and the generator breakers consist of two 1,500,000-circular-mil, paper-insulated, shielded, lead-covered cables per phase, run in transite ducts, embedded in the building structure. Each generator oil circuit breaker is the vertical-lift disconnecting type enclosed in an outdoor steel cubicle. A transformer bus is between the two breakers, connecting the breakers to the transformer bank. The bus consists of bare copper tubing on porcelain insulators in concrete and steel enclosures. A third breaker in an outdoor cubicle is connected to this bus to control the station service transformer bank associated with units 1 and 2. Current transformers are within the generator housing and the breaker cubicles. Potential transformers are in a masonry compartment below the transformer bus. The neutral lead from each generator to its neutral breaker consists of one 500,000-circular-mil varnished cambric, shielded cable, run in transite conduit. A neutral bus is provided between the two neutral breakers of generators 1 and 2 and is grounded through a 0.858-ohm reactor. The two neutral breakers are interlocked in order that only one may be closed at a time.

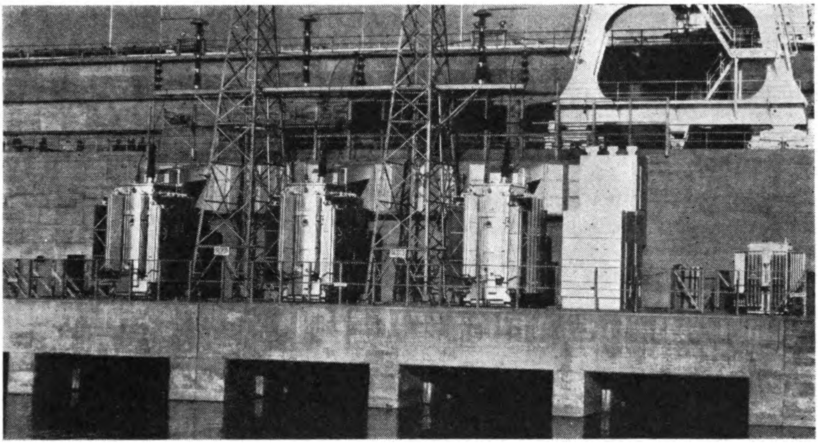

FIGURE 45.—*Main transformers.*

Main transformers.—Generator units 1 and 2 are electrically connected through their oil circuit breakers and a short transformer bus to a 72,000-kilovolt-amperes, 13,200/154,000-volt transformer bank, consisting of three 24,000-kilovolt-amperes single-phase transformers, which are adjacent to the generator deck on the downstream side of the powerhouse. The transformer connections are delta-star. The 154,000-volt neutral is solidly grounded initially with provision for a future reactor.

The transformers are outdoor type, oil-insulated, self-cooled, with inert gas seal.

They are designed for rated load continuously at not over 55° C. and will withstand a 5-second short circuit with full sustained primary voltage without injury. A resistance temperature detector in the low-voltage winding of each transformer is connected to a temperature recorder in the main control room, and annunciation is provided for high temperatures. Annunciation is also provided for high oil temperature and for low inert gas pressure. Each transformer is equipped with eight flanged wheels and rests on rails embedded in concrete piers over a well-drained oil sump. Fire protection is provided by a dry pipe water spray system. Lightning arrestors are close to the transformer 154-kilovolt terminals.

Switchboards and controls.—The glassed-in control room at the east end of the top floor of the control building houses four switchboards and furnishes centralized control of the power plant and switchyard. These four switchboards in the control room are: the main control benchboard, the instrument board, the relay board, and the auxiliary control board. An auxiliary vertical switchboard opposite the benchboard controls the main auxiliary power. Upon this board is mounted the related annunciator group and station recording equipment. The operator's desk is between the benchboard and auxiliary board. The general control scheme is such that the switchboard operator may start the main generator and all necessary unit auxiliaries with or without the aid of the hydraulic floor operator. The generators may likewise be stopped wholly by the main control operator, but stopping the auxiliaries is under the supervision of the hydraulic operator.

The benchboard contains controls for the two generators, one main transformer bank, and the high-voltage switchyard, including the four 154-kilovolt feeders. The main connections are indicated by miniature busses using distinguishing colors for the various voltages. The instrument switchboard contains instruments and meters for the circuits, and upon this the related annunciator group is mounted above the main panel. General or miscellaneous instruments and equipment, such as temperature recorders, recording wattmeters, and totalizing equipment are mounted on the auxiliary board.

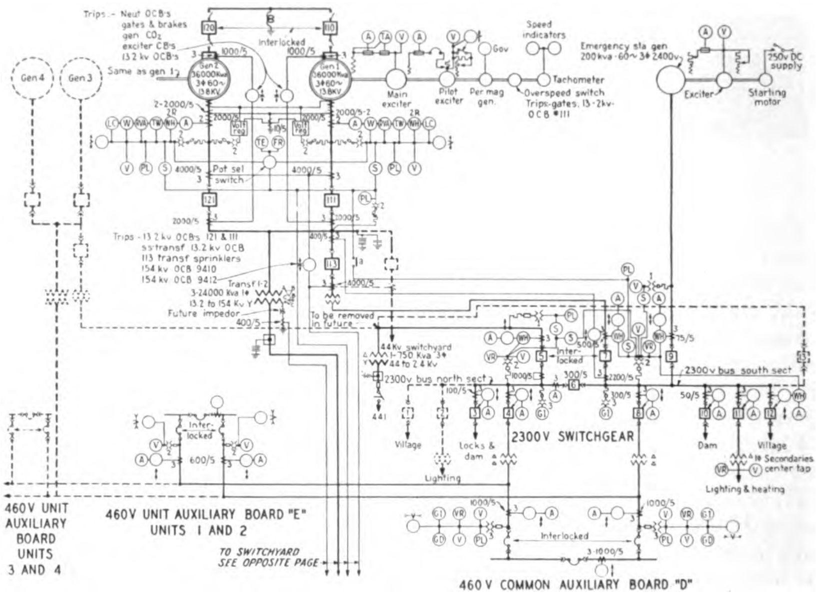

FIGURE 46.—*Single-line wiring*

In order to simplify the wiring connections at the switchboard, and to terminate properly the station control cables, terminal boards enclosed in steel compartments are in the terminal room on the floor immediately below the main control room. The station control cables are brought to a spreading room below the terminal room, thence pass through the floor of the terminal room to the terminals. From the terminal, switchboard wire is carried up through the switchboard floor to the switchboard panels.

Mounted at the left end of the instrument board, and facing the benchboard, is a swinging bracket panel provided with a synchroscope, an indicating voltmeter for incoming voltage, an indicating voltmeter for running voltage, and a transfer switch to cut out the automatic synchronizing features and leave the synchroscope and voltmeters in the circuit for manual synchronizing.

The excitation panels for each unit, including rheostats, field switches, and other equipment, are on the hydraulic operating gallery floor in a concrete and steel enclosure.

The starting controls are so interlocked that it is impossible to start the unit unless the following conditions have been fully met;

1. Sufficient cooling water must be flowing in generator thrust bearing coolers.
2. Sufficient cooling water must be flowing to turbine lower guide bearing.
3. Generator field circuit breaker must be closed.
4. Governor oil pressure must be up to normal.

When starting, the generating unit is brought up to no-load speed by the wicket gate control switch, after which the governor takes control and the gate limit stop is set at the desired maximum operating position. When shutting down, the load on the generator is

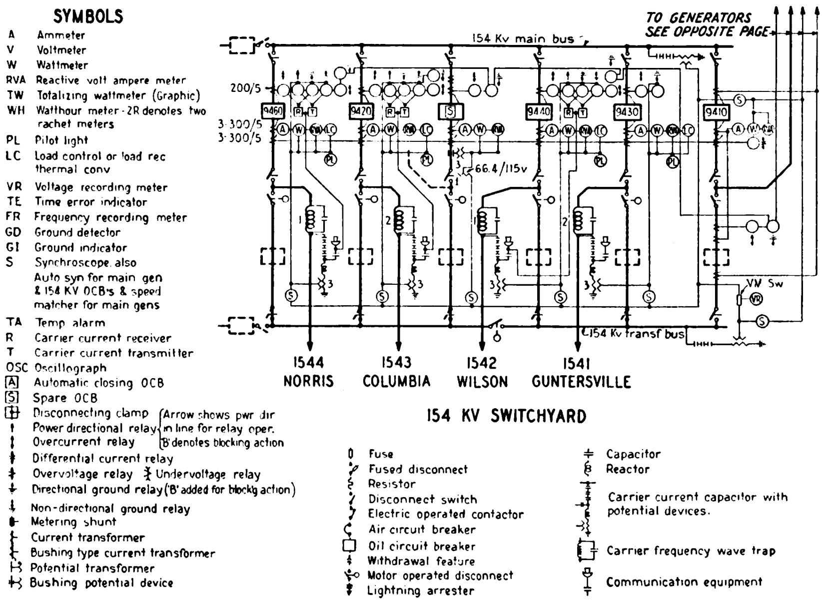

diagram of principal power circuits.

reduced and the unit is brought to no-load speed by the governor speed adjusting control switch, and stopping from this position is accomplished by the gate limit control switch and brake.

Load and frequency control equipment, consisting of unit load controllers, a master load-frequency controller, a frequency and time error recorder, a tie-line load recorder-controller, a station load recorder, and other auxiliary apparatus, is supplied for each generating station in order to obtain the maximum flexibility of station operation. Control is accomplished by sending raise or lower impulses to the governor speed level motor, thus making the unit respond to variations in load or frequency which the governor is either not sufficiently sensitive to detect, or is not inherently equipped to measure. The equipment supplied permits operation under any of the following types of control:

1. Unit base load control, in order to hold on each generating unit a fixed load, adjustable at will.
2. Station base load control, in order to maintain a fixed station load by means of unit control.

3. Flat frequency control, in order to regulate the system frequency with an instantaneous sensitivity of measurement not to exceed 0.005 cycles, within the load change capacity of the generating units under control.

4. Flat tie-line load control, in order to maintain a selected tie-line load within predetermined adjustable kilowatt limits, within the limits of the generating capacity under control.

5. Selective frequency control, in order to absorb by the plant the load changes originating in its particular area within the limits of the generating capacity under control, and thus minimize unnecessary power transfer over the lines.

6. Tie-line load bias frequency control, in order to permit automatic parallel regulation with a master frequency controlling station. In this type of operation, the station under control momentarily assists the master station in the frequency regulation regardless of where the swing originates, and then gradually restores its tie line to the scheduled interchange. The amount of assistance contributed and the time it is contributed are adjustable at will. The "bias" of frequency control by tie-line load may be varied over the entire range from flat frequency control to practically flat tie-line load control.

7. Frequency condensing control, in order to permit the operation of one or more generators as synchronous condensers with automatic control for picking up load in case of tie-line failure.

Either manual or automatic time-error correction may be used under conditions (3) and (7) above. Automatic time-error correction requires the use of a standard frequency source.

Unit hydraulic gage boards.—Each generating unit has an enclosed cubicle-type gage board on the operating gallery floor and adjacent to the governor actuator cabinets. These gage boards contain instruments for recording thrust and guide bearing temperatures, and for indicating generator and exciter air temperatures, thrust bearing oil levels, and cooling water pressures and temperatures.

Station battery system.—Energy for station emergency control and lighting is supplied by a 250-volt storage battery having 120 cells. The battery has a discharge capacity of 50 amperes for 8 hours, or 650 amperes for 1 minute to a final voltage of 1.75 volts per cell. Two motor-generator sets are provided for trickle charging the battery. Each set consists of a 30-horsepower, 440-volt, 3-phase, induction motor direct-connected to a 20-kilowatt, diverter-pole type direct-current generator.

Annunciator system.—The annunciator system provides for audible and visible signals in the main control room and on the turbine operating floor to indicate the occurrence of abnormal conditions in equipment. The visual signals are the illuminated type and are above the switchboards in the main control room and on the unit auxiliary switchboards on the turbine operating floor. A separate 48-volt storage battery is provided for the annunciator system.

Carrier current relay system.—Carrier current relaying equipment is provided for protecting the 154-kilovolt transmission lines. The carrier relay equipment utilizes phase A and ground, with coupling capacitors connected to phase A at both ends of the transmission line. The relay group is so designed that the direction of power flow at one end of the line in relation to the direction of simultaneous flow at the other end determines whether or not an electrical fault has developed on the protected line. The relay system will function under the following conditions:

1. Will energize the oil circuit breaker trip circuit in 0.05 second on both ends of the line when a fault occurs.

2. Will not cause tripping of the breaker during instability of the system unless a fault occurs during the instability period.

The transmitter-receiver unit of the carrier relay system is mounted in a weatherproof housing in the switchyard. The relay equipment is mounted on the relay switchboard in the main control room. Potential for the relays is supplied by resonant type capacitor potential devices connected directly to the transmission line, and the corresponding current values are obtained from bushing type current transformers on the 161-kilovolt oil circuit breakers in the switchyard.

Carrier current telephone system.—The carrier current telephone system utilizes phases B and C of the 154-kilovolt line. At the time Wheeler plant was placed in operation, the carrier channel arrangements permitted two-way telephone communication with Wilson, Guntersville, Chickamauga, Norris, and Columbia, Tenn., but this system has since been extended. The system is used for general purposes, including load dispatching, and for giving orders and instructions to transmission line patrolmen who have patrol cars equipped with carrier current telephone receivers. Selective ringing equipment is so designed that any extension or telephone station may be selectively called from any other station on the same carrier channel by the usual method of dialing.

Automatic telephone and signal system.—Local communication and signal facilities are provided by automatic telephone and signal equipment which is interconnected with the carrier current telephone system. The key type turret and exchange desk are located in the main control room in front of the benchboard, and the automatic switching equipment is located in the telephone and carrier current room one floor below the main control room.

The exchange provides continuous day and night service without manual operation except for toll or other similar service. Executive right-of-way service is provided on one line of the main control room to permit the head operator to make emergency or important calls to busy telephones by breaking in on established connections.

Conference service for a maximum of six telephones is provided. The signal for conference calls is a steady uninterrupted ringing of the called station, and if the station called should be engaged, an identifying tone signal is automatically put on the line to indicate that a conference is being called. The existing conversation is not interfered with in any manner. Code call signaling of the usual type is secured from any telephone station by dialing a code call prior to dialing the station number.

Auxiliary power supply.—The three present and four ultimate sources of auxiliary power supply are shown in the single-line diagram of figure 46. The transformer energized from the 13.2-kilovolt bus for units 1 and 2 is a 2,000 kilovolt-amperes, 3-phase transformer which provides sufficient capacity for all anticipated requirements of the initial and ultimate plant. A second duplicate transformer will be provided with units 3 and 4. The transformer energized from the 44-kilovolt outside source is a 750-kilovolt-amperes, 3-phase transformer located in the switchyard. This outside source comes from Wilson Dam. It now substitutes for the second 13.2 kilovolt source and will also probably be retained as a stand-by source after units 3 and 4 are installed. The emergency station generator is driven by a gasoline engine. It has a rating of 200 kilovolt-amperes and is provided primarily as an emergency source which can be used for operating the spillway gates and providing a limited amount of station lighting in case of such a major catastrophe that no other auxiliary power source is available.

All these sources of auxiliary power supply are connected to the 2,300-volt main auxiliary power bus which is sectionalized at the middle. The two main sources are interlocked so that only one may be used at a time, thus reducing short circuit severity and circulating currents. In conformity with established practice for all TVA plants, this interlocking is carried throughout the 440-volt system also and is arranged in each case so that in transferring from one main source to the other a momentary interruption is accepted instead of a momentary paralleling. The 2,300-volt switching consists of oil circuit breakers in steel enclosures of the truck type, dead-front construction. This board controls, besides the incoming sources of power, the main circuits to the lock, the spillway, the village, and the station power and lighting transformers.

Station power and centralized heating are served at 440 volts, 3 phase, from the 2,300-volt bus through two duplicate, 1,000-kilovolt-ampere transformer banks, each having sufficient capacity for ultimate station requirements. The 440-volt switching is in the form of fused switches and magnetic contactors enclosed in dead-front steel cubicles.

Station lighting and local heating are served at 115/230 volts, 1 phase, from the 2,300-volt bus through appropriate transformers. The switching consists of air circuit breakers on a dead-front steel switchboard for the main feeders and numerous dead-front distribution cabinets for the branch circuits.

Low-voltage wiring.—Cables for plant auxiliaries and miscellaneous indoor station equipment are insulated with performite rubber compound containing at least 35 percent by weight of best grade new rubber. The compound is highly moisture-resisting, superaging, and capable of operating continuously in a satisfactory manner with a maximum copper temperature of 75° centigrade. The cables are braid-covered with moisture-resisting flameproof cotton braid except where the cables are subject to excessive moisture, such as for the dam and switchyard, in which case lead-covered cables are used. In a few special instances, armored cables and asbestos-covered cables are used. The cables are generally run in rigid steel conduit.

Cables for controls, instruments, and signals are generally of the construction described in the preceding paragraph and are 3- to 19-conductor size as required. Because of the many cables required and the inherent need of utmost flexibility for future changes and additions these cables are run on open shelves insofar as practicable. Concrete tunnels extend from the terminal room below the main switchboard throughout the length of the powerhouse and the switchyard. Shelving is transite lumber on steel brackets. Final routing of the cables between the tunnel and the equipment in the powerhouse and between the tunnel and the equipment in the switchyard is through comparatively short runs of embedded steel conduit.

Grounding system.—The powerhouse grounding system consists of an interconnected copper cable system throughout the plant to which all equipment is connected. The cables are grounded to three ground mats, two of which are in the bottom of the reservoir upstream from the dam and the other in the tailrace. Separate connections are run from the lightning arresters on the draft tube deck to the tailrace mat almost directly below.

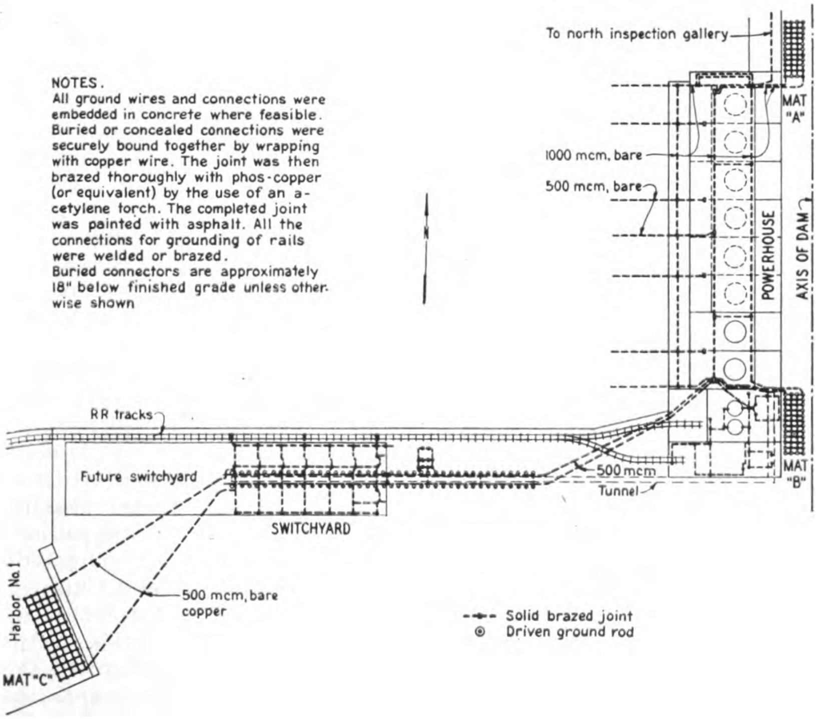

FIGURE 47.—*Grounding system for dam, powerhouse, and switchyard.*

The switchyard grounding system consists of a copper cable grill covering the entire yard, just below the surface, to which all structures and equipment are connected. It is grounded to a downstream mat and also to a long mat along each side of the cable tunnel. The powerhouse and switchyard mats are interconnected, and test points are provided in all connections to grounding mats.

Lighting system.—The normal lighting system is 115-volt alternating current. An emergency group of lights is automatically lighted, on failure of the alternating-current system, through independent circuits having a 250-volt direct-current supply from the station storage battery. The number of emergency lights is limited, and they are placed only in the more important locations for necessary operation of the plant and to insure safety. Typical high-bay, direct-type, overhead lighting units illuminate the generator room.

The main control room is lighted by totally indirect units, while the remainder of the plant is lighted with various standard types of commercial fixtures. Illumination for the public lobbies and reception room is from fixtures of special design harmonizing with the architectural features and arranged to suit the specific requirements in each case.

Water system.—The raw water supply is obtained from the forebay through a 10-inch, cast-iron pipe. From this main header a 3-inch branch line extends to the air compressors and the 500-gallon cooling water tank for the gasoline engine in the control building at elevation 513.3. A 6-inch branch parallels this line and extends to the filtration plant south of the control building. A branch from the 6-inch line serves the air-conditioning and refrigeration equipment. The main header also supplies the generator air and bearing cooling water emergency pump.

The water purification plant for the sanitary supply is adjacent to the south side of the control building. This plant serves the needs of both the dam and village. The operating floor is at elevation 552.3, and the plant is covered by the roadway and parking area slab. The plant consists of mixing, coagulating, and filter basins. Equipment on the operating floor consists of two chemical feed machines, two chlorinators, and a raw water pump and motor. The control valves are manually operated. The plant has a capacity of 100,000 gallons per 24-hour operating day.

Sewerage system.—Disposal of the sewage from all the sanitary facilities is by a single septic tank located at the southwest corner of the control building. The tank discharges into the tailrace.

Oil purification and storage system.—Only two grades of oil are used in the power station: transil oil for transformers and oil circuit breakers, and lubricating oil for governors and generator bearings. The oil purification and storage equipment is in the control building at elevation 498.8. It consists of a 1,200-gallon-per-hour oil purifier with centrifuge and filter press, one 50-gallon-per-minute pump for lubricating oil, and one 100-gallon-per-minute pump for transil oil, and the storage tanks. There are two 12,000-gallon tanks, one for filtered and another for unfiltered transil oil; one 7,500-gallon tank for unfiltered circuit breaker oil; and two 5,000-gallon tanks, one for unfiltered and the other for filtered lubricating oil. Lubricating oil is supplied for various uses about the plant from a 220-gallon tank with a compressed air connection to maintain the pressure.

Compressed air system.—Three stationary air compressors with aftercoolers and air receivers are included in the plant equipment. Two of these compressors have a capacity of 330 cubic feet of free air per minute and the third 95 cubic feet per minute. From the air receiver a 4-inch main header supplies air to the various services, one line going to the air pressure tank in the oil pump and purification room in the service bay. Two-inch lines are provided for the evacuation system in each unit. A 1½-inch riser in the northwest quadrant room of unit No. 1 branches to serve the governor actuator cabinets in the turbine gallery, and ¾-inch lines from these cabinets serve the generator brake and jack machinery. A 4-inch riser from the compressor room leads up to the control cable gallery and provides miscellaneous air service from outlets in the gallery.

Miscellaneous hoist and transfer equipment.—A 75-ton traveling crane is in the west end of the control building and used for transformer untanking. It is the indoor, overhead traveling type with a 75-ton main hoist trolley and a 1-ton auxiliary electric hoist mounted on an I-beam fastened to the underside of the bottom flange of the south bridge girder. The main trolley and bridge are electrically operated and arranged for floor control. The crane has a 23-foot 2¾-inch span and a 40-foot lift. The 1-ton electric auxiliary hoist is operated by pendant cords from the floor, and the trolley is operated by chain from the floor.

In the service bay there are two single I-beam type cranes with a low headroom electric hoist. The trolley and bridge travel are manually operated from the floor by means of hand chains. Each crane has a bridge travel of approximately 70 feet. The capacity of each hoist is 4 tons, and the hoisting speed is between 15 and 25 feet per minute. The 3½-horsepower hoist motors are the fully enclosed high-torque wound-rotor type for operation at 440 volts, 3-phase, 60 cycles, 1,570 revolutions per minute.

There are two 2-ton chain hoists, one with a trolley beam about 45 feet long for the machine shop and the other for handling the unwatering pump for units 1 and 2 in the unwatering gallery.

A 100-ton transfer car is provided for moving the transformers and circuit breakers from their positions on the transformer deck of the powerhouse into the untanking room. The car has four wheels and operates on the upstream rail for the 20-ton gantry crane and a rail midway between gantry rails. The 20-ton gantry crane pushes or pulls the car with its load at a speed not exceeding 100 feet per minute. A geared hand-power winch is mounted on the transfer car for the purpose of hauling the transformers on and off the car.

Because there is no direct rail access to the dam, heavy equipment and supplies are shipped to Wilson Dam by rail and then by barge to Wheeler. Therefore, as part of the permanent equipment at Wheeler Dam, a 100-ton steel, self-slewing, fixed-radius stiff-leg derrick was installed for handling freight. A similar derrick is located at the rail terminal at Wilson Dam.

Air conditioning, ventilating, and heating.—An air-conditioning plant is provided for year-round service to the visitors' reception room, control room, lobby, and administrative offices. The system is designed to clean and circulate air continuously throughout the conditioned spaces, to heat and humidify it in cold weather, and to cool and dehumidify it in warm weather. The degree of cooling or heating of air delivered to the supply ducts varies according to the temperature existing in the conditioned spaces and is regulated by thermostats in the common return air duct, the temperature of which is indicative of the average temperature in the combined spaces. Temperatures in the individual conditioned spaces are further regulated by thermostat-controlling dampers which admit quantities of conditioned air necessary to maintain the required temperature. A mechanical refrigerating system using Freon 12 [13] as a refrigerant cools and dehumidifys the air during the cooling season. It consists of two refrigerant compressors, one water-cooled condenser common to both compressors, and a dehumidifier.

[13] Thompson, R. J., *The Technical Aspects of "Freon" Refrigerants,* Refrigerating Engineering. April 1937.

The central heater for the air-conditioning plant consists of thin heating strips mounted in eight sections of 40 kw. each. Each section of heaters is complete in a steel subframe, and the separate sections are assembled together to form the central heater. The heater has an aggregate capacity of 320 kw. for 440-volt, 3-phase operation and is divided into six circuits, two of which are 80 kw. each, and four of which are 40 kw. each. An auxiliary electric blast heater of 30-kw. capacity is in the supply duct serving the observation room. A similar heater is mounted in the supply duct serving the control room. In the offices and lobbies, auxiliary 220-volt recessed 5-kw. electric convection heaters are supplied to supplement the heating by the central air-conditioning system and for greater flexibility of control. In the lower floors of the control building and in the service and unit bays, direct resistance electric heaters are in the spaces to be heated. The heaters are two principal types: 440-volt fan-type unit for the more severe heating requirements, and 220-volt convection type for lighter duty.

A separate fan and duct system having a capacity of 3,000 cubic feet per minute is provided for the ventilation of the oil purifier room and oil-tank enclosure. A separate exhaust system ventilates the battery room. The toilets for both visitors and employees are ventilated by exhausting air from them to an exhaust fan in the penthouse, which discharges to the outside. Air to replace that exhausted is drawn into the toilets from the lobby through grilles in the wall. Toilets which adjoin air-conditioned spaces are partially air conditioned since the air drawn through them has been treated in the central air-conditioning plant—no air is mechanically supplied. By confining the ventilation to exhausting, a slight negative pressure is maintained in the toilets, which minimizes the possibility of exfiltration of odors into the building.

Elevator.—The elevator in the control building is an automatic push-button control electric elevator designed for a live-load capacity of 5,000 pounds at 250 feet per minute. The hoisting machinery, motor-generator set, and control panels are installed in the machinery room at elevation 587. Elevator entrances at each landing are two-speed, hand-operated sliding doors. The elevator is provided with a floor-finding, self-leveling device which will automatically bring the platform to a position within one-half inch of the floor for which the corresponding button has been pushed.

Switchyard equipment.—The oil circuit breakers are each rated 1,200 amperes, 161 kv., 60 cycles, with an interrupting capacity of 2,500,000 kva. The breakers are triple-pole, single-throw, 250-volt, d. c. control, solenoid-operated, trip-free, and full automatic. Each phase is an individual tank with all poles of one breaker operated as a unit by a common enclosed mechanism. The three tanks of one unit are mounted on one concrete foundation mat. The breaker tanks are 3/8-inch steel plate with fabricated steel tops. Bushing current transformers are provided for bus differential relays, metering, and carrier current relays. Each bushing has taps, arranged for the installation of potential devices. An electric heater is placed inside the housing of each control mechanism, and the housings are ventilated in order to eliminate as far as possible any condensation of moisture which might cause disturbing grounds in the control circuits.

The disconnecting switches are rated 161 kv., 600 amperes, 60 cycles, triple-pole, single-throw, and gang-operated. The isolating switches for the oil circuit breakers are manually operated and the selective switches and sectionalizing switches are motor operated. The buses are of 3-inch standard iron pipe size hard-drawn copper tubing, and are supported by pedestal type insulator assemblies spaced 32 feet apart. Phase separation is 8 feet 6 inches. Connections to the buses are made with 500,000-circular-mil bare copper cable with bolted connectors. The cables are supported by strain insulators.

The 154-kv. yard has overhead ground wires for each of the outgoing feeders and for the transformer circuits from the powerhouse. The ground wires are grounded solidly to the steel structure. Spill-gaps are installed on each circuit. The gaps consist of 36-inch-diameter rings made of 1-inch standard galvanized steel pipe which are mounted directly on caps and bases of the supporting insulator assemblies. The steel structure is thoroughly grounded to the grounding system described elsewhere.

Oil piping consists of a 3-inch common supply line for circuit breakers and transformers and a common 4-inch drain line, both of which are connected in the power plant to pumping and purifying equipment and accessory storage tanks.

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CHAPTER 4

HOUSING AND ACCESS

EMPLOYEE HOUSING

The employee housing problem for this project was solved by building a small group of permanent houses to be occupied first by construction executives and later by the plant operators, a group of temporary low-cost houses for other workmen and their families, and a compact group of dormitories of temporary construction for unmarried workmen. A separate camp consisting of temporary low-cost houses and dormitories was constructed for the Negro employees. The Wheeler camp was the second housing project undertaken by the Authority, and although it differed in many respects from the one at Norris, the experience gained there was very helpful.

PRELIMINARY STUDIES

Soon after work on the dam was started in November 1933, it became evident that because of its remoteness and the inadequacy of existing access roads a large number of employees would have to be housed at the dam site. However, since Wheeler Dam was only 25 miles from the "Tri-Cities" (Florence, Sheffield, and Tuscumbia) and the TVA-owned Muscle Shoals villages, there was no apparent need for a large permanent village near the dam. Hence it was decided that, except for a small group of houses intended for eventual occupancy by the permanent operating force, the housing should be constructed to last only for the duration of the construction work. In general the design of the buildings was influenced by the experience gained at Norris but differences in climatic and local conditions were taken into consideration in adapting plans to that locality. Many sections of the camp and village were specially designed for Wheeler.

Few privately owned modern houses were available for rental in the outlying areas near Wheeler or in the nearby towns of Courtland, Town Creek, and Decatur. Although there was a number of vacant houses in the villages on the Government reservation in the Muscle Shoals area 25 miles from the job, they were in poor repair and were expected to be occupied by workers in the Muscle Shoals area. However, about the middle of April 1934, 67 reconditioned houses in the Muscle Shoals villages were assigned for occupancy by Wheeler workers.

In the beginning the size of the initial power installation was uncertain. For this reason it was felt that the number of permanent houses should conform to the minimum operating requirements that could be foreseen rather than to a larger number based upon a specu-

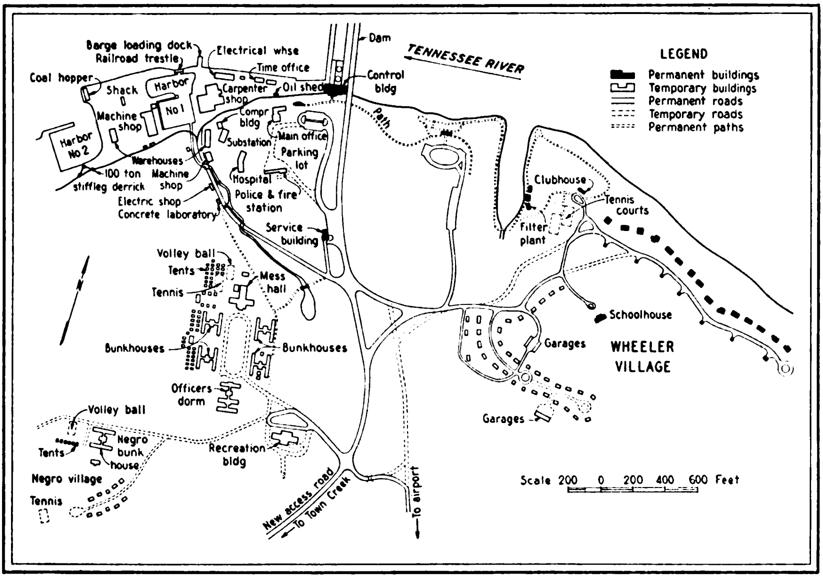

FIGURE 48.—*Camp and village layout.*

lative need. The building of 15 permanent houses was based upon the assumption that not more than one generator, or perhaps none, would be installed initially, and it was estimated that this number would be sufficient to accommodate the small force of operators likely to be stationed at Wheeler. It was also expected that certain members of Wheeler personnel, such as administrative officials and heavy repair crews, would live at Muscle Shoals, since much of their work would be centered around Wilson Dam.

At first it was estimated that only 60 to 100 white men and 20 to 25 Negroes out of the 2,000 men expected to be employed would require living accommodations at the dam site. Later, however, it was found that several hundred men would require housing accommodations, and the preliminary plans had to be revised. As a result of this revised estimate, six 120-man dormitories were built. One of the dormitories was for engineering and construction staff members, four for other white employees, and one for the Negro employees. No women's dormitory was built as all of the women employees could be housed in one of the larger permanent houses. Adequate cafeteria and recreation facilities were planned as a part of the construction camp. The Authority also built 44 low-cost temporary houses for married supervisory employees and their families. Thirty-four of these houses were for the use of white employees and 10 for Negro employees.

During the period of peak employment in the summer of 1935, over 1,000 men were accommodated in the camp. Many of the men employed for this period only were housed in tents erected near the white and Negro dormitories.

The south bank of the river was the only logical location for the camp and village, since the greater part of the work would be on that side. In addition, the dam construction offices and all of the construction plant except the floating equipment were on the south side. The principal access roads from the Tri-Cities also connected the south side of the site with the railroad at Town Creek and Alabama State Highway No. 20.

Although considerable land was available adjacent to the south approach to Wheeler Dam, choice of sites for the housing development was considered to be somewhat limited because of the desire to reserve much of the land for permanent demonstration parks. The site chosen for the permanent houses is along the shore line about 2,000 feet upstream from the south end of the dam, convenient to the control building. Here begins a gradual slope downward which terminates in bluffs and steep declivities along the margin of the reservoir. The site is well wooded with a number of splendid oak and chestnut trees and offers attractive views across the water and along the shore line. The steep topography and the desire to preserve the fringe of existing trees required considerable study of house sites, roads, and utility lines. The low-cost temporary houses were placed south of the permanent houses about midway between them and the road from the dam to Town Creek. The area chosen for the white camp was about 1,500 feet from the south end of the dam. This site was fairly level and well adapted to the H-shaped, one-story dormitories. The Negro camp and village was about 600 feet southwest of the white camp. Several parks, paths, and shelters built in the vicinity by the National Park Service Civilian Conservation Corps units greatly increased the attractiveness of this area. Little landscaping was required in the camp or around the houses. The natural grades were altered but slightly and planting was kept to a minimum. Considerable improvement of wooded sections was necessary throughout the developed areas. Dead wood, thick underbrush, and blighted chestnut were removed where necessary; but healthy trees were preserved and protected where possible.

UTILITIES

The roads and streets of the camp and village were laid out to fit best the topography and scenic advantages of the areas and yet to provide easy and direct access to any part. Permanent roads were built from the access road to the village, park, dam, and construction plant areas. Temporary roads were built to the temporary police and fire stations and to the white and Negro construction camps. All roads were surfaced with chert.

Construction of two parks was started by the Civilian Conservation Corps in April 1934. The one nearest the camp is along the shore of Wheeler Reservoir between the dam and the permanent houses. The other is southwest of the dam along the shores of Big Nance Creek and Wilson Reservoir. The park between the dam and the permanent houses is reached by steps leading up from the road to the dam or by a footpath from the permanent houses. Two overlook parapets and a picnic shelter were built. The picnic shelter contains a large picnic space with chert floor, fireplaces, and a storage room for park maintenance equipment. The terrace adjoining this shelter affords a broad view of the dam and reservoir. The second

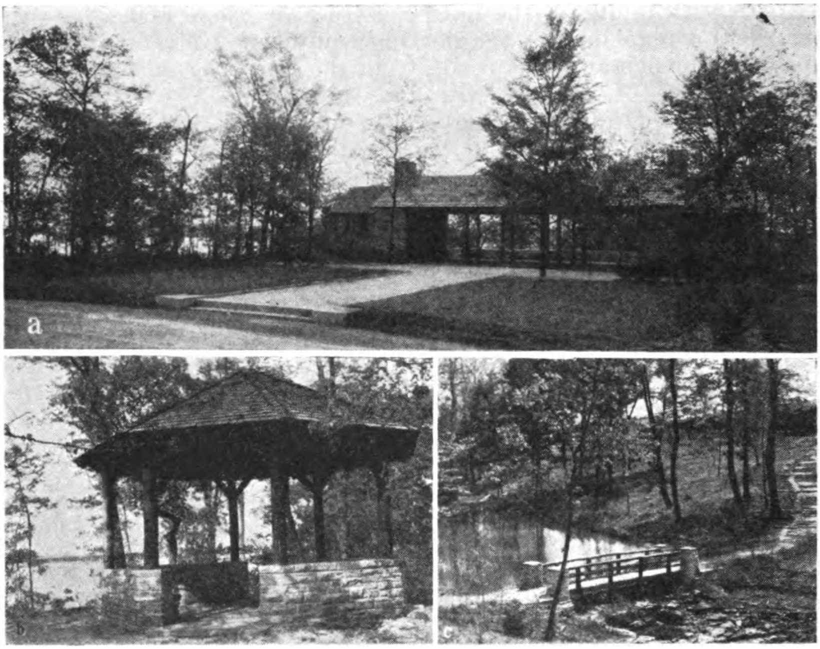

FIGURE 49.—*Park developments (a) picnic shelter, (b) overlook shelter—Big Nance Creek Park, (c) small recreation pool.*

park, Big Nance Creek Park, is less accessible but can be reached by a road from the south access road to the dam and also by trails from the dam and powerhouse. Besides an overlook shelter, a large picnic shelter with fireplaces and a building with modern rest rooms were constructed. A one-acre pool, adding considerably to the attractiveness of the area, was formed by damming the flow of several small springs. The overflow from this pool formed a stream which ran adjacent to the south access road near the construction camp. A walk crossed the bridge over the spillway. Both dam and bridge were constructed by the Civilian Conservation Corps.

Water-supply system.

Three possible sources of water supply were investigated. These were: a natural spring about one mile south of the camp, a deep well driven near the camp site, and the Tennessee River. Measurements indicated that the flow from the spring or a deep well might prove inadequate to meet the demands of the camp and village. The general uncertainties of these two sources, coupled with the prevailing need for a readily available source of water supply to meet the requirements of the expanding project, precipitated the decision to utilize water from the river. Standard treatment by coagulation, filtration, and chlorination was adopted for purifying the river water. The water was distributed to the camp and village through a system of cast-iron mains. An elevated tank which "floated" on the distribution system gave a pressure of 60 to 80 pounds per square inch at the extreme ends of the system. During the construction period the water was supplied to the distribution system through a temporary filtration plant located about 1,000 feet upstream from the dam. A permanent plant was later incorporated in the power-house structure at the south end of the control building.[1]

A complete sewerage system was built to serve both the camp and village. The system was designed to collect the sewage, treat it, and discharge the effluent into the river below the dam. An Imhoff tank located about 2,000 feet downstream from the dam was used for treatment. This type of treatment prevented the accumulation of sludge banks in the still water of Wilson Reservoir.

HOUSING FACILITIES

Housing facilities for white and Negro employees were provided in proportion to the probable ratio of white and Negro employment. The facilities for white employees consisted of four dormitories for workmen, a dormitory for single staff members, a cafeteria, a recreation building, 15 permanent houses, 34 temporary houses, and a school building. The construction camp and village for Negro workers included one dormitory, a cafeteria and recreation building, 10 temporary low-cost houses, and a school.

White construction camp.

Because the construction camp was to be used only three years, the buildings were built of wood framing on open wood post foundations and covered with asphalt rolled roofing. The interiors of the

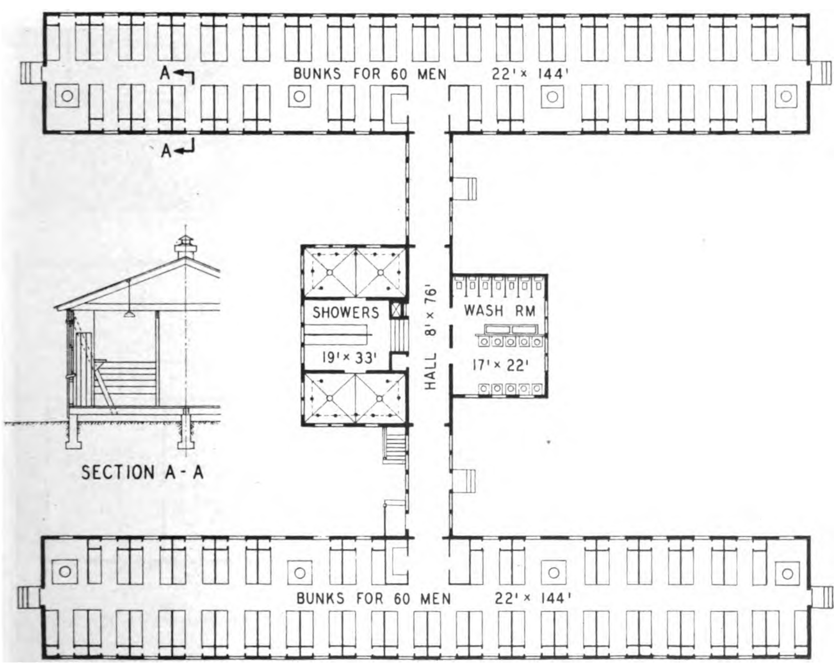

FIGURE 50.—*Typical dormitory.*

[1] See p. 92.

buildings were left unfinished except in the cafeteria and the recreation building—these were finished with flush shiplap. These buildings were one-story frame structures with creosoted wood posts to support the floor girders, joists and frame walls, partitions, and roofs. All buildings were provided with warm-air, steam, or stove heating, electric lighting, and standard plumbing fixtures.

One dormitory for 120 staff men was constructed with enclosed two-man cubicles. It was provided with a warm-air heating system and a centrally located shower and toilet room within the building. Four other dormitories to accommodate 120 men each were built similar in plan and construction to the staff dormitory, except that they were arranged with open cubicles. Stoves were provided for heating. Warm-air heat was provided for the centrally located shower and toilet room.

Cafeteria.—The cafeteria was similar in design to the one at Norris. The food preparation room, mechanical refrigerators, general storage room, and central steam-heating plant were in the basement.

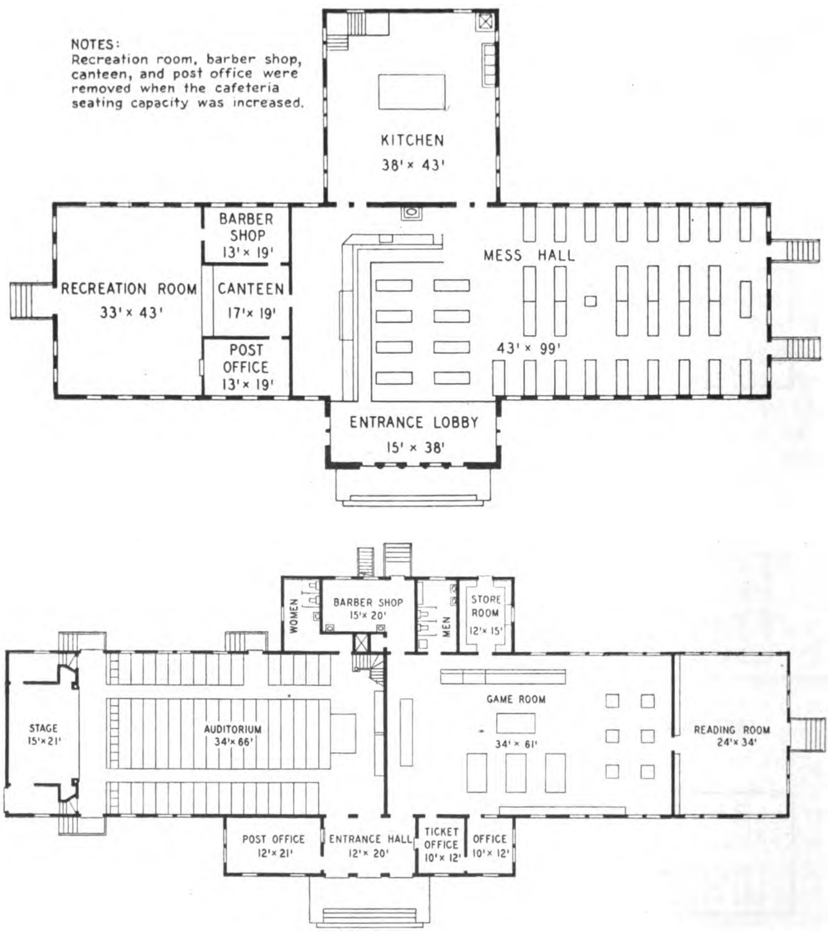

Figure 51.—*Cafeteria and recreation buildings.*

The equipment in the kitchen and food preparation room was electric. The building was finished inside with shiplap and heated with steam unit heaters. The cafeteria as originally constructed and arranged seated 304 men in the dining area and was equipped with a cafeteria counter and a large kitchen. As the camp grew, however, the seating capacity was increased to 516 by removing the commissary, recreation center, and post office from one wing of the building. During the peak of construction operations in May 1935, about 3,000 meals were served daily.

Recreation building.—As the number of employees and the size of the camp increased, space in the cafeteria that had been reserved for recreation purposes was used to make larger dining quarters and a separate recreation building was constructed. The recreation building housed a commissary with a soda fountain, game tables, reading room, barber shop, post office, recreation office, toilet facilities, and an auditorium with a stage and equipment for showing sound-on-film moving pictures. The exterior was finished in boards and battens, and the interior was finished with flush shiplap. The roof trusses and rafters were left exposed. The building was heated by a warm-air system.

White village.

Temporary houses.—Thirty-four temporary houses were built in the white village to house a portion of the dam construction workers. An inexpensive type of construction was used since the houses were to

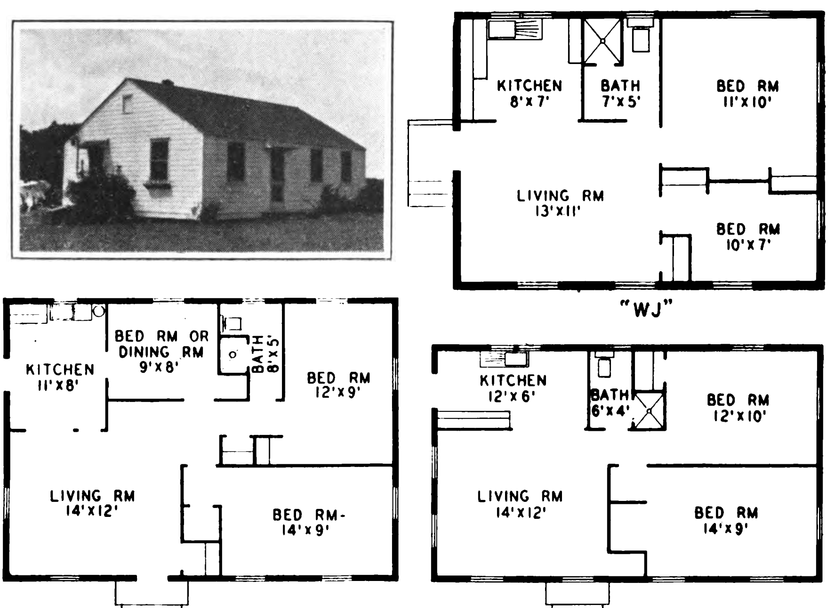

FIGURE 52.—*Temporary low-cost houses—Types WJ, WK, and WL—Type WL shown in the photograph*

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be occupied only 2 or 3 years. They were supported on wooden posts capped with galvanized iron termite shields. Exterior walls were of open-stud construction with one layer of building paper on the outside covered with 1- by 6-inch tongue and groove vertical sheathing. Later the houses were again covered with building paper and refinished with horizontal boards. The roofs were crimp-jointed galvanized iron. Interior partitions were single thickness, 1- by 6-inch tongue and groove vertical sheathing; floors were double thickness with building paper between; and ceilings were ½-inch insulation board.

These houses were provided with a combination living-dining room, bedrooms containing closets, and a bathroom with toilet and a galvanized iron shower stall but no lavatory or tub and were heated with combination heater-ranges.

TABLE 4.—*Low-cost temporary houses*

House type	Number houses	Cost each	Area each square feet	Volume each cubic feet	Rooms each	Cost per cubic foot
WJ	5	$953.00	432	5,256	4	$0.182
WK	19	989.00	606	7,496	4	.132
WL	6	1,119.00	767	9,654	5	.116
KF	13	2,204.02	814	8,477	4	.26
KF	1	2,505.88	960	9,638	6	.26

Permanent houses.—Fifteen permanent houses were built to house first a portion of the construction forces and later the operating forces. These houses vary in size from 3 to 10 rooms and all have three-fixture bathrooms, screened porches, and laundries. They are brick and insulated throughout with rock wool. Brick foundations and reinforced concrete slab-and-girder construction were used throughout the first floor as a precaution against termite damage. Roofs were covered with asphalt shingles. The interior walls were finished with V-jointed shiplap up to the window-sill level and with plywood above this level. The ceilings were made of V-jointed insulation boards. Oak flooring was used throughout except for the mastic finish for kitchen, laundries, and bathrooms. In most of the houses the attics were left unfinished and no basements were provided, but any available storage space owing to the slope of the ground under the houses was made accessible by means of outside doors. These houses are all electrically heated and are equipped with an electric refrigerator, an electric range, and an automatic electric water heater.

TABLE 5.—*Permanent houses*

House type	Number of houses	Cost each	Area each square feet	Volume each cubic feet	Rooms each	Cost per cubic foot
31	5	$6,746.49	1,287	17,730	3	$0.38
41	5	7,301.30	1,444	17,835	4	.41
52	3	8,459.32	1,139	20,522	5	.41
51A	1	12,667.84	1,142	27,707	7	.46
100	1	14,956.40	1,399	29,756	10	.50

Two of the houses, type 51–A and type 100, in the permanent house group were more elaborately finished than the other 13 houses. Among other features, both of these houses have a room and toilet in the basement, have load-bearing hollow tile backing for the brick-veneered walls, and have partitions and ceilings plastered throughout with plaster on metal laths.

Garages.—One single and six double garages were built across the road from the permanent houses. The construction was very simple, but the appearance of these scattered small buildings is not unattractive. The walls are clapboarded frame construction on brick

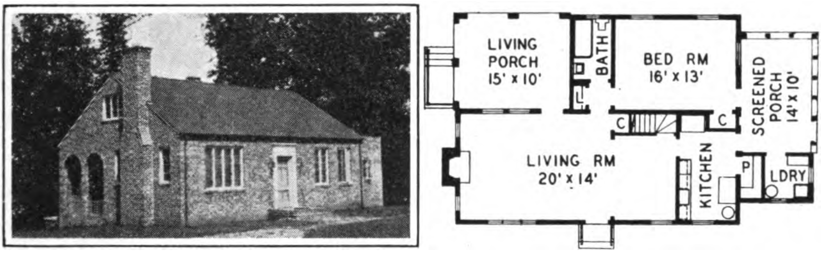

Figure 53.—*Permanent house type 31.*

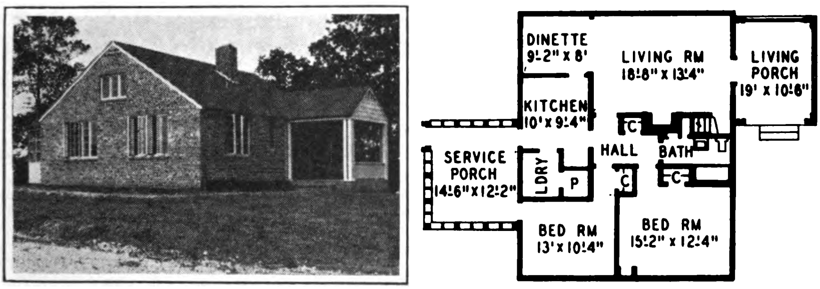

Figure 54.—*Permanent house type 41.*

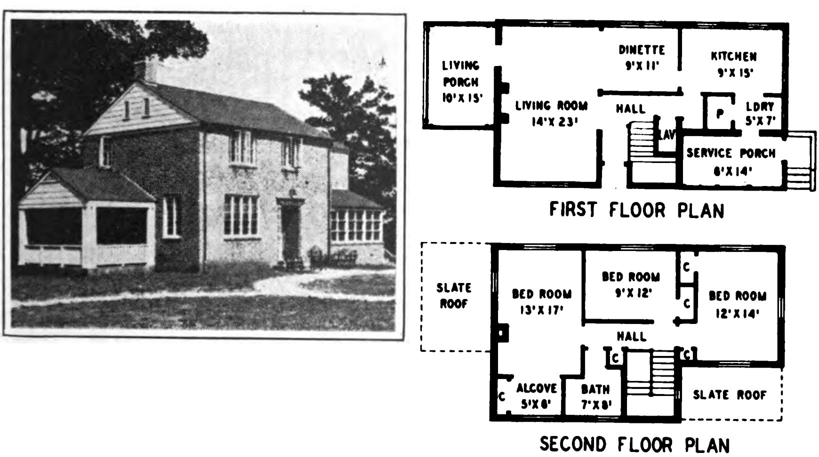

Figure 55.—*Permanent house type 52.*

foundations; the floors are loose gravel, and doors were omitted. One 11-car and two 8-car frame garages were built for the use of the occupants of the temporary houses.

Negro camp and village.

Dormitory.—A combination dormitory, mess hall, and recreation building was constructed for the single Negro employees. This building was arranged in an -shaped plan and was of frame construction with exterior walls of flush shiplap and interior walls unfinished. The dormitory wing was arranged with open cubicles and accommodated 120 men sleeping in double-tier bunks. Added height was provided in the dormitory wing to provide adequate ventilation. The other wing was equipped with a cafeteria with dining space for 68 men, a kitchen, a storage room, a laundry, and a recreation room. Shower and toilet facilities were provided in a centrally located portion of the building. The dormitory portion of the building was heated with open stoves and the remainder of the building by a central warm-air system.

Temporary houses.—Ten temporary houses of types shown in figure 53 were constructed for Negro employees. These houses were identical to the temporary houses for the white employees.

SERVICE BUILDING

A service building for the permanent village was built in the summer of 1937. It was placed to be convenient to visitors as well as village residents and designed to harmonize with the architectural scheme of the permanent village. This building is a one and one-half-story brick-veneer structure with a partially excavated basement. The first floor includes an entrance porch, a commissary, post office, reception room, offices for the police department and town manager, and visitors' toilet facilities. The second floor contains two bedrooms and a bath.

Where brick veneer was used, building paper was applied directly over the exterior surface of the studs. All walls and ceilings were thoroughly insulated with rock wool. Double floors with building paper between were used except in toilets where the floors were concrete. All walls were finished with ¼-inch V-jointed plywood, ceilings were of ½-inch celotex, roofing of asbestos shingles, gutters and downspouts of galvanized iron, and electrical heating was used throughout.

TABLE 6.—*Camp operation costs*

	Sales and other income	Net expenses [1]	Net operation costs
Dormitories and other buildings	$130,718.75	$105,306.32	[2] *$25,412.43*
Cafeteria, commissary, barber shop, filling station, and garages	588,920.47	587,288.00	[2] *1,632.47*
Utility services	12,792.61	8,605.27	[2] *4,187.34*
Administration of camp, fire and police protection, public safety, roads, streets, garbage and trash removal, and environmental sanitation	23.00	91,023.78	91,000.78
Employee training and recreation	9,059.23	142,679.27	133,620.04
Total before depreciation	741,514.06	934,902.64	193,388.58
Depreciation on buildings and equipment		[3] 349,474.41	349,474.41
Total	741,514.06	1,284,377.05	542,862.99

[1] Included here are (1) all operation expenditures and (2) interdepartmental credits ($139,731.60) which represent charges to other departments and operations for use of buildings and roads, policing, electricity, water, and similar services.

[2] Denotes credit.

[3] Includes the entire first cost of temporary camp facilities and normal depreciation during the construction on the permanent town facilities.

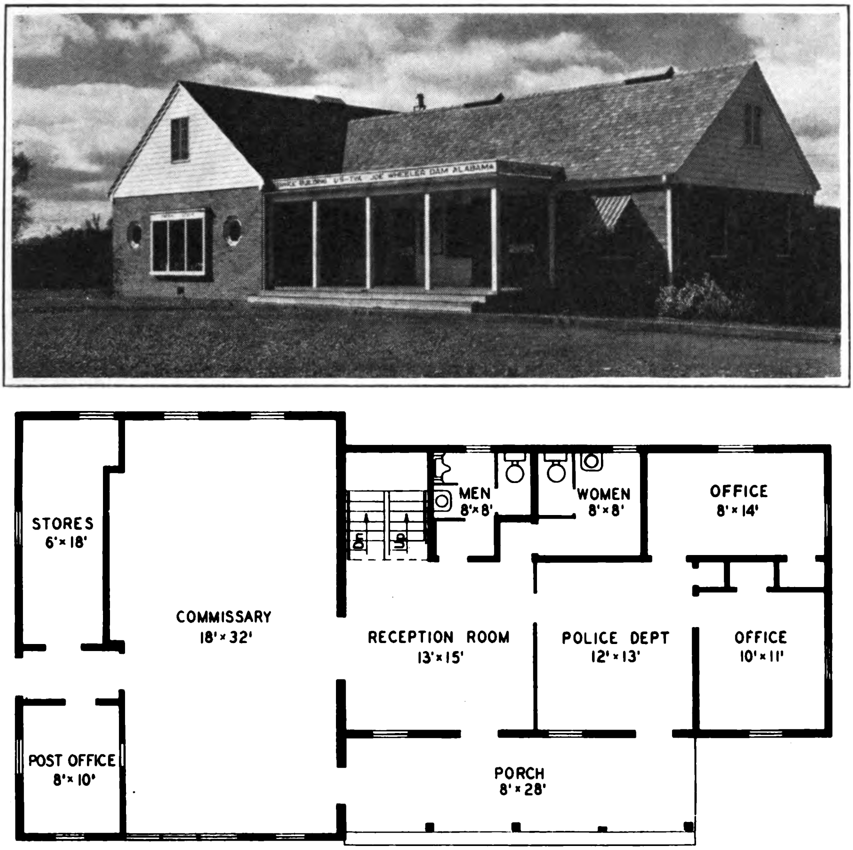

FIGURE 56.—*Permanent service building.*

Access To The Dam Site

Soon after the dam site was selected, various methods of shipping equipment and materials for the construction of the dam were studied. The location of the site, 15.5 river miles above Wilson Dam and 29.5 river miles below Decatur, placed it in an advantageous position for utilizing river transportation.

Facilities for transporting equipment and materials by water were available to serve the project. Decatur, Ala., had a railway incline for loading freight cars on barges and ferries and Sheffield, Ala., had two boat landings. During favorable navigation seasons, railway inclines at Hobbs Island and Guntersville, Ala., and a car-ferry between those points, operated by the Nashville, Chattanooga & St. Louis Ry. as a connecting link in their railway system, were also available for use.

The Southern Ry. passed within 8.5 miles of the site, and the Louisville & Nashville R. R. passed through Decatur and Athens. The Nashville, Chattanooga & St. Louis Ry. crossed the Southern Ry. at Huntsville, Ala.

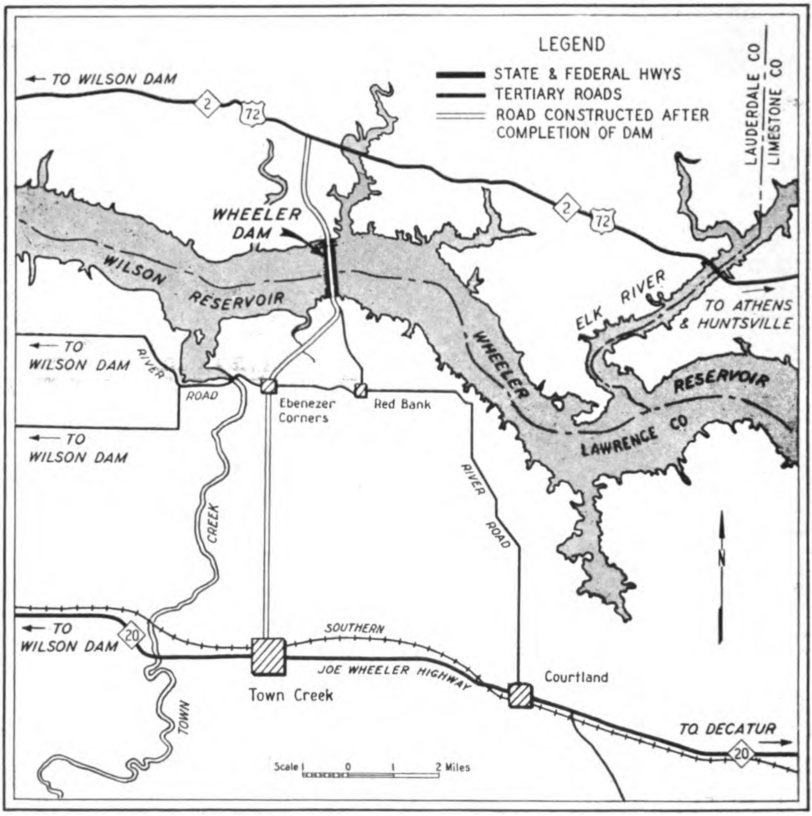

Figure 57.—*Roads in the vicinity of Wheeler Dam.*

On the south side of the river State Highway No. 20 paralleled the Southern Ry. from Decatur west and on the north side, U. S. Highway No. 72 crosses Elk River above the dam site and generally paralleled the Tennessee River, passing within 2 miles of the dam.

Access by water.

It was decided as a result of early studies that most of the construction equipment and material should be shipped by rail to Wilson Dam and transferred to barges for delivery to Wheeler. Wilson Reservoir, extending only 15.5 miles from Wilson Dam to the site of Wheeler Dam, was navigable for the entire distance; and since the maximum pool fluctuation was only 4½ feet in the event of unusual floods, no unusual trouble was experienced in loading the barges at Wilson or in unloading them at Wheeler.

The Authority used 3 towboats, 3 tenders, and 14 barges for all of the major construction transportation operations except for the transportation of the aggregates. The aggregates, furnished by contract, were obtained by dredging in the general vicinity of Riverton, Ala., 35 miles below Wilson Dam, and were transported to the dam in the contractor's barges.

Access by land.

The consideration of access directly to the site by rail was discarded almost immediately because of high cost. Although a high-grade access highway was not built to the dam site during the construction period, a few of the existing public roads leading to the site were slightly improved so that employees from Decatur, Sheffield, Florence, Tuscumbia, and other neighboring towns would have access to the site, and also in order that a relatively small amount of material, notably lumber, could be hauled to the site by truck.

When the project was started in 1933, Lawrence County, Ala., reconditioned an existing road from Red Bank to the south side of the river, where the construction plant buildings and camp were to be located. This road served as the principal access road during the entire construction period, and was maintained by the Authority from Red Bank to the dam site.

The Authority designed a first-class road to the south end of the dam from Ebenezer Corners, but did not build it because the expenditure of its estimated cost of $63,000 did not appear justified in view of the fact that most of the material was to be transported to the site by barges. However, in 1936, this road was finally constructed under a PWA project sponsored by the State and county. At the same time an access road connecting the north end of the dam with U. S. Highway No. 72 was designed by the Authority and constructed under the same PWA project.

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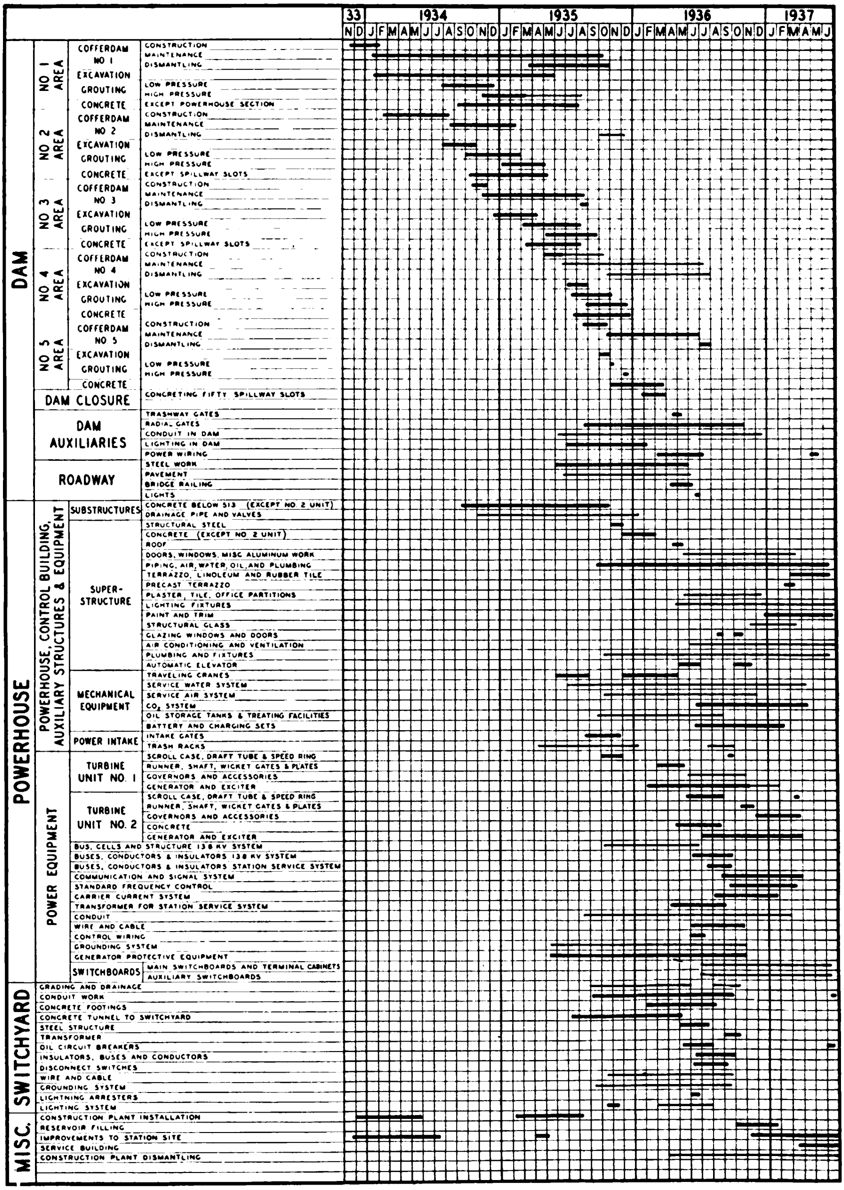

FIGURE 58.—*Construction Schedule.*

CHAPTER 5

CONSTRUCTION

The construction of Wheeler Dam was started November 21, 1933, and completed July 31, 1937. The reservoir was filled during the fall of 1936, the upper miter sill having been completed in October; the first power was generated on November 9, 1936; and the project was first operated for flood control in January 1937. Construction of the dam and power plant involved 548,000 cubic yards of excavation; 631,000 cubic yards of concrete; 8,500 tons of reinforcing steel; and 3,000,000 square feet of formwork. Employment at the dam reached a peak in July 1935 when over 4,700 men were at work. In addition to the construction problems, such features as the construction plant and the procurement of materials and personnel are discussed in this chapter.

CONSTRUCTION PERSONNEL

Construction was carried out by a "force account" organization working in four 5½-hour shifts per day. A construction organization was first set up functioning under two main heads—the Construction Engineer and Construction Superintendent, who in turn coordinated their work through the Assistant Chief Engineer. As the Authority widened its scope of activities and other projects were undertaken, a chief construction engineer was appointed, after which the construction engineer and construction superintendent reported directly to him.

The construction engineer was assisted by a staff composed of an assistant construction engineer, field engineer, office engineer, materials engineer, electrical engineer, and cost engineer. The construction superintendent was assisted by a staff composed of four assistant construction superintendents, master mechanic, chief electrician, structural steel foreman, general carpenter foreman, excavation foreman, concrete foreman, river transportation foreman, general labor foreman, storekeeper, and chief clerk.

Employment procedure.

Immediately after the passage of the Tennessee Valley Authority Act in May 1933, letters of application for employment were received at a rate of more than 1,000 per day. Regardless of the geographical residence of the applicants, formal blanks were sent to those whose letters indicated that they were qualified to render some service to the Authority. Therefore, when the Authority was requested by President Roosevelt in October 1933 to start construction of Wheeler Dam immediately, a large file of applications was already at hand.

As assistants were needed by the construction engineer and construction superintendent, the best-qualified applicants were selected from the files and were recommended for employment. All those selected were required to pass a thorough physical examination.

The TVA Act [1] specifies that all appointments be made on the basis of merit and efficiency without any political test or consideration. In the selection of employees for service with the Authority, the spirit as well as the letter of this particular provision was observed. It was recognized from the very beginning that the success of the project depended largely upon the ability to obtain the services of competent personnel at all levels.

When offices were opened in Knoxville in August 1933, the Authority was overwhelmed by thousands of applicants for labor and trade positions. Careful appraisal of the applicants interviewed in such large numbers was well-nigh impossible. To solve this problem, the United States Civil Service Commission, at the request of the Authority, held a series of examinations in which all applicants for nonprofessional positions (laborers and tradesmen) were required to participate in order to qualify for such positions. As a means of forestalling an influx of unemployed workers from other parts of the country, residence in the Tennessee Valley and certain adjacent areas was required of examinees. Announcements of the examination were published in newspapers and posted in post offices in the area.

In order to be certain that many well-qualified persons would file applications and thus be eligible to take the examination, personnel representatives visited 175 counties in the area, which included all of the State of Tennessee and parts of Kentucky, Virginia, North Carolina, Georgia, Alabama, and Mississippi. These representatives explained the examination procedure to leaders in the counties and asked them to urge competent persons in their locality to participate in the tests. Application forms were made available in the post offices in the area. The Civil Service Commission issued admission cards for the examination to all candidates whose blanks were filed on time and who met the residence requirements.

The examination consisted of a test of ability to follow oral and printed instructions, a simple reading test, and a mechanical aptitude test. A nonlanguage test enabled persons with little formal education to make some score on the examination. In this manner, persons of good native intelligence but without facility in reading and writing were not eliminated from consideration. The fact that less than half of the persons who received application blanks actually participated in the examination indicates that the examination requirements operated in themselves as a means of selection in eliminating many who were not particularly interested in employment with the Authority or who feared they might not qualify. When the examination results were made available in percentile rank distribution by the Civil Service Commission, personnel representatives were sent into the field to interview prospective employees selected from the top of the register. These interviewers were men familiar with the requirements of various trades, and they endeavored to judge

[1] See sec. VI, Tennessee Valley Authority Act, Public, No. 17, 73d Cong., and amended by Public, No. 412, 74th Cong.

accurately the qualifications of specific individuals. In addition they talked with leaders and employers in various communities to check the experience, qualifications, and general competency of the applicants, as well as to inquire into their standing in the community. The interview records were placed in the files of the applicants, and as men were requisitioned for work, candidates were selected with due regard to special skill, experience, and character. In many instances the candidates actually employed had acquired experience in construction work in other sections of the country.

For the most part, laborers and tradesmen were selected from the western portion of the examination area previously mentioned. Thus, Wheeler workers came from central and western Tennessee, northern Alabama, northeastern Mississippi, and a few from that portion of Kentucky that lies in the Tennessee Valley. Definite quotas of employment were not assigned to these counties, but a number of employees were selected from nearly every one. Only emergency requisitions could be filled immediately as the details of this procedure ordinarily required approximately one week. However, this difficulty was largely overcome by the construction supervisors' anticipating their labor requirements as far in advance as possible, and working in close cooperation with the personnel officers. Through this employment procedure a superior corps of workers was secured. This was especially important because many foremen and other supervisors were promoted "from the ranks" and, at the completion of the Wheeler project, transferred to other Tennessee Valley Authority projects.

Salary and wage schedules.

Salaries paid to annual employees of the Authority during the first few months were in general accord with rates then prevailing in local private industry. However, an Executive order [2] issued by the President on November 18, 1933, established a salary schedule to be followed by the newly created unemployment relief agencies. This schedule largely approximated the rates prevailing for the regular governmental departments, although there were certain differences in the higher levels. While the Authority was not bound by this Executive order,[3] the Board of Directors determined to follow it for the most part. The job supervisors prepared descriptions of duties for all positions, and these were compared with positions described [4] by the personnel classification board and the Civil Service Commission. Since Wheeler Dam was authorized October 16, 1933, employment at that location was largely in accordance with this preliminary classification of positions. In this manner, comparable salaries were paid for comparable duties and responsibilities. Wage schedules for the nonprofessional workers were established and adjusted from time to time to agree with the prevailing wages for similar occupations in the surrounding area. The hourly wage schedules are included in appendix D.

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[2] Executive Order No. 6440.

[3] Executive Order No. 6746 specifically exempted the Tennessee Valley Authority from the requirement of Executive Order No. 6440.

[4] Preliminary Class Specifications of Positions in the Field Service, Government Printing Office, 1930.

Labor relations.

During the first 2 years of construction, the maintenance of proper individual and collective relations between supervisory personnel and supervised employees was given much consideration, especially regarding working rules, regulations, established rates of pay, and working hours. At the instigation of either supervisors or supervised employees complaints were investigated and differences settled. The Authority also undertook the responsibility for seeing that employees understood their rights with respect to employee organization and cooperated in coordinating the activities of various organizations. Representatives of the Authority maintained contacts with organized labor groups in the area and with agencies of the Government relating to organized labor. Representatives also undertook to make certain that labor provisions of the Authority's contracts were fulfilled.

With the formal adoption of an employee relationship policy [5] in August 1935, the approach to these problems was changed. Previously, employees frequently came directly to the personnel department with their grievances, and members of that staff served as intermediaries between employees and their supervisors. The employee relationship policy, however, specifically requires that employees work through established supervisory channels, only those cases coming to the personnel department which are appealed after failure of satisfactory settlement on the job.

Training and recreation.

The Wheeler construction camp was rather quickly built in a somewhat isolated rural area with the result that adequate training and recreational facilities were not available for the workmen. A need, as well as an opportunity, for training and recreation was created by this condition, by the increasing scarcity of skilled workmen in certain trades, and the relatively large amount of leisure.

Training and recreational activities were conducted for both white and Negro employees and included job training, recreation, library service, and educational and commercial motion pictures. Job training courses which included such subjects as mathematics, blueprint reading, electricity, concrete, pipe fitting, estimating, use of special tools, and welding were related to the work on the dam whenever possible. The recreation activities were for the most part handled through an employees' recreation association.

The objectives of the program were to increase the efficiency of men on the job, to allow employees to prepare for other jobs inside and outside the Authority, to provide for the general educational, recreational, and social needs of employees and their families, and to provide elementary-school facilities for the children of employees living in the camp. The program arranged to meet these objectives was centered in the community building which housed the lounge, auditorium, gymnasium, and library, and separate schools for the white and Negro children.

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[5] See appendix I, *The Norris Project,* Tennessee Valley Authority Technical Report No. 1.

Labor turn-over.

In general the labor turn-over was low and was probably due to:

1. The care used in the original selection of employees.
2. The favorable wage scale of the Authority.
3. Good housing and working conditions.
4. The training and recreation program.
5. Lack of opportunities for private employment.

Based on a special study over a period of 19 months, there were 4,999 exits, of which 49 (1 percent) were discharged; 3,490 (69.8 percent) were laid off due to completion of the work; 529 (10.6 percent) resigned; and 931 (18.6 percent) left the project for other reasons, including for the most part transfers to other TVA jobs. This transfer procedure, of course, reduced the labor turn-over in the professional and skilled-labor positions and effected a definite saving to the Authority.

Safety.

In many respects conditions affecting safety of employees at Wheeler Dam were comparable to those at Norris.[6] Most of the keymen on the project had never worked together prior to their employment at Wheeler and their attitudes toward a definite safety program varied considerably. The supply of local labor skilled in heavy construction methods was inadequate, necessitating considerable in-service training. However, to counterbalance these unfavorable conditions and render the project a comparatively safe place in which to work even before the safety program was inaugurated, there were favorable factors such as the extensive use of modern construction equipment and methods, high type supervisory personnel, detailed project planning, advanced personnel policies for selection and placement of personnel, good employee-management relationships, detailed job training, and complete medical and first-aid facilities and service.

A safety engineer was assigned to the project shortly after the organized safety program was begun by the Authority in June 1934, and an additional safety officer was assigned in February 1935 at the request of the project management. Work of the safety engineer included the holding of crew and job safety meetings, the establishment of interdepartmental safety competition, the safeguarding of mechanical equipment, the investigation of accidents, the making of regular safety inspections, and the placing of safety signs and posters at strategic locations about the project. First-aid instruction was given by representatives of the United States Bureau of Mines, and 328 employees satisfactorily completed the basic course.

No full-time safety personnel was assigned to the reservoir clearance operations although contact was maintained with these units and with auxiliary construction operations in the reservoir area throughout the Muscle Shoals district. The accident experience for dam construction, reservoir clearance, auxiliary reservoir construction, town management, and miscellaneous construction operations is shown in table 7.

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[6] See *The Norris Project,* Tennessee Valley Authority Technical Report No. 1.

TABLE 7.—*Lost time accident experience*

[As of May 1, 1938]

Units	Fiscal year ending June 30	Man-hours of exposure	Number of disabling injuries					Rates	
			Fatal	Permanent total	Permanent partial	Temporary	Total	Frequency[1]	Severity[2]
Dam construction	1934[3]	496, 393				38	38		
	1935	4, 073, 383	1		16	245	262		
	1936	4, 391, 531	4		6	251	261		
	1937[4]	966, 652				9	9		
		9, 927, 959	5		22	543	570	57. 4	7. 49
Reservoir clearance	1934[5]	481, 576			2	60	62		
	1935	4, 198, 744	2		4	440	446		
	1936	1, 448, 581			2	131	133		
	1937	91, 548				11	11		
		6, 220, 449	2		8	642	652	104. 8	6. 74
Auxiliary reservoir construction	1934[6]	263, 411			2	17	19		
	1935	219, 586			1	11	12		
	1936	472, 648				37	37		
	1937	280, 169				20	20		
		1, 235, 814			3	85	88	71. 2	4. 52
Town management	1936	248, 772				1	1		
	1937	76, 634			1		1		
		325, 406			1	1	2	6. 1	. 93
Miscellaneous	1934[6]	81, 996				3	3		
	1935	328, 705				3	3		
	1936	163, 368				2	2		
	1937	109, 082				1	1		
		683, 151				9	9	13. 2	. 25
All operations	1934[3]	1, 323, 376			4	118	122		
	1935	8, 820, 418	3		21	699	724		
	1936	6, 724, 900	4		8	422	434		
	1937	1, 524, 085			1	41	42		
		18, 392, 779	7		34	1, 280	1, 321	71. 8	6. 66

[1] Number of lost time injuries per million man-hours worked.
[2] Number of days lost per thousand man-hours worked.
[3] Period began November 1933.
[4] Wheeler Dam was designated "Honor Roll Company" by the National Safety Council by virtue of having achieved the lowest accident experience in its industrial classification.
[5] Period began February 1934.
[6] Period began December 1933.

In addition to the industrial safety work, a public safety unit was organized to provide police and fire protection, to guide visitors, and to perform miscellaneous management functions. Despite the large number of visitors and a camp population of approximately 2,000, it was necessary to arrest very few persons for civil offenses occurring on TVA property during the construction period. No major crimes were committed and no automobile accidents occurred which resulted in personal injury or major property damage.

Medical services.

Medical service during the construction period was provided through two medical service units, one located at the dam and the other centrally located in the reservoir area. These units were administered through a central office in Knoxville.

The medical service program offered the following principal services:

1. Employment physical examinations, including preemployment, transfer, termination, and special physical examinations and appraisals.
2. Immunization of employees against typhoid fever and smallpox.
3. Care of service-connected injuries, including treatment and handling of employees injured while in the performance of duty.
4. Emergency care of illnesses, non-service-connected, occurring to employees while on duty.
5. Venereal disease control, including prophylaxis and treatment.
6. Medical care, prevention, and treatment of non-service-connected illnesses occurring to employees and their dependents remote from other medical facilities.
7. Preparation and handling of compensation claims.
8. Emergency care of injuries and illnesses occurring to non-employees in which the Authority might become involved.

Medical service was provided at the dam from January 15, 1934, to April 15, 1937. Personnel for this unit was headed by a doctor responsible for the immediate administration and operation of the medical unit. He was assisted during the peak of construction activities by one or more associate medical officers, graduate nurses in sufficient number to provide 7-day, 24 hour nursing service for hospital patients and to assist medical officers in out-patient services; sufficient medical aides to render first aid and to re-dress minor injuries at the immediate site of operations and during all hours of construction activities; X-ray and clinical laboratory technician; compensation and record clerks; and miscellaneous hospital help such as cooks and orderlies.

Medical facilities were increased as construction activities grew. These facilities ultimately included an emergency operating room, X-ray and clinical laboratory, kitchen, staff dining room, space for 14 hospital beds, and a small isolation ward. A portable dressing station was located adjacent to actual construction operations for treatment of minor injuries and initial emergency care of major injuries. A small, lightweight ambulance and a small speedboat were used for the transportation of seriously injured employees. First-aid equipment was placed at convenient locations throughout the entire job area.

Medical service was provided at Decatur, Ala., for the employees in the reservoir area. This service was started February 25, 1934, and continued to April 1, 1937. The organization, though smaller, was similar to that at the dam. Facilities for this unit consisted of a physician's office, treatment rooms, small clinical and X-ray laboratory, clerical office, and physical examination rooms. A small trailer-type mobile medical unit was used for field service to concentrated groups of employees. Medical aides who made daily contact with the decentralized units of field employees were supplied with semi-ambulances equipped for prone transportation of seriously injured employees. Local hospitals in Decatur, Ala., and the Guntersville Dam hospital were used for hospitalization and treatment of injured employees needing attention. Table 8 shows the services performed by the dam construction and reservoir construction medical units.

Table 8.—*Medical services*

	Dam construction	Reservoir construction	Total
Employment examinations	11,669	10,079	21,748
Immunization services	15,124	18,911	34,035
Treatments of service-related injuries	47,257	28,429	75,686
Emergency services for minor illness:			
Nonservice related	16,352	5,281	21,633
Venereal disease treatments	3,474	58	3,532
Services in medical care program (out-patient and infirmary)	6,123		6,123
Total medical services	99,999	62,758	162,757

PROCUREMENT OF MATERIALS

The procurement program was recognized as one of the vital factors in the efficiency and economy of the construction project. The requirements of the purchasing program were that for each purchase the quality would be consistent with that desired in the final structure; the delivery would conform to the construction schedule and the needs of the organization; the price would be consistent with the cost of efficient manufacture; the activities of the procurement forces would be in accordance with existing regulations; and the records of all transactions would be preserved for future reference.

Development of purchasing system.

Originally the Tennessee Valley Authority Act contained no regulations for the purchase of materials and equipment. The Board of Directors took the position that the general Federal Purchasing Statute, section 3709, was not applicable to the Authority. They decided as a matter of policy, however, to comply with its provisions where feasible.

A director of purchases was appointed and an organization was created to work out procurement policies and procedures and to perform the purchasing activities of the Authority. The most desirable features of the purchasing systems which had been investigated were adopted, and new ideas incorporated as needed. The result was a system designed specifically to meet the requirements of the Authority.

Purchases were made on the basis of least final cost to the Authority. The practice of award to the lowest responsible bidder meeting all conditions of bid and complying with the specification was followed.

Purchasing system under the amended act.

The act[7] as amended in 1935 gave the Authority the discretionary powers necessary to direct its purchasing for the greatest benefit to the project and allowed for the full exercise of procurement and engineering judgment in making purchases on a businesslike basis. Under the amended act the acquisition of sound and dependable equipment was simplified and it was possible to secure for the permanent structures materials, machinery, and workmanship which would result in the greatest service per dollar expended.

The general authority to purchase and contract for materials, equipment, and supplies was vested in the Board of Directors. Much of

[7] Section 9b.

this authority was delegated by the Board to the director of purchases and his authorized assistants.

The three general methods of purchasing materials, equipment, and supplies were to purchase after advertising, to purchase in the open market, and to purchase small items without competition. The method used depended upon the circumstances surrounding the individual purchase.

Advertising was employed for original contracts involving more than $500 which did not entail an emergency or require repair parts or supplemental equipment or services. Requisitions were sent to the budget officer for authorization, and to the director of purchases for approval. Specifications were prepared or approved, and invitations to bid were drawn up, advertised, and distributed to prospective bidders. Advertising was conducted through newspapers, trade periodicals, circulars or letters sent to vendors, and by notices posted in public places. The minimum time of advertising varied from one to three weeks, depending upon the nature of the purchase, and was extended when necessary. Formal sealed bids received after advertising were opened, read, and recorded in public. Informal sealed bids, however, were not necessarily read and recorded in public. Analyses of complicated bids requiring engineering judgment and recommendations of award were submitted by the engineer in charge. Awards were approved by the director of purchases, and notices of awards sent to the successful bidders. The final contracts were prepared and distributed to the successful bidder for execution and return. Shipping instructions were prepared and sent to the contractor. The necessary inspection and testing were carried on, and final acceptance was made by the office initiating the requisition. Final payments were made by the Authority according to the terms of the individual contract.

Purchases involving less than $500, purchases of an emergency nature, repair parts, supplemental equipment, or services were made in the open market when necessary in much the same manner employed by businessmen. Reorders were handled likewise. The procedure followed was the same as used in formal contracts except that bids were requested by circular or letter rather than by advertising to the general public.

Small items costing less than $25 were purchased from wholesalers and jobbers without competition. A staff of purchasing agents was maintained to handle this work along with other purchasing operations.

Each of these methods had its own particular advantages and disadvantages. Purchase after advertising gave access to the widest market and made possible the lowest prices. This method, however, required considerable time. Purchase in the open market saved time but restricted the market. The purchase of small articles without competition eliminated much of the procurement cost on items where printing charges alone would have been a large proportion of the total cost. The three methods, used discriminately, offered an excellent means of satisfying the purchasing requirements.

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Procurement activities.

The first step in the procurement program for the project was to ascertain the immediate needs and to make provisions for supplying them. A field purchasing office was established at the site to coordinate the work at that point. Provisions were made for warehouse and storage facilities, and studies were conducted to determine the most suitable transportation arrangement.

Items to be purchased were catalogued and procurements scheduled to conform with the construction schedule. This scheduling had to be revised frequently because much detailed design work was not completed until construction was advanced.

Studies were made to determine the most economical and practical means of transporting materials and equipment. Since there were no satisfactory railway or highway connections to the dam site, arrangements were made to utilize the docking facilities at Wilson Dam and to transport shipments from there to the dam site by water. This method not only saved the expense of building a heavy-duty access road, but also made it possible to start delivery almost immediately. Negotiations with the railroad company serving the area resulted in a reduction in some rates on bulk material. Attention was given to freight cost reduction possible through the use of Government bills of lading on land-grant railroads.

Standardization studies were conducted and the Authority's requirements were adapted to regular market items when it was feasible to do so. Standard specifications were prepared for items which were to be required in considerable number, either at Wheeler or future projects. Federal, ASTM, and other standard specifications were adopted for stock items. This work made it possible to secure earlier deliveries and better facilities for interchange of equipment, as well as lower bid prices.

Provisions were made for necessary inspection and testing of items to be purchased. Stock items were inspected upon delivery to see that they conformed to the specifications. Special items, such as turbines, gates, and other equipment designed particularly for the project were tested and inspected during the manufacturing period so that discrepancies could be discovered and corrected with a minimum loss in time and money. Acceptance tests were conducted at the project to supplement this work and to insure the fulfillment of performance requirements.

One outstanding feature of the procurement activities was the purchase of cement.[8] In the original bidding for a small portion of the cement needed for Norris and Wheeler Dams, identical bids were received from all companies participating. This price was analyzed and considered excessive. Studies were conducted to determine the actual production cost of cement as well as the possibility of the Authority's manufacturing its own cement. Negotiations were then carried on with the cement companies for furnishing the remainder of the estimated quantities for all projects then under consideration. New bids were asked and identical prices were again quoted by all bidders. However, on this rebid the cost was considered satisfactory and cement contracts were allocated to 10 plants in the Tennessee Valley area. The cement contracts finally made resulted in a saving

[8] See appendix E, *The Norris Project,* Tennessee Valley Authority Technical Report No. 1.

to the Authority of approximately 38 cents per barrel on the 840,700 barrels of cement used in Wheeler Dam.

Another unusual feature was the purchasing of some materials on existing contracts of the United States Treasury, the Navy, and other governmental departments. This practice saved money and eliminated duplication of effort.

The major purchases by contract for materials and equipment used in the dam, powerhouse, and switchyard are listed in table 9.

TABLE 9.—*Major material and equipment contracts*

Item	Contractor	Date of award	Cost [1]
Aggregate and cement			
Aggregate	Cumberland River Sand Co	May 7, 1934	$1,166,059.01
Do	do	Nov. 5, 1935	3,029.32
Do	do	Dec. 11, 1935	3,328.29
Do	do	Jan. 10, 1936	2,521.36
Do	do	Aug. 28, 1936	4,101.30
Cement	Alpha Portland Cement Co	May 15, 1934	32,446.79
Do	Cumberland Cement Co	May 4, 1934	37,106.87
Do	Lehigh Portland Cement Co	May 8, 1934	70,148.16
Do	Alpha Portland Cement Co	Oct. 22, 1934	162,691.58
Do	Universal Atlas Cement Co	do	247,124.55
Do	Marquette Cement Co	do	141,923.60
Do	Lehigh Portland Cement Co	do	163,014.80
Do	Hermitage Portland Cement Co	do	157,989.55
Do	Cumberland Cement Co	do	95,241.24
Do	Lonestar Cement Co	do	257,044.48
Do	Penn-Dixie Cement Co	do	160,103.91
Structural steel			
Roadway bridge steel	Duffin Iron Co	Apr. 26, 1935	17,153.00
Do	Mt. Vernon Bridge Co	do	83,548.80
Control building steel	Wisconsin Bridge and Iron Co	Aug. 12, 1935	41,744.62
Roadway bridge steel	McClintic Marshall Steel Co	Nov. 7, 1935	76,866.00
Switchyard steel	Bethlehem Steel Co	Jan. 28, 1936	16,384.32
Lumber			
Lumber	Moore Newton Lumber Co	Nov. 28, 1934	8,497.04
Do	Laumer Griffith Co	do	6,632.94
Mechanical and hydraulic equipment			
Seat frames, bronze liners for draft tube gates.	Reuter Bros. Iron Works	Oct. 15, 1934	11,304.31
Turbine No. 1 and spare parts	Baldwin-Southwark Corporation	Oct. 22, 1934	343,996.58
Gate frames and parts	Virginia Bridge and Iron Co	Nov. 15, 1934	44,010.00
Trashrack metal work	Bartlett Hayward Co	Dec. 5, 1934	27,580.00
Inlet service gates	do	Feb. 26, 1935	200,499.92
Bulkhead gates	Virginia Bridge and Iron Co	do	7,364.00
Inlet emergency gates, skimming screen.	Ingalls Iron Works Co	do	21,561.10
Draft tube pier nose castings	Machined Steel Casting Co	Apr. 3, 1935	[2] 11,930.00
Draft tube bulkhead gates	Converse Bridge and Steel Co	Apr. 26, 1935	5,850.00
Oil Purifier	Delaval Separator Co	May 20, 1935	5,880.00
Deep well pumps	Aurora Pump Co	May 31, 1935	10,579.01
Radial gates	Lakeside Bridge and Steel Co	June 17, 1935	102,693.12
Trashway gates	Pittsburgh Des Moines Steel Co	do	2,467.27
Compressors	Worthington Pump and Machine Co	do	1,900.93
Do	Ingersoll Rand Co	do	6,222.97
Oil storage tanks	Stacey Manufacturing Co	Aug. 3, 1935	2,804.25
Turbine governors	Woodward Governor Co	Aug. 23, 1935	36,327.00
Turbine No. 2	Baldwin-Southwark Corporation	Nov. 18, 1935	314,697.67
Equipment for inactive gates	Bartlett Hayward Co	Feb. 3, 1936	24,745.20
Trashrack	Newport News Shipbuilding and Dry-dock Co.	Dec. 17, 1936	[2] 8,184.98
Floating bulkhead gate	Lakeside Bridge and Steel Co	Feb. 13, 1937	[2] 8,880.00
Portable deep well pump	Aurora Pump Co	Mar. 10, 1937	1,846.55
Electrical equipment			
Generator No. 1	General Electric Co	Jan. 17, 1935	[3] 424,661.08
Main power transformers	Westinghouse Electric and Manufacturing Co.	Aug. 29, 1935	155,567.27
Oil circuit breakers	do	Sept. 12, 1935	[3] 187,265.00
Outdoor switchgear	do	Oct. 19, 1935	40,518.93

For footnotes, see end of table.

TABLE 9.—*Major material and equipment contracts*—Continued

Item	Contractor	Date of award	Cost [1]
Electrical equipment—Continued			
Gasoline electric generator	Buffalo Gasoline Motor Co	Nov. 14, 1935	[2] $5,007.00
Generator No. 2	General Electric Co	Nov. 18, 1935	[3] 409,425.03
Battery charging sets	Electric Products Co	Dec. 12, 1935	2,630.00
Disconnect switches	Delta Star Electric Co	Jan. 18, 1936	32,779.43
Switchgear	do	Jan. 27, 1936	36,343.79
CO_2 equipment	Walter Kidde Co	Feb. 3, 1936	15,824.44
Switchboards, annunciators, oscillographs.	Westinghouse Electric and Manufacturing Co.	Feb. 7, 1936	77,194.00
Switch housings	do	Feb. 8, 1936	4,111.37
Generator reactor	General Electric Co	do	1,343.00
Generator housings	Lakeside Bridge and Steel Co	Feb. 29, 1936	[2] 13,762.00
Auxiliary switchboards	Westinghouse Electric and Manufacturing Co.	Mar. 10, 1936	45,975.00
Bus structure, transformer housings	Delta Star Electric Co	Apr. 13, 1936	4,484.87
Switchgear	Westinghouse Electric and Manufacturing Co.	Apr. 27, 1936	3,230.18
Do	do	May 15, 1936	22,523.19
Frequency source, load frequency control.	Leeds and Northrup	May 26, 1936	12,478.75
Electric motor, solenoid valve	Wallace Tiernan Co	June 11, 1936	[3] 110.00
Cranes, hoists, and transporting equipment			
85-ton gantry crane	Harnischfeger Sales Corporation	Nov. 13, 1934	[2] 36,195.41
50-ton guy derrick	American Hoist and Derrick Co	May 20, 1935	[2] 9,837.19
270-ton, 20-ton gantry cranes	Alliance Machine Co	May 31, 1935	[2] 87,920.00
Hoisting assembly for radial gates	Clyde Sales Co	June 17, 1935	171,312.89
Chain assemblies	S. G. Taylor Chain Co	do	15,146.67
4-ton low head cranes	Euclid Armington Corporation	Nov. 1, 1935	[2] 2,680.00
Rotor lifting beam	Morgan Engineering Co	do	[2] 4,304.09
100-ton transfer car	Alliance Machine Co	Nov. 14, 1935	2,268.31
75-ton traveling crane	Bedford Foundry and Machine Co	Nov. 25, 1935	[2] 11,219.92
Elevator	Warner Electric Elevator Co	Feb. 29, 1936	[3] 21,935.15
Station service equipment			
Radial drill	American Tool Works Co	Aug. 22, 1935	[2] 4,899.19
Power hacksaw	Woodward, Wight and Co	Sept. 9, 1935	505.00
Communication and code call equipment.	American Automatic Electric Sales Co.	Mar. 23, 1936	10,340.98
Ventilating systems	Scholl, Choffin Co	Apr. 27, 1936	[3] 6,598.54
Carrier current system	General Electric Co	May 5, 1936	42,655.92
Water filter equipment	E. W. Bacharach and Co	June 23, 1936	[3] 3,390.00
Telephone cable	General Cable Corporation	July 8, 1936	4,179.38
Air-conditioning ducts	Scholl, Choffin Co	July 16, 1936	[3] 8,581.21
Air conditioning	York Ice Machinery Co	Nov. 2, 1936	[3] 18,761.00
Control building superstructure work and equipment			
Aluminum doors	Kawneer Co	Mar. 4, 1936	[2] 14,265.54
Aluminum sash	Detroit Steel Products Co	Mar. 5, 1936	14,788.10
Metal doors	General Bronze Corporation	Apr. 1, 1936	[2] 12,512.00
Do	Superior Steel Door and Trim Co	Apr. 25, 1936	[2] 8,273.67
Steel partitions	E. T. Hauserman Co	May 7, 1936	[3] 4,316.60
Governor gallery tile floor and base	George Wallace, Jr	May 14, 1936	[3] 551.58
Glazing	Marlin Glass Co	May 25, 1936	[3] 2,486.79
Terrazzo work	Caldwell Marble and Tile Co	July 13, 1936	[3] 3,651.26
Plastering	A. G. Hopton	Aug. 11, 1936	[3] 2,547.30
Linoleum and rubber tile floor	Southern Flooring and Insulating Co.	Aug. 13, 1936	[3] 2,547.30
Tile floor	George Wallace, Jr	Aug. 25, 1936	[3] 882.01
Structural glass	Sanitary Construction Co	Sept. 4, 1936	[3] 8,215.40
Precast terrazzo	Marcus Marble and Tile Co	Sept. 14, 1936	[3] 3,055.50
Venetian blinds	Knoxville Awning, Tent, and Tarpaulin Co.	Jan. 19, 1937	[3] 750.68
Floor finishing	Darby Hardware Co	July 12, 1937	[3] 190.00
Reception room furniture	Fowler Bros. Co	July 13, 1937	[2] 574.39
Road construction and equipment			
Handrails for roadway	Blaw Knox Co	Dec. 12, 1935	29,685.00
Road gravel	A. A. Ballew	Oct. 15, 1936	3,590.48
Hard surface roadway, south approach to dam.	Carter Construction Co	Apr. 19, 1937	[3] 3,628.06

[1] All deliveries f. o. b. Wheeler Dam, or Sheffield, Alabama, at Authority's option, except as noted.
[2] F. o. b. manufacturer's plant.
[3] Installed at Wheeler Dam.

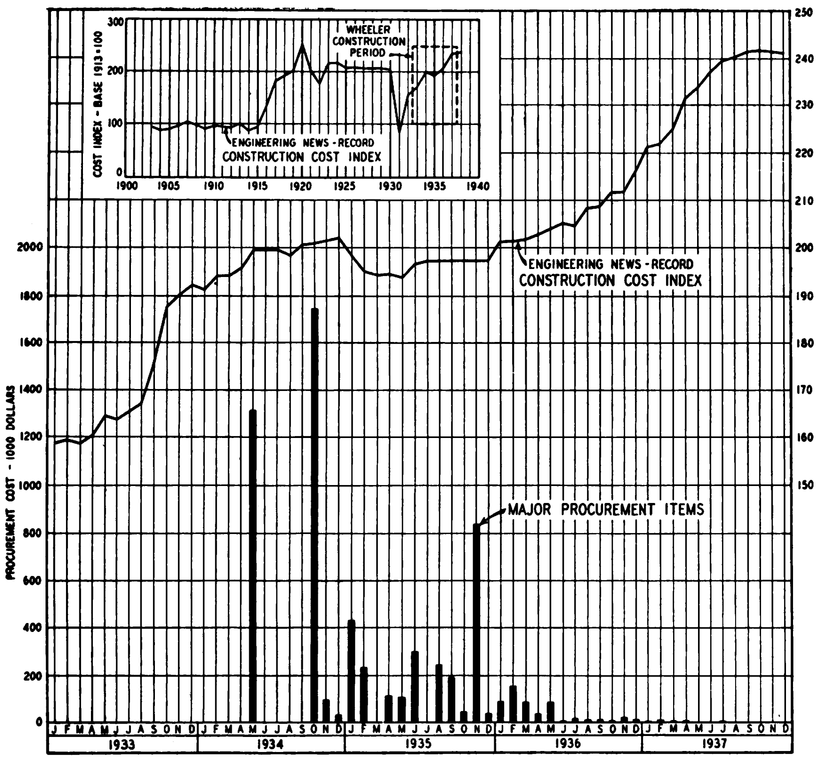

Figure 59.—*Relation of major procurement*[9] *to the* Engineering News-Record *Construction Cost Index.*[10]

Market conditions during construction.

Procurement of major items began in the latter part of 1933 when construction costs were just rising from their lowest point in 15 years, reached in 1932. The *Engineering News-Record* Construction Cost Index average throughout the procurement period was approximately 190 as compared to 158.4 at the beginning of 1933. These figures, while not entirely applicable to the Wheeler project, reflect general market conditions and show the consistent upward trend during the construction period.

A large part of the purchases was therefore made in a rising market, and manufacturers naturally included a margin of protection in their bids. Invitations to bid were designed to alleviate this condition where possible, but could not entirely eliminate it.

Identical bids were received in a number of cases, especially during the existence of the National Recovery Administration. If they were acceptable, the successful bidder was determined by drawing between bidders.

COFFERDAMS

It was recognized that the construction period would include two full years before permanent river closure could be made. Because

[9] See table 9 for list of major material and equipment contracts.
[10] Engineering News-Record, 122: 80–81, April 27, 1939.

of the great width of the river at the dam site, the problem arose of determining the most suitable number of cofferdams. Short cofferdams would not restrict the flow in the river so greatly during the early stages and would permit lower cofferdam heights. The areas exposed in each would also be less and would reduce the unwatering and maintenance expense should high water flood out an area at any time. High river velocities owing to constricted channels would also be reduced until the later construction stages. After considering these factors, it seemed desirable to use five short cofferdams rather than a smaller number of longer ones.

The first cofferdam projected into the river from the south bank, and the others were continued successively across the river until

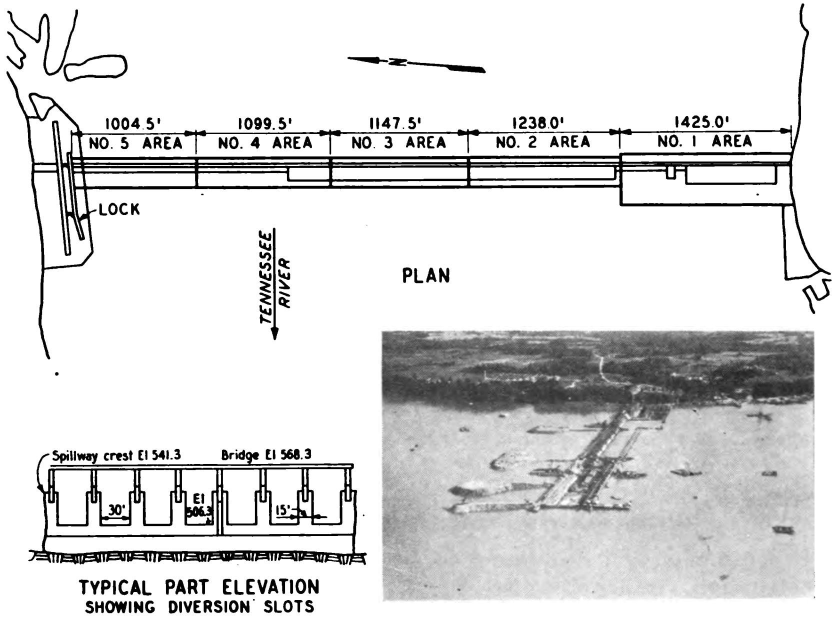

FIGURE 60.—*Cofferdam plan for river diversion.*

the fifth cofferdam effected the river closure at the completed navigation lock near the north bank.

Prior to the construction of cofferdam No. 5 the stream flow was confined to the river channel north of the line of cofferdams. By the time the last cofferdam was scheduled for construction, the intakes in the powerhouse had been completed and the stalls for the six future units were utilized to pass the ordinary flow of the river. Meanwhile the downstream arms of the second and third cofferdams were removed. Floods in excess of the capacity of the intakes overtopped the upstream arms of these cofferdams and flowed through fifty 30-foot openings which had been left temporarily in the spillway section.

The spillway section was constructed in alternate 15- and 30-foot blocks. Twenty-two of the 30-foot blocks were stopped at a height about one foot above the normal elevation of Wilson pool, elevation

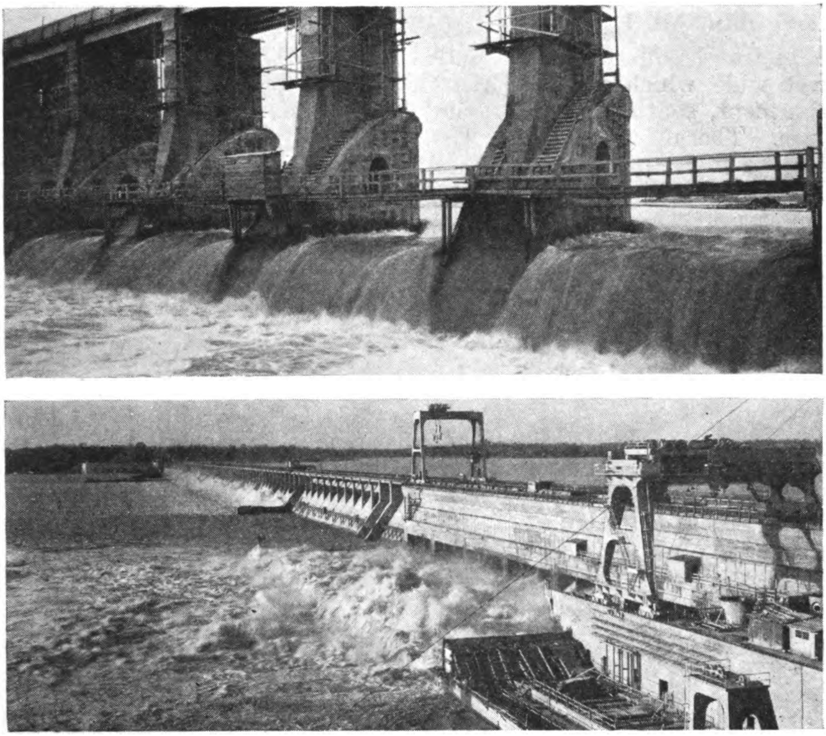

Figure 61.—*River diversion—Water passing over low spillway blocks and through the powerhouse intake section during high flow.*

505.4, and the remaining 28 about 8 feet above that elevation. The intermediate 15-foot blocks were continued to the full height to permit construction of the piers and the roadway across the dam. After completion of the work in the fourth and fifth cofferdams, all temporary spillway openings were concreted to the finished crest, elevation 541.3, and the tainter gates were installed.

The use of five cofferdams gave an opportunity to study the effect on the height of the water surface above and below the structure for various river discharges. Data gathered during early stages were used in designing the cofferdams for later stages and in regulating navigation in the construction area during flood stages. As soon as the first cofferdam was completed, gages were established on both the upstream and downstream sides. Readings were taken twice daily during periods of normal flow and hourly during flood periods. Figure 62 shows a typical map of water surface contours during a flood. Similar maps were made of all important floods for each cofferdam.

All cofferdams except No. 2 were the Ohio River type, No. 2 being constructed of waste rock obtained from the powerhouse excavation. Construction was begun at the south bank of the river since construction of the lock was already under way and further work there would have meant congestion of the area already in use by the lock contractor.

Cofferdam No. 1.

The first cofferdam surrounded an area 1,425 feet long and 416 feet wide which included the intakes, powerhouse, tailrace, south abutment, south trashways, and south nonoverflow section of the dam. The upstream arm was about 50 feet upstream from the face of the dam. This structure consisted of an earth fill confined between two lines of wood sheathing spaced 20 feet apart and held in place by timber wales and steel tie rods. A small rock fill was placed at the inside toe of the upstream wall to prevent this wall from sliding.

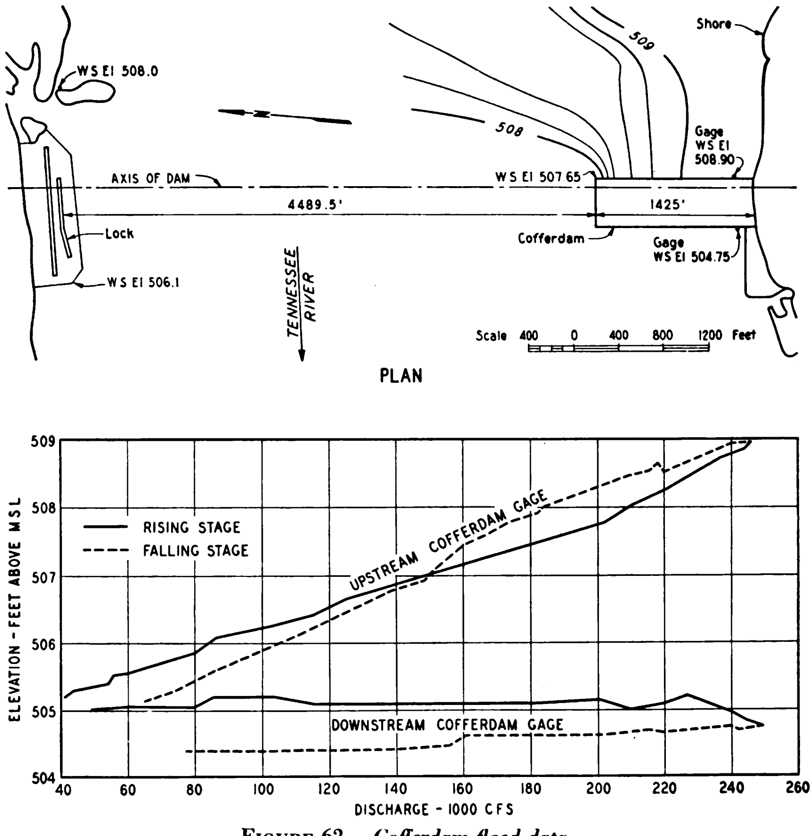

FIGURE 62.—*Cofferdam flood data.*

The timber framing was started on November 20, 1933, at Wilson Dam, and as completed it was transported by barge to Wheeler Dam site. At the site the sections were assembled on small wooden barges and placed in final position by a small derrick. After the sheathing had been driven in place it was nailed to the top frame. A large portion of the lumber used in this construction was obtained from timber cut in the reservoir-clearing operations.

Cofferdams were constructed entirely with marine equipment, most of which was rented from the United States Army Engineers. Major items consisted of:

2 Dipper dredges, steam powered, 1½ cubic yards capacity
4 Large derrick boats, steel barges, steam powered

2 Small derrick boats, wooden barges, wooden mast and boom
1 Diesel electric towboat, of capacity to move several barges
3 Small towboats, with capacity to move one loaded barge
8 Steel materials barges, average size 25 feet by 100 feet
4 Small wooden barges of various sizes

Clay fill for the cofferdam was obtained from the north bank of the river about 600 feet upstream from the lock and from several small towheads near the axis line. It was excavated with a dipper dredge and transferred to the cofferdam by barge and towboats and placed in position by derrick boats equipped with clamshell buckets. Work on the cofferdam progressed at the rate of about 65 feet of wall per day. Final closure was made on January 16, 1934, 51 days after the start of construction.

Rock-fill cribs for pumps were placed inside the downstream arm at the point of deepest water. Three 12-inch and two 6-inch electrically driven centrifugal pumps were used (working 53 hours, from January 16 to January 18, 1934) for the unwatering operation.

Cofferdam maintenance covered a period of 628 days from January 8, 1934, to October 11, 1935. Little water leaked through the structure. The pump requirements for maintenance of the water level were:

Pump size and type	Power	Number used	Pump days used
12-inch centrifugal	Electric	1	156
6-inch centrifugal	do	3	1,200
4-inch centrifugal	do	2	780
2½-inch rotary sump	Air	4	Continuous.
3-inch centrifugal	Gasoline	1	Intermittent.

Since the parts of the trashway and nonoverflow sections of the dam within cofferdam No. 1 were completed before the powerhouse, cut-off cofferdam walls were constructed at right angles to the upstream and downstream arms to eliminate the necessity of maintaining a dry cofferdam in the section where concreting had been completed. For these cut-offs the same type of construction was used as in the cofferdam. The total length of both cut-off walls was 276.4 feet.

After the cut-off walls had been completed, the upstream wall of the remainder of the cofferdam was raised 7 feet from elevation 513.3 to elevation 520.3 to withstand increased head-water elevation caused by the construction of additional cofferdams. This was accomplished by adding mud-filled box cribs 7 feet high and 8 feet wide to the original wall. These extended 953 feet, from the south abutment to the cut-off wall, and along the cut-off wall 38 feet to the upstream face of the dam.

One dipper dredge, three derrick boats, and several steel barges were used to remove the cofferdam. When the cofferdams were broken up, much of the timber and some of the fill was carried off by the flow of the river. The remainder was loaded onto barges for disposal. The entire structure had been removed by the end of October 1935.

Tables 10 and 11 show comparative construction and maintenance costs for all cofferdams.

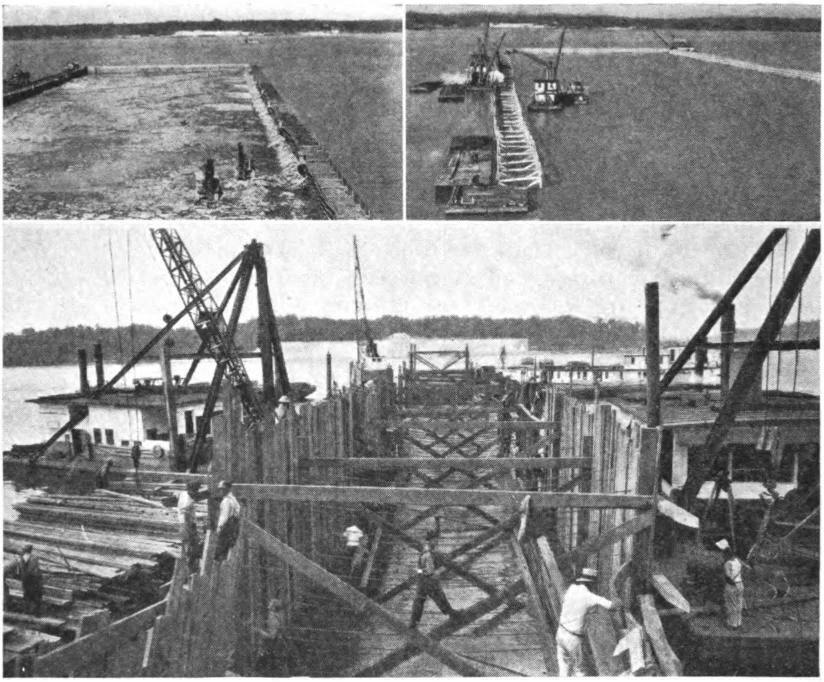

FIGURE 63.—*Cofferdam construction.*

Cofferdam No. 2.

This structure was a rock fill covered with a clay blanket. It enclosed an area 1,238 by 260 feet, long enough to permit the construction of the first 25 bays at the south end of the spillway structure and wide enough to include the spillway apron. The upstream arm was 50 feet upstream from the face of the dam, was built to elevation 512.3, and was 24 feet wide on top. The downstream arm was built to elevation 509.3 and had a 14-foot top width. Rock surface of the river bed was approximately at elevation 497.

All of the rock for this cofferdam was obtained from the powerhouse excavation. Trucks of 10-cubic-yard capacity started the fill by extending the upstream and downstream arms of cofferdam No. 1. Most of the grading was done by hand because room was lacking for a tractor equipped with a bulldozer on these fills. Extra fill was placed at three locations to provide turning points for the trucks.

Material excavated from the south approach highway was used for the blanket fill, and some additional borrow adjacent to this area was also required. A special loading dock permitted trucks to dump directly onto barges which transported the fill to the cofferdam where it was placed by derrick boats. When better control of placing operations was desired or when rough water prevented the loading of barges, the trucks carried the material over the ramps of cofferdam No. 1 and placed it directly in the fill of cofferdam No. 2.

The position of the structure exposed the clay blanket to wave action. A wooden mat, extending from below the water line to above a line of wave action, was placed on the fill to protect the clay blanket. The fill was leveled to the correct slope by hand labor to obtain a uniform surface for the mat. The mat was made of rough lumber 1 inch thick nailed with 2-inch cross battens. A rock toe fill was used to hold the mattress in place.

Leaks developed when the first attempt was made to unwater the cofferdam. The mats were removed and additional material was placed on the fill. A row of wooden sheeting was driven along the toe of the main fill. This sheeting was protected on the outside by a toe fill of rock, and between the sheeting and the main rock fill additional clay was placed. This treatment proved adequate except in a few places where more material was required because of settlement.

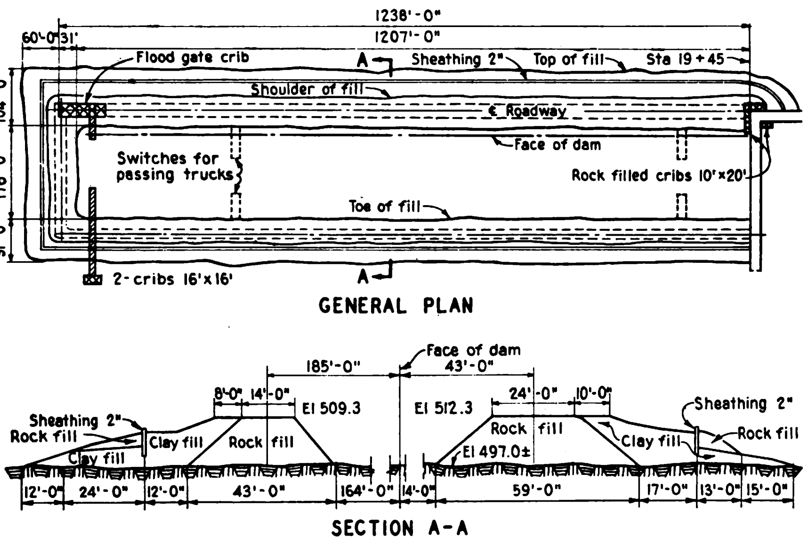

FIGURE 64.—*Cofferdam No. 2 details.*

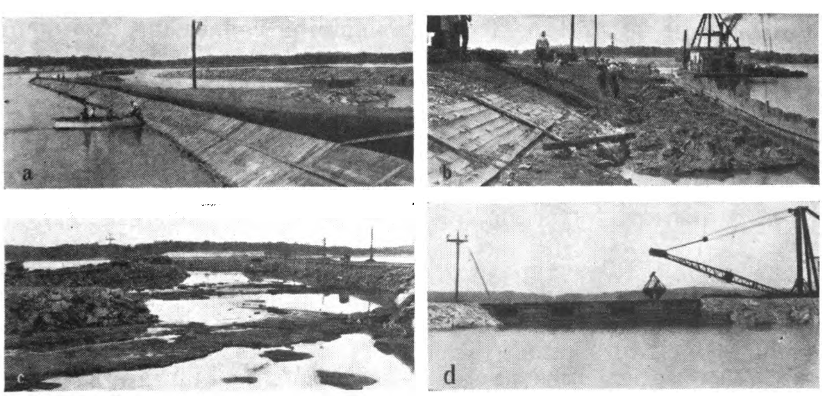

FIGURE 65.—*Construction of cofferdam No. 2 (rock fill), (a) wooden mat to protect clay fill from wave action, (b) placing clay blanket, (c) cofferdam unwatered, (d) emergency spillway.*

Timber cribs were placed at the north end of the upstream arm to support emergency flood gates. Additional cribs projected from the north end of the cofferdam to serve as a breakwater and prevent scour of the clay blanket. These additional cribs were also used as a tie-in for cofferdam No. 3.

The first attempt at unwatering was made on June 4, 1934, but leakage was so great that the pumps were stopped and the additional clay fill was added. On June 28, the cofferdam was unwatered in 26 hours, using one 6-inch and three 12-inch centrifugal pumps.

Constant replenishment of the earth blanket was required during the period the cofferdam was unwatered. Earth was obtained from borrow pits on the south shore and transported to the site by barge. Mud was secured from the north shore and loaded onto barges by the dipper dredge.

The cofferdam was fairly tight until excavation was started, after which there were some bad leaks through solution cavities in the underlying bedrock. An attempt was made to grout these seams, but this did not reduce the inflow sufficiently. Further treatment, using sacks to block the openings plus additional grouting, made it possible to keep the cofferdam dry by a reasonable amount of pumping. Sumps were excavated in the rock to catch this inflow and rows of sand bags used to direct water to the sumps.

During the latter part of cofferdam use, 20 pumps were used almost continuously to keep the water level low enough to permit shovels and trucks to work. Six 12-inch, six 6-inch, and miscellaneous smaller pumps operating continuously were required in most cases for this work. A total pump time of 3,270 pump hours was recorded during the 233 days the cofferdam was in use.

Tables 10 and 11 give comparative cost and maintenance data for all five cofferdams.

Cofferdams Nos. 3, 4, and 5.

Ohio River type cofferdams similar to the first cofferdam were used for the three remaining river-diversion stages. This type cofferdam was adopted following the experience gained by the use of the two types and after studies had been made regarding the feasibility of the use of either type for the remaining cofferdams. Although leakage was excessive in cofferdam No. 2, it was due largely to the foundation rock conditions. Studies comparing the rock fill and the Ohio River types revealed the following conclusions:

1. Although the rock-fill cofferdam was apparently cheaper than the Ohio River type, any possible saving in later cofferdams was eliminated because of the increased height required and because of the large increase in the amount of material that had to be removed later.
2. The Ohio River type was considered more desirable from the standpoint of speed of construction and removal.
3. The time element was especially important in the construction of cofferdam No. 3. Excavation started in No. 2 in August 1934, and it would have been necessary to start cofferdam No. 3 at once to handle economically the rock from the excavation if the rock-fill type were to be used. Due to foundation conditions that developed in cofferdam No. 2, the definite amount of excavation was not known.

If it increased, construction would have been delayed and the spring floods would have done considerable damage in cofferdam No. 2, since the restriction in the river caused by cofferdam No. 3 would have resulted in overtopping of cofferdam No. 2.

4. The use of the Ohio River type permitted the start of construction of the third cofferdam to be delayed a month, and at the end of that time more definite knowledge of the excavation progress in cofferdam No. 2 was available and gave more assurance that it would be safe to construct No. 3 at that time. In the event of adverse information, construction of No. 3 could have been postponed until after the spring floods.

Cofferdam No. 3.—An area 1,147 feet long and 236 feet wide was enclosed by this cofferdam, inside which the second 25 sections of the spillway and apron were constructed. The top of the upstream arm was at elevation 515.3, and the top of the downstream arm was at elevation 509.3. After the cofferdam had been unwatered and excavation well started, several bad fissure leaks developed which caused suspension of operations for a short time and resulted in heavy pumping demands during most of the period of cofferdam use.

Cofferdam No. 4.—The fourth cofferdam was 1,099 feet long. It was 240 feet wide at the south end where it enclosed the north 10 sections of the spillway, beyond which the width was reduced to 166 feet to enclose the south portion of the north nonoverflow section. The top of the upstream arm was at elevation 520.3. Later, a 5-foot box crib was added to increase the height to elevation 525.3. The downstream arm was built to elevation 509.3.

TABLE 10.—*Summary of cofferdam unit costs*

[Cofferdams No. 1 and No. 2 are comparable; Nos. 3, 4, and 5 are not comparable to No. 1 or No. 2 because of increase in height]

Unit	Cofferdam No. 1	Cofferdam No. 2	Cofferdam No. 3	Cofferdam No. 4	Cofferdam No. 5
Wall length.......linear ft..	3,607	[1] 2,678	2,511	2,454	2,025
Face.......sq. ft..	64,241	36,225	42,379	50,716	44,356
Wall.......cu. yd..	46,999	27,949	31,433	42,936	42,250
Maintenance.......days..	628	233	275	121	278
Unit costs per linear foot of wall					
Construction.......	$24.27	$16.40	$22.32	$44.08	$42.96
Unwatering.......	.47	.52	.69	.60	.91
Maintaining cofferdams.......	.56	5.00	2.08	.74	.87
Maintaining water level.......	18.04	17.96	28.59	8.40	16.00
Removal of cofferdam [2].......	5.11	8.05	6.01	1.26	3.18
Total unit direct costs.......	48.45	47.93	59.69	55.08	63.92

[1] For purposes of comparing the rock fill type with the Ohio River type, the square foot of face of the former is based on the vertical height. Also the cubic content is obtained from the vertical height times a width equivalent to the Ohio River type.

[2] Unit costs for removal are based on the same quantities as used for construction. Cofferdam No. 5 includes the construction of the breakwater upstream from the structure.

Cofferdam No. 5.—The fifth cofferdam, 1,012 feet long and 166 feet wide, enclosed the remainder of the north nonoverflow section and the north trashway structure to the navigation lock. Its height was the same as for cofferdam No. 4. It was necessary to construct a rock breakwater upstream from the cofferdam in order to retard the current through this restricted passage sufficiently to permit setting the cribs. When the portion of the spillway section constructed inside cofferdam No. 4 was completed to a sufficient height to permit the flooding of the cofferdam, a cut-off wall was constructed at the north end of the spillway, and the north end of cofferdam No. 4 was eventually opened into cofferdam No. 5 for the purpose of water maintenance.

Pumping in cofferdam No. 4 began June 24, 1935, but only continued as an independent operation for a few months. Pumping in cofferdam No. 5 started September 25, 1935; and from that time on until the spring of 1936 pumping was done to service both cofferdams. Tables 10 and 11 show cofferdam cost as well as unwatering and maintenance data.

Cofferdam costs.

In comparing the costs shown in table 10, consideration should be given to the varying conditions under which the different cofferdams were built. For cofferdam No. 1 the channel was wide and there was practically no current. Fissure leaks gave but little trouble. As the succeeding cofferdams advanced, water velocities increased with each additional constriction of the channel. This necessitated the use of higher and wider cofferdams and materially increased the difficulties of construction. In cofferdams Nos. 2 and 3, fissures in the foundation rock delayed unwatering and increased the cost both of pumping and maintenance.

TABLE 11.—*Summary of cofferdam maintenance*

Cofferdam	Date of construction	Time channel open days	Total channel open feet	River face per cofferdam square feet	Pumping							
					Unwatering				Maintenance			
					Dates	Elapsed time hrs.	Number of pumps	Size of pumps inches	Dates	Time in service days	Size of pumps inches	Total days
No. 1	Nov. 27, 1933 to Jan. 16, 1934.	108	4,497	63,167	Jan. 16, 1934 to Jan. 18, 1934.	53	3 2	12 6	Jan. 18, 1934 to Oct. 11, 1935.	628	12 6 4 2	156 1,200 780 120
No. 2	Feb. 28, 1934 to June 4, 1934.	164	3,259	36,225	June 4, 1934, to June 29, 1934.	85	3 2	12 6	June 29, 1934, to Feb. 16, 1935.	233	12 6 4 2	930 900 720 720
No. 3	Oct. 22, 1934, to Nov. 15, 1934.	221	2,112	42,379	Nov. 15, 1934, to Nov. 17, 1934.	37	2	12	Nov. 17, 1934, to Aug. 19, 1935.	275	12 10 6 4	1,050 150 690 360
No. 4	May 15, 1935, to June 24, 1935.	107	1,012	50,716	June 22, 1935, to June 24, 1935.	31	2	12	June 24, 1935, to Oct. 23, 1935.	121	12 6	3 520
No. 5	Aug. 28, 1935, to Sept. 24, 1935.	....	......	44,356	Sept. 24, 1935, to Sept. 25, 1935.	30	2	12	Sept. 25, 1935, to June 29, 1936.	278	12 8 6 4	4 4 450 90

CONSTRUCTION PLANT

The selection of the type of construction plant best suited for the construction of Wheeler Dam involved the consideration of a number of factors. The location of the site at the head of Wilson Lake assured an almost constant depth of water, the maximum rise in river level of 4 to 4½ feet occurring only in the event of extreme floods. The 6,000-foot width of the river presented the problem of access to the various parts of the job. Railroad terminal and transfer facilities were already available at Wilson Dam, 15.5 miles downstream. No suitable quarry site was available near Wheeler for the manufacture of aggregates, but there was a sufficient supply of river gravel for this purpose below Wilson Dam, within reasonable hauling distance of the site. River gravel had the further advantage of being of proven quality since concrete structures had been built of this material for a number of years.

In view of the existing conditions, a construction plant utilizing floating equipment wherever practical was selected. Contracts were let for dredging, sizing, and delivering the river gravel to the site. With the possible exception of four floating mixing plants, the Wheeler construction plant embodied no unusual features. Most items of equipment were of standard design and manufacture.

Two all-electric shovels equipped with 2½-yard dippers were used for the greater part of the excavation work, revolving cranes, small shovels, and guy derricks accounting for only about 10 percent of this work. Pneumatic wagon drills were utilized for all blast hole and line drilling in the foundation area; holes for the little secondary blasting required were drilled with jackhammers. Ten-yard Boulder Dam type dump trucks transported waste rock from the place of excavation to spoil dumps immediately upstream from the cofferdams. Considerable material was also deposited below the dam along the south shore of the river, forming a fill on which shops, warehouses, permanent tracks, and the switchyard were built. A tractor equipped with a bulldozer kept the spoil areas level.

Foundations were treated by pumping cement grout under pressure into the rock through wagon and core drill holes. Three 5½-inch shot core drills were used for exploration work and in addition a 36-inch shot drill was used for drilling observation holes. Mechanically agitated mixers and reciprocating group pumps were used for grouting.

All concrete was mixed in barge-mounted floating mixing plants which were moved along the cofferdams by towboats as required. Each plant consisted of a 2-cubic-yard mixer, necessary weighing and batching equipment, and cement and aggregate bins. A crane transferred aggregates from supply barges moored alongside the plant to the storage bins, and bulk cement was pumped from the supply barges to the storage silos.

Mixed concrete was transferred in 2-cubic-yard concrete buckets by whirley cranes from the mixer plant to the point of placement in the forms. Where necessary these cranes were mounted on gantry frames which operated on tracks inside the cofferdam areas. In the powerhouse where placement areas were inaccessible to the cranes, some concrete was placed by a pumpcrete machine and some concrete was transferred by derrick boats and guy derricks. The concrete was vibrated by electric vibrators. In addition to placing concrete, the revolving cranes handled heavy materials, forms, excavation, and clean-up.

All general plant buildings were below the dam on the south shore of the river. Most of the warehouses, shops, and also the locomotive-crane tracks were built on the fill formed from waste rock. The main office buildings were above the flood plain on a bluff overlooking the dam site.

A considerable part of the plant consisted of marine equipment, such as towboats, barges, launches, and a derrick boat. During the early construction period, marine equipment was rented from the United States Army Engineers, and some items of equipment were rented as required to supplement the TVA-owned equipment during the entire duration of the job.

Tables 12 and 13 show, respectively, the major construction plant equipment used throughout the entire job, and the marine equipment rented from the United States Army Engineers. Figure 66 shows a schedule of the actual use of the various items making up the construction plant.

A decided saving was effected in the net cost of the construction plant for the dam through planning of continued equipment use on other Authority projects combined with a study of market conditions

ITEM	NO.	VALUE
SUBSTATION BUILDING	1	$ 6,852
ELECTRICAL SERVICE LINES		26,973
PERSONNEL OFFICE (Structures only)	1	2,528
FIRE TRUCKS	2	464
WATER CURING PLANT		41,047
CONSTRUCTION POWER LINES		101,929
TRANSFORMERS	117	20,052
COMPRESSOR PLANT AND LINES		66,021
STATIONARY COMPRESSOR	3	7,300
COMPRESSOR ACCESSORIES	15	2,516
GENERAL OFFICE (Structures only)	1	14,250
ENGINEERS OFFICE (Structures only)	1	
TIME OFFICE	2	5,688
WAREHOUSE & STORAGE YARDS	2	8,841
CONCRETE LAB BUILDING & EQUIPMENT		8,334
PORTABLE OFFICES		3,346
DRILL SHARPENER SHOP		6,120
DRILL SHARPENERS & FURNACES	7	5,194
NEW MACHINE SHOPS	2	16,799
CARPENTER SHOP & EQUIPMENT		16,300
GARAGE - SHOP & EQUIPMENT		8,704
TRUCKS		
MISCELLANEOUS ½ TON	3	2,429
STAKE 2½ & 3 TON	2	3,325
DUMP 10 CU YD	9	72,238
DUMP 4 CU YD	2	7,733
TRACTOR - CATERPILLAR 70 WITH BULLDOZER	1	6,153
FLAT CARS	4	1,700
CORE DRILLS	4	9,293
DRILLS		
JACKHAMMERS & PAVEMENT BREAKERS	45	8,673
CHIPPING HAMMERS	33	1,545
WAGON DRILLS	18	18,600
PORTABLE COMPRESSORS 240 CFM	2	4,523
HOIST I-R	4	1,780
SHOVELS		
MARION 4101 2½ CU YD	1	39,557
BUCYRUS - ERIE 75-B 2½ CU YD	1	37,930
MARION 450 1½ CU YD	1	9,587
CRANE - OSGOOD CRAWLER	1	8,179
CRANES		
CLYDE WHIRLEY (5 with gantries)	6	233,055
LOCOMOTIVE	2	28,994
HOISTS		
VARIOUS	6	3,827
DERRICK 20 T GUY	1	11,380
DERRICK 50 T GUY	1	26,637
PUMPS (6" & over: 23; less than 6": 46)	69	23,313
TOWBOATS		
JOHN M WARNER 88 TON GROSS		34,301
JOE WHEELER 50'X12'-6"X4'-2"		22,828
CLARA BOND 110'X26'X4'		14,021
TVA 129 75HP 30'X9'X3'-6"		3,679
MILHUT 85 HP DIESEL		2,231
BLUEWATER 50 HP 34'X8'-3" (Motor only)		1,118
DAISY 40 HP		RENTED
DERRICK BOAT - STEAM 75' BOOM	1	33,959
BARGES	14	71,582
LAUNCHES	4	3,058
PUMPCRETE & ACCESSORIES	1	28,423
PORTABLE MIXERS	4	2,743
VIBRATORS	38	18,129
GROUT MACHINES 2 Pumps: 3 mixers: 1 mach.		1,809
FLOATING MIXER PLANT & ACCESSORIES	4	242,238
MISCELLANEOUS EQUIPMENT		39,604
MISCELLANEOUS PLANT		177,745
GRAND TOTAL PLANT & EQUIPMENT		$1,627,177

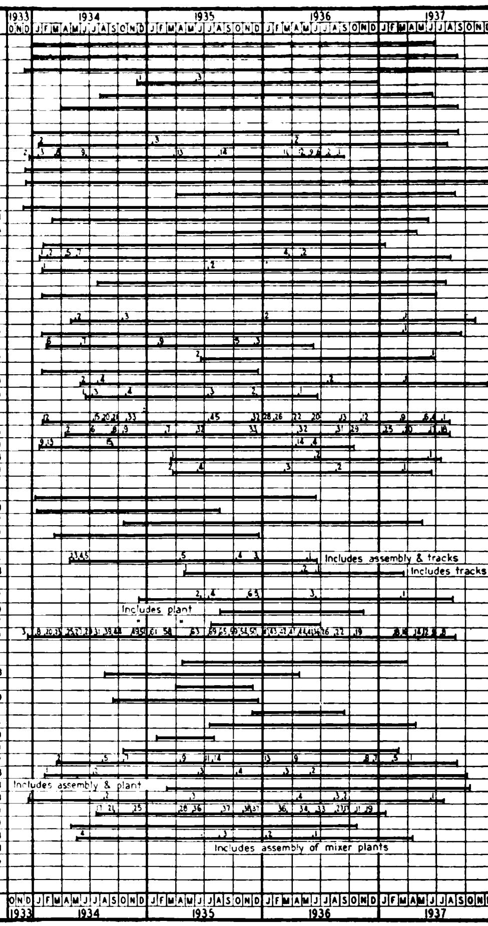

FIGURE 66.—*Schedule of construction plant and equipment.*

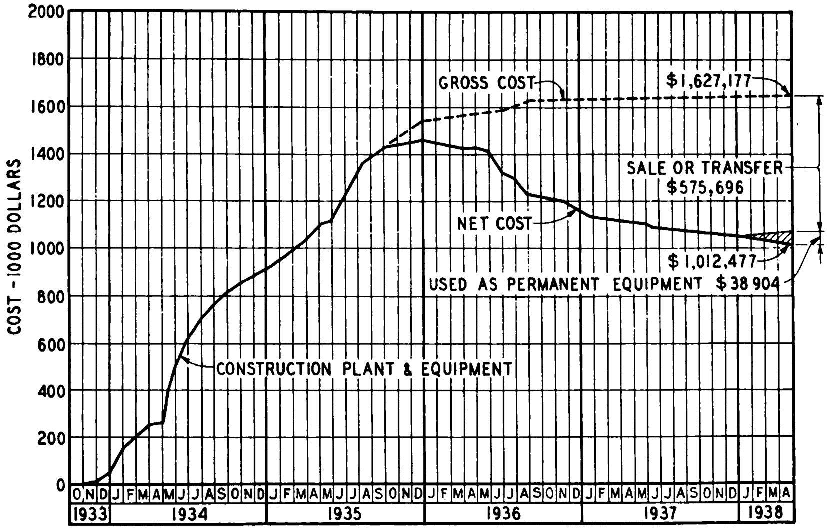

FIGURE 67.—*Plant and equipment costs—Gross and net*

for possible sale of released equipment. Transferred equipment was appraised on a basis of physical condition and use value. Equipment sales were consummated when a study of physical condition and possible use showed a bid price greater than the value of the items to the Authority. Figure 67 gives the gross cost of the construction plant for the dam and includes those items which had no salvage, sale, or transfer value.

TABLE 12.—*Construction plant equipment*

Unwatering equipment:	
1 pump, horizontal, electric	14-inch.
8 pumps, horizontal, electric	12-inch.
2 pumps, horizontal, electric	10-inch.
2 pumps, vertical, electric	10-inch.
12 pumps, horizontal, electric	6-inch.
5 pumps, centrifugal, electric	4-inch.
4 pumps, centrifugal, electric	3-inch.
3 pumps, centrifugal, gas	3-inch.
16 pumps, rotary sump, air	2½-inch.
10 miscellaneous small pumps.	
Excavating equipment:	
2 shovels, all electric, revolving	2½ cubic-yard.
1 shovel, gas electric, revolving	1¼ cubic-yard.
1 shovel, gas, revolving	1 cubic-yard.
9 trucks, Boulder Dam type	10 cubic-yard.
2 trucks, dump	4 cubic-yard.
Concrete mixers:	
4 floating plants complete with:	
Davis tilting mixers	2 cubic-yard.
Steel barges	90 by 40 by 7 feet.
American revolver whirley	75-foot boom.
Cement silo	500 barrels.
Batcher—Blaw-Knox	2 cubic-yard.
Water tank	100-gallon.
Also small mixers for miscellaneous use.	
7 clamshells—rehandling	2 cubic-yard.

Table 12.—*Construction plant equipment*—Continued

Cement pumps:	
4 Fuller-Kinyon portable	4-inch.
Compressors:	
3 Sullivan stationary WN-4	2,448 cubic feet per minute.
2 Worthington portables	240 cubic feet per minute.
1 Air Receiver, horizontal	6-foot 3-inch diameter by 25-foot 4-inch.
2 Air Receivers, horizontal	6-foot diameter by 24-foot.
Concrete placing equipment:	
6 revolving cranes—Clyde	95-foot boom.
1 Rex dual pumpcrete	60 cubic yards per hour.
4 vibrators	Puddler type.
26 vibrators	Spade type.
8 vibrators	Flexible shaft.
2 frequency converters	15-kilowatt.
7 concrete buckets—Union	60 cubic feet.
1 concrete bucket—Steubner	1 cubic yard.
9 concrete buckets—Insley	2 cubic yards.
5 concrete buckets—Blaw-Knox	62 cubic feet.
1 concrete bucket—Dravo	2 cubic yards.
18 concrete carts	6 cubic feet.
Drill shop equipment:	
3 drill sharpeners.	
3 oil furnaces.	
1 shank grinder.	
Drilling equipment:	
16 wagon drills.	
30 jackhammers.	
15 paving breakers.	
Marine equipment:	
1 launch, *Shoal*	19 feet by 6 feet 2½ inches, 45-horsepower.
1 launch, *Nance*	21 feet by 6½ feet, 70-horsepower.
1 launch, *Spring Creek*	24 feet by 7 feet.
1 towboat *Duck*, wood	30 feet by 9 feet by 3½ feet, 75-horsepower.
1 towboat *Joe Wheeler*, steel	50 feet by 11½ feet by 4 feet 2 inches, 180-horsepower.
1 towboat *Elk*, wood	110 feet by 26 feet by 4 feet 2 inches, steam.
1 towboat *Blue Water*, wood	30 feet by 8 feet by 2½ feet, 78-horsepower.
1 towboat, *Hiwassee*, steel	111 feet 7 inches by 26 feet 8 inches, by 4 feet, 400-horsepower.
1 towboat *Paint Rock*, steel	42 feet 5 inches, by 10 feet 3 inches by 4 feet 2 inches, 85-horsepower.
2 barges, wood	16 feet by 32 feet by 3 feet 2 inches.
3 barges, steel	100 feet by 26 feet by 6½ feet.
2 barges, steel	120 feet by 28 feet by 7 feet.
2 barges, steel	100 feet by 25 feet by 5 feet 7¼ inches.
1 barge, wood	10 feet by 26 feet 7 inches by 2½ feet.
2 barges, steel	70 feet by 20 feet by 4 feet.
2 barges, steel	110 feet by 26 feet by 6½ feet.
1 derrick boat—steel barge—22-ton American crane	62 feet by 34 feet by 5 feet.
(See table 13 for rented marine equipment.)	
Foundation exploration and grouting equipment:	
4 grout mixers.	
3 grout pumps, air	7 inches by 2½ inches by 10 inches.
3 core drills, shot, gas	5½ inches.
1 core drill, shot, electric	36 inches.
Concrete laboratory equipment:	
1 concrete compression machine	300,000 pounds.
2 testing sieve shakers	¼-horsepower motor.
3 balance scales	20 kilogram.
1 platform scale	500-pound.
2 torsion balance scales	10-pound.
1 mixer	7 cubic feet.

TABLE 12.—*Construction plant equipment*—Continued

Miscellaneous equipment:	
7 electric welders	20-horsepower, 300 amperes.
2 hoists, 2-drum electric	60-horsepower.
1 light plant	800-watt.
1 bar bender—electric	15-horsepower, 1¼-inch bar.
2 conduit benders, hand	¾ inch to 2 inches.
4 flat cars, standard gage	60,000-pound.
1 locomotive crane, 60-foot boom (short use)	12-ton.
1 locomotive crane, 60-foot boom	25-ton.
2 bar cutters, hand	6-inch.
1 guy derrick, 60-foot, 100-horsepower, 2-drum	50-ton.
1 guy derrick, 50-foot, 80-horsepower, 2-drum	20-ton.
3 portable concrete mixers (gas)	7, 10, and 14 cubic feet.
2 trucks, stake body	3-ton.
2 trucks, stake body	1½-ton.
1 fuel tank, horizontal	1000 gallons.
1 fuel tank, horizontal	2000 gallons.
1 fuel tank, horizontal	3000 gallons.
Carpenter shop equipment:	
1 woodworking planer.	
1 moulder, 4 side	8 inches by 4 inches.
2 woodworking machines	5-horsepower, 16 inches by 20 inches.
1 sander, portable.	
1 surfacer.	
1 jointer	5-horsepower, 16 inches.
1 saw, rip and cut-off	5-horsepower.
1 rip saw	10-horsepower.
1 rip saw	belt drive, 6 inches.
1 saw—swing cut-off	5-horsepower.
1 timber cutting saw—DeWalt	10-horsepower.
2 power saws, portable	1-horsepower, 8 inches.
3 saws, comb, rip, and cut-off	16 inches.
1 saw, band	36 inches.
1 saw mill	20 horsepower, 20 feet.
5 wood borers	1 to 1½ inches.
Machine shop equipment:	
1 metal lathe	8-foot bed, 16-inch swing.
1 metal lathe	9-foot bed, 18-inch swing.
1 metal lathe	10-foot bed, 18-inch swing.
1 metal lathe	12-foot bed, 24-inch swing.
2 power hacksaws	6 inches by 6 inches.
1 shaper	16 inches.
1 electric drill, portable	
1 drill press	20 inches.
1 drill press	24 inches.
1 steam hammer	800 pounds.
1 power hammer, electric	100 pounds.
2 chain hoists	5-ton.
2 chain hoists	12-ton.
4 air hoists	1-ton.
1 grinder, 2-wheel	belt drive.
1 grinder, knife	belt drive.
2 grinders, portable	air.
1 grinder, surface	air.
1 grinder, bench	electric, 6 inches.
1 grinder, tool	belt, 12 inches.
2 bolt threaders	1½ inches to 2 inches.
1 bolt threader	½ inch to 1½ inches.
1 pipe threader	1 inch to 4 inches.
2 pipe threaders	1 inch to 2 inches.
1 pipe threader	4 inches to 12 inches.
1 pipe threader	1 inch to 6 inches.

TABLE 13.—*Rented marine equipment* [1]

	Rental rate per day
2 dredges—dipper—*Virginia* and *Kentucky*	$121 and $130.
4 derrick boats—Nos. 4, 6, 7, and *Kwasind*	$40 to $90.
2 derrick boats—wood—Nos 13 and 14	[2] $30 each.
1 towboat—*Tennessee;* Diesel	$155.
1 towboat—*Colbert;* steam	$123.50.
1 towboat—*Ocoee;* steam	[2] $38.
1 launch—*Drifter;* gasoline	$29.
1 quarterboat—No. 21	$4 30.
1 tender.	
1 coal barge.	
1 supply barge.	
4 barges; steel—110 feet by 24 feet by 5 feet	$3.50 to $5 70.
4 barges; steel—100 feet by 20 feet by 5 feet	$4.40 to $6 80.
4 barges; wood—Nos. 92, 93, 98, and 4	$1.60 to $3.30.

[1] Rented from U. S. Army Engineers as required.

[2] Equipment rental only—operating labor or material not included. Rates shown are approximate, including equipment rental, operating labor, and materials except quarterboat and barges, which are equipment-rental rates only, as no labor or materials are required on these items.

The *Hiwassee,* the *Elk,* and the *Paint Rock* were all rented at first, but later purchased. (See pp. 195 to 197.)

FIELD ENGINEERING

Preliminary field survey work was completed by the United States Army Engineers in connection with the location of the navigation lock. In the construction of the dam it was necessary for the Authority to establish a triangulation system that tied in with the points used for the lock and to establish bench marks for the dam construction, the elevations of which were referenced to the lock datum. Elevations were based on the 1912 General Adjustment of the United States Coast and Geodetic Survey.[11] Three concrete monuments, two on the north bank and one on the south bank, were established by the United States Army Engineers along the axis of the dam. In addition, on the north bank an accurately chained base line about 2,300 feet long was extended west from the main axis at an angle of about 85 degrees for triangulation work. Distances along the axis were determined where practicable by chaining. Where chaining was not practicable, distances were determined by ordinary triangulation methods using the base line on the north bank. In many cases both methods were used, one to check the other.

The necessary base lines and bench marks for detail construction work were established in the various cofferdams as the work progressed. In all but the intake section of the dam a main control line marked with 12-inch square concrete posts located at 100-foot intervals was established a few feet upstream from the face of the dam. Secondary lines were laid out from the main upstream base line where necessary. Of these the principal one was a downstream base line along the center of the downstream gantry track extending from the north end of cofferdam No. 1 to the lock wall. This line was required since it was impossible to locate points below the dam from the upstream base line after concrete had attained any height. In the powerhouse area concrete monuments placed on the transverse center line of each unit 15 feet upstream from the face of the dam were used for reference in laying out all powerhouse construction

[11] See page 221.

work. A second base line in this area was established near the downstream wall of the cofferdam, and a third line on the north-south center line of the units.

Field engineering work included many other activities. Cross sections were taken of the original river bed at maximum intervals of 10 feet before and after excavation to determine excavation quantities. All embedded pipe such as grout pipes and miscellaneous material not specifically dimensioned on the plans were located and field records made. Layout and inspection service was furnished continuously for foundation treatment and exploration, all necessary records being made by the field parties. Engineering service involving layout work and inspection was furnished for all construction operations, such as construction of cofferdams and installation of construction plant equipment, and for the installation of all permanent equipment, such as spillway and trashway gates, powerhouse intake and draft tube gates, and turbines and generators. Topographic maps of the reservation and nearby grounds were made, and all underground piping, conduits, and sewers, as well as structures above the ground, were located. Installation of the powerhouse piping system required constant inspection and involved the keeping of a large number of records. In the control building inspection was required of all architectural details such as floor finishing, plaster, terrazzo and other trim work, installation of sash and doors, and miscellaneous metal trim work. At the completion of the job permanent reference points were established and bench marks set in various locations for future use.

The field engineering organization was headed by a field engineer who was assisted by two shift field engineers and one chief surveyor. A six-man crew, supervised by a shift inspector, was assigned to each of the mixing plants and inspected the mixing and placing of concrete for each shift. In addition, six general construction inspectors were used to inspect complicated pieces of work, installation of embedded parts, structural steel, equipment installation, and bridge erection. Three gravel inspectors each working 8 hours per day were stationed at the gravel dredge to inspect the concrete aggregates. Inspection of grouting work was handled by three grout inspectors, each working 8 hours per day. Core drilling and rock excavation inspection work was done by three engineer inspectors. From three to six layout parties were used as required for the construction layout work.

FOUNDATION EXCAVATION

Foundation excavation extended horizontally for a distance of 6,139 feet between the lock wall and the south abutment. Approximately 548,000 cubic yards of rock were removed. Of this, about half came from within cofferdam No. 1. The greatest depth of excavation was 53 feet. The excavation was somewhat shallower under the dam, averaging 16 feet in the spillway and 6 to 10 feet in the nonoverflow sections. The width of cut for the spillway was 125 feet and for the nonoverflow sections 55 feet.

Foundation drilling.

Foundation drilling extended over a period of 21 months. Wagon drills completed 97 percent of the rock drilling; jack-hammers accounted for the rest. Four wagon drills started operation on January 27, 1934, more drills being added from time to time until 16 machines were in use. All equipment was purchased new at the start of the job.

The neat or dimension lines of the excavation were in all cases line drilled and in a few instances "broached" so as to preserve the foundation outline. This procedure was necessary on account of the horizontal rock stratification and to prevent damage to the surrounding rock as the overbreak in blasting, however light the charge, was considerable—usually amounting to about 2½ times the depth of the shot.

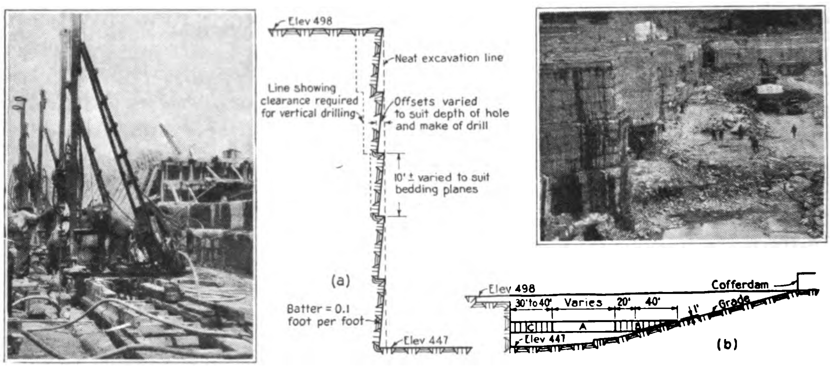

Figure 68.—*Drilling and blasting methods, (a) line drilling, (b) blasting.*

Line drilling consisted of 2¼-inch wagon drill holes on 4½- to 6-inch centers drilled to a depth of from 4 to 12 feet. The drills worked in groups of from three to five drills each. They operated on sectional steel tracks supported by portable timber runways. For this work the wheels were removed and a single channel bolted to the bottom of the frame near the front of the mounting. This channel fitted over the front rail and functioned as a guide to assure a straight line of holes. The rear of the mounting rested directly on the second rail, bolted shims under the front channel permitting the mast to tilt backward to give the whole a slight dip from the vertical. A depth greater than 15 feet proved impractical because of slack in the drill guides. This slack caused the bottom of the holes to be out of line.

Where it was necessary to excavate more than one lift, the line drilling was sloped 0.1 foot per foot which resulted in practically a vertical face to compensate for the 6- to 12-inch clearance required by the drill when set up against the wall formed in the preceding lift. The results obtained by this method are shown by figure 68 (a).

In the powerhouse excavation the rock was removed in lifts which varied from 5 to 12 feet in depth. A typical lift lettered "A," "B," and "C" as shown in figures 68 (b) was taken out as follows: that marked "A" was removed first and in the most economical method from the viewpoint of speed and cost, because it was far enough from the final faces of the cut that blasting would not endanger their solidity. The portions marked "B" and "C" were drilled and blasted out in strips, each strip shown representing one line of drill holes.

In blasting the rock away from the line-drilled faces, no holes were loaded nearer than 30 feet from this line for the first shot. This precaution was taken because it was not considered safe to allow these large blasts any closer. The break resulting from these shots usually extended about halfway, and in some cases entirely to the line. It was usually necessary, however, to shoot an additional single line of holes between the blasted muck pile and the line-drilled face to complete this operation. This 30-foot dimension was decreased to a minimum of 12 feet on minor shots or for small depths.

In the excavation for the spillway and nonoverflow sections of the dam, a procedure similar to that followed in the powerhouse area was used. Considerable difficulty was experienced, however, in obtaining a sloping excavation under the spillway apron section, owing to the horizontal stratification of the rock. In this area the rock tended to loosen up as each layer of the foundation was excavated. The most satisfactory solution to this problem was that of line drilling additional holes parallel to the axis of the dam to produce a stepped surface area rather than the sloping surface originally planned. Care was taken at all times to prevent blasting too close to the line-drilled surfaces, but in the nonoverflow section in particular it was not practical to hold to the 30-foot limit that had been adhered to in the wider excavation areas. In the narrow sections of the excavation, this plan would have produced a blasted area too narrow to accommodate the large shovels. Therefore, blasts were permitted at a minimum distance from the line drilling equivalent to 2½ times the depth of the hole. This resulted in a safe method of excavation, and at the same time provided sufficient width for the operation of the heavy excavating equipment.

Drill performance.—While blast holes and line drilling constituted the major portion of the drilling program, 62,357 feet of high- and low-pressure grout holes were also drilled. High-pressure grout holes varied in depth from 20 to 60 feet, while the depth of low-pressure holes seldom exceeded 20 feet. The 60-foot holes were started with a bit gage of 3½ inches. They were not satisfactory and only a few were completed; core drills were used for the remainder. Channeling, an operation consisting of line drilling a row of holes on about 4½-inch centers and broaching, was done only in special cases in sumps, keyways, and in the pier nose foundations where parallel sides were desired and where the volume of rock to be removed was small.

Drill operations were carried on during four 5½-hour shifts daily for the first year and four 6-hour shifts for the remainder of the time. The normal drill crew consisted of one operator at $0.75 and one helper at $0.60 per hour. The same crew was required for either drilling or broaching.

The total footage drilled by all drills was:

	Linear feet
Blast holes	720, 100
Line drilling	858, 027
Low-pressure grout holes	29, 335
High-pressure grout holes	33, 022
Dowel holes	1, 163
Total drilling	1, 641, 647
Channeling, square feet (wall area)	30, 463

Rates of progress for each of the various types of drilling amounted to:

	Linear feet per net hour
Grout holes	11.89
Blast holes	25.18
Line drilling	25.18
	Square feet per net hour
Broaching (square feet actual web area between line drill holes)	4.65

Operating costs were charged directly to each feature prior to August 1, 1934, and are not available for that period. After that time, however, detailed costs of all drilling were kept, and the totals for the period from August 1, 1934, to November 1935 are shown below. Unit field costs for the various types of holes drilled were:

	Per foot
Grout holes	$0.545
Blast holes	0.258
Line drilling	0.278
	Per square foot
Broaching	1.491

The total track cost charged to line drilling and broaching amounted to $0.02 per linear foot of hole. It includes all materials, maintenance, and moving charges for the line drilling track. The unit cost shown above includes such items as operating labor, compressed air, miscellaneous supplies and expense, repairs to equipment, small tools, air-line maintenance, and depreciation on the drilling equipment. Based on the condition of the equipment when work was completed, depreciation was estimated to be 52 percent of the first cost of the wagon drills.

Drill sharpening.—All wagon drill and jackhammer steel was sharpened in a central drill shop, on the south bank of the river near cofferdam No. 1 during the drilling operations in cofferdams Nos. 1 and 2. A fill was constructed near the north end of cofferdam No. 3 to which the drill shop was moved to serve the operations in cofferdams Nos. 3, 4, and 5. Drill steel was usually transported by truck, but towboats also performed this work, and during the early and later periods of the job hand labor assisted. As a result, transportation cost was comparatively high.

Drill sharpening during peak periods was carried on during four 5½-hour shifts daily. The crew for each shift consisted of three drill sharpeners, two helpers, and one laborer. Hourly wages for these men were $1.00, $0.60, and $0.45, respectively. Table 14 lists the amount of drill steel and broaching steel purchased for the entire job for use on wagon drills and jackhammers, and a record of the work done is shown for the period between January 25, 1934, and February 28, 1935, during which time only blast hole and line drilling was being done. Sharpening costs chargeable to the wagon-drill operation amount to $0.04 per linear foot of hole drilled. This included such items as operating labor, materials, repairs to the equipment, small tool charges, transportation, equipment installation, and depreciation on the building and sharpening equipment.

TABLE 14.—*Drill steel purchased for entire job*

Size	Use	Total	Total
		Pounds	*Feet*
¾ by 3 inches	Broaching	12, 593	1, 690
1½-inch hollow round	Wagon drills	27, 342	4, 971
1-inch hexagonal hollow	Jackhammers	37, 982	13, 712
1¼-inch hexagonal solid	Paving breakers	10, 035	2, 182

Drill sharpening shop record

(13-month period)

	1¼-inch round wagon drill steel		1-inch hexagonal jackhammer steel	
	Total	Per day	Total	Per day
Sharpened	147, 536	464	48, 913	185
New bits made	5, 076	16	1, 943	6
Shanks dressed	3, 725	12	1, 288	4
New shanks made	4, 835	15	2, 926	9
Total, all operations	161, 172		65, 070	

Air-compressing plant.—The compressed-air plant consisted of three Sullivan WN-4 twin compressors each with a 2,448-cubic-foot-per-minute rating. They were obtained from the TVA-owned Nitrate Plant No. 2 near Wilson Dam. Two units were placed at the south end of the dam to operate as the main plant, while the third was set up on the fill near the north end of cofferdam No. 3 for supplying the air requirements in cofferdams Nos. 3, 4, and 5. An 8-inch supply line carried air from the compressors to the work location. An air receiver outside the main compressor plant and two additional receivers on the cofferdams provided sufficient storage capacity.

Blasting.

Blast holes averaged 6 to 8 feet in depth and were drilled in a diamond pattern with spacing equal to about two-thirds of the depth. Normally these holes were drilled to about 6 inches above the nearest bedding plane that would give an 8- to 10-foot lift. This method usually left a smooth, nearly level surface on which to operate the shovels and drills.

No "springing" of any blast holes was permitted because of the tendency to damage the surrounding rock by keeping the powder too low in the hole. In most cases no limit was placed on the size of the shot allowed in blasting the rock, but the depth and general condition of the surrounding strata were invariably taken into consideration before permitting a very large shot. Normal charges varied between a minimum of 1,200 pounds and a maximum of 6,600 pounds of dynamite at any one time. The amount of powder required to blast a cubic yard of rock varied between one-half and 1 pound. The practice of not blasting before line drilling had been completed was followed, the line drilling invariably preceding any blasting operations. For the entire job a total of 12,318 pounds of

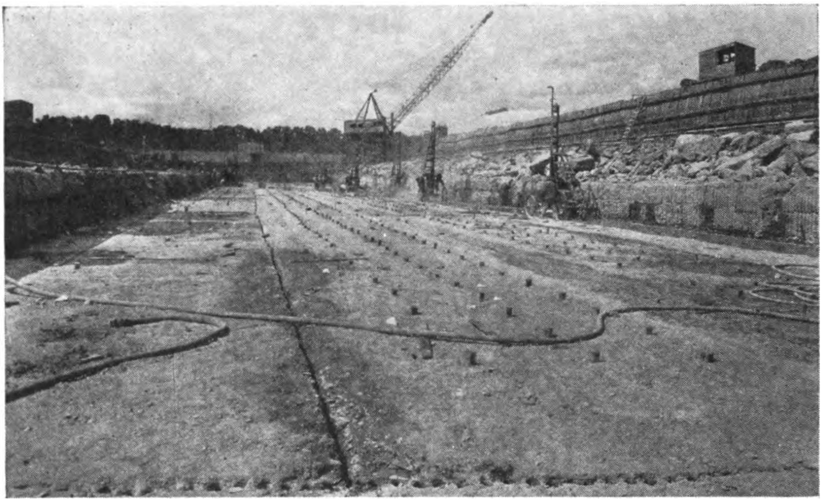

FIGURE 69.—*Completed line drilling and blast holes prior to shooting.*

60 percent dynamite and 291,846 pounds of 40 percent dynamite was used to blast the 548,000 cubic yards of rock—a ratio of 0.556 pound of dynamite per cubic yard of excavation. During the early stages of the job, 60 percent dynamite was used, but it was soon decided that the 40 percent material was more suitable for the existing conditions.

Excavation shovels.

The major loading equipment required for handling blasted rock consisted of two all-electric shovels. They were supplemented from time to time by small shovels, cranes, and guy derricks; but the material handled by these auxiliary methods was not more than 10 percent of the total. Both shovels were essentially the same in size and type, each being fully enclosed and equipped with crawler mountings, full revolving turntables, and 2½-cubic-yard dippers.

The rock was loaded directly by the shovels into 10-cubic-yard dump trucks. Performance of the two shovels compared favorably. Shovel A averaged 54.8 cubic yards per net operating hour, compared to 51.9 for shovel B; a maximum hourly production of 118 cubic yards was obtained with the former and 123 cubic yards with the latter. Delays amounted to 10.3 percent of gross operating time for shovel A and 14.2 percent for shovel B.

The standard crew per shift on each shovel was:

Crew:	*Hourly wage*
1 Operator	$1.50
1 Oiler	.60
1 Groundman (regular)	.45
1 Electrician helper	.60
Total	3.15

An electrician and a mechanic were present at all times in the area where the shovels were working and part of their time was prorated to shovel operation. Unit shovel operating costs were:

	Shovel A	Shovel B
Per net operating hour	$16.25	$15.11
Per ton of rock excavated	.1323	.1296
Per cubic yard of rock excavated	.2963	.2904

The above costs include such items as operating labor, repairs, maintenance, power charges, transportation, removal of ramps, and depreciation. Depreciation was computed at a rate of 2 percent per month for the period of use at Wheeler which amounted to 34 percent of the first cost of shovel B and 40 percent of the first cost of shovel A.

Excavation trucks.

During the first year of the excavation program the major truck equipment consisted of seven 10-cubic-yard dump trucks equipped with Boulder Dam type bodies and dual telescoping hoists. Each truck was equipped with tandem dual rear wheels and driving axles, providing a total of eight driving wheels, all with pneumatic tires. In January 1935, two new trucks were added.

One make of truck was equipped with standard and auxiliary transmissions providing eight speeds forward and two speeds reverse, while the other make had standard and auxiliary transmis-

FIGURE 70.—*Shovels for foundation excavation.*

sions which provided 12 speeds forward and three speeds reverse. The maximum load capacity of each was 30,000 pounds. The wheel base for one make truck was 185 inches, and for the other make truck was 175½ inches.

Figure 71 shows the cofferdam layout for the entire job, the position of the spoil dumps and the routes taken by the excavation trucks in order to dispose of their loads. Ramps were constructed in each cofferdam to permit entrance and exit of the trucks to the working area and in most cases they were very rough because, with the exception of one ramp in cofferdam No. 1, they were either the exposed river bed in the unwatered area or were constructed of waste rock. It was necessary to grade all truck roads in order to save tires and reduce hauling costs.

Most of the rock in the various spoil dumps was transported over previously dumped rock. Ramps in the powerhouse area had a maximum grade of approximately 10 percent, but those in the dam area were much steeper due to limited space. There, a maximum

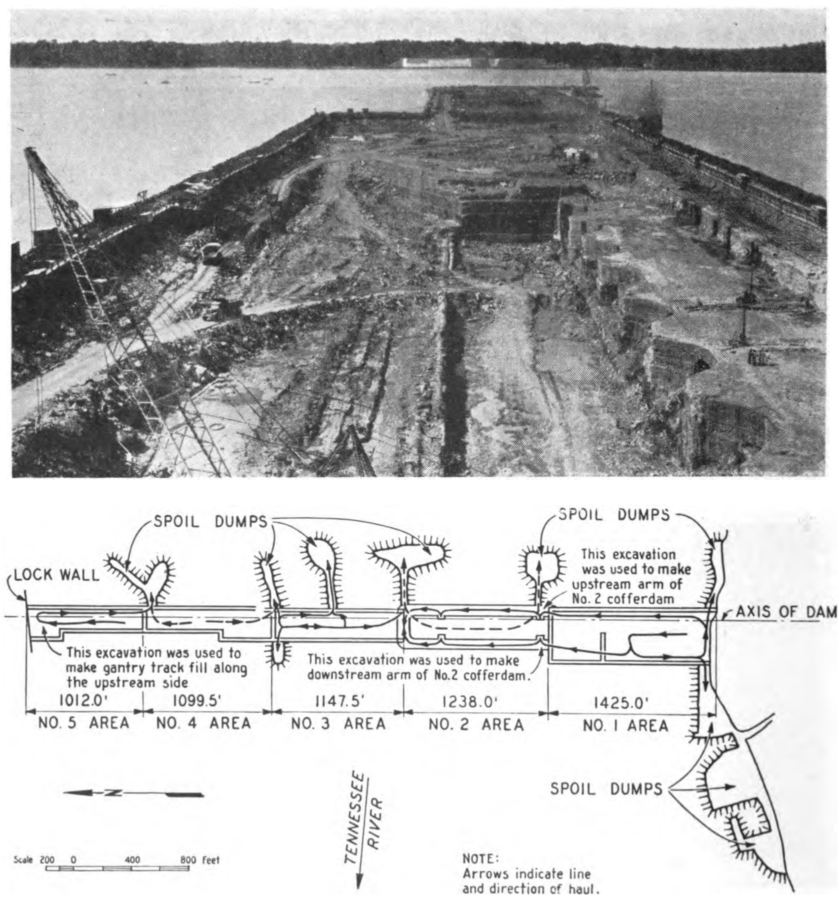

FIGURE 71.—*Cofferdam and spoil bank layouts and truck routes.*

grade of 20 percent was encountered with an average of 12 to 15 percent. In some places where turns could not be made, it was necessary for trucks to back halfway up the ramp in one direction and go forward the other half. On these steep ramps reduced loads only could be carried.

Some material from cofferdam No. 1 was dumped upstream and adjacent to the cofferdam along the south bank of the river. A considerable portion of the excavation was used to form the fill downstream from the dam along the south shore. Additional material from this cofferdam was used to construct the rock fill cofferdam No. 2. In the remaining cofferdams the spoil was placed immediately upstream from the working area. Spoil dumps and ramps were kept level by a tractor equipped with a bulldozer. This tractor was also used to concentrate material for the shovels in the cofferdam area.

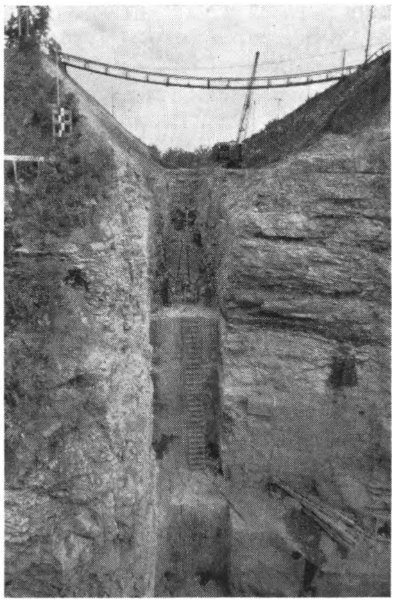

FIGURE 72.—*Final excavation in the south abutment area.*

During the excavation period from February 1934 through December 1935, all truck units operated approximately 42,000 net operating hours, of which 90 percent was on rock excavation and the remainder on handling miscellaneous materials, such as reinforcing steel, forms, and lumber. The haul from cofferdam No. 1 averaged 0.57 mile and from the other cofferdams 0.25 mile. Average production amounted to 2.63 loads per net operating hour or 149 cubic yards per net operating hour. For the powerhouse excavation the average load amounted to 13.6 tons as compared to 12.0 tons per load for the dam excavation. Total non-operating time amounted to approximately 53 percent of the possible working hours for all units. An analysis of this item shows that approximately 82 percent was due to idle time, 15 percent to mechanical delays, and 3 percent to miscellaneous delays.

Unit costs for truck operations were as follows:

	Number of cubic yards	Cost per cubic yard
Powerhouse excavation	249, 814	$0. 293
Dam excavation	273, 947	. 276
South abutment excavation	6, 551	. 872
Weighted average		. 292

The longer haul was one reason for the unit cost in the powerhouse section being greater than in the dam section. The comparatively small amount of rock handled and difficult working conditions resulted in the high unit cost for the south abutment area.

Depreciation included in the above costs amounted to about 50 percent of the first cost of the equipment, or $0.858 per operating hour. Operating labor amounted to $1.109 and repairs to $1.652 per net operating hour. These costs form approximately 82 percent of the total truck operating cost which totaled $4.414 per net operating hour.

One make of truck was equipped with 10.50- by 24-inch (12 ply) tires and the other make with 11.25- by 20-inch (14 ply) tires. Tire costs per cubic yard of rock averaged for both makes approximately $0.026 per cubic yard. Tire life averaged approximately 24,000 cubic yards per set of tires.

Excavation costs.

The total unit excavating costs for the entire job are shown in table 15. These costs are broken down between the powerhouse and dam excavation. Excavation for the powerhouse amounted to 47 percent of the total and for the dam 53 percent; unit field costs were $1.81 and $2.67 per cubic yard, respectively. The lower excavation costs obtained in the powerhouse area are attributed to the following facts:

The area of the dam excavation was about 2⅓ times greater than the powerhouse area and the cut only half as deep, which required considerably more moving of equipment per cubic yard of excavation.

The linear feet of drilling in the dam area was 1.71 times as great as that in the powerhouse area per cubic yard of excavation as the drilling in the dam area was for 6-foot depths or less while in the powerhouse the drilling depth averaged about 10 feet. This required more movement of drilling equipment and consequent increase in cost.

All of the powerhouse excavation was in cofferdam No. 1, which was dry. Excess leakage was encountered in cofferdams Nos. 2 and 3; this made the operation of excavating equipment difficult and thereby increased the costs.

TABLE 15.—*Foundation excavation summary*

	Powerhouse	Dam [1]
Excavation quantities:		
Length along axis ... feet	613	5,526
Average depth ... do	31	16
Area of cut ... square feet	211,485	487,566
Drilling:		
Line drilling ... linear feet	317,849	540,178
Blast holes ... do	264,185	455,915
Broaching ... square feet	8,503	21,986
Dynamite:		
60 percent ... pounds	11,643	675
40 percent ... do	116,820	175,026
Stripping ... cubic yards	6,019	3,832
Rock ... do	250,298	285,004
Total excavation ... do	256,317	288,836
Cubic yards per load	6.10	5.36
Tons per load	13.60	12.00
Length of haul ... miles	.57	.25
Pounds dynamite per cubic yard	.501	.608
Excavation unit costs:	*Per cubic yard*	*Per cubic yard*
Drilling and blasting	$0.69	$1.29
Loading (including hand labor)	.62	.73
Hauling (including ramps)	.35	.31
Bulldozer operations	.02	.04
Marine operations	(2)	(2)
Preparation of foundation	.07	.22
Miscellaneous	.06	.08
Total unit cost	1.81	2.67

[1] Includes south abutment.
[2] Less than 0.005.

FOUNDATION INVESTIGATION

The rock formation underlying the various parts of the dam is composed of two limestone members. (See chapter 2.) The position of these members and the stratification in the underlying beds are shown on figure 73 which was prepared from the core boring records.

There is wide variation in the types of rock composing these members, particularly the Fort Payne. Early in the work a standard classification for types was made from chemical and physical analyses. The two types best suited for foundation rock were the fine-grained compact siliceous limestone and the coarse crystalline limestone, both of which stood up well under exposure to the atmosphere and to weathering. The other types, while structurally sound and of sufficient compressive strength to make an entirely safe foundation material in the undisturbed condition, were unable to stand exposure even for short periods. They would air-slack, crack from expansion and contraction, and the surface would soon become loose and scaly.

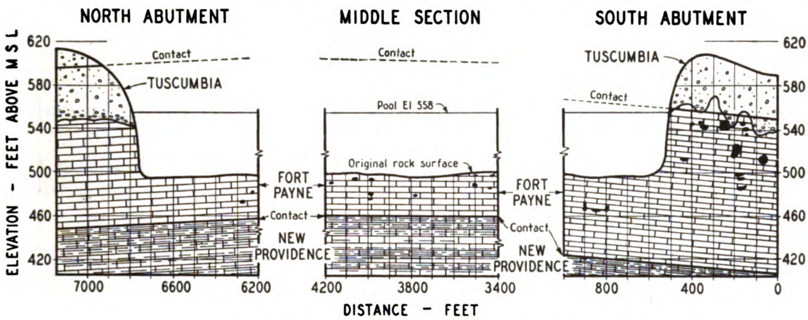

FIGURE 73.—*Rock formations underlying the dam.*

When exposed for more than a few hours prior to concreting, the surfaces of the shaly rock had to be kept constantly wet by means of sprinkler pipes, and even with this precaution in some instances the exposed layer cracked or became loose and had to be removed to the next bedding plane. This formation occurred in fairly thin strata, varying in thickness from a minimum of 9 inches to an average maximum of 60 inches, and lying in an almost horizontal plane.

At frequent intervals across the entire length of the foundation area, the horizontal bedding planes were intercepted by vertical seams or joints occurring in an approximate upstream and downstream direction, which, at the dam site, is from east to west. The condition of the vertical joints or seams varies from tight calcite-filled joints to loose mud-filled seams, the latter varying in width from between 1 or 2 inches up to a maximum of 12 inches. These joints gradually pinched out to fine ribbons of calcite at an average depth of about 25 feet below the bottom of the completed excavation and gave little or no trouble except that each one had to be investigated and plugged.

Above the level of the river bed in the south abutment section, these vertical joints in most cases led down to a solution channel and small cavities beneath the surface. (See fig. 74 (a).) Below the river bed in the intake area, joints at units 3 and 4 opened into a fairly large mud-filled cavity running completely through the foundation section.

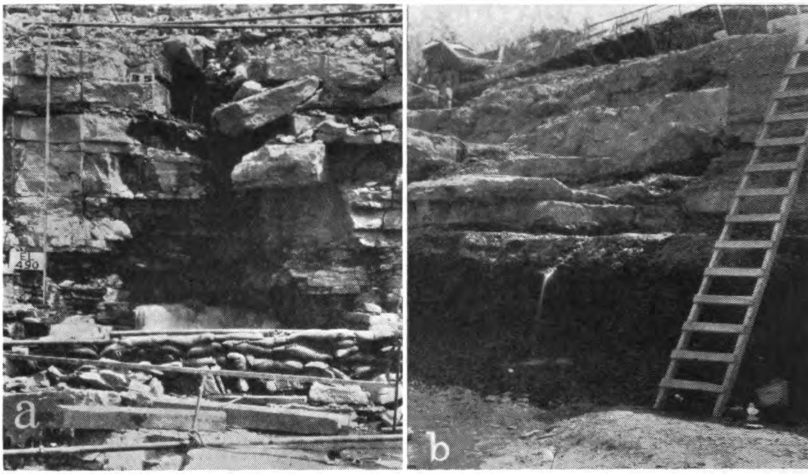

FIGURE 74.—*Foundation conditions uncovered by excavation, (a) a solution cavity, (b) a mud seam.*

Both of these cavities terminated in a large horizontal mud seam at approximately elevation 471 in the tailrace areas from units 1 to 4, inclusive. (See fig. 74 (b).) This seam had a general bearing from northeast to southwest, but as most of it lay above finished grade in the tailrace excavation, no serious difficulties were experienced from its presence, except that a high concrete wall or lining had to be placed against the south face of the tailrace excavation for the entire height, and a low concrete retaining wall was necessary from units 1 to 4 near the downstream edge of the tailrace slope.

The exposed surface of the top or cap rock was hard and pitted owing to the leaching out of all soluble compounds. This cap rock varied in thickness from about 18 to 24 inches and occurred in loose and broken layers. In most cases a thin horizontal seam of mud lay directly under the cap rock.

Just below the cap rock there occurred usually about three more layers of variable thickness, all of which had thin mud seams interbedded. This made it impossible to use these strata for foundation purposes owing to the fact that they could not be made tight by grouting and were always heavily jointed and loose.

This condition of loose bedding planes occurred from the surface at elevation 498 down to elevation 490 at the highest level and elevation 474 at the lowest level before tight bedding planes were encountered.

The rock above these elevations was usually structurally sound but could not be used for foundations on account of the looseness of the bedding planes. Attempts were made early in the period to use these higher strata of rock for foundation purposes by grouting; but this was not practical, principally because of the fact that before the loose seams could be properly grouted, they were displaced by even a slight grouting pressure which rapidly spread out under the large horizontal area, heaved the thin layers, and caused cracking.

At the beginning of the foundation work in the powerhouse area, no additional exploratory drilling was started because fairly deep excavation was necessary, and it was decided to formulate future plans from the results of observations on this work. When the draft tube and tailrace excavation had reached elevation 471, the large solution cavities and mud seams previously mentioned were uncovered, and it was then determined to make additional studies by exploration over the entire river bottom. At this time also several consulting geologists were engaged to advise on the general condition of the foundation. It was decided to explore the foundation north of the powerhouse area ahead of the regular drilling and blasting.

Drilling equipment.

The equipment purchased for the exploration program consisted of three gasoline-driven shot core drills and one electric-driven 36-inch calyx drill. The only essential difference in the operation of the large and small shot core drills was the method used in removing the core. In the small drills the core was twisted off by the action of the drilling tools after it had been firmly wedged in the barrel by means of coarse sand and fine gravel injected through the hollow drill steel. The 36-inch cores were broken off by placing one or two very light charges of dynamite in the bottom of the shot groove. Once broken, the core was raised to the surface by fastening the hoist line to an eyebolt wedged into a jackhammer hole in the top of the core.

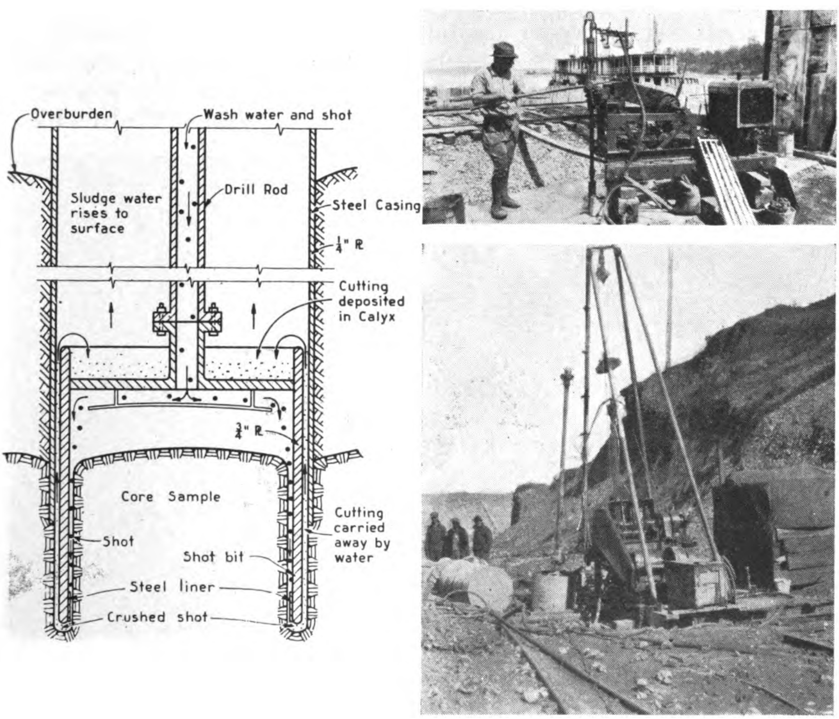

Figure 75.—*The 5½- and 36-inch shot core drills.*

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Drilling procedure.

It was decided to drill vertical holes to elevation 471, which was determined as the bottom of the probable solution channel zone. These holes were to be spaced about 2 feet from each vertical joint or seam in the rock formation on the assumption that these joints indicated the possible location of underground solution channels or cavities similar to the ones uncovered at units 3 and 4 in the power-house area. If these joint planes occurred at greater intervals than approximately 20 feet, intermediate holes were drilled so as to give a continuous line of holes spaced at 20- to 25-foot intervals along the entire length of the dam.

This program of core drilling was followed throughout cofferdams 2 and 3, but was altered in cofferdams 4 and 5 where the spacing of the holes was changed from approximately 22 feet to about 40 feet, as it was considered unnecessary to use a closer spacing except where the existence of vertical jointing planes warranted.

After the first few holes were drilled, using 6-inch bits, the size was changed to give a 4½-inch-diameter core; but the holes drilled at various locations varied from 4½ to 6 inches, as indicated by the results required.

Because of a heavy inflow of water through the loose bedding planes of the upper strata (see fig. 76), the small holes were of little value in determining the exact depth to which the excavation should be carried, since, with a few exceptions, it was impossible to observe the rock formation in place by the use of mirrors, periscopes, and other devices. A feeler rod was used to locate the bedding planes to some extent, but after the loose upper strata were passed, this pro-

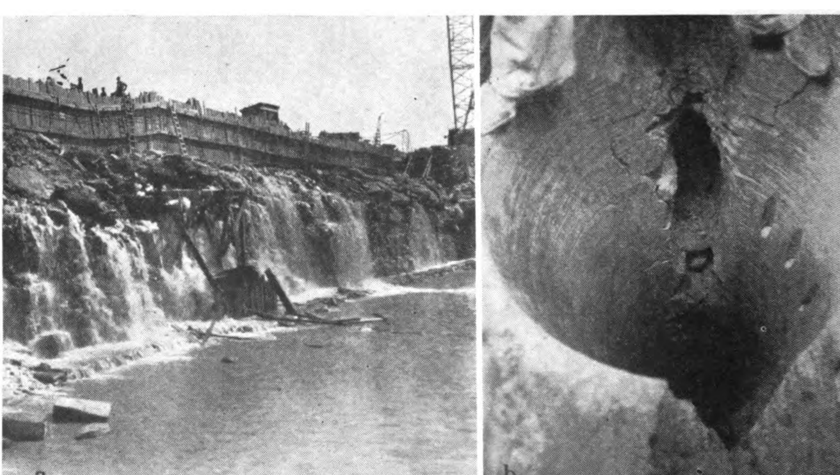

FIGURE 76.—*Foundation investigation using 36-inch core drill, (a) inflow of water through the loose bedding planes of the foundation, (b) a 36-inch core hole drilled in a vertical seam.*

cedure was not very satisfactory and was abandoned. It developed, therefore, that the principal importance of this size core drill hole was to determine definitely whether solution channels or cavities existed that might not be uncovered during excavation.

As soon as a cofferdam had been unwatered and the small shot drills started, three or four 36 inch holes equally spaced over the entire length of the cofferdam were drilled before any excavation was begun. These holes proved exceedingly valuable since a man could be lowered into them to make a visual inspection of the foundation. From this information it was quickly decided to what depth the excavation in that particular area should be carried. From the results of these observations the depth to which the line drilling would be carried was predetermined and excavation then followed accordingly.

An additional advantage of this method was that an inspection could be made of the bedding planes so that the number of lifts and the depth to which the rock should be excavated could be determined accurately.

The last stage in the foundation investigation work immediately followed the completed rock excavation. In this stage also the 36-inch drill was used almost entirely. All vertical-jointed planes of questionable appearance were drilled near the upstream face of the excavation—the depth of drilling being determined by conditions encountered. If the vertical joint continued watertight, drilling was soon abandoned; but in most cases these vertical joints were sufficiently wide to an average depth of about 25 feet to allow river water to leak into the drill hole. This drilling was continued until it was proved that these joints pinched out to a tight calcite-filled seam. Later the hole was filled with concrete to form a plug which sealed the joint.

A few 5-inch holes were drilled in the powerhouse draft tube area as a final check on the general condition of the rock underlying the point of deepest excavation. In the south abutment area preliminary investigation consisted first of drilling a series of holes with the small core drills 97 to 100 feet deep to elevation 473. Later a more thorough investigation was made consisting of a line of 36-inch holes drilled from the surface of the rough foundation cut at elevation 568 to an average bottom elevation of 498.

Another economical, fast, and convenient method of investigation was by means of the ordinary wagon drill. This method was used quite extensively when it was necessary to investigate hurriedly a piece of questionable foundation which had not been previously investigated by core drilling. With this method a hole was drilled and a very accurate observation made of the behavior of the drill. The relative hardness of the rock strata, the location of the bedding planes, open horizontal seams, mud seams, and other general conditions were determined by measuring the downward speed of the drill in the drill guides. This method, however, required the services of a thoroughly competent drill operator and observer, since inaccurate observations of air pressure, condition of drill, and many other factors would be misleading.

Core drill performance.

Drilling operations were carried on continuously in four 6-hour shifts. The work was supervised by one day foreman, and one night foreman. The following crew [12] was required for each drill for each shift:

	Small drills (hourly wage)	36-inch drills (hourly wage)
1 driller	$0.75 to $1.00	0$.75 to $1.00
1 helper	.45 to .60	.60
1 helper	----------	.45

The character of the foundation rock made drilling difficult. Shot consumption averaged 11.43 pounds per foot of 36-inch holes, and 7.97 pounds per foot of 5½- and 3-inch holes.

The 36-inch bits purchased from the Ingersoll-Rand Co. for $195 each had an effective barrel length of 44 inches, a top diameter of 34 inches, and a 1-inch taper which produced a wider shot groove. Four bits were also made in the shop from boilerplate, with a welded seam, at a cost of $65 each. These were 60 inches long with no taper. They seemed to work better than the tapered bits and had the added advantage of cutting a longer core. All bits were discarded when worn to 20 inches in length. An attempt was made to weld three 20-inch sections together to form a new bit, but owing largely to the difficulty of aligning the various worn sections the results were not satisfactory. The average life of each bit was 210 feet for the 36-inch and 140 feet for the smaller bits.

Performance and costs for the various size drilling were as follows:

Holes	Average rate in feet per gross hour	Field cost per foot of hole
3-inch	1.30	$3.50
5½-inch	.77	5.95
36-inch	.46	10.84

These costs include operating labor, tools and supplies, moving drills and cores, repairs, and depreciation. Depreciation on the small drills was approximately 50 percent of first cost, while that on the 36-inch drill was about 25 percent of first cost.

FOUNDATION PREPARATION AND GROUTING

Foundation preparation consisted of two separate operations: Preparing the surface of the excavation for concreting, in order to insure sound, clean rock surfaces; and grouting to consolidate the foundation rock below the surface of the excavation. The grouting consisted of two classes: Low-pressure grouting, prior to pouring of concrete; and high-pressure grouting, after the concrete had attained some height.

In general, rock was excavated slightly above foundation grade and the surface left in unfinished condition until immediately prior to concreting. At the south end of the dam, the trench for the cut-off wall was excavated to its proposed level. At this elevation a very thin layer of shaly material covered a thick stratum of limestone.

[12] See table 36, pp. 353–354 for changes in rates of pay during the job.

Immediately before placing the concrete this soft material was removed with paving breakers, and the surface was washed with water and air.

In preparing the rock foundation for the intake section of the powerhouse, the first four units were founded on a clean stratum of limestone, but north of this point a two-inch layer of soft greenish shale adhered to the limestone formation below. This shale was left as a protective coating on the rock until immediately before concrete was placed, at which time it was removed by paving breakers, jackhammers with large-gage bits, and ordinary hand picks. This soft deposit gave considerable trouble because it would not chip or scale readily and had to be cut and ground out by the pneumatic tools, it being so thin that no other method could be used.

In the nonoverflow sections of the dam, after the rock had been excavated to within a few inches of the bedding plane, the loose rock was scaled by wedges and bars except where, in blasting the rock, it was broken through to the seam. Where this condition existed, it presented a smooth bedding plane of rock that in some instances had to be scored parallel to the dam. This area also required continuous sprinkling with water to stop the rapid decomposition of the rock by expansion and contraction and air-slacking which resulted if exposed to the atmosphere for a short time.

Under the spillway apron sections of the dam, preparation of the foundation became rather complicated owing to the necessary slope of the rock excavation and the resultant unraveling of the thin bedding planes. This condition required the constant use of bars and wedges just ahead of the concreting operation in preparing these areas for concrete. Under the main spillway section, however, the usual procedure of scaling or watering was followed throughout.

For grouting, plunger-type grout pumps were used, operated by compressed air at about 100 pounds per square inch pressure. These were equipped with removable composition rubber disc valves and valve seats since constant replacement of worn parts was necessary. When in operation, these pumps required cleaning and dismantling about once every 6 hours, when all the caked grout was removed from the valve chambers and around the valves. The frequency of cleaning depended on the temperature, the pressure, and the rate of flow of the grout. High temperature and pressure combined with a slow rate of flow would clog the pump more quickly than the reverse condition of low temperature and pressure with a high rate of flow.

Grout mixers were the mechanically agitated paddle type, the paddle wheel being operated by a small air-driven motor. At the beginning of the grouting operations one pump and one mixer were used, the mixer discharging into a receiver which was connected to the suction of the pump. This arrangement was modified to use the receiver as a mixer and provide two mixers for each pump. A second grout pump of the same size was purchased during the early stages of the work and a double-compartment paddle-type mixer was used with this equipment. The pump and mixers were mounted on skids to form a compact, mobile unit.

All grouting was carried on throughout the four daily shifts. A total of 11,800 barrels of cement was used in the low- and high-pressure grouting operations.

For low-pressure grouting in the powerhouse intake area, two lines of holes were drilled with wagon drills. One line was located 1 foot upstream and the other 19 feet downstream from the axis of the dam. These holes were spaced on 20-foot centers and staggered, and were drilled $2\frac{1}{2}$ inches in diameter and 20 feet deep. Upon completion of the drilling, the holes were blown out with air, using a pressure of 100 pounds per square inch. Three-foot lengths of 2-inch pipe were inserted to a depth of 2 feet in the holes and grouted with neat, high early strength, rapid hardening cement. After these pipes had set for a sufficient length of time, the holes were washed with clear water and blown dry.

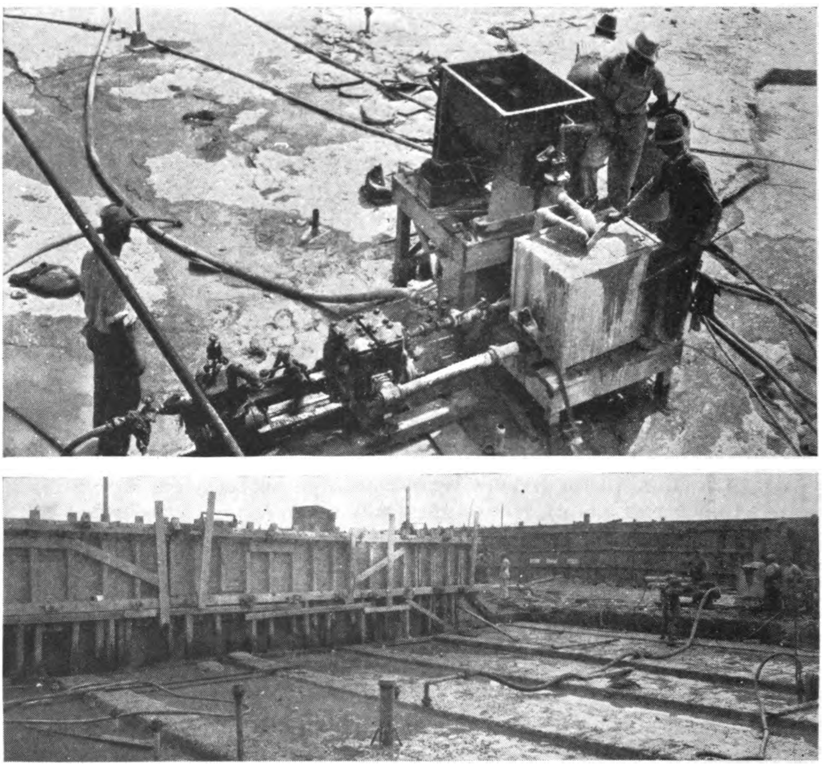

FIGURE 77.—*Grout mixer and pump.*

Before injecting grout, each hole was given a hydrostatic test. After the water pressure had been raised to 38 pounds per square inch, the supply was turned off and the time required for the pressure to drop from 38 pounds to zero was noted; or in the event it did not fall completely to zero, the pressure was held on for several minutes and the rate of pressure drop noted.

After the hydrostatic test, a neat cement grout composed of 30 gallons of water per sack of cement was pumped into the hole. During this grouting the caps were removed from adjacent holes in order to release any trapped air and to permit a flow of grout through any seams in the rock. This venting was probably unnecessary as only a few holes showed any grout flowing from adjacent holes. An initial pressure of about 20 pounds per square inch was used for this work. Upon completion of the grouting of the first two lines of holes, intermediate holes 20 feet deep were drilled half way between the original holes. These holes were then grouted in a similar manner. In addition, wherever a vertical seam was located, one or more 20-foot grout holes were drilled along the seam and grouted.

In the nonoverflow and spillway sections low-pressure grouting consisted of two lines of 2½-inch-diameter wagon drill holes 20 feet deep spaced on 20-foot centers and staggered. The upstream holes, which were inclined 20 degrees from the vertical, were about 8 feet upstream from the dam axis; the downstream holes were on a line 12 feet downstream from the axis of the dam. Additional holes were drilled when the behavior of the first set indicated they were needed. Whenever a vertical seam or joint running in an upstream and downstream direction was discovered, one or more additional holes were drilled to an average depth of 20 feet. These holes were washed, capped, tested, and grouted in a manner similar to that used in the intake section. The grout mix used was determined from the results of the hydrostatic tests and ranged from a minimum of 7½ to a maximum of 30 gallons of water per sack of cement. Pumping on each hole was continued until it took grout at a rate of one-tenth of a cubic foot per minute or less.

Because of the thin, horizontal stratification, extreme care was taken to make certain that the rock was not lifted. A constant check was kept on the area surrounding the hole that was being grouted. Four target rods were mounted on large square bases and continuous readings taken with a level as grouting progressed. A slow increase in pump pressure was maintained while starting a new hole, and if no rock movement took place it was gradually raised to a usual maximum of 40 pounds per square inch. Some holes were sealed at 50 pounds, but the lower pressure was more usual. In the majority of low-pressure grout holes, the amount of grout taken was just enough to fill the volume of the holes.

Some trouble was experienced in the spillway areas from grout leaking out through the spillway apron excavation. This condition was overcome by temporarily stiffening the grout and pumping very slowly at a reduced pressure to prevent opening the bedding planes through which the grout flowed.

High-pressure grouting followed essentially the same procedure as low-pressure grouting. Drilling was started after the low-pressure grouting had been completed, except in the intake area. The high-pressure holes were drilled 30 feet deep with wagon drills and spaced at 5-ft. intervals along the axis of the dam. After the foundation area had been prepared for concrete, 2-inch pipe sleeves were grouted in the holes, and after a sufficient lapse of time the hole was washed, cleaned, and capped.

As soon as concreting began, pipe extensions were added to the sleeve that had previously been grouted in place. As concrete progressed, these pipes were curved outward to meet the upstream face of the dam at elevation 504. By terminating these pipes at the upstream face, it was possible to continue grouting at any time without interfering with other construction work.

The high-pressure grouting in the intake area was done by first setting a 4-inch-diameter pipe on the rock surface before concreting had started and later drilling through this casing pipe with 3-inch shot drill to a depth of 60 feet. The holes for the intake area were drilled on 5-foot centers along a line 39 feet downstream from the axis of the dam.

The mixes used in high-pressure grouting were the same as those for low-pressure work, and the safe maximum final grouting pressure in all cases was determined to be 100 pounds per square inch.

In the high-pressure grouting, considerable trouble was experienced from grout seeping along the foundation surface and entering the tile drain system. This occurred in the spillway and intake sections. In these areas a supply of clear water was continuously circulated through the drains to wash out any grout that might enter. A little trouble was also encountered during the intake grouting from grout seeping out through the bedding planes and running down the vertical face of the draft tube excavation. This was easily corrected, however, by using a stiffer mix for a short period and then returning to the thin mix after the leakage had been effectively stopped.

CONCRETE AGGREGATES

Surveys made by the United States Army Engineers in the section of the river between Sheffield and Riverton indicated that sufficient sand and gravel for the construction of the dam could be obtained in a 4-mile stretch immediately below Seven Mile Island. This was located within an average towing distance of about 30 miles from Wheeler Dam. Bids for supplying the aggregate were requested, and the Cumberland River Sand Co. was low bidder at a price of $0.895 per ton delivered at the dam site. It was considered advisable to have this work done by private contract at that price rather than secure equipment and do the work by force account. Accordingly, a con-

FIGURE 78.—*Testing equipment for gravel survey.*

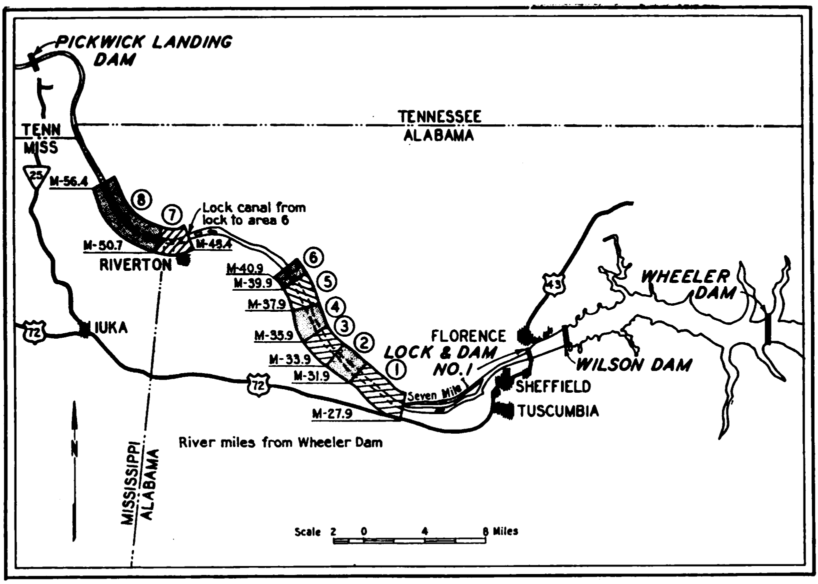

FIGURE 79.—*Gravel dredging areas.*

tract was let calling for 1,000,000 to 1,400,000 tons of aggregate made up of sand and three sizes of gravel. All material that could be currently used was delivered directly to the dam and the surplus was stored in a yard which the contractor maintained at Florence, Ala. Approximately a tenth of the total aggregate used went through the storage pile. The contractor arrived with his equipment in May 1934 and loaded the first barge on July 12, 1934.

Surveys for additional gravel.

Actual dredging operations developed only about 30 percent of the material indicated by the original survey, and it became necessary to locate further workable bars. The gravel exploration equipment consisted of a 4-inch wash drill outfit mounted on a 14- by 40-foot wooden barge and a 24-inch testing outfit mounted on an 18- by 80-foot wooden barge. The 4-inch drill was used to determine the extent and depth of the bars, and the 24-inch outfit to determine the quality and grading of the material. A 1,500-cubic-inch orange-peel bucket was used to remove the material from inside the 24-inch pipe. Sampling was done on the barge, utilizing a set of portable screens and a scale. No additional material was found above Riverton, Ala., but some good bars were located below the Riverton lock between river miles 221 and 226. This new location was 20 miles further downstream or 50 miles below Wheeler Dam. About 70 percent of all the material used came from the area below Riverton.

Supplemental contract.

When it became evident that the original source would be inadequate, a supplemental agreement was made with the aggregate contractor providing for the extension of operations farther downstream and for additional compensation for the longer haul.

The revised unit prices, which were expressly intended to cover the increased cost of towing, were as follows:

	Per ton
Area No. 1—Original contract. From lower end of Seven Mile Island to 4 miles below the island: 29.9 miles from Wheeler	$0.895
Area No. 2—0 to 2 miles below original contract: 32.9 miles from Wheeler	.925
Area No. 3—2 to 4 miles below original contract: 34.9 miles from Wheeler	.955
Area No. 4—4 to 6 miles below original contract: 36.9 miles from Wheeler	.985
Area No. 5—6 to 8 miles below original contract: 38.9 miles from Wheeler	1.015
Area No. 6—8 miles below original contract to head of Riverton Canal: 40.4 miles from Wheeler	1.045
Area No. 7—Riverton Lock to 2.3 miles below the lock: 49.5 miles from Wheeler	1.185
Area No. 8—2.3 miles below Riverton Lock to 8 miles below the lock: 53.6 miles from Wheeler	1.300

Surveys failed to locate any workable bars in areas 2–6, and all material supplied at the prices covered by the supplemental agreement came from areas 7 and 8 in nearly equal proportions.

Dredging equipment.

The *Kentucky*, an all-steel ladder-type dredge, 155 by 44 by 8 feet, with a capacity of from 200 to 270 tons per hour under the existing conditions, was entirely self-contained. It was capable of digging, screening, washing, and accurately sizing the sand and three sizes of gravel required, and delivering them conveniently to four barges, two on each side of the dredge. The dredge was capable of digging effectively to a depth of about 55 feet and was equipped with four 36-inch by 65-foot-square steel spuds. The digging buckets were 4.3-cubic-foot capacity and moved at a rate of 31 buckets per minute. These buckets dumped the material over grizzly bars with 7-inch openings where an elevator with 3¾-cubic-foot buckets conveyed the passing material to the primary screen a 4- by 8-foot Simplicity, with 3¼-inch-square opening. Material passing went to a second elevator, and that which was retained went through a 7-inch Allis-Chalmers, Newhouse crusher and back into the material line. The second elevator discharged over a pair of Robbins triple-deck Gyrex screens, each 4 by 8 feet. The top deck of each had a 1½-inch-square opening, the middle deck ⅞-inch clear opening wire cloth, and the bottom deck ⅜-inch clear opening wire cloth. Material from the ⅜-inch screen passed over a single-deck 4- by 8-foot Simplicity screen with 3-mesh screen cloth which removed the excess ¼- to ⅜-inch material and wasted it into the river. Two auxiliary elevators completed the sand and ¾-inch gravel distribution to the chutes. The material was thoroughly washed at every stage of production.

One 10-inch and one 8-inch pump, rated at 2,400 and 1,600 gallons per minute, respectively, furnished water for the dredge. The digging buckets and the two bucket elevators were operated by direct power from a 16- by 16-inch "Skinner Universal Uniflo" engine. Auxiliary power for screens, secondary elevators, and other uses was

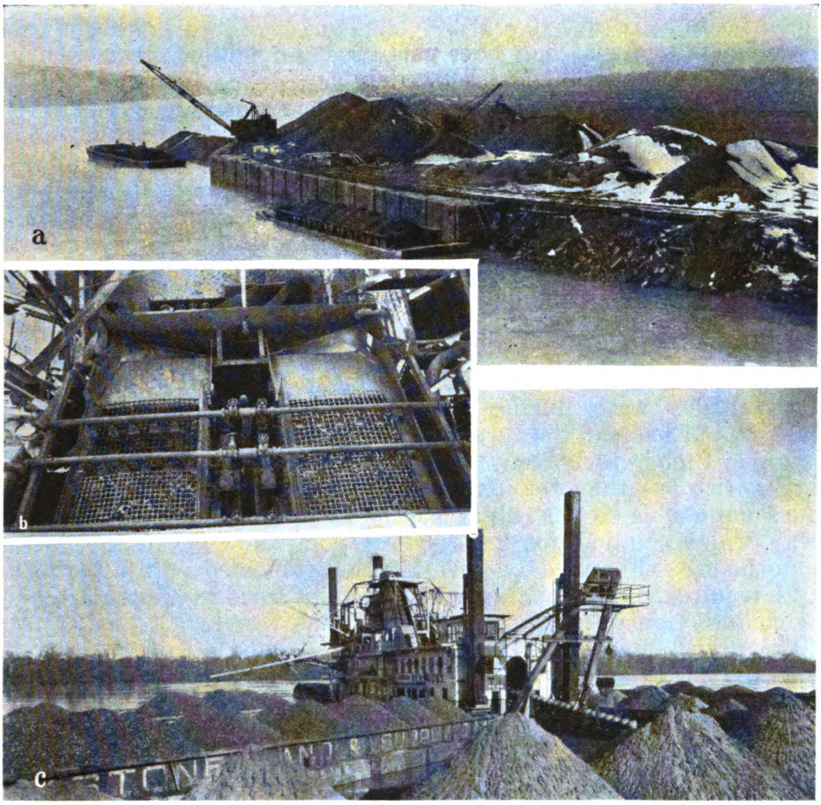

FIGURE 80.—*Dredging operations: (a) Contractor's aggregate storage yard at Florence, Ala., (b) Screens, (c) Dredge "Kentucky."*

supplied by a 50-kw., 250-volt Westinghouse generator driven by a second 16- by 16-inch Skinner engine. There were two 21-foot by 75½-inch vertical fire tube boilers. The dredge had living accommodations for about 25 men and was built in 1928.

Towing equipment.

One of the main problems confronting the aggregate contractor was that of towing barges upstream, from 30 to 50 miles, through some of the worst water in the lower river. Numerous cuts and chutes with swift water made towing extremely difficult, and at one location it was necessary to maintain a power puller for a distance of a half mile. Conditions were more difficult during periods of low water when at several points there was not sufficient depth to permit towing fully loaded barges. There were three locks to be passed—Riverton, Lock and Dam No. 1, and Wilson (a double-lift lock).

The sand company sublet the towing through the greater part of the job to the Walter G. Hougland River Transportation Co., which operated two steamboats rated at 250 and 275 horsepower and two

Diesel boats rated at 240 and 300 horsepower. Two 120-horsepower Diesel boats were also in service part time and another steamboat was held for emergencies. Approximately 50 barges furnished by the sand company were in service, mostly of the steel cargo box type, 135 by 27 by 8 feet in size, with a capacity of about 400 tons each under good towing conditions. The principal problem was to maintain a flow of material from the dredge to the mixing plants, so the latter were amply supplied without tying up too many loaded barges at the mixing plant thus crippling dredge operations. This was done in a fairly satisfactory manner without undue dependence on the storage supply at Florence.

Dredging conditions.

Both dredging and towing operations were greatly hampered by extremely low water during the late summer and fall of 1935. The river was at or near the zero stage on the Florence gage for a considerable time, which restricted traffic below Florence to boats and barges with less than four-foot draft. This made it difficult and at times impossible to get empty barges to the dredge or loaded ones away and accounted for a period of poor showing in the dredge performance. Fortunately, this situation coincided with a period of light requirements at the dam and did not seriously interfere with the construction program.

The gravel survey indicated bars with depths up to 16 feet, and generally the dredge found material about as indicated, both regarding quality and depth, and secured from 8 to 14 feet of gravel. The dredge had a draft of 8 feet but required about 12 feet of water for satisfactory digging. This depth was usually available with the river at normal stage.

A shortage of sand and an overrunning of ¾- and 1½-inch gravel in the bars necessitated the unloading and wasting of some of the gravel to maintain a balance in the different sizes of material. After the start of construction at Pickwick Landing Dam, which was also supplied by this dredge, the contractor found it desirable to purchase about 30,000 tons of sand at Montgomery, Ala., in order to maintain this balance and place the gravel in storage at Pickwick Landing.

Taken as a whole and eliminating from consideration the period of low water, which was not totally unexpected at that season, dredge performance was fairly uniform and met the requirements of the construction program with only moderate cause for delay. The job sometimes ran out of one and occasionally two sizes of coarse aggregate for short periods of time as the result of poor coordination of dredging and towing. In these instances mixes were adapted [13] from whatever size materials were available in order to keep the job running.

Dredge performance.

Table 16 summarizes dredge performance from January 1935 to March 1936, inclusive. Figures are not shown prior to 1935, since dredge records before that time were not kept in sufficient detail to

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[13] See p. 173.

be useful. Shipment of some of the dredged material to Pickwick Landing began in November 1935 and continued in increasing proportions until April 1, 1936, when the Wheeler contract was practically completed and nearly all material went to Pickwick Landing. No division between the two jobs is shown in this record. The percentages of time indicated to have been consumed by the various operations and delays are only approximately correct because different activities were often in progress at the same time. In this table the intent has been to charge time to the activity which seemed to be the governing factor.

Table 16.—*Performance record—Gravel dredge "Kentucky"*

[From January 1935 to March 1936, inclusive]

	Hours	Percentage of gross
Gross operating time	10,944.0	100.00
Delays:		
Move dredge	241.5	2.21
Shift changes and meals	461.0	4.22
Change barges	523.5	4.77
No empties	1,602.8	14.65
Repairing	1,936.0	17.69
Wash boilers	528.4	4.83
Barges or dredge aground	146.6	1.34
Holidays	57.1	.52
Fog or frozen equipment	24.0	.22
Out of coal	24.8	.22
Total delays	5,545.7	50.66
Net operating time	5,398.3	49.34
Total tons dredged	1,120,071	
Total barge loads	2,959	
Tons per gross hour	102.3	
Tons per net hour	207.4	
Tons per month	74,671.4	
Tons per barge	378.5	
Barges per month	196.3	

Aggregate tests.

All aggregates were tested by material inspectors at the dredge before the barges left for the dam site or storage pile. If the material was rejected, the contractor had the privilege of sending it to the dam for final inspection if he so desired. Some of this rejected material was accepted and used for sandbags or fills.

The inspector's work at the dredge included tests for such factors as grading, organic impurities, dust particles, and silt. The inspector kept a log of the dredge operations and barge movements and had the right to decide where the material was to be dug. After the barges were received at the dam, they were sampled and the material tested again as a check on the tests made at the dredge. In general, the two groups of tests checked quite closely. Tables 17 and 18 give the average monthly grading of the different sizes of aggregates. Figure 81 shows the variation in grading of the sand for the period of the job.

TABLE 17.—*Average screen analysis of concrete aggregate sand*

	Percent passing							Fineness modulus	Number of tests
	⅜ inch	No. 4	No. 8	No. 16	No. 30	No. 50	No. 100		
1934									
August	100.0	99.6	87.1	70.0	43.0	7.2	0.2	2.93	26
September	100.0	99.4	86.3	69.9	42.9	7.4	.2	2.93	65
October	100.0	99.1	86.3	69.3	41.6	6.9	.2	2.96	99
November	100.0	99.4	86.9	69.2	39.1	6.8	.3	2.98	113
December	100.0	99.4	85.9	69.7	41.1	5.9	.3	2.98	117
1935									
January	100.0	99.7	89.1	77.6	51.6	7.9	.5	2.74	97
February	100.0	99.6	88.7	76.0	53.2	9.1	.4	2.73	56
March	100.0	99.7	85.4	69.2	46.5	7.4	.4	2.91	49
April	100.0	99.6	82.2	63.5	39.6	5.1	.3	3.09	107
May	100.0	99.6	83.5	65.8	44.3	6.8	.4	2.99	157
June	100.0	99.7	84.9	68.3	48.9	9.2	.4	2.88	159
July	100.0	99.7	85.7	69.4	49.4	9.1	.6	2.86	113
August	100.0	99.7	85.7	69.1	50.5	6.4	.8	2.87	161
September	100.0	99.5	81.1	60.5	38.7	8.2	.6	3.11	115
October	100.0	99.5	79.8	56.5	35.8	6.5	.5	3.21	81
November	100.0	99.8	83.5	60.2	39.3	8.5	.4	3.08	103
December	100.0	99.6	83.1	61.1	39.9	7.3	.4	3.08	85
1936									
January	100.0	99.7	83.6	60.7	39.9	8.3	.4	3.07	40
February	100.0	99.6	85.6	65.0	45.0	7.5	.4	2.97	34
March	100.0	99.4	88.8	75.8	51.6	11.9	.4	2.72	31
Average or total	100.0	99.6	85.2	66.8	44.1	7.7	.4	2.95	1,806

NOTE.—Tests made at dredge of Cumberland River Sand Co. and at Wheeler Dam laboratory.

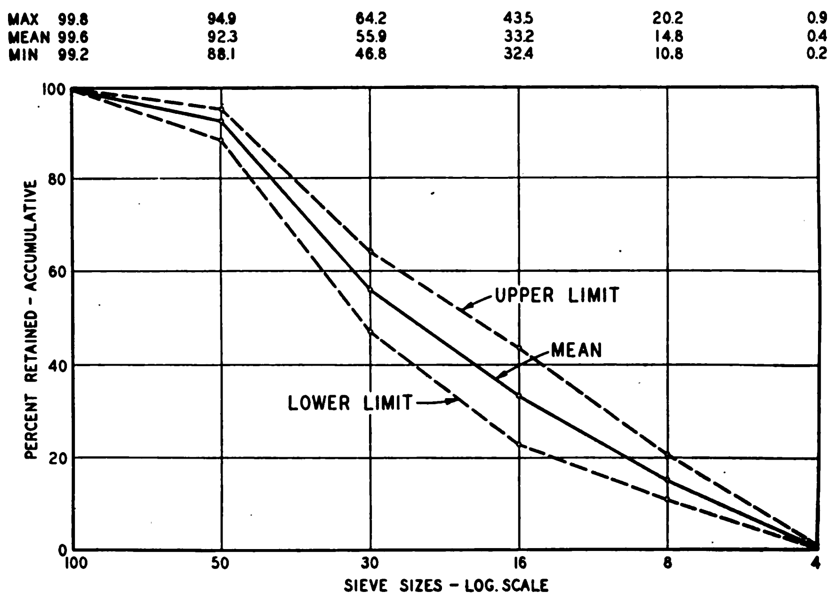

FIGURE 81.—*Sand grading variation.*

TABLE 18.—*Average screen analysis of concrete aggregate fine gravel*

	Percent passing					Fineness modulus	Number of tests
	1½ inches	¾ inch	⅜ inch	No. 4	No. 8		
1934							
August	100.0	89.3	23.2	1.1	0	6.86	15
September	100.0	89.2	23.9	.5	0	6.86	42
October	100.0	91.0	26.5	1.6	0	6.80	64
November	100.0	92.3	28.4	1.2	0	6.78	70
December	100.0	92.4	27.6	1.1	0	6.79	62
1935							
January	100.0	90.0	24.4	.5	0	6.85	49
February	100.0	89.1	21.8	.5	0	6.88	36
March	100.0	91.7	25.5	.8	0	6.82	25
April	100.0	92.1	29.0	.5	0	6.78	80
May	100.0	92.8	29.9	.7	0	6.66	99
June	100.0	92.7	29.0	.7	0	6.77	112
July	100.0	92.1	26.9	.6	0	6.80	71
August	100.0	90.3	23.0	.3	0	6.86	112
September	100.0	90.8	24.6	.6	0	6.84	66
October	100.0	90.4	22.4	.6	0	6.86	31
November	100.0	92.3	22.5	.7	0	6.84	99
December	100.0	89.5	23.0	.7	0	6.86	78
1936							
January	100.0	88.2	25.1	.8	0	6.85	52
February	100.0	88.4	24.0	.1	0	6.87	17
March	100.0	89.3	24.7	.7	0	6.85	13
Average or total	100.0	90.7	25.3	.8	0	6.83	1,213

NOTE.—Tests made at dredge of Cumberland River Sand Co. and at Wheeler Dam laboratory.

TABLE 19.—*Average screen analysis of concrete aggregate medium gravel*

	Percent passing				Fineness modulus	Number of tests
	3 inches	1½ inches	¾ inch	⅜ inch		
1934						
August	100.0	92.7	4.7	0	8.02	25
September	100.0	88.3	5.1	0	8.06	52
October	100.0	91.6	6.1	0	8.02	75
November	100.0	93.1	5.5	0	8.01	89
December	100.0	94.3	4.9	0	8.00	101
1935						
January	100.0	92.8	6.2	0	8.01	84
February	100.0	93.4	5.8	0	8.00	44
March	100.0	93.5	7.6	0	7.98	41
April	100.0	90.0	6.5	0	8.03	69
May	100.0	93.2	5.4	0	8.01	98
June	100.0	94.3	5.9	0	7.99	117
July	100.0	95.3	5.8	0	7.98	75
August	100.0	96.0	5.4	0	7.98	110
September	100.0	96.4	5.3	0	7.98	65
October	100.0	95.5	6.2	0	7.98	26
November	100.0	93.2	7.7	0	7.99	89
December	100.0	93.3	6.5	0	7.99	75
1936						
January	100.0	93.2	7.0	0	7.99	59
February	100.0	93.0	8.0	0	7.99	15
March	100.0	92.1	12.0	0	7.96	5
Average or total	100.0	93.3	6.4	0	8.00	1,314

NOTE.—Tests made at dredge of Cumberland River Sand Co. and at Wheeler Dam laboratory.

Table 20.—*Average screen analysis of concrete aggregate coarse gravel*

	Percent passing				Fineness modulus	Number of tests
	4 inches	3 inches	1½ inches	¾ inch		
1934						
August	100.0	97.1	11.3	0	8.91	12
September	100.0	98.8	9.9	0	8.91	31
October	100.0	99.0	9.3	0	8.91	47
November	100.0	98.0	8.9	0	8.83	60
December	100.0	98.9	9.5	0	8.91	58
1935						
January	100.0	97.6	11.4	0	8.91	40
February	100.0	97.7	11.4	0	8.91	28
March	100.0	95.9	13.6	0	8.90	19
April	100.0	97.7	8.5	0	8.93	31
May	100.0	98.6	8.6	0	8.92	48
June	100.0	98.2	8.8	0	8.93	56
July	100.0	97.8	12.6	0	8.89	44
August	100.0	98.8	13.1	0	8.88	55
September	100.0	97.3	12.7	0	8.90	46
October	100.0	97.9	13.0	0	8.89	14
November	100.0	97.2	13.8	0	8.89	58
December	100.0	90.3	10.8	0	8.98	38
1936						
January	100.0	91.0	10.6	0	8.88	33
February	100.0	94.1	12.9	0	8.93	12
March	100.0	100.0	21.1	0	8.79	4
Average or total	100.0	97.1	11.6	0	8.91	734

Note.—Tests made at dredge of Cumberland River Sand Co. and at Wheeler Dam laboratory.

CONCRETE MIXING

The four floating mixing plants were probably the most unusual items of equipment used on the entire job. In the design of these plants it was necessary to limit the over-all height of the storage bins to a minimum; otherwise the width of the barge required to prevent overturning would have been excessive. The size of the barge was 90 by 40 by 7 feet. Also, it was essential to embody in the structural design, as well as in the weighing scales, provision for stability and accuracy under variable tilting of the barge.

Mixing plant.

During operation of the plants, four aggregate barges were tied to the sides of each mixing plant barge: two with coarse aggregates on one side, and one with coarse aggregate and one with sand on the other side. The material was unloaded from the aggregate barges to the overhead bin on the plant barge by means of an American Revolver whirley type crane equipped with a 2-cubic-yard clamshell bucket.

The plant had an over-all height from the deck to the top of the aggregate bin of 38 feet 10 inches. Aggregate bins, batching equipment, mixers, and the cement silo were at the forward end of the barge and the crane was aft. Both the aggregate bins and batching equipment were furnished by the Blaw-Knox Co., of Pittsburgh, Pa. The bins and batchers were originally built for three sizes of aggregates but were rebuilt on the job to care for four sizes of material. The bins had a total capacity of about 150 tons.

Kron springless-type scales having a capacity of 10,000 pounds were used for weighing aggregates. The sand and coarse aggregate

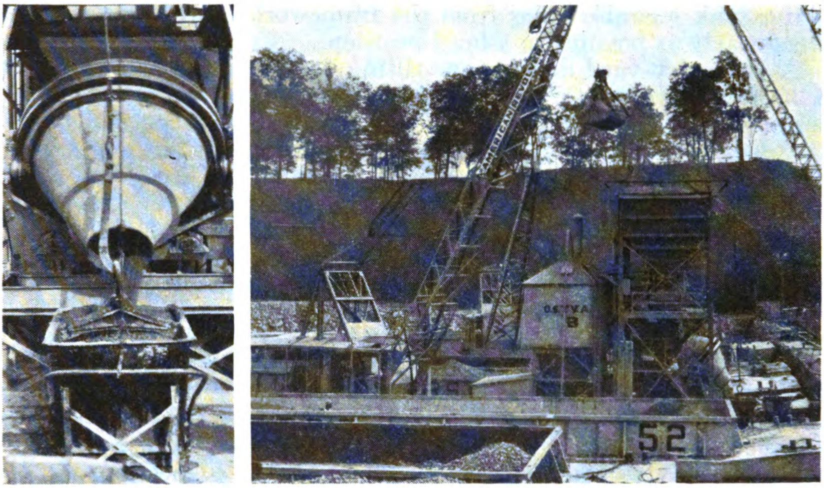

FIGURE 82.—*Floating concrete mixer plant.*

required for a batch were weighed accumulatively, the sand being weighed last, in order to make adjustments in weight more easily for variation in moisture content. The aggregates were weighed in a single batcher divided into two sections, for sand and coarse aggregate. The gates leading from the aggregate bin to the batcher were manually operated.

Cement was unloaded into the silo by a Fuller-Kinyon pump from a cement barge tied to the stern of the mixer barge. These barges contained from 1,200 to 1,400 barrels of cement. The cement silo, cylindrical in shape and 15 feet in diameter, had a capacity of 550 barrels. It was set low in order to concentrate as much weight as possible near the deck of the barge. The cement was discharged by gravity from the bottom of the silo through a 14-inch rotary valve, air jets being provided to loosen the cement if arching or packing prevented its flow. The cement discharged through the valve into a horizontal screw conveyor about 8 feet long, which in turn discharged into the bottom of a vertical conveyor. This elevated the cement to a sufficient height so that it could be discharged by gravity through a chute directly into the cement weighing batcher. Both of the conveyors were driven by the same motor located on the deck of the barge. A louver gate was used to control the feeding of cement from the gravity chute into the batcher. The gate was operated by a small lever which also controlled the motor driving the screw conveyor. Kron springless-type scales of 2,000-pound capacity were used for weighing cement. Cement and aggregate scales were suspended from the framework in order to be plumb regardless of any tilt of the barge.

A water-measuring tank holding 100 gallons was provided with a mechanical interlock on the discharge valve to prevent its opening until the filling valve had been closed. The water measurement was controlled by an adjustable overflow pipe located centrally inside the tank to prevent errors caused by the listing of the barges. The

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water tank was also hung from the framework in order to maintain it as nearly as possible in a level position.

A two-cubic-yard double-cone tilting 56–S Davis Mixer driven by a 35-horsepower motor at a speed of about 10 revolutions per minute was mounted on each plant. It was mounted on a pedestal base on the deck of the barge, high enough to permit discharging directly into a two-cubic-yard concrete bucket placed on the deck of the barge. Approximately 15 seconds were required to charge the mixer from the batchers above. The mixing time was not less than 1½ minutes, and the discharge required approximately 15 seconds. The timing device and batchers were interlocked with the gates on the batcher discharge chutes so that the timing of the mix did not start until this gate was closed.

After a lift had been completely poured, the mixer barge and the cement and aggregate barges were moved as a single unit by a towboat to a point opposite the cofferdams where the next pour was to be made. About 30 minutes were required for such a move. It was necessary to make and break the electrical connection with the main feeder line on the cofferdam each time the plant was moved. Vibrators used for placing concrete were also connected to lines coming from the mixer, and it was necessary to disconnect them at each move. The water pipes to the mixer also had to be changed. One of the mixers was dismantled early in 1935, and its crane was used in the Nos. 3 and 4 cofferdam areas.

Mixing plant performance.

A record of mixing plant performance was kept for each plant by means of a batch counter in addition to a tabulation of operating time and delays. The meter readings included the 2-cubic-yard concrete batches, 1-cubic-yard grout batches, and all batches of water passing through the mixer for washing purposes. During long pours, it was necessary to clean the mixer drum occasionally by charging the mixer with large aggregate and water, mixing the usual cycle, and wasting the used aggregate. The maximum hourly rate of pouring was 54 cubic yards, and the average for the job was 37.12 cubic yards. The following tabulation gives the quantities mixed and the operating time:

Total quantity of concrete handled by the floating mixer plants including the lock miter sill, cubic yards		631,150
Operating time (hours):		
Gross operating time		20,492.75
Delays:		
Idle (with crew)	820.75	
Moving to other pours	667.75	
Waiting for forms	969.75	
Waiting for cranes	315.25	
Repairs to plants	261.75	
Miscellaneous	360.00	
Total delays		3,395.25
Net operating hours		17,097.50

Mixing plant costs.

The normal crew used in actual operation of these mixing plants and the rates [14] of pay per hour were:

[14] See table 36, pp. 353–354, for changes in rates of pay during the job.

Material handling		*Mixer*	
Crane operator	$1.50	Weigher	$0.55
Oiler	.60	Dumper	.55
Flagman	.60	Concrete laborer (2 at $0.55)	1.10
Cement pumpman	.60	Laborer	.45
Standard crew	3.30	Total	2.65

A shift foreman's wage at $1.25 per hour, 3 percent of the general concrete foremen's salaries, and a pro rata of the miscellaneous helper and maintenance gang costs are included in the operating labor costs. A number of the following listed mechanics and helpers are also included in the operating forces. The number of these men used depended on such factors as the number of plants in use and their position in relation to the different cofferdams and to each other.

	Per hour
Mechanic	$1.10
Electrician	1.10
Pipe fitter	1.10
Pipe fitter helper	.60
Rigger	1.00
Relief man	.60

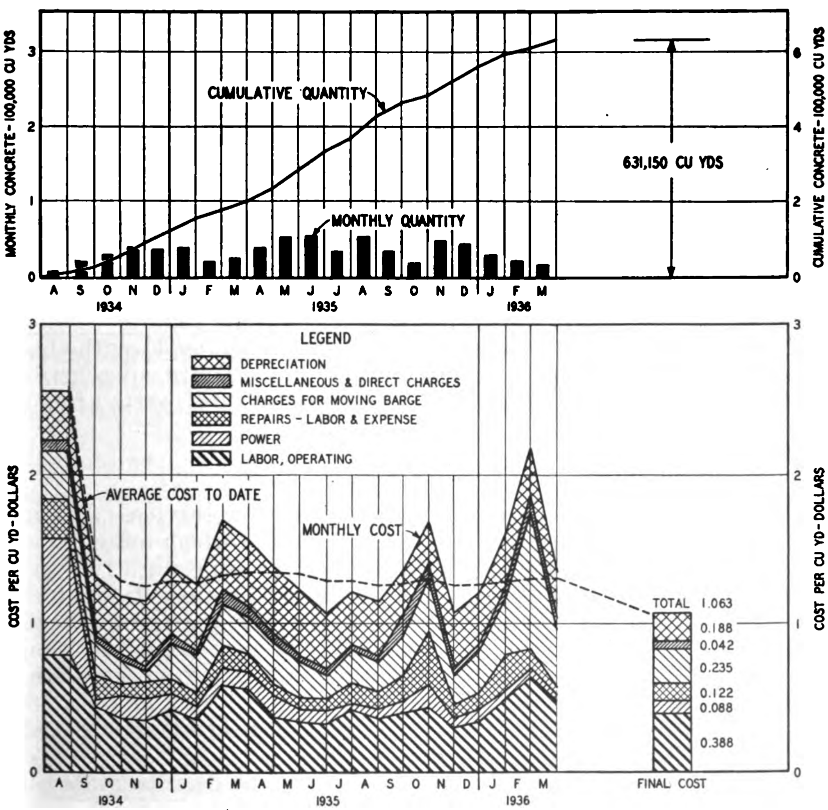


FIGURE 83.—*Average operation costs on floating concrete mixer plants.*

An analysis of the unit production costs for the mixer plant operations during the entire job is shown in the following tabulation:

	Total costs	Cost per net hour	Cost per cubic yard
Labor	$244, 698. 18	$14. 312	$0. 388
Power	55, 508. 40	3. 247	. 088
Miscellaneous supplies and expense	11, 903. 03	. 696	. 019
Repairs	77, 193. 90	4. 515	. 122
Depreciation	118, 809. 24	6. 949	188
Direct charges	14, 678 94	. 859	. 023
Moving equipment	148, 433. 91	8. 682	. 235
Total	671, 225. 60	39. 260	1. 063

The moving charge includes the cost of moving the plant from one location to another, and moving loaded and empty barges to and from the mixing plant. The labor cost item includes the direct operating labor as well as other charges previously mentioned. Depreciation charge includes first cost plus dismantling charges, less transfer or salvage credits.

Repairs.

The aggregate bins required frequent repair due to damage caused by the clamshell buckets. Because a considerable amount of foreign material—nuts, bolts, and nails—found its way into the cement, the cement pumps required frequent repairs. Electric vibrators were placed on the aggregate charging hoppers to increase the speed of delivery of materials to the mixer. The charging side of the mixer blades was cut down to permit the mixers to take aggregate faster.

The opening between the mixer blades and the drum was increased to about 2½ inches to reduce the sticking of concrete and to improve the mixing. Much material was wasted at the charging end of the mixer until these changes were made and the jackknife chute discarded for a sliding one. Blades and drum lining of the mixers were occasionally built up by hard surfacing welding rods to counteract abrasion. Some damage was caused by the crane hook or the bucket hooking the discharge end of the mixer. Cast-iron sprockets and gears in some cases were replaced with heavier steel parts after they were broken or worn.

Conclusions and recommendations.

The mixing plant as a whole gave satisfactory service. The manually operated aggregate weighing batchers worked quite satisfactorily. Batcher operators always weighed up to or slightly over the required amounts, and for this reason and also because of slight losses in handling, the scale weight of the surface dry aggregates averaged about 1½ percent less than the contractor's barge displacement weight, and the cement batcher weight averaged about 2 percent less than the railroad weight.

Improvements as listed below would be beneficial and should be considered in duplicating this plant.

1. An additional aggregate bin would have made it possible to use a finer sand or an admixture when the regular sand became too coarse.

2. The speed of placing concrete would have been increased at times had the concrete been first discharged into a hopper mounted on the deck of the barge rather than directly into the bucket. To accomplish this, the mixers would have had to be raised an additional 5 to 6 feet above the deck of the barge. It should be noted, however, that the barges should be of different design because of a higher center of gravity if the mixers were so located. This would probably require a wider and stronger barge.

3. The canvas cloth connection between the cement weighing batcher and the discharge chute was not satisfactory. The cement would accumulate and harden, both on the inside and outside of this connection, because of its looseness and bagginess. This accumulation would register as much as 25 pounds on the scale so that, in weighing a batch, allowance had to be made for this weight, which was not constant but would vary from batch to batch. An impervious connection that would not collect the cement would have been more satisfactory.

4. A more efficient method of heating materials on the plant would have been more economical and satisfactory than using one of the towboats for supplying steam and hot water. Each barge should be equipped with an oil-burning boiler of 75 to 90 horsepower for economical and safe winter operation. This addition would apply in colder climates and would apply in the Tennessee Valley if continuous placing of concrete during winter weather is in any way essential.

DESIGN OF CONCRETE MIX

A number of different mixes were considered before any mixes were finally adopted. A study was first made of the proportions used in structures similar to Wheeler Dam in which aggregates had been obtained from the same source. These studies showed that the mass concrete in Wilson Dam varied in cement content from a minimum of five sacks to about six sacks per cubic yard, and a half a bag of hydrated lime per cubic yard had also been added. The aggregate had been separated into two sizes, sand and gravel. The sand had been graded from 0 to No. 4, and the gravel from No. 4 to 3 inches, with a considerable amount of aggregate coarser than 3 inches at certain times. The specification for the quantity of water in the mix was as follows:

> The quantity of water for each batch will be that minimum quantity required by the standard quaking test for freshly mixed concrete. On this work this quake test will require that men dumping concrete buckets do not track deeper than 10 inches or less than 2 inches in freshly mixed concrete.

This specification indicates that a slump of 3 to 6 inches was used. The average compressive strength at 28 days for the five-bag mix was about 2,000 pounds per square inch and about 2,500 pounds per square inch for the six-bag mix. Results of special tests made at the time indicated that a water-cement ratio of 1.00 was required for a 2,000-pound strength and a water-cement ratio of 0.90 which gave a strength of 2,500 pounds per square inch at 28 days. These results are in accord with strengths obtained with these water-cement ratios and cement used at that time (1921–26).

Studies of the concrete placed in the lock at Wheeler Dam (1933–34) showed that the minimum cement content was five bags per cubic yard of concrete and the average water-cement ratio was 0.85 by volume. The slump ranged from 2 to 3 inches. A strength of 4,000 pounds per square inch was obtained in 28 days. The aggregate was separated into two sizes, sand and gravel, the sand being graded from 0 to No. 4 and the gravel from No. 4 to 3 inches.

Several tons of the aggregate were sent to the United States Bureau of Reclamation laboratory, where a series of complete tests were made on the material. The final results of these tests were not available, however, until several months after actual concrete placing had started at the project. In the meantime a series of tests was made in the laboratory at Wheeler Dam and a number of mixes designed for use in the work. The mixes were designed by trial, and, when the results[15] of the Bureau of Reclamation tests were available several months later, it was found that the greatest difference in cement content between any two mixes of the two groups was not more than 0.03 barrel per cubic yard.

In designing the mixes the following factors were considered:

1. The strength of portland cement had been increased since the time of the building of Wilson Dam. For a given water-cement ratio and cement content, the strength of concrete had increased 50 percent. This improvement in strength, however, did not necessarily mean that all of the other qualities, such as durability, watertightness, and wear, had been increased in the same proportion. Therefore it was decided that although the cement content could be reduced somewhat below that used at Wilson Dam it should not be reduced to give a minimum strength of less than 3,000 pounds per square inch.

2. The finer grinding of portland cement compared with that used 10 years previously produced more workable mixes, which permitted the reduction of the cement content without affecting the workability.

3. The use of vibrators in placing concrete permitted the use of drier mixes with a reduction of cement content without an increase of the water-cement ratio.

4. The high absorption, low specific gravity, and generally poor appearance of the coarse aggregate did not favor using a very low cement factor until the results of the durability tests made by the United States Bureau of Reclamation were known.

As a result of the above studies, it was decided to use a minimum cement content of 1.20 barrels per cubic yard for the mass concrete until the durability studies gave some indication of the quality of the concrete being produced. This cement content produced a workable mix which could easily be placed by use of vibrators.

The results of the durability or freezing and thawing tests made by the United States Bureau of Reclamation indicated that a high-quality concrete could be made with the available aggregates in spite of their seemingly bad characteristics. The cement content was therefore reduced several months later to about 1.15 barrels for mass concrete without appreciably increasing the water-cement ratio

[15] Vidal, E. N., and Meissner, H. S., *Concrete Investigations for Wheeler Dam, Tennessee Valley Authority*, February 25, 1935, U. S. Bureau of Reclamation Technical Memorandum No. 440.

or lowering the strength. Again, several months later, the cement content was reduced to about 1.10 barrels per cubic yard. This was accomplished by increasing the speed of the vibrators from 3,600 to 4,080 revolutions per minute.[16] This permitted the placing of drier, harsher mixes by the addition of more aggregates to the mix without increasing the water-cement ratio.

Although specifications permitted the use of a maximum water-cement ratio of 0.90 by volume (0.60 by weight), it was decided to design all mixes for a somewhat lower water-cement ratio in order to allow for inconsistencies in the mix such as variations in strength between different brands of cement and different lots of the same brand of cement, and also variations in grading of sand. Reducing the cement content below 1.10 barrels and increasing the water-cement ratio occasionally caused additional free water to collect on the surface of the concrete during placing. The actual water-cement ratio averaged about 0.80 by volume and produced concrete that was both impermeable and durable.

The water-cement ratio was lower than average for concrete of this type, while the cement content was less than that used at either Wheeler lock or Wilson Dam, where the same type of aggregate was used. The average strength, on the other hand, was practically double that obtained at Wilson Dam and from 10 to 20 percent higher than obtained at the Wheeler lock.

Aggregate grading.

The coarse aggregate used in the production of concrete was graded as follows:

Size of aggregate	*Grading limits (percent by weight)*
No. 4 to ¾ inch:	
No. 4	0–10
¾ inch	95–100
¾ inch to 1½ inches:	
¾ inch	0–10
1½ inches	95–100
1½ inches to 3 inches:	
1½ inches	0–5
3 inches	100

These limits were used as a guide and were followed as closely as practicable. If material varied greatly from the limits set so that additional cement was required to produce workable concrete, it was rejected.

In order to meet the specifications on sand and No. 4 to ¾-inch coarse aggregate, a large percentage of No. 4 to ⅜-inch material was wasted into the river during dredging operations. The uniformity of the small sizes of coarse aggregate was maintained closely because variations in the small sizes had a much greater effect on the uniformity of the concrete than did variations in the larger sizes.

An accurate record of the amount of each size produced during dredging operation was made so that as much of the material as was practicable would be used in the various mixes. For this reason the proportions of the various sizes of coarse aggregates used in the

[16] See "Electric concrete vibrators," p. 189.

mix would change from time to time. Certain limits, however, could not be exceeded without increasing the cost of the concrete. For example, an excess of No. 4 to 3/4-inch material in the mix used in mass concrete required the use of additional cement to maintain the water-cement ratio at the proper limit; a reduction of 1½- to 3-inch coarse aggregate in the same mix had the same effect.

The grading of the sand could not be closely controlled, and, therefore, two or three alternate mixes were designed to allow for variations in the grading of the sand. The mix to be used was determined by the mixing plant inspector who obtained his information on the grading of the sand from a tag placed on each barge by the aggregate inspector at the dredge. This tag indicated whether the sand was "fine," "average," or "coarse." The plant inspector then used the proportion called for on the "mix sheet" for the particular grading of sand being used. In general, the finer the sand the lower the proportion of sand used in the mix. The concrete used in mass work was designed for the three grades of sand. The cement content and water-cement ratio were the same for all three mixes. This method also permitted the control of workability within a reasonable limit.

With no aggregate storage capacity at the project other than the material contained in the barges, a shortage of certain sizes of coarse aggregates existed at various stages of the work. This shortage would sometimes last for several hours, and in order that the work might continue without delay it was necessary to use mixes lacking one or two sizes of coarse aggregates. This particular condition called for special mixes designed to take care of the situation. In general, the same or a slightly greater cement content (about 0.06 barrel additional) was used to maintain the required workability and water-cement ratio. When the above condition existed, 3-inch aggregate was used without 1½- or 3/4-inch aggregate or both, while the 1½-inch aggregate was used without 3/4-inch coarse aggregate in the mix.

Consequently the mixes used in various parts of the work were constantly being changed; but in spite of these changes the quality of the concrete was uniformly high.

Concrete testing.

Testing was carried on in the main laboratory located on the left bank, in two small field portable laboratories located on a cofferdam, and at the dredge located about 50 miles from the project. Concrete samples were taken for approximately each 350 to 400 cubic yards of concrete placed. Three 6- by 12-inch cylinders were made from each sample, one for testing at 7 days and one or two for testing at 28 days. A specimen was made from each second or third sample for testing at 3, 6, or 12 months. The specimens were molded on the mixer barges and removed to the laboratory approximately 24 hours after molding. The results of tests made with a 300,000-pound hydraulic compression testing machine are summarized in table 21.

TABLE 21.—*Summary of concrete tests Aug. 20, 1934, to Mar. 1, 1936*

Month	Number of tests	Cubic yards placed	Average cement per cubic yard, barrels		W/c by volume		Average F. M. of sand	Compressive strength—pounds per square inch				
			Mass	Powerhouse and piers	Mass	Powerhouse and piers		7 days	28 days	3 months	6 months	1 year
1934												
August	18	1,407	1.19		0.83		2.96	3,030	4,830	5,190		
September	150	19,133	1.21	1.37	.80	0.80	2.91	2,920	4,810	5,330	5,860	6,330
October	240	29,519	1.21	1.38	.79	.78	3.04	2,600	4,080	4,700	5,580	5,600
November	282	37,382	1.20	1.35	.77	.76	3.00	2,920	4,750	5,970	5,980	6,660
December	282	34,914	1.22	1.41	.75	.77	2.92	3,220	5,000	5,930	6,250	6,280
1935												
January	272	34,880	1.19	1.36	.76	.74	2.74	2,890	4,700	5,380	5,680	6,430
February	228	19,916	1.19		.78		2.73	2,900	4,960	5,430	5,520	5,950
March	237	21,555	1.19	1.30	.78	.78	2.91	2,870	4,650	5,500	5,870	6,210
April	312	36,760	1.16	1.36	.78	.77	3.09	2,720	4,810	5,650	5,990	6,780
May	390	52,161	1.14	1.36	.79	.80	2.99	2,830	4,560	5,630	6,270	6,520
June	441	54,906	1.14	1.35	.80	.84	2.88	2,570	4,530	5,700	5,910	6,030
July	333	33,720	1.16	1.34	.80	.80	2.86	2,690	4,410	5,560	5,830	6,260
August	477	53,677	1.14	1.34	.80	.81	2.87	2,720	4,350	5,590	5,910	5,990
September	294	33,610	1.14	1.32	.81	.80	3.11	2,930	4,830	5,600	5,900	6,250
October	141	16,032	1.14	1.39	.80	.80	3.21	3,180	5,040	6,170	6,250	6,250
November	327	45,167	1.12	1.41	.79	.75	3.08	2,960	5,030	5,750	6,020	6,550
December	306	42,197	1.10	1.34	.78	.75	3.08	2,870	4,890	5,840	6,170	6,200
1936												
January	222	27,063	1.12	1.61	.79	.65	3.07	2,640	4,720	4,800	5,900	6,060
February	126	19,965	1.27	1.34	.74	.78	2.95	3,500	5,350	6,450	7,150	6,790
Average			1.17	1.37	.78	.78	2.97	2,890	4,750	5,590	6,000	6,300
Number of tests[1]								1,693	2,790	205	195	196

[1] Total number of tests during above period: concrete tests 5,079, aggregate tests 4,040.

Total cubic yards concrete placed during this period: 613,964.

Compression tests of 6- by 12-inch concrete cylinders. Specimens tested in the field and then removed to the laboratory approximately 24 hours after molding and cured in water at 70° until tested. Tested damp. Eight brands of modified portland cement were used. The brand of cement was changed approximately every 15 days. One set of three cylinders was made for each 350 cubic yards of concrete placed. Thirty-two different inspectors were used in making test cylinders.

TABLE 22.—*Typical mixes for dam and powerhouse concrete*

Location	Mix	Slump, inches	Quantities for 1 cubic yard				
			Cement, barrels	Sand, tons	¾ inch gravel, tons	1½ inch gravel, tons	3 inch gravel, tons
Highway bridge	1-1.9-2.7	2-3	1.72	0.614	0.872		
South abutment	1-2.6-5.2	½-2	1.14	.550	.385	0.495	0.220
Nonoverflow mass	1-2.6-5.2	½-2	1.14	.550	.385	.495	.220
Nonoverflow piers	1-2.4-4.2	2-3	1.28	.575	.475	.535	
Overflow mass	1-2.6-5.2	½-2	1.14	.550	.385	.495	.220
Overflow training walls	1-2.45-4.75	2-3	1.22	.550	.375	.428	.231
Overflow piers	1-2.6-4.0	3	1.28	.625	.450	.510	
Energy dissipators	1-2.65-4.55	3	1.22	.595	.358	.410	.255
Trashway	1-2.6-5.2	1-2	1.14	.550	.385	.495	.220
Intake	1-2.45-3.6	6	1.36	.577	.457	.567	
P. H. sub.—Draft tube and gantry deck	1-2.45-4.75	2-3	1.22	.595	.358	.460	.205
P. H. sub.—Elbow section	1-2.25-3.8	4-6	1.36	.578	.458	.518	
Recess lining for future units	1-2.3-4.0	3-4	1.32	.573	.468	.528	
"C" and "B" line walls and transfer deck	1-2.25-3.8	4-6	1.36	.578	.455	.518	
Retaining wall units 2 and 8	1-2.3-4.0	3-4	1.32	.573	.468	.528	
Tailrace slab and retaining wall	1-2.3-4.0	3-4	1.32	.573	.468	.528	
Service and contact bay below elevation 513	1-2.25-3.8	4-6	1.36	.578	.458	.518	
P. H. sub—Equipment foundations	1-2.25-3.5	1-2	1.38	.648	.905		
P. H. superstructure	1-2.25-3.8	4-6	1.36	.577	.458	.517	
P. H. sub.—Unit No. 2	1-2.1-3.0	4-6	1.56	.613	.413	.465	
Station filter plant	1-2.2-2.4	4-6	1.68	.695	.760		

The average water-cement ratio used for the mixes shown above was 0.52 by weight with the exception of the bridge concrete and a few small pours where greater durability and strength were desired and a lower water-cement ratio was used.

The changes in the mass concrete mixes to take advantage of the modified conditions were as follows:

Period mixes were used	Typical mixes by weight	Slump, inches	Water-cement ratio by weight	Barrel cement per cubic yard
From beginning of work until aggregate tests were available	1-2. 4-4. 8	1-2½	0. 52	1. 20
After aggregate tests were available until vibrator speed was increased	1-2. 6-5. 2	1-2	. 52	1. 15
After vibrator speed was increased from 3,600 to 4,080 revolutions per minute	1-2. 7-5. 5	½-1	. 52	1. 10

Because of an increase in water gain with the 1.10-barrel mix during cold weather, the cement content was increased near the end of the job in an effort to cut down this free water.

Table 22 gives a list of mixes used for the dam and powerhouse. These are typical but include only a portion of the total number used. As previously stated, constantly changing conditions in the materials, placing methods, and other factors required a constant redesign of the mixes being used.

The average water-cement ratio for the mass concrete was practically the same as that for the reinforced concrete, which contained smaller maximum size aggregates and more cement. The variations in water and cement were somewhat greater for the reinforced concrete, however, because certain sections were purposely made stronger.

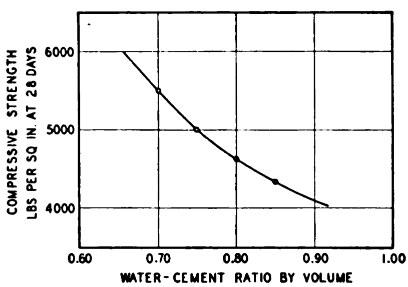


FIGURE 84.—*Water-cement ratio—strength relation.*

The curve in figure 84 shows the average water-cement ratio strength relationship for the entire job and includes tests from both mass and reinforced concrete made with the eight brands of type B, or modified portland cement, used on the project. It indicates that the 28-day concrete strengths obtained with modified portland cement are equal to or slightly greater than strengths obtained with the majority of regular or standard portland cements. This factor, together with the lower heat of hydration and temperature rise, made it an excellent cement for use in mass and most other types of concrete. The average age-strength relation for all concrete used on the project is shown in figure 85. The increase in strength with age is about the same as that obtained with the normal portland cement of the same water-cement ratio.

Figure 86 shows the monthly age-strength relation. While the later ages show the same general trend as the 7- and 28-day tests, this trend is not as marked as the relation between the 7- and 28-day tests, because of the considerably fewer number of tests made at the later ages.

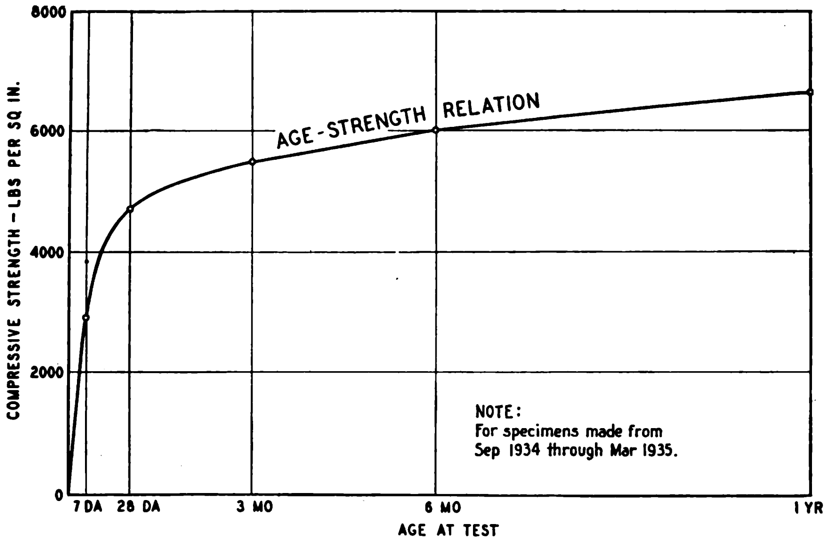


Figure 85.—*Age-strength relation of Wheeler concrete.*

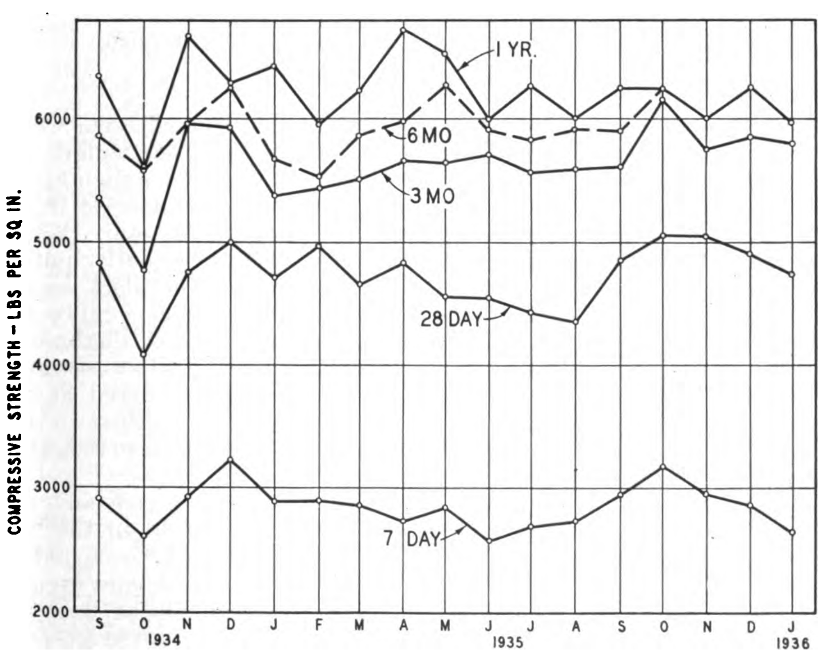


Figure 86.—*Monthly age-strength relation of concrete.*

The data plotted on figure 87 show the relationship between the mortar and the concrete strength for the various brands of cement used on the project. The mortar tests were made in the Authority's laboratory at Knoxville, while the concrete tests were the regular field tests made from samples taken on the job. The mortar tests were made on 2-inch cubes and 2- by 4-inch cylinders using a 1:2.88 graded Ottawa sand-mortar plastic mix, while the concrete tests were made on 6- by 12-inch cylinders moulded in the field.

The concrete curves for both the 7- and 28-day tests followed the same general trend as the 7-day mortar test. The curves also indicated that the concrete strength could be predicted with a fair degree of accuracy from the results of the cement-mortar strength tests. The 7-day mortar strengths showed a range from 1,600 to 3,200 pounds per square inch, while the 7-day concrete strengths ranged from 2,100 to 3,300 pounds per square inch. Although the cement tests showed a considerable range in strength, all of the values were well above the minimum specified strength of 1,500 pounds per square inch at 7 days and 750 pounds per square inch at 3 days.

In general, strengths obtained from different lots of the same brand of cement checked reasonably well, but several brands varied more than the average. This variation is shown in figure 88, which shows the water-cement ratio–strength relation for different lots of the same brand of cement.

Numerous special tests were made in the laboratory to determine the quality and characteristics of the concrete, cement, and aggregates. These included permeability tests, soundness tests of coarse aggregates by use of sodium sulphate, and concrete temperature tests, all of which are given in detail in appendix C to this report.

Cracking of concrete.

The three common types of cracks occurring in the concrete were:

1. Cracks caused by width or length of section, particularly in mass concrete where the outside cooled considerably faster than inside of the block, was due to stresses set up within the concrete itself. These cracks occurred at right angles to the axis of the dam. While unsightly in the early stages of the job, they eventually pulled together or healed as the concrete at the center of the block cooled and the temperature became more uniform throughout. Since the filling of the lake, no leakage has been observed through such cracks. Cracks of this type do not occur in the 15-foot sections. In sections up to 30 feet wide, a crack generally occurred at the center of the block. In blocks 45 feet or more in width, two and sometimes more cracks occurred. Where two cracks occurred, they were generally located near the third points.

2. The second type of cracks occurred in mass concrete and ran parallel to the axis of the dam. These cracks occurred in the vertical keys on the inside face of the blocks. The cracks occurred in blocks of all widths and as all of them were on the inside and eventually were covered with concrete of the intervening blocks it is not possible to tell whether they have all closed.

3. The third and most troublesome type of cracks occurred in the arched roof of the inspection gallery and at corners of openings, such as doors and windows. These cracks occurred in mass sections

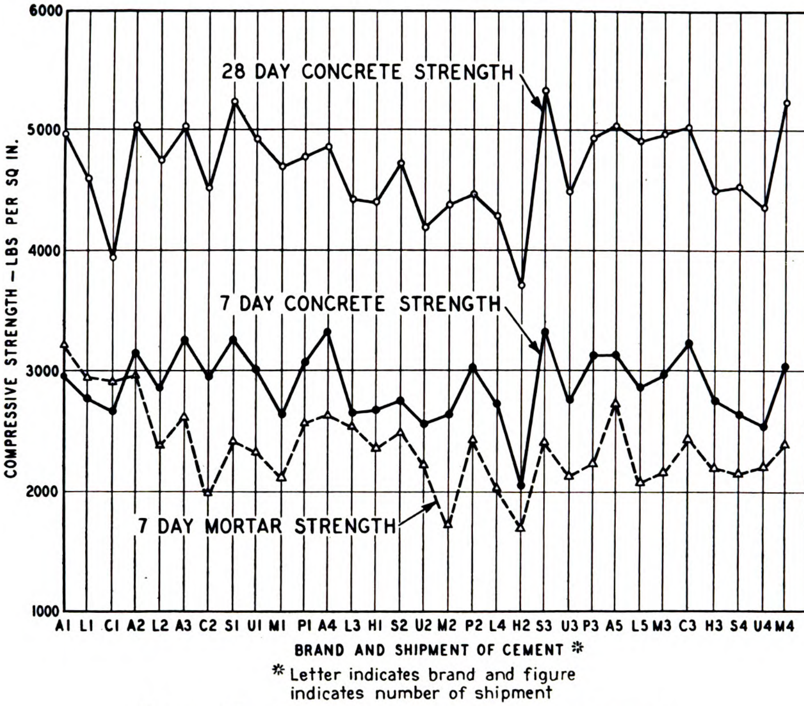


FIGURE 87.—*Comparative mortar and concrete strengths.*

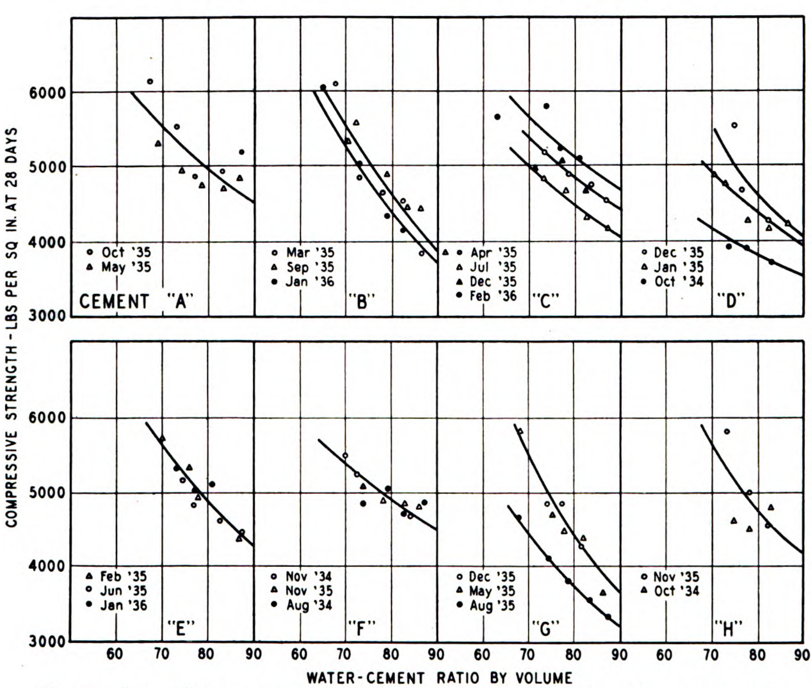


FIGURE 88.—*Water-cement ratio—strength relationships for eight commercial brands of cement.*

and thin sections alike. In the thin sections, they are no doubt due to shrinkage caused by drying out after the curing period had been completed. Adequate reinforcing at these points, and also a loosening of the forms as soon as possible after placing, will no doubt tend to cut down this type of cracking.

Factors tending to reduce shrinkage or cracking in concrete are: low cement content, low water-cement ratio, low heat cement, and considerable curing. At Wheeler all of these factors were taken into consideration and no doubt tended to minimize the cracking.

Concrete costs.

The quantities and costs of forms, reinforcing steel, and concrete used in various parts of the dam and powerhouse are summarized in table 23, and quantities and costs of concrete mixing and placing are shown in figure 89.

TABLE 23.—*Concrete quantities and costs*

	Quantity			Cost			
	Concrete	Forms [1]	Reinforced steel [1]	Concrete	Forms	Reinforced steel	Total cost
	Cubic yards	*Square feet*	*Pounds*	*Cubic yards*	*Square feet*	*Pounds*	*Cubic yards*
South abutment	4,117	1.560		9.16	0.88		$10.54
Overflow mass	274,265	2.995	1.369	6.56	.69	0.060	8.70
Nonoverflow mass	179,278	3.209	0.879	6.39	.55	.068	8.21
Trashways	16,292	3.722	4.562	6.52	.54	.072	8.84
Energy dissipators	7,130	6.285	17.142	6.72	.55	.035	10.75
Substructure (powerhouse)	58,704	7.491	77.962	9.39	1.23	.046	22.20
Intake structure	57,127	7.318	146.346	7.86	1.25	.039	22.68
Training walls	3,928	8.598	110.657	6.47	0.60	.034	15.43
Overflow piers	6,828	18.106	82.241	10.25	1.31	.066	39.46
Nonoverflow piers	2,172	25.054	191.409	16.32	1.46	.055	63.41
Highway bridge	5,456	36.445	206.503	13.51	0.49	.040	39.81
Superstructure (powerhouse)	3,940	41.753	152.619	21.46	.66	.053	57.02
Total	619,237						
Average		4.750	27.322	7.14	0.84	0.043	$12.32

[1] Per cubic yard of concrete.

NOTE.—The above costs include all field charges including depreciation.

These costs include all equipment operations such as mixing plants, vibrators, transfer cranes, and other transfer facilities, and also cost of aggregates, cement, reinforcing steel, formwork, and all other items chargeable to formwork and concreting. The cost of the individual equipment operations making up the entire concreting operation are included under those sections of this report dealing with such equipment operations.

CONCRETE PLACING

Major equipment used in the placing of concrete consisted of revolving cranes, a concrete pump, concrete buckets, concrete buggies, and electric concrete vibrators.

Revolving cranes.

One American revolver and six Clyde Wiley cranes were used to place the 631,150 cubic yards of concrete (exclusive of the lock,

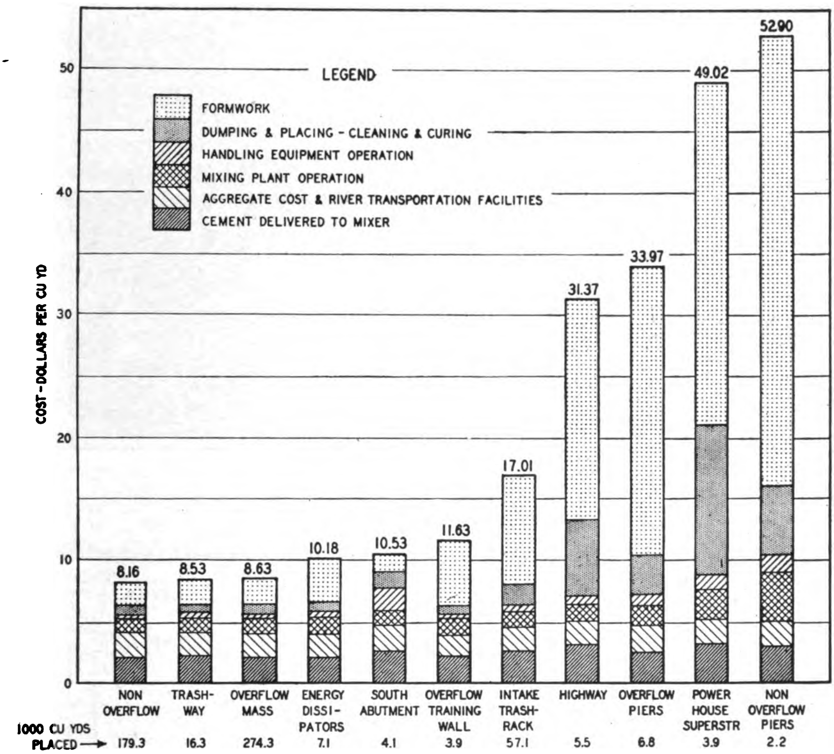


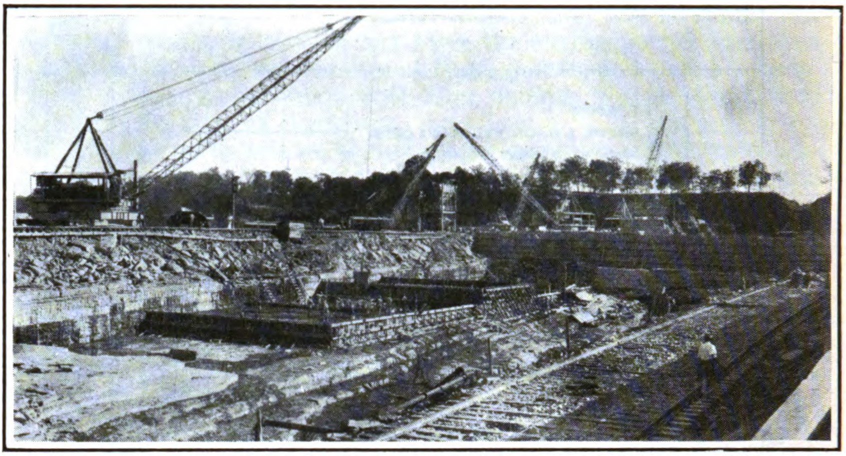
FIGURE 89.—*Summary of average unit concrete costs (cost does not include reinforcing steel—see table 23).*

built by the U. S. Army Engineers) and to handle the major portion of reinforcing steel, form lumber, and miscellaneous materials.

The Clyde Wiley cranes cost approximately $18,500 each, including three transformers per unit, and erection. In addition five gantries, four propelling devices, and a spare boom were purchased. Each unit was equipped with a 95-foot steel boom and a 150-horsepower, three-drum main hoist. The cab was supported on a 17-foot-square box frame with a 17-foot-diameter rail circle. Swinging was done by a 40-horsepower electric motor. Gantries for these machines were 16 by 16 by 14½ feet high. Propelling units were added after the first year of operation. They consisted of two 7½-horsepower motors connected to the gantry trucks and permitted a traveling speed of about 110 feet per minute.

One American revolver crane was taken from a mixer and was equipped with a 95-foot boom to operate with the Clyde Wileys. It worked about 5 months and operated on trucks without gantry or propelling mechanism.

All cranes operated on 16-foot-gage track which ran parallel to the axis of the dam. One track was located on the floor and about 10 to 20 feet inside the upstream side of all cofferdam areas except No. 2, where the track ran along the top of the cofferdam. On the downstream side, except in a portion of No. 1 cofferdam, another track ran



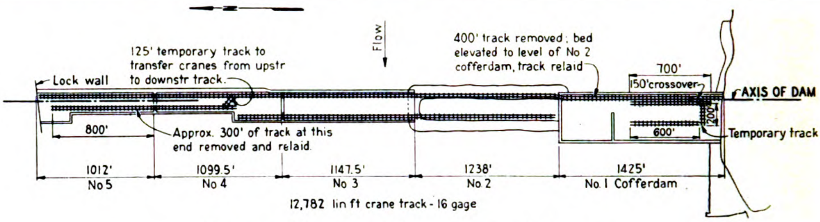


FIGURE 90.—*Revolving crane track layout.*

the full length of the dam. A total of 12,782 linear feet of 16-foot-gage track was used.

Five Clyde Wileys were mounted on gantries and were able to reach the roadway on the dam except on the viaduct ramp for which a 20-foot section was added to one Clyde Wiley boom for work at this point.

Most of the work performed by these cranes was the transfer of concrete from the floating mixer plant to the forms, a single crane with a 2-cubic-yard bottom-dump bucket usually serving each plant. Generally the crane could be so located that concrete could be placed with a single swing of the boom with little or no traveling. The mixing cycle permitted the crane to place its load of concrete and swing back in time to receive the next batch of concrete without delaying the mixer.

Other materials brought to the cofferdams by barge were also unloaded by these cranes. Some of the excavation was handled by skip pans which were picked up by crane and dumped into trucks.

The normal operating crew [17] of each crane consisted of:

	Rate of pay per hour
One operator	$1.50
One oiler	.75
One flagman	.60
One electrician's helper	.60
	3.45

[17] See table 36, pp. 353–354, for changes in rates of pay during the job.

During the first year of operation, the cranes were moved by boom and cable. For this work a rigger at $1 per hour was added to the regular crew.

Complete individual production records for each crane were not kept, but the group placed a total of 631,150 cubic yards of concrete at an average rate of 37.1 cubic yards per net hour. Approximately 34,000 cubic yards of clean-up rock excavation were handled. Tonnage of other materials handled was not reported, but most of the 8,000 tons of reinforcing steel used, as well as the major portion of form lumber, other construction machinery, and miscellaneous materials, were handled by the cranes.

During September 1935 a study was made of the operation of Clyde Wiley cranes in the placing of concrete. It was found that the crane cycle was considerably faster than the mixer cycle, and the cranes were never extended to full capacity—an average of 20.5 percent of the cycle being spent waiting at the mixer. A summary of the study is shown in table 24.

Operating costs from August 1934 to May 1936 amounted to $12.029 per net hour. These costs include labor, power, repairs, depreciation, and track charges. Track costs include depreciation, installation, and maintenance charges and amounted to $7.15 per linear foot of 16-foot-gage track.

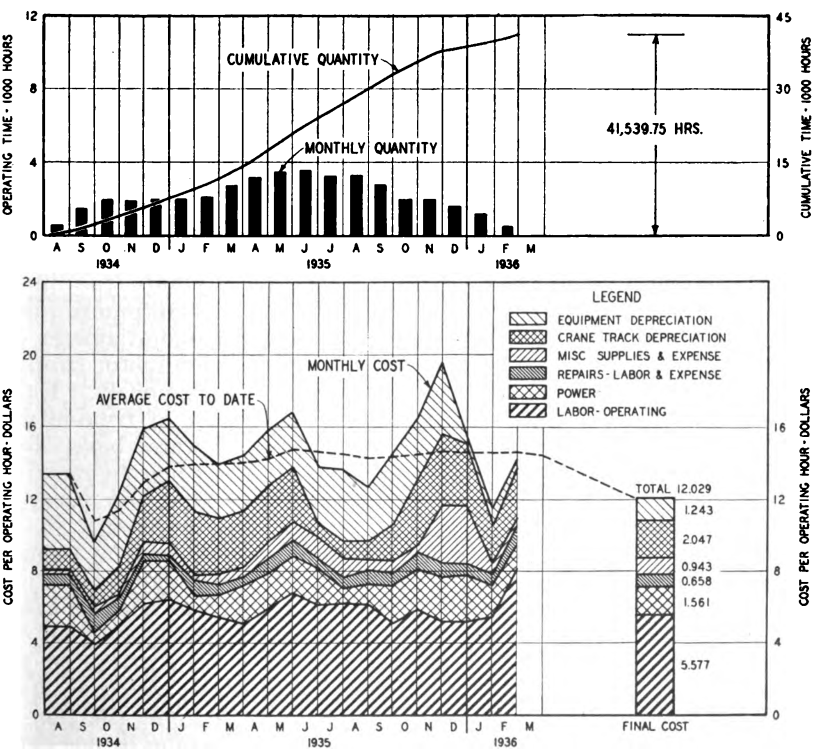


FIGURE 91.—*Average operation costs for seven revolving cranes.*

TABLE 24.—*Summary—whirley crane swing studies*

Number of swings	Difference in elevation	Swing angle, degrees	Boom angle, degrees				Time in seconds						
			At mixer	At forms			Filling bucket	Swinging to form	Spotting bucket	Dump bucket	Return to mixer	Waiting at mixer	Total
10	10.1	155	35	60	-----	-----	16.0	29.0	18.5	11.0	26.0	24.5	125.0
7	10.1	180	50	70	-----	-----	17.5	37.0	10.5	16.0	26.5	21.0	128.5
31	35.9	180	45	70	-----	-----	14.5	29.5	14.5	12.5	25.5	28.5	125.0
18	10.7	180	30	40	55	-----	13.5	40.5	26.5	13.0	27.5	13.5	134.5
26	−11.2	160	40	45	75	-----	15.0	32.5	10.0	4.5	25.5	37.5	125.0
15	−16.9	180	30	45	60	80	15.0	33.5	20.0	6.5	31.0	19.0	125.0
Average time							14.9	33.1	16.3	9.9	26.7	26.0	126.9
Percent of total							11.7	26.1	12.8	7.8	21.1	20.5	100.0

Study made: Clyde Wiley "J," Aug. 22, 1935, to Sept. 20, 1935, inclusive; Clyde Wiley "H," Sept. 25, 1935—Type of work: Placing concrete—Length of boom: 95 feet—Average elevation of barge: 511.6—Average elevation of cofferdam: 520.0.

While these time studies give a true indication of the work, this information should not be relied on too greatly in arriving at a figure representing the capacity of this equipment. The crane cycle is considerably faster than the concrete mixer cycle, and at practically no time are the cranes pressed in any way by the mixers. Thus, there is no object in working the crane at top speed with the accompanying wear and tear on machinery unless required by mixer operation.

No major repairs were made on the American revolver crane. On the Clyde Wiley crane the major trouble was in the swing engine. The bearings on the driving worm shaft were apparently too light to withstand the strain occasioned in stopping the swing of the machine.

Pumpcrete machine.

A Rex dual pumpcrete machine, shown in figure 92, was obtained at a delivered cost of $17,668, including the necessary pipe and accessories. It was a model 200, double pumpcrete—rated capacity 50 cubic yards of concrete per hour—complete with a remixing hopper. Essentially, the unit consisted of two 8-inch single-acting concrete pumps with the remixing hopper mounted vertically over the two pumps and discharging through two outlets into the intake valve of each. Separate discharge pipes from each pump joined about 8 feet from the machine to form a single 7-inch-diameter discharge pipe to the forms. A 50-horsepower induction motor supplied power to drive both pumps which operated continuously. Either pump could operate independently, however, in case of trouble with the other. The paddles in the remixing hopper were driven by a 10-horsepower induction motor to insure that the concrete remained well mixed as it entered the pump. With 7-inch-diameter pipe, the machine could handle 2½-inch maximum size aggregate.

From the machine the concrete was transported by the transfer pipe to the point of placement where it was discharged through various distributing arrangements or into a hopper for further transfer to buggies. Pipe sections varied in length from 1 to 10 feet and were easily attached or removed by means of a patented coupling device. When pumping concrete was finished, both the pumpcrete and pipe line were flushed clean by washing out the remixer hopper and pump chambers and by pumping a cleaning device called a "go-devil"[18] through the pipe with air or water to force the re-

[18] Two metal disks with rubber cups attached, joined together by a short rod.

mainder of the concrete out of the pipe. Concrete was transferred by whirley crane from the mixing plant to the pumpcrete, where it was placed in the remixing hopper of the machine.

The pumpcrete unit was purchased because the shortage of cranes and the necessity of placing concrete at times when the cranes were needed elsewhere made it advisable to keep up with the concreting schedule by some other method. In addition, there were many small blocks of concrete which were out of reach of the cranes and for which some other method of placement had to be used. With the pumpcrete located near the cofferdam where concrete from the mixing plant could be easily transferred to the pump hopper by crane, running a pipe line to remote places around the job was in most cases a simple matter.

The pumpcrete method of placing concrete was used only in the powerhouse and south abutment, where it placed about 21 percent of the total concrete in that area. Figure 92 shows the main area of placement but does not include the control building or keyway in the south abutment. The majority of pours were made from two locations on the cofferdam as indicated.

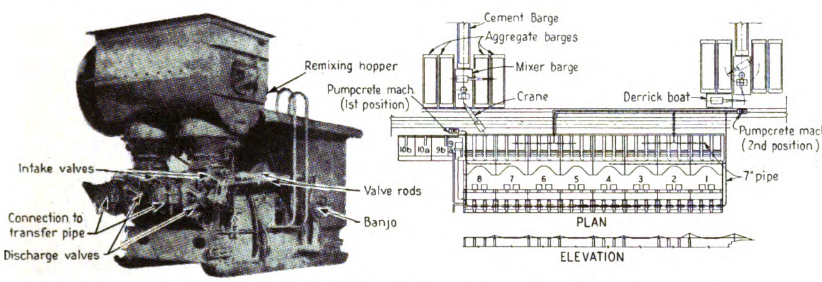


FIGURE 92.—*Pumpcrete machine.*

The operating costs of the crane and supporting equipment amounted to about 23 percent of the total operating cost of the pumpcrete machine. In the original plans for the use of this machine a tentative set-up proposed placing the pumpcrete machine on the deck of a mixer barge so as to be fed directly from the mixer bowl. This scheme involved a change in the construction of a mixer plant and required the use of a flexible pipe connection to the shore and was never carried out.

Before the most efficient method of placing concrete with the pumpcrete machine was developed, several different methods were tried as follows:

1. Pumping through the pipe line, which was carried 20 to 30 feet over the form and turned down into the form through 90° bends, and discharging near the surface of the concrete.
2. Discharging from the pipe line into elephant-trunk chutes, varying in length up to 30 feet, the lower end being moved about in the form.
3. Discharging from the pipe line into a receiving hopper and using open metal-lined wooden chutes for distribution in the form.
4. Discharging from the pipe line into a receiving hopper and using concrete buggies for distribution.

The first method caused a great many pipe changes and also resulted in some segregation of concrete ingredients because of the long vertical drop through the pipes. A great deal of such segregation also resulted in the second method of placement. The mortar flowed slowly along the lower side of the chutes, while the gravel flowed more freely and was entirely separated. This method required considerable mixing and spading in the forms in addition to many changes in the pipe line. Method No. 3 proved more satisfactory; segregation was practically eliminated if the chutes were not too steep. Considerable chute changing was necessary, however, and resulted in much lost time. An output of about 40 cubic yards per net hour was attained with this method of distribution. The best output was obtained by method No. 4 in which the pipe could discharge continuously into a receiving hopper, it being necessary only to have enough buggies on hand to take care of the output of the mixer. About 50 cubic yards of concrete per net hour could be placed in this way since no pipe changes were necessary and very little time was lost.

Pours were made with the machine at irregular intervals. The operating crew was assigned to miscellaneous tasks about the job when pumping was not in progress. The regular crew varied in number from a minimum of three to a maximum of about six, depending on pumping conditions. Although the construction of the transport pipe was such that pipe changes could be made by laborers as needed, labor union requirements caused this to be done by pipe fitters, which resulted in a somewhat higher cost for this work.

A well-trained crew of operators and maintenance men was necessary for continuous operation. The plant had to be operated by men who were able to detect signs of impending trouble. It was necessary for pump operators to keep in touch with the mixing plant and to know at all times what kind of concrete was being made and when it would arrive at the pump. The majority of operating delays was due to mechanical troubles and was attributed to the inexperience of operating crews.

During the period of use the machine poured 26,461 cubic yards of concrete at a gross pumping rate of 39.3 cubic yards per hour. The largest pour was 980 cubic yards, and the highest rate of production for any one pour was 51.7 cubic yards per gross hour. The average length of equivalent straight pipe line per hour was 423 feet. The quality of the concrete placed by the pumpcrete was probably about equal to that placed by other methods. The strength at 28 days was as follows:

	Pounds per square inch
Maximum	6,070
Minimum	2,940
Average	4,568

Delays averaged 14.2 percent of gross operating time. Nearly all of the mechanical difficulties may be traced to three causes: inexperienced operators, light design of the machine, and inability of the machine to handle the 3-inch aggregate which was used at first.

The following table shows pumpcrete operating costs for the period of March to October 1935, during which time 26,461 cubic yards were placed. These costs include operating labor, power charges, repair and maintenance, scaffolding and pipe changes, miscellaneous supplies, direct charges, transportation, and depreciation.

	Per cubic yard
Labor	$0.233
Power	.027
Miscellaneous supplies and expense	.025
Repairs	.182
Plant depreciation	.418
Equipment depreciation	.604
Small tools, direct charges	.016
Supporting equipment (mixer to machine)	.310
Total cost of placing	1.815

Concrete buckets.

Several different types of buckets were used for transferring mixed concrete from the floating mixing plants to the forms. Except in small forms or where a very slow pouring rate was being used, these buckets were of 2-cubic-yard capacity. In the exceptional cases mentioned, a bucket of 1-cubic-yard capacity was used. The following types of buckets were used:

Make	Number used	Size, cubic yards	Average cubic yards placed per bucket
Stuebner	2	1	
Insley	9	2	1,100
Unionloc	7	2	22,000
Blaw-Knox	5	2	82,000
Dravo	1	2	48,000

The Stuebner and Insley type buckets were procured second-hand from Wilson Dam, where they had been used several years before in the construction of that project. Their use was limited to a small yardage poured during the early construction stages.

Prior to February 12, 1935, about 150,000 cubic yards of concrete were placed by the seven Unionloc two-bail buckets. Slightly sloping sides and large doors permitted quick dumping. They could be dumped easily by the crane operator by releasing the line holding the dumping bail, and when the doors opened they swung clear of the falling concrete. For pouring concrete under water, this was an excellent type bucket. Several accidents resulted, however, from dumping these buckets too early, and they were ultimately discarded because of this danger. Cost of maintenance was low. The Unionloc buckets were best suited for open forms where it was not necessary to control the flow of concrete from the bucket. Leakage around the doors when transporting grout was remedied by mixing pea gravel in the grout.

In contrast to the square bucket, the Blaw-Knox buckets are round with the bottom of the hopper shaped like an inverted, truncated cone. The gate consists of a rubber apron supported on roller bearings. The dumping levers are manually operated, and the flow of the concrete from the bucket is variable. Concrete worked past the rubber apron and into the roller bearings causing undue wear and delay and made the dumping of the buckets more difficult. One

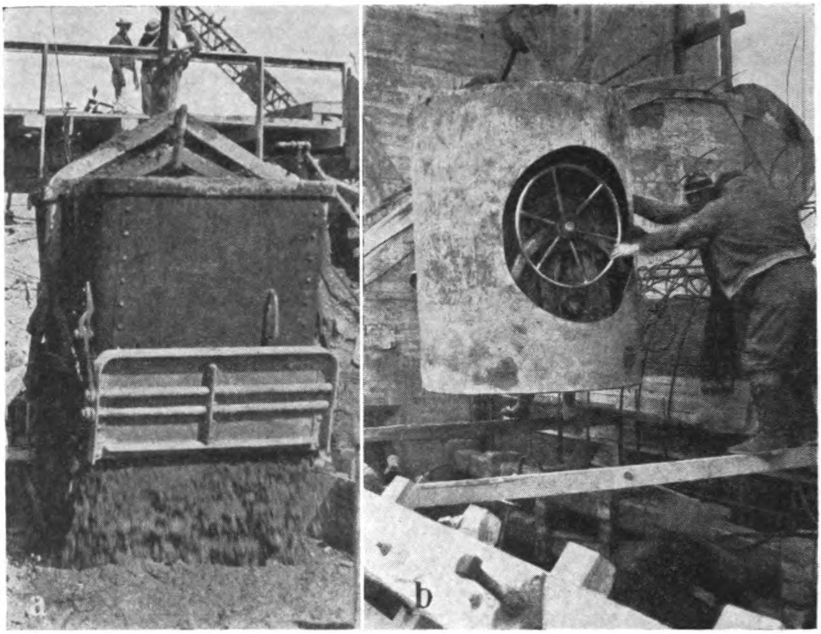


FIGURE 93.—*Buckets for concrete, (a) Unionloc, (b) Dravo.*

man could dump the buckets easily when they were new, but most of the time 2 or more men were required. No leakage of wet concrete or grout was experienced with this type bucket. The amount of concrete was easily controllable, but the rubber apron remained in such position that the falling concrete striking upon it caused the bucket to move away from position over small forms. Some trouble was experienced in getting stiff concrete to pass through the dumping opening. Major repairs consisted of 21 conveyor belt aprons, 100 roller bearings, and 2,800 felt retaining washers.

In October 1935 a Dravo bucket was tried and worked so satisfactorily that it was purchased. It had the same general shape as the Blaw-Knox bucket but had a radial steel gate. It had no roller bearings, nor did it utilize the rubber apron for sealing the opening. Dumping was controlled by a wheel in the side of the shell instead of the levers. The only repairs necessary were the hard surfacing of the throat for a distance of 12 inches and the welding of a small angle. Low repair costs and ease of operation seemed to favor this bucket. No leakage was encountered, and the bucket did not move from over small forms when the concrete was being dumped. Handholds should have been provided in the shell near the bottom so this bucket could be more easily twisted into place. Some locking device should also have been placed on the dumping mechanism to eliminate the tendency of the bucket to open when filled with very wet concrete or grout.

Electric concrete vibrators.

Three different electric vibrators were used to consolidate the concrete: the flat or puddler type, the spade type, and the flexible-shaft type.

Four puddler-type vibrators were used. They consisted, fundamentally, of a motor mounted on an 18- by 36-inch float which received its vibration from an eccentrically placed weight on the motor shaft. They were used chiefly to tamp the surface of a completely filled form.

Twenty-five spade-type vibrators were used. They consisted of a motor mounted within a spade or bullet-shaped shell to which handles of various lengths were directly connected. The operator stood within the large open form and merely jammed the spade into the concrete. Vibration was likewise produced by an eccentrically located weight on the motor shaft.

Eight flexible-shaft units were used. In this type the motor was separate from the vibrating section and was connected by a flexible cable approximately 21 feet in length. The vibrator section was a bullet-shaped shell with an eccentrically placed weight within. This type was used in deep forms too narrow for an operator to enter. While standing near the top of the form, the operator directed the vibrating section by means of the cable housing into the desired portions of the concrete mass and controlled the power.

All of the three types were 110-volt, 3-phase, 60-cycle machines. During the latter part of the job, frequency changers were installed which were capable of increasing the frequency to 80 cycles. After study, however, a frequency of 68 cycles was chosen as the most desirable.

Vibrator speeds of 3,600 revolutions per minute were used in placing about 70 percent of the concrete. During the latter part of

FIGURE 94.—*Placing mass concrete.*

the job, the speed was increased to about 4,080 revolutions per minute by increasing the frequency, and this speed was used until the work was completed. It was found that the increase in speed allowed the placing of drier, larger mixes and lower cement content without affecting the water-cement ratio or ease of placing. This ultimate speed was selected more as a safe physical limit for the equipment rather than as the desired limit for concrete vibration.

Normally three vibrators were used to service the concrete placing requirements for each mixer plant, one extra vibrator being kept in the form as a stand-by unit. Two men at $0.55 per hour were required for each vibrator. All work other than actually operating the vibrators such as moving and relocating the lines, disconnecting and connecting the vibrators, was done by the electrical forces.

A rigid inspection and repair schedule was maintained. Each vibrator was brought to the shop during peak concrete placing periods after from 30 to 40 hours of service to be greased and inspected. Special attention was given to electrical connections to prevent wires breaking and causing an open circuit or single-phase operation which would seriously damage the motor. This was particularly true in the spade type, for at the flexible connection between the handle and the vibrator the wires were weakened from vibration. As these units are operated in wet concrete, care was exercised to prevent faulty insulation. The ends of the power cables were repaired promptly as necessary, and if the motor windings did not test properly, they were repaired before being returned to service.

The flexible-shaft vibrators required the most repair work—chiefly on the flexible-shaft and vibrator units. In terms of cost per vibrator per month, the flexible-shaft vibrators cost $120 for repairs and maintenance as compared to $27 for the spade- and puddler-type vibrators. Cost studies of vibrator operations indicate that the average cost was $0.209 per cubic yard of concrete. This includes depreciation, repairs, supplies, transportation, and pro rata of labor, power, and small tools.

The use of vibrators saved approximately one-half bag of cement per cubic yard of concrete. This figure is based on a comparison between lock concrete where vibrators were not used and the dam concrete where vibrators were used, taking into consideration improved mix design. The thickness and size of sections were approximately the same for both jobs. The lock concrete was necessarily somewhat wetter and also produced considerably more sand streaking on the surface. The elimination of sand streaks in the dam was also due to a certain extent to the use of more finely ground cement. Where it was impossible to use either the internal- or surface-type vibrator, as in heavily reinforced concrete walls, it was possible to use a form vibrator. In using a vibrator of this type, forms needed to be tight and well built, because this machine was hard on formwork and had a tendency to pull the mortar out of the concrete through any small openings.

The volume of concrete that could be effectively handled per vibrator varied with the size and type of the machine. In general, the quantity varied from 12 to 36 cubic yards per hour.

The consistency of concrete placed by vibration could be considerably drier than that of concrete placed by ordinary methods such as booting or spading. For example, concrete placed in large open forms and booted in place required a slump of 2 to 3 inches, whereas placing with vibration in a similar form required a slump of only ½ to 1 inch. Similarly, concrete requiring 6- to 8-inch slump for placing by ordinary methods could be placed with a 2- to 4-inch slump by vibration.

For vibrator concrete the same proportions could be somewhat reduced, particularly for mass work. For example, a mix using 35 to 37 percent sand by weight with ordinary methods of placing could safely be reduced to 31 to 33 percent when the concrete is vibrated. This reduction could be accomplished by holding the mortar proportions constant and increasing the coarse aggregate content.

The generally higher quality of the finished product must also be considered with the lower cost of vibrated concrete. The drier concrete used with vibration eliminates to a great extent the excess water, laitance, and segregation so common in the past with standard methods of placing. Vibration assures a more uniform quality of the concrete throughout the entire height of a lift. This avoids to a great extent a plane of weakness or a porous concrete at the top of a lift which would be subject to disintegration by weathering. The lower water-cement ratio gives greater strength and durability and the decreased cement content causes smaller volume change.

Panel and built-in-place forms.

Forms used on overflow and nonoverflow mass concrete construction were almost entirely the panel type and design shown on figure 95. Only 10 percent of the nonoverflow and 5 percent of the overflow forms were built in place. Approximately 48 percent of the total contact area was formed by panels.

Panels for the downstream face of the overflow section required a curved face mold. Since the curvature of the face for each succeeding lift was different, the panels could not be stripped and used for the next lift above. As a result, forms from adjacent blocks had to be stripped and moved to the blocks being formed.

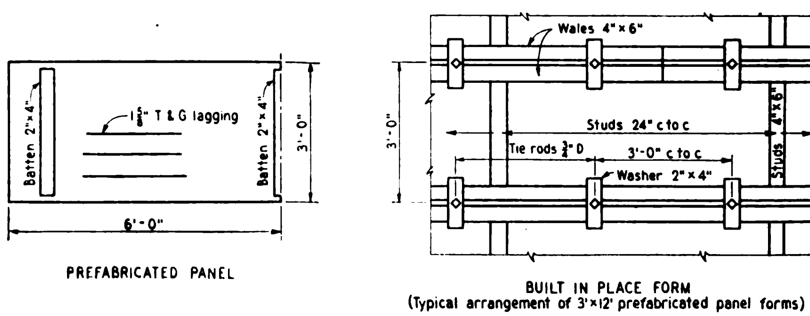
The panels shown on figure 95 were designed to withstand a maximum pressure of 500 pounds per square foot at the bottom of the form. Sheathing and studs were sufficient for substantially higher pressures and no deflection or failure of these members occurred. Substantially higher pressures were encountered, however, in small forms near the top of the block in which the rate of lift was high. Under such conditions movement of the forms was sometimes noted.

Much of this movement may be attributed to the hooked rod connection and lack of vertical support of the wale, as the rod undoubtedly had a tendency to straighten out and the wale to move downward under impact loading, permitting the form to move outward at the top. It will be noted from figure 95 that the load applied to the 4- by 6-inch corbel block has a much longer arm than the resisting arm. After several uses the wood bearing on wood had a tendency to crush, permitting some movement at the bottom of the form. In general the panels were readily adaptable to any straight surface for a 5-foot lift and could be moved and erected rapidly. A large number of panels was required because it was necessary to have three rings of forms on each block. This was occasioned by the fact that thrust at the bottom of the form broke the form below loose from the concrete and there was nothing to resist its swing down and out about the point at which the diagonal rod came through the concrete unless the third form was left in place.

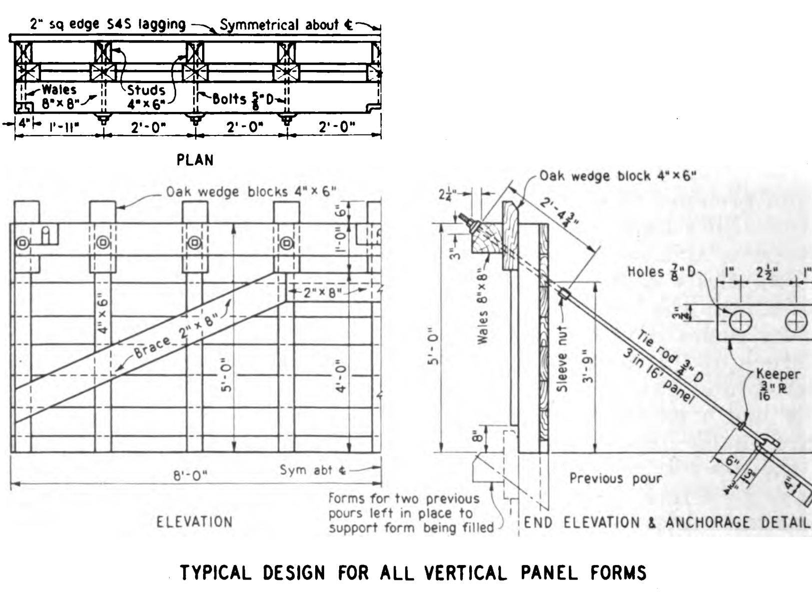
Forms for mass concrete are estimated to have been used 10 times while those in the intake, powerhouse, parapet walls, and south abutment were used on an average of 5 times. All forms were oiled before filling with concrete.

Panels were fabricated in two lengths, 15 and 16 feet. Vertical panels were made 5 feet high, and sloping panels varied depending on the width required to form a 5-foot lift.

Forms were built at the carpenter shop on the south bank and transported by boat or truck to the cofferdam at which they were to be used. They were then transferred by revolving cranes to the



FORMS FOR INTAKE PIERS



TYPICAL DESIGN FOR ALL VERTICAL PANEL FORMS

FIGURE 95.—*Typical panel form and anchorage details.*

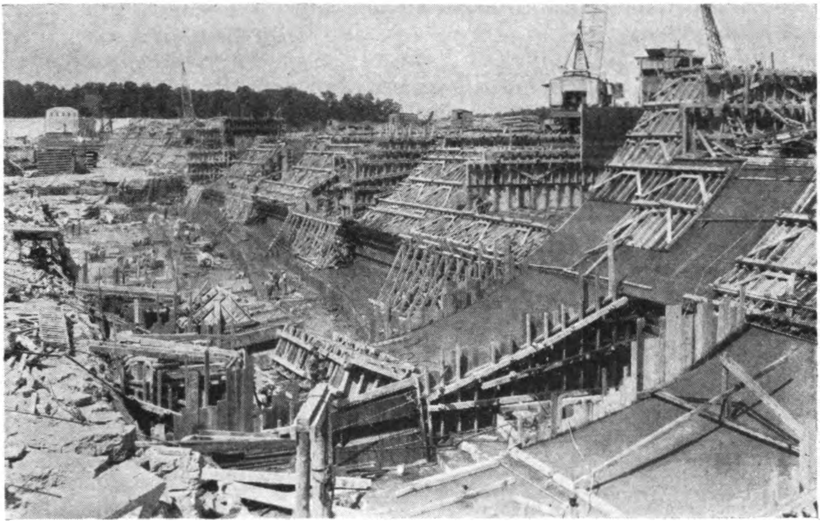


FIGURE 96.—*Panel forms.*

point of use in the block. A typical form-stripping crew [19] exclusive of the crane crew was as follows:

Crew	Hourly wage	Total per hour
1 foreman	$1.00	$1.00
2 carpenters	1.00	2.00
1 rigger	1.00	1.00
1 carpenter helper	.60	.60
10 laborers	.45	4.50

The normal form erecting crew [19] exclusive of the crane crew consisted of:

Crew	Hourly wage	Total per hour
1 foreman	$1.25	$1.25
10 carpenters	1.00	10.00
1 rigger	1.00	1.00
4 carpenter helpers	.60	2.40

The total form cost per square foot of contact area amounted to $0.55 for the nonoverflow mass concrete and $0.69 for the overflow mass concrete. A summary of form costs and quantities is given in table 25.

Cold weather placing.

During the winter of 1934–35, coil heaters were used to heat the mixing water. They were not satisfactory, however, because they could not raise the temperature of the concrete more than about 10° Fahrenheit. After placing, the concrete was covered with tarpaulins and kept warm by salamanders. In some instances live steam was

[19] See table 36, pp. 353–354, for changes in rates of pay during the job.

injected under the tarpaulin in order to protect the concrete for several days after placing, and to prevent freezing before the final set. When the temperature fell below 20° Fahrenheit, the aggregate froze in the barges and bins and concreting operations were discontinued. Since the time lost from this cause was never more than 2 or 3 days, it was considered more economical to suspend construction operations than to provide equipment for thawing aggregate.

The winter 1935–36 was considerably colder than the winter of 1934–35. Live steam supplied by stationary boilers mounted on barges and by one of the towboats or steam derricks was used to thaw the aggregate so that operation could be continued. If predictions indicated possible freezing weather, concreting in thin walls or slabs was postponed until warmer weather.

TABLE 25.—*Panel and field-fabricated form operations*

General data	Nonoverflow mass	Overflow mass
Square feet contact area—panel and field-fabricated	575,356	812,442
Percent of total—panels	90	95
Percent of total—field fabricated	10	5
Square feet forms per cubic yard concrete	3.21	2.995
FORM COSTS		
Labor:	*Per sq. ft.*	*contact area*
Erection	$0.230	
Stripping	.027	
Cleaning, stacking, etc	.029	
Repairs	.004	
Total labor	.290	$0.352
Material:		
Lumber	.027	.060
Bolts, nuts, nails, etc	.045	.078
Total material	.072	.138
Panel depreciation: Fabrication, repairs, transport	.096	.085
Expense:		
Equipment operation: Cranes, trucks, marine	.086	.107
Shop proportions	.027	.034
Total expense	.113	.141
Total form costs	.571	.716

Approximately 90 and 95 percent of the nonoverflow and overflow mass concrete, respectively, was formed using panels, although only about 48 percent of the entire 2,884,214 square feet of form contact area used in the entire job was formed with panels.

Each mixer barge should have been equipped with an oil-burning boiler of 75 to 90 horsepower for the economical heating of concreting materials for winter operation. This addition would be essential in colder climates and would also apply in the Tennessee Valley if continuous placing of concrete during winter is required.

Cleaning concrete surfaces.

All unfinished concrete surfaces were cleaned with an air-water jet just before the final set. The length of time before cleaning depended on the brand of cement being used and the weather conditions. Some of the eight different brands of cement had a slower final set than the others, and also different lots of the same brand varied in setting time. The time at which cleaning could be started varied from about three hours in hot weather to as much as 24 hours when the temperature was 10° to 15° below freezing. Determination of the time at which cleaning should start was largely a matter of judgment on the part of the cleaning foremen. When cleaned at the proper time, the mortar or laitance was scoured from the top surface of the concrete with the coarse aggregate left projecting. This gave the top of the lift, a rough, gravelly surface of the same color as the coarse aggregate. Just before placing concrete on an old surface it was again broomed and cleaned with an air-water jet. Brooming the surface before using the jet was an important factor in producing clean surfaces.

Curing.

Practically all of the upstream and downstream faces were cured for 14 days with water applied by perforated pipes. Roadway slabs and all floor surfaces were protected with wet burlap for the first 24 hours, and then cured with damp sand for the required period of time. A regular crew was employed to take care of curing.

MARINE EQUIPMENT

A considerable amount of marine equipment was used in the construction work. In order to begin construction as soon as possible, most of the early cofferdams were installed by use of rented equipment. Later, however, most of the necessary marine equipment was purchased by the Authority. Table 13 lists the rented equipment and the rental rates paid.

Marine equipment owned by the Authority is listed in table 12. It consisted principally of 4 large towboats, a derrick boat, 2 small towboats, 3 launches, and 3 wooden and 11 steel barges. The towboats were used to move the mixing plants and derrick boat, transfer cement and gravel barges, and transport construction material and equipment from Wilson Dam.

Towboats.

The launches and two small towboats were used for various kinds of work such as ambulance service, transportation of workmen and equipment, and engineering work. During peak operations one of these boats, the *Bluewater*, was used for towing the small ferry barge between No. 1 harbor and the cofferdams. Materials from the shops and warehouses were transported by the same means. Occasional trips were made to Wilson Dam by the *Bluewater* and the other small towboat, the *Duck*, but the majority of such trips were made by the larger boats, which could handle several barges in one tow.

The towboat *Hiwassee* was rented, with option to purchase from the Wolf River Sand Co., in April 1935. A rental of $3,360 was paid before purchase and this sum was credited on the $20,000 purchase price. This boat has a steel hull 112 feet long, a 26-foot beam, and an over-all draft of 4 feet 3 inches. A wooden superstructure and pilothouse were built upon the steel hull. The boat has a displacement of 88 tons and a speed of 7 miles per hour in still water under full load. Fuel consumption averaged 2½ tons of coal per 6-hour shift. The *Hiwassee* was used primarily for transporting material from Wilson to Wheeler. In October 1935 it was partially withdrawn from service and completely overhauled. Shortly after completion of these alterations in December 1935, the boat was withdrawn from work at Wheeler and assigned for interproject towing.

The *Elk* was a stern-wheel steam-driven towboat, of wood construction throughout, 110 feet long, with a 36-foot beam and an over-all draft of 4 feet 2½ inches. It had a displacement of 109 tons and a speed of 10 miles per hour in still water. The *Elk* was purchased for a price of $14,020, of which $2,975 was paid in rental and applied on the purchase price. At Wheeler $3,000 was added to the capitalized value in additions and betterments. Until May 1935 this boat was used for general towing service at the dam site, and at that time it was withdrawn from service for six weeks and practically rebuilt. Repairs consisted of an entire lower deck, gunwales, cross beams, head block, rudder beam, and the rebuilding of the upper deck and pilot house. The cost of these repairs was $8,092. The boat was again placed in regular towing service in June 1935 and was used until November when it was transferred to Pickwick Landing Dam. The fuel consumption was 2 tons of coal per 6-hour shift.

The towboat *Joe Wheeler* was built for the Authority by the Nashville Bridge Co. in 1934 at a cost of $22,310. It was of all-steel construction, 50 feet long, with a 12½-foot beam and a draft of 4 feet 2 inches. A 180-horsepower, at 800 revolutions per minute, Buda marine Diesel engine provided the power to turn the 50-inch-diameter propeller. The boat had a pushing capacity of 900 tons, a displacement of 25 tons, and a speed of 10 miles per hour in still water. Immediately after its delivery in August 1934, it was placed in service and was used for general towing, making frequent trips to Wilson Dam. It was also used for moving the floating mixer plants, derricks, and cement and aggregate barges. Operation was from 6 to 24 hours per day depending on job requirements.

Purchase of the *Paint Rock* was made in March 1935 from the Stevens Brothers and Miller-Hutchinson Company for $5,031. The boat was of all-steel construction, 42 feet long, with a 10-foot beam and 4-foot 2-inch draft. It was provided with a 32-inch, 3-blade propeller powered by a Cummins marine Diesel engine rated at 85 horsepower, 1,200 revolutions per minute. It was used from March 1935 to September 1936, at which time it was transferred to Guntersville Dam. Work done at Wheeler was the same as for other towboats, including towing between Wilson and Wheeler Dams. Three gallons of fuel oil were used per operating hour.

Towboat costs. Table 26 lists the hourly operating costs for the four towboats:

TABLE 26.—*Towboat operating costs*

	Hiwassee Apr.'35–Mar.'36	Elk Dec.'34–Nov.'35	Joe Wheeler Aug.'34–Apr.'36	Paint Rock Mar.'35–Sept.'36
Operating data:				
Crew time (hours)	5,297	6,060	10,279	5,420
Idle hours	2,003	2,022	2,331	1,926
Repairs (hours)	62	[1] 216	782	381
Net operating hours	3,232	3,822	7,166	3,113
Operating costs per net hour:				
Labor	$7.36	$6.62	$3.88	$3.73
Fuel	1.99	1.95	.34	.46
Miscellaneous supplies and expenses	.11	.14	.35	.36
Use rate—repairs and depreciation	6.75	[2] 4.54	2.66	2.75
Direct charges	1.05	.69		
Total	17.26	13.94	7.23	7.30

[1] Does not include time spent in major overhaul—288 hours.
[2] Includes general overhaul ($6,500.00).

On the *Joe Wheeler* frequent repairs were made to the reduction gears, reverse gears, and clutch. This trouble was largely eliminated by the installation of heavier reduction gears, the shortening of the propeller shaft, and the installation of a safety coupling to prevent breaking of the propeller blades. The *Paint Rock* also required the removal and replacement of several reduction gear units. One complete overhaul of the hull and the engine was made at a direct cost of about $1,000.

Derrick boat.

Material-handling equipment was required which could be moved about the job as necessary and could work independently of shore connections. For this purpose a derrick boat consisting of a whirley crane mounted on a barge was purchased. The barge was built by the Nashville Bridge Co. and was 62 by 34 by 5 feet in size with four spuds 18 inches square and 24 feet long. An American Revolver whirley crane was mounted on this barge and was equipped with a 75-foot boom and 68-horsepower vertical oil-fired boiler. A 2½-cubic-yard clamshell bucket was used for handling aggregate and other loose material.

During the construction and removal of the main cofferdams and the cofferdams for the power line crossing piers, this unit proved very valuable. In addition it placed some concrete, handled aggregate and excavation, and did miscellaneous work. Movement from one point to another was by towboat.

During the peak construction period the unit worked four 6-hour shifts, while at other times two 6-hour shifts per day were the usual work period.

For normal operation the crew required was:[20]

	Per hour
1 Crane operator	$1.50
1 Fireman	.75
1 Deck hand	.60

Occasionally, when the work required, a signalman at $0.625 per hour was added to the normal crew.

No record of quantities or work done by the derrick boat is available because of the variety of work and large amount of moving required. The unit operated the following hours from July 1934 to October 1936:

Gross operating hours	9,257
Delays:	
Idle with crew	1,238
Repairs	74
Net operating hours	7,945

Delays due to mechanical trouble were few, with minor repairs and adjustments being made by the job-repair crew during idle time. Shortly after the boat arrived at Wheeler, the whirley crane was moved forward several feet to a point slightly to the rear of the center of the barge. To October 1, 1936, operating costs amounted to approximately $12.60 per net hour.

[20] See table 36, pp. 353–354, for changes in rates of pay during the job.

POWER PLANT CONSTRUCTION

Power plant construction was simplified considerably by the adoption of the outdoor type powerhouse structure. With the exception of the service bay, at the south end of the line of units, the construction problem was mainly one of forming intricate parts of the substructure, such as the draft tubes and scroll cases, and the erection of the turbine and generating equipment. With few exceptions the construction for each one of the two units initially installed was similar. The intakes, the horizontal legs of the draft tubes, and the draft tube gate crane runway were constructed for the future installation of six additional units. The exposed portion of the foundation rock of the turbine recesses was covered by concrete to protect it from weathering until such time as the units would be installed.

Intakes.

After the foundation excavation was completed in the powerhouse area, the next step was the construction of the eight intakes. Work on this structure was not unique. The mass concrete was poured in 5-foot lifts and the pier concrete in 10-foot lifts.

Draft tubes.

Construction of the draft tubes was begun soon after work had been started on the intake structures. They were constructed in two stages. In the first stage, the three discharge openings of each

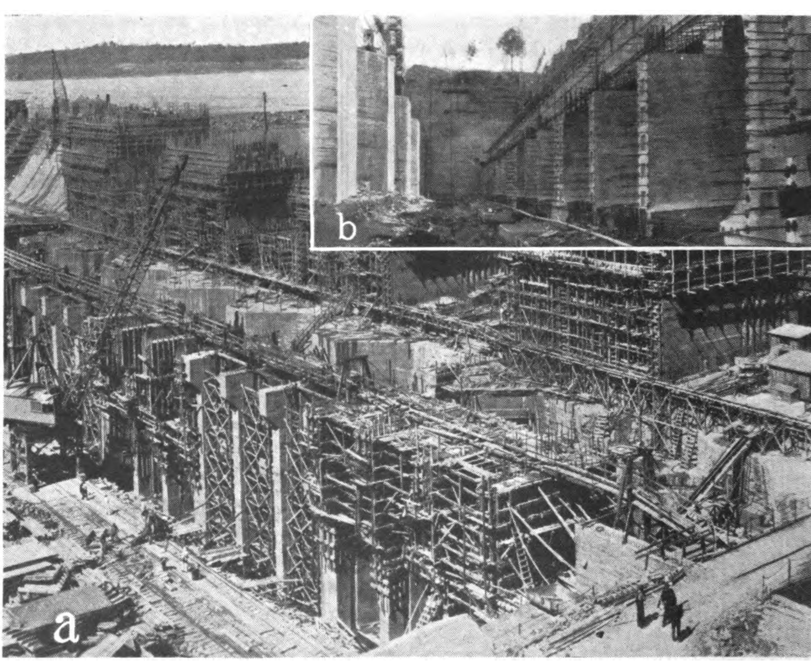


Figure 97.—*Provision for future units, (a) showing downstream openings for draft tubes, (b) space for installation of units.*

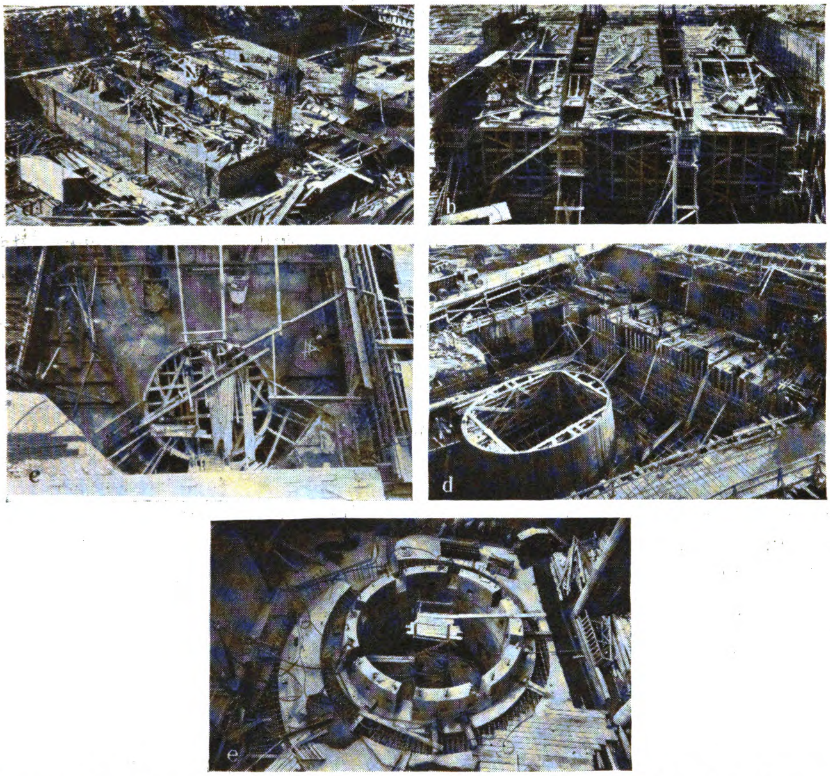


FIGURE 98.—*Draft tube construction, (a) floor slabs poured, (b) forms for piers and roof, (c) form for transition section, (d) form for throat section, (e) throat section poured.*

draft tube were constructed to a point just downstream from the two pier nose castings; and in the second stage, the remainder of the tube up to the throat ring of the turbine was constructed. This latter stage included the installation of the pier nose castings.

The discharge tube forms for each unit were built in three sections, each being 46 feet 6 inches long, 24 feet 6½ inches high at the downstream end, and tapering to 16 feet 6 inches at the upstream end. These were erected on the bottom draft tube slab which had been previously poured. Ribs in the straight sections were built of two pieces of 2- by 12-inch dressed lumber, while for the curved portions three pieces of 2- by 12-inch material were used. They were spaced 16 inches on centers and the curved sections were bolted to the straight sections to permit removal of the form. The vertical, horizontal, and diagonal bracing was made of 4- by 6-inch, 2- by 8-inch, and 2- by 10-inch dressed timber, respectively. Vertical bracing was spaced on 3-foot centers and the horizontal bracing on 5-foot centers. Diagonal bracing was placed as needed. Lagging was laid parallel to the flow using a double thickness of 1- by 3-inch and 1- by 4-inch dressed lumber. For curved surfaces lagging cut into strips 1 inch wide with double thickness was used.

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The form for the second stage of draft tube construction was built in one piece on keel timbers set to the same grade as the bottom of the draft tube. The pier nose castings were checked for dimensions prior to any work on the form. Facsimiles of those portions of the castings that were to connect with the form were built of timber and set in place on the carpenter shop erection platform. Ribs, bracing, and lagging were built of the same size timbers as had been used for the discharge tube forms.

After the form had been completed and checked for alignment, it was cut into sections to facilitate handling and placing. From the barge at the cofferdam each section was moved into place by a 50-ton guy derrick located on the 568.3 elevation of No. 1 intake.

Railroad rails were used to support each section of the draft tube forms. Where this rail came in contact with the form, a box 18 inches square was constructed. The opening left by this box was filled with concrete after the form had been removed. To prevent the formation of air pockets 1-inch holes were drilled in the bottom of the form. Concrete was poured around the discharge tubes of the first draft tube stage and continued upward to form the slab for the draft tube gate gantry crane. For the second stage, concrete was poured in three lifts—the top of the third lift being formed into eight blocks for the support of the turbine speed ring.

Turbine embedded parts.

From this point concreting and installation of the embedded parts of the turbine continued as an interlocking operation. The first part installed was the draft tube liner which was composed of six sections. These sections were placed in the draft tube, bolted together, and jacked into approximate position. The liner was centered by means of tape measurements from a plumb bob suspended on the center line of the turbine. The four sections of the throat ring were next assembled, and the ring was adjusted to exact location by jacks and jack bolts. The liner was then raised slightly, realigned carefully, and fitted to the ring—the top being marked for the proper cut. The liner was next lowered and cut to the final line, after which it was again raised to its proper position and riveted and welded to the throat ring. The joints between the sections of the liner were also riveted and welded at this time.

The speed ring was shipped to the site in six pieces. These sections were placed on wooden blocks on top of the draft tube concrete and several bolts in each vertical joint placed to hold the ring prior to alignment and final bolting. A series of 12-ton jacks was placed between the concrete and bottom of the ring to elevate each section where necessary in order to accomplish final bolting of the vertical joints.

Upon completion of this bolting, the entire ring was rotated, shifted, and leveled to line and grade. Eight pipe jacks—two at each quadrant point—were placed diagonally to rotate the ring. Eight concrete blocks were then poured on top of the supporting wall, and after being allowed to set, the weight of the ring was transferred to them from the wooden blocks. A timber spider, from the center of

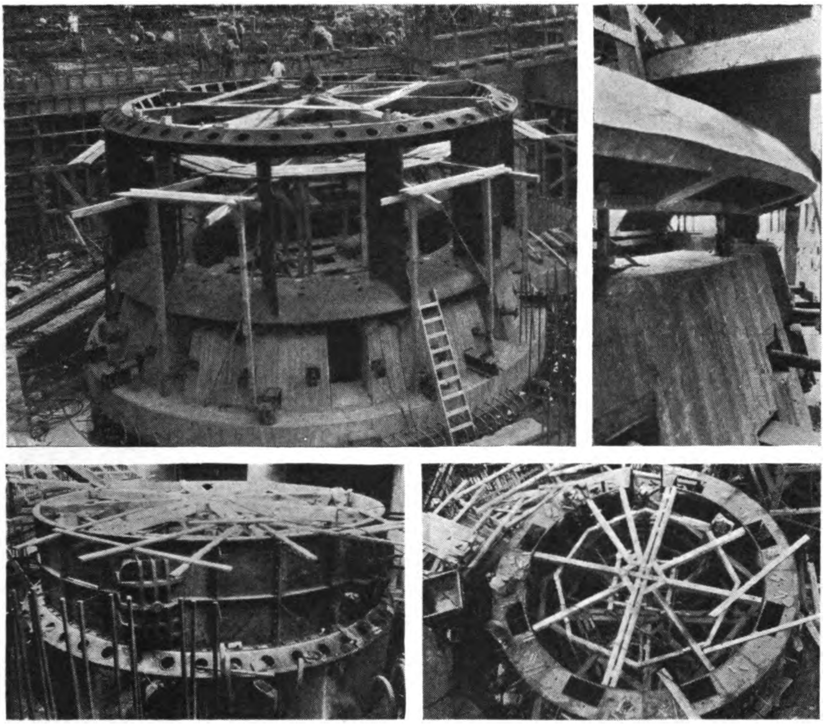

FIGURE 99.—*Speed ring and pit liner erection.*

which was suspended a plumb line, was erected in the plane of the top of the stay ring. The ring was shifted to the exact center line of the unit as determined by the plumb line and then the throat ring and draft tube liner were shifted to final position as determined from the same line. The four sections of the distributor ring were next bolted to the speed ring and throat ring.

Concreting or grouting in of the various parts that had been assembled to this point was then completed. The first concrete was placed at the bottom of the draft tube liner to elevation 487.5 and on the next day concrete to elevation 496 was added, or about one foot above the top of the draft tube liner. Pouring was then continued through 5-inch holes in the bottom flange of the speed ring until the space around the throat ring and under the distributor and speed ring was completely filled. To minimize the danger of disturbing the turbine parts, pouring was done at a slow rate. No vibrators were used and the concrete was tamped with wooden poles inserted through the 5-inch holes in the speed ring. Care was taken to avoid air pockets and one-inch vent holes were drilled in the distributor and speed rings to free any air that might become trapped. These holes as well as the 5-inch holes were later plugged and welded.

The lower pit liner, which had been shipped in six sections, was next erected on top of the speed ring. Two sections were lined accurately since they contained the operating cylinder bays. The bottom and top sections of the upper pit liners were then erected. Each came in four sections which were bolted together during the erection. Alignment was obtained from the plumb line which had been extended upward from the speed ring. All joints were welded after the final alignment had been obtained.

Scroll cases.

Construction of the scroll cases was carried on simultaneously with the erection of the pit liners. The bottom slab for the scroll case, as shown in figure 98, was poured prior to setting the lower half of the scroll case form. After the lower half of the form had been erected, concrete was placed around it. The top of the form was then placed and concrete poured in thin layers to the final elevation of the generator deck. Electric vibrators were used to consolidate this concrete, but they were not permitted to be operated too close to the pit liners.

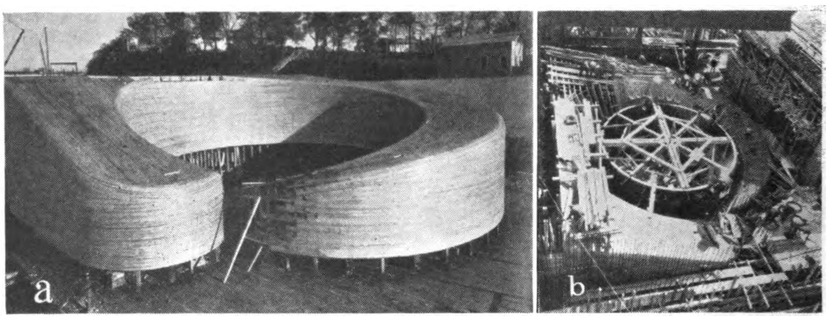

FIGURE 100.—*Upper half of the scroll case form (a) being constructed and (b) being placed.*

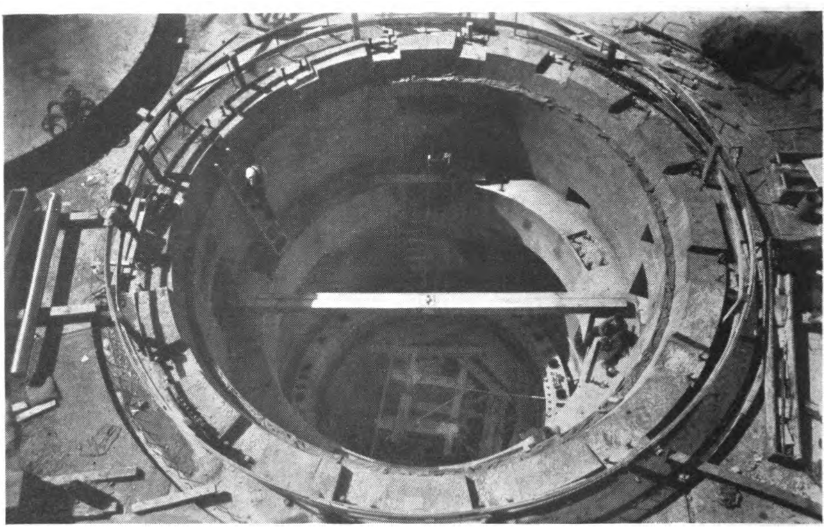

FIGURE 101.—*All embedded parts in place.*

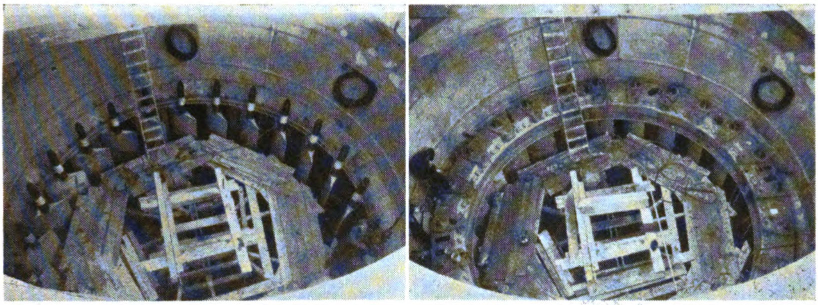

FIGURE 102.—*Guide vane and outer head cover erection.*

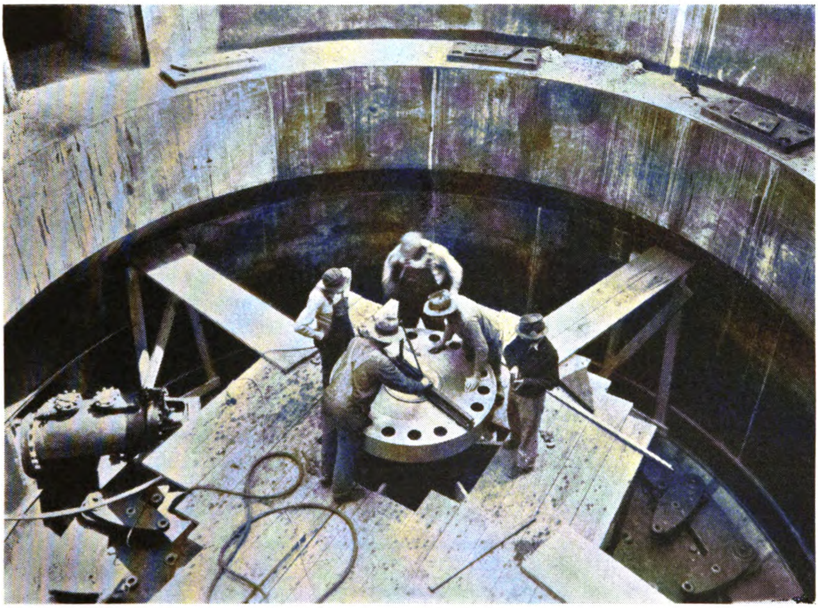

FIGURE 103.—*Preliminary leveling of turbine shaft.*

Turbine erection.

After the erection of the embedded turbine parts had been completed, the assembly of the mechanical parts was begun and completed with little difficulty. With the exception of the turbine runner and shaft and their associated parts which were assembled in the erection bay, all parts were placed directly in position in the turbine pit.

A similar procedure was followed on both units with the exception of the installation of the speed ring. In No. 2 unit, prior to the setting of the speed ring sections, 12 concrete blocks were placed on top of the wall topping out the draft tube. These blocks replaced the wood blocks used on No. 1 unit and speeded up the erection since no time was lost waiting for the concrete to harden.

Generator erection.

The two generators were installed under the supervision of and by the forces of the General Electric Company. At the time the contract was let, the first unit constituted the largest diameter unit of its kind. The shipping weight was 1,738,757 pounds as compared to the installed weight of approximately 1,000,000 pounds. A total of 32 railroad cars was required for the complete shipment. To handle the heavy pieces of the generator and also of the turbine, two 100-ton derricks were installed—one at Wilson and the other at Wheeler.

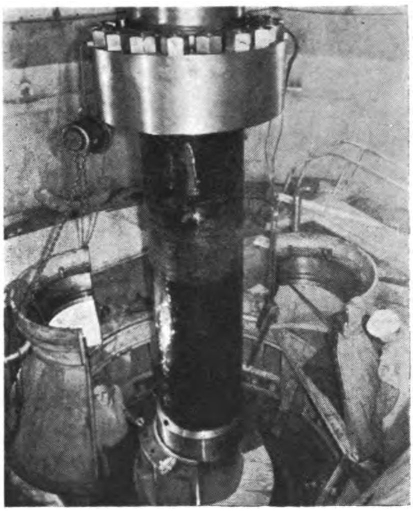

FIGURE 104.—*Assembly of the guide bearing. (Note water bearing chamber removed)*

At Wilson the generator parts were transferred from the railroad cars to barges. These were towed to Wheeler where the parts were again transferred to flatcars for transportation over the permanent railroad track to the service bay of the powerhouse. The permanent power-

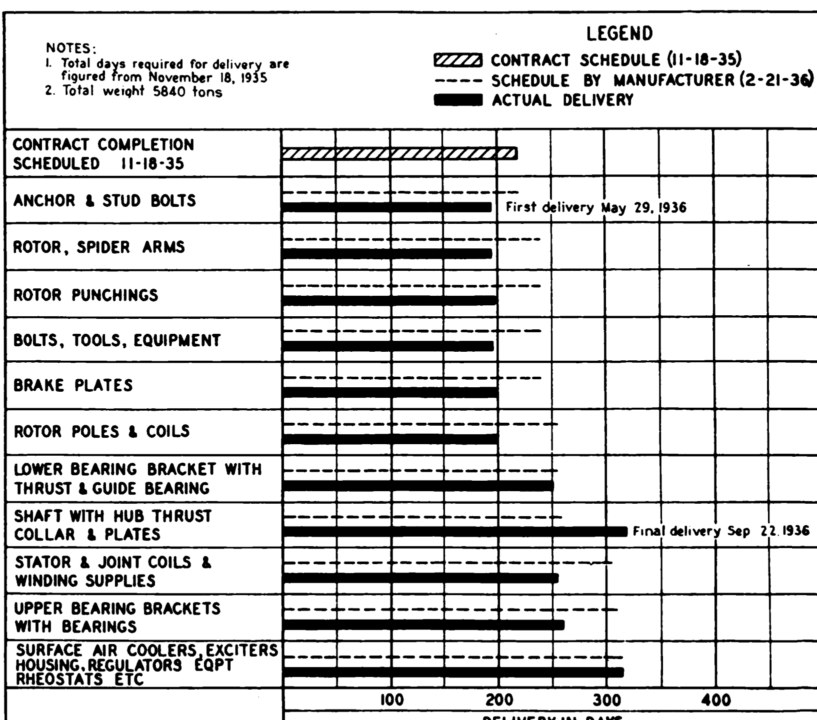

FIGURE 105.—*Shipping schedule for generator No. 1.*

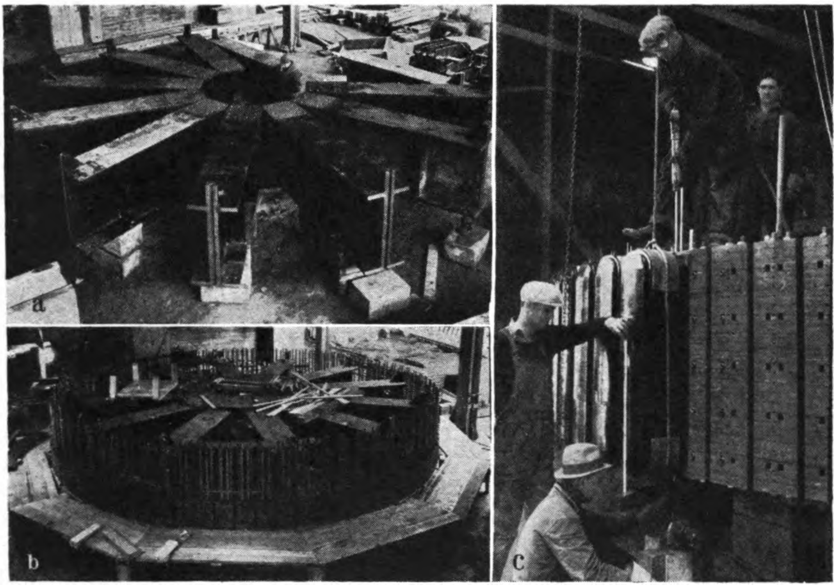

Figure 106.—*Generator rotor erection, (a) assembly of spider, (b) stacking laminations, and (c) field coil erection.*

house crane was used to handle the parts during erection.

To coordinate the erection with job conditions and manufacturing schedules, a careful study and follow-up was made of the sequence of shipments from the factory.

Assembly of the first unit was started February 11, 1936, with the erection of the rotor in the erection bay at the south end of the line of units. The spider rings and arms were placed on wood blocks and leveled. To facilitate the stacking of the 2,152 sections of laminations, a platform was built around the periphery of the rotor. Each sector of lamination weighed approximately 68 pounds and the 4-ton overhead traveling crane was utilized in lifting them.

Figure 107.—*Lower bracket and shaft assembly.*

While the rotor was being assembled in the assembly bay, the lower bearing bracket was being assembled on the deck at elevation 540.3.[21] When the bracket had been completed, the shaft was placed in the bracket but was supported by separate cribbing on the deck to keep the weight of the shaft off the upper bearing plate. The upper and lower collars of the guide bearings, consisting of two sections each, were then set and all springs placed.

[21] 1929 General Adjustment, see page 221.

During the time the lower bearing bracket was being erected, the sole plates for the bracket were set and grouted in place. The anchor bolts had been placed when the concrete foundation was poured. The lower guide bracket with the shaft and associated parts was next placed on the sole plates. This total assembly weighed 167 tons. Jackscrews were used for alignment with the turbine shaft. After placing the heavy assembly on the sole plates and properly adjusting the generator and turbine shafts, micrometer measurements were taken for the milling of the chock plates between the faces of the bearing bracket and the sole plates.

The stator was shipped in four sections which were assembled on the job. Although the stator was placed and the coils installed, the final centering of the face of the windings could not be completed until the rotor had been placed and bolted to the turbine shaft. However, a very close approximation of its relation to the shaft was obtained by tramming, using the water wheel shaft as a center.

When the stator had been approximately aligned, the assembly of the upper bearing bracket was begun. As soon as the turbine erector completed the bolting down of the inner head cover of the turbine and aligning of the shaft, preparations were made to place the rotor. The rotor, including the lifting beam and bell, weighed 228 tons. Measurements of the deflection of the lower bearing bracket were taken as the weight was applied to it, but no appreciable deflections were noted.

After the rotor was placed, it was bolted to the hub of the generator shaft. Checks were made not only for air gap distance but for the location of the correct magnetic centers—whether or not the bottom of the laminations of the stator and rotor were in the same plane. It was discovered that the rotor was thirty-five sixty-fourths of an inch above the stator punchings. Further checks

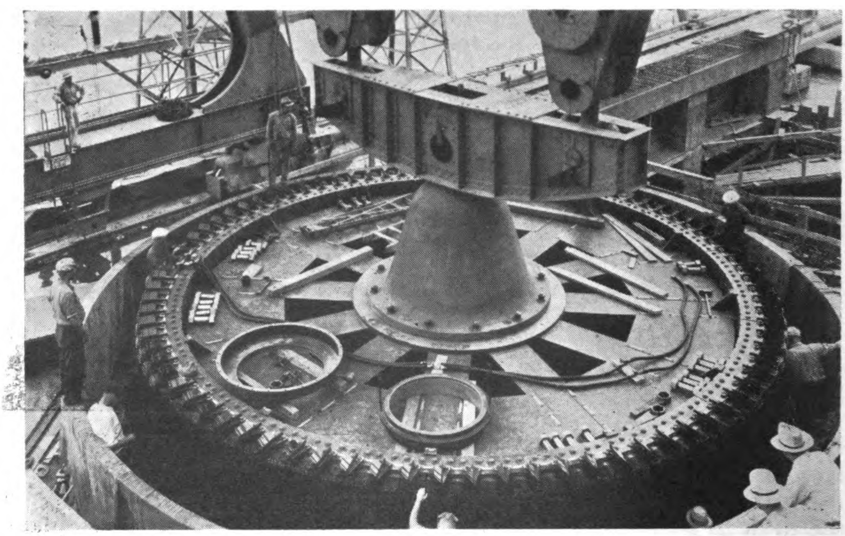

FIGURE 108.—*Rotor being placed.*

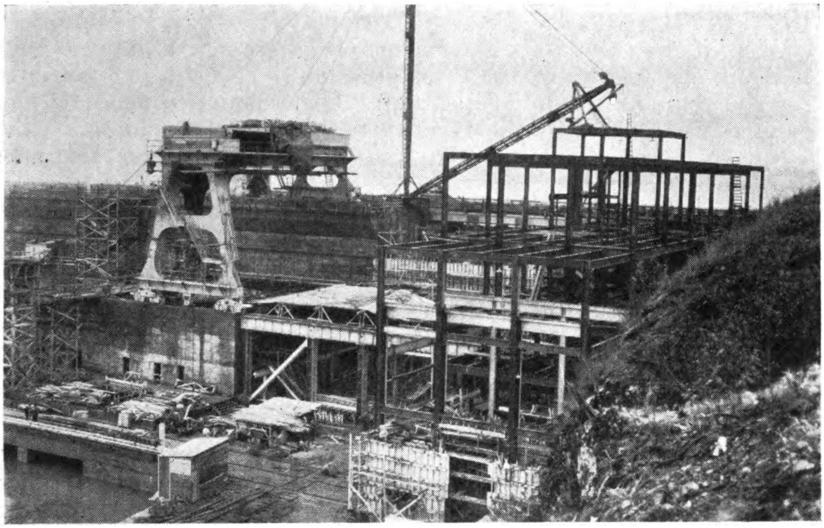

FIGURE 109.—*Control building construction.*

were made to locate the discrepancy and it was found that the generator contractor had made an error in the shaft dimension. After the upper bearing bracket was set, the exciter was placed temporarily, and this showed specifically that the stator was low. To rectify this the grout under the stator sole plates was removed and the plates and stator were raised by jack bolts. The sole plates were not regrouted until after the machine had been thoroughly rechecked for magnetic center. When the plates were grouted in, the jack bolts were replaced with push-pull jacks preparatory to aligning the stator and rotor for final air gap measurement.

While the upper bearing bracket and the heavier parts were being properly aligned, the main exciter was being assembled. The exciter armature was bolted to the shaft as soon as final adjustment of the generator air gap had been completed. The field pole frame was then adjusted to give the proper air gap. After the main exciter had been installed, the pilot exciter was mounted.

Completion of the erection of the unit consisted of the installation of the generator air coolers. After this had been done, the generator covering was installed by the Authority. Considerable difficulty was encountered in making the covering watertight. Installation of the necessary electrical control equipment proceeded in a routine manner. Little trouble was experienced in this work, and since much of it came assembled from the factory, it remained only for the job forces to wire and connect the associated parts.

Control building.

Structural steel for the control building was erected by the job forces, with a 20-ton guy derrick. The forms for the floor slabs and floor beam reinforcing were made up as panels and, since beam spacing was the same for all floors, each panel could be reused several times.

Intake gates.

Seven of the twenty-six gates assembled were active gates, 6 being placed in the intakes of the 2 installed units and 1 being placed in 1 of the remaining intake openings. The remainder of the intake openings were closed by the 17 inactive gates. (See chap. 3.) Two emergency gates were also erected and placed in reserve for emergency use.

Recesses were provided in the concrete of the intake piers to permit the grouting of the track seats and gate seals at a later date. On the upstream side of each recess, anchor bolts were embedded during construction. These bolts were used to support brackets spaced at 3-foot intervals which in turn supported the track for final alignment and grouting.

Plumb lines were dropped from the top of the intake structures and secured so that check measurements could be made as the grouting progressed. After the tracks had been grouted in final position, the lower gate seat was bolted to the track seat plates and brought to grade by means of jack bolts. The top channel section was then filled with babbitt and the seat grouted in place.

The gates were assembled in a horizontal position on a series of concrete piers placed on top of the intake structure as shown by figure 110. Thirty-inch 180-pound CB sections were placed on these piers, and the four sections of the active gates aligned on top of the CB sections. Jack bolts between the top flanges of the CB sections were used to level the various sections of the gate. The four sections were first bolted together. Welding was then started, but considerable trouble was encountered in maintaining a correct alignment of the gate during the welding operation. Various methods were used, the most successful including the use of two strongbacks to hold the four gate sections in place. The step-back method of welding was used—the top and bottom of the gate being welded at

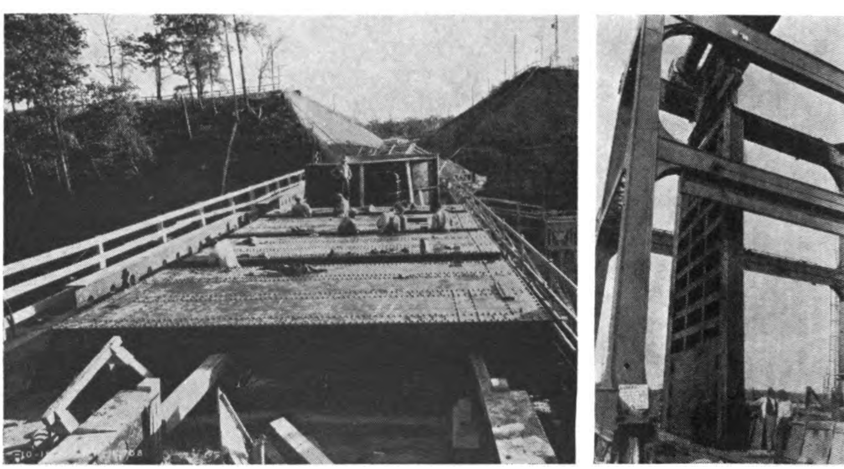

FIGURE 110.—*Intake gate erection.*

the same time. After welding was completed, all necessary riveting was done.

This was followed by bolting the metal seals to the top and sides of the gate and the assembly of the wheels. The wheels were properly aligned by measurement from the side seals. A special lug and shackle was made to lift the gate and place it in the gate slots. It was then removed and replaced by the regular lifting hooks. A final check was made to assure that the gate latches operated properly and the stop blocks were satisfactory. The erection of the 17 inactive service gates was similar to that just described.

The gate guides for the emergency gates are made up of a plate similar to the active gate track seat plate. This plate was aligned and grouted in a manner similar to that used for the active gates. These gates also were fabricated in four sections and were assembled on the concrete pedestals; the skin plate was used for alignment of the sections.

Draft tube gates.

Seats and guides for the draft tube gates were aligned and grouted in the recesses provided at the discharge end of the draft tubes. The gates were fabricated in three sections and assembled in the tailrace area below the draft tubes. After the joints had been welded, the gates were riveted, the rollers bolted, and the seals installed. When lowered into final position, the gates tended to lean away from the side seal when the head against both faces was balanced. Four automobile spring leaves were installed at the back of each gate to exert pressure against the back guide and hold the gate against the seal.

SWITCHYARD

The main switchyard is on the rock fill about 600 feet downstream from the control building on the south bank of the river. No unusual features developed during the construction. All foundations for the equipment and steel columns were based on the rock fill at elevation 503. The top of the rock fill was at elevation 513. Under the foundations for the structures the rock fill was slushed with grout before the concrete footings were poured. During the pouring of these footings Wilson pool was lowered several feet. The steel erection consisted principally of bolting the various fabricated sections together.

The cable tunnel which connects the switchyard and the control building was constructed in several sections. That portion adjacent to the control building was constructed at the same time as the control building. That portion from the building passing through cofferdam No. 1 was completed later when Wilson Lake could be lowered, since the bottom of the tunnel was below normal lake level.

The main power transformers are located on the powerhouse substructure at elevation 516.3. These units were shipped in sections and assembled in the transformer assembly room in the control building, utilizing a 75-ton crane provided for that purpose.

The transformer windings were shipped in the lower sections of the transformer cases which were sealed to prevent condensation.

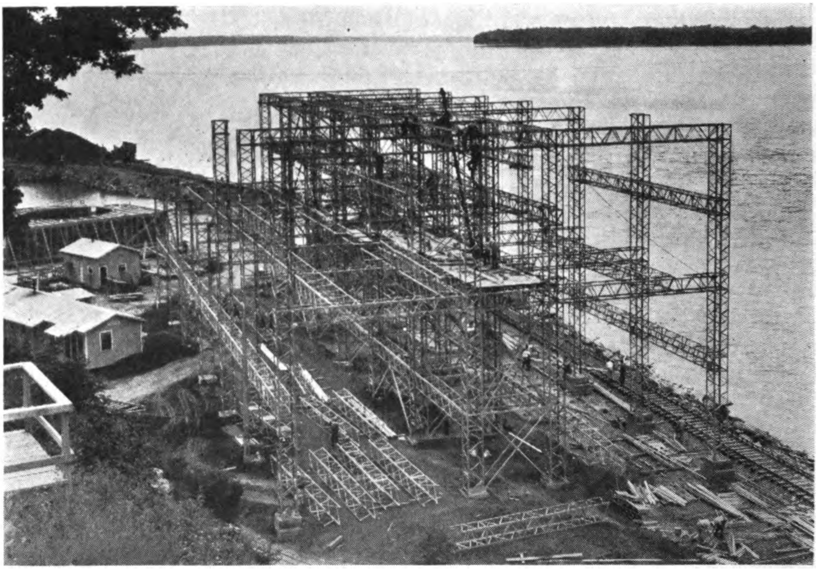

FIGURE 111.—*Switchyard steel erection.*

The case sections were welded together after arrival at the job and the completed unit placed opposite its final location by the 20-ton draft tube gantry crane. A transformer transfer car carried the unit for this operation. It is equipped with a hand winch to move the transformer from the car to the permanent operating location.

Grounding mats were placed on the bottom of No. 1 harbor (see fig. 47), one on the bottom of the tailrace, and two on the lake bottom upstream from the intakes. All connections were brazed and when underground were painted with asphalt. Because of the possibility of the turbine discharge tearing out the mat, the tailrace grounding system was embedded in shallow trenches excavated in the river bed. These were filled with coke after the wire was laid and the entire surface covered with a porous concrete slab.

ROADWAY AND BRIDGE

The steel for the roadway and bridge across the dam was erected by the Authority's forces. Over the spillway, nonoverflow sections, and intakes, the roadway girders were placed by cranes on concrete piers between the spillway sections. Steel for the north ramp over the lock was handled by crane also.

Each bridge truss was assembled in a horizontal position on the downstream arm of cofferdam No. 5, rolled onto barges, and towed to the lock chamber. From this position they were picked up by two guy derricks—one mounted on each lock wall—and hoisted into position where they were temporarily braced until the bracing, floor beams, and stringers could be connected.

For the short span between the bridge and the north bank of the river an improvised cableway was used. This utilized a guy derrick

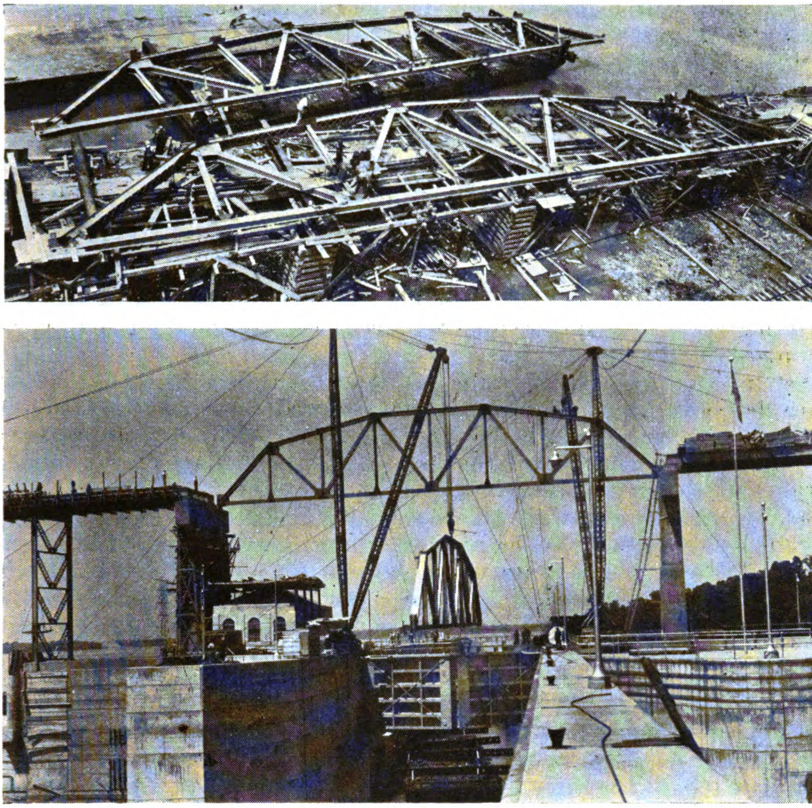

FIGURE 112.—*Bridge truss erection.*

on the land wall of the lock and a cable passing through a sheave anchored on the north bank. By this means the girders, floor beams, and bracing were erected for this span.

Forms for the roadway slab consisted of panels held in position by bracing from the lower flanges of the roadway steel. The slab was poured for the full width and in sections 135 feet long. Steel expansion dams served as end forms for each section. The curb form consisted of a wood bulkhead of the proper height. Curb panels were constructed and braced similarly to those for the roadway concrete. Boxes were formed at the outer edge of each sidewalk in which the guard-rail posts were later grouted. This rail consisted entirely of rolled structural shapes which were erected with riveted connections. Curb forms were removed 2 hours after pouring concrete and the curb lines given a sand finish while still green.

TAINTER AND TRASH GATE ERECTION

The job forces devised several items of equipment and novel construction methods for setting the 60 tainter gates. No difficulties were experienced in the erection of the two trashway gates. They were placed in the guides provided in the trashway piers and the brass seal strips secured to complete the erection.

Most of the parts of the tainter gates were erected after the roadway slab over the spillway piers had been poured. Erection of the pin-bearing grillages was started after the roadway steel had been placed and continued with the pouring of the roadway concrete. Some of the side-seal wall plates were placed before the 30-foot closure blocks were brought up to finished crest, and some after. The upper sheaves and the gates proper were installed after the spillway crest had been topped.

Recesses had been provided on the downstream faces of the piers for the pin-bearing grillages, in the north and south faces for the wall plates, in the spillway crests for the sills, and in the tops of the piers for the upper sheaves.

Setting the pin-bearing grillages was begun July 13, 1935. These were delivered to the dam on barges, hoisted into place, set on the anchor bolts, and brought to line and grade. No difficulties were encounted in this operation, the recesses for the grillages being amply large and the reference points for correct alignment conveniently near. Similarly, the wall plates were hoisted into place, set on the anchor bolts provided in the recesses, and aligned. For radial alignment, the wall plates were set by means of a tram of the correct length and lag screwed to a timber bolted to the grillage on the center line of the pins. By these means clearance distances between the plates varied no more than one-eighth inch from the plan dimensions. This greatly facilitated the fitting of the tainter gates to the openings.

After installation and adjustment of the grillages and wall plates, the plate recesses were filled with grout and the grillage recesses with concrete of the same mix as that used in the piers.

Some difficulties were encountered because of the narrow clearance between the top of the sheaves and the bottom of the roadway slab, which had been poured prior to the erection of the sheaves. The assemblies had to be dismantled before they could be set in place. This dismantling was done on the roadway bridge. The sheaves were reassembled in place, brought to correct line, and the recesses filled with grout. After this operation the crest seals were placed in the recesses provided and were grouted in after being checked for position from established grade points.

The greatest problem encountered was that of setting the gate-hoisting mechanism in the inspection gallery. Several adits had been provided leading to the gallery and most of the operating hoists were taken in through these openings. A few of these, however, were moved into the tunnel in the 15-foot blocks after the 30-foot closure section had been poured up to gallery floor level and before tunnel forms were set. Hoists and sheaves were brought to the dam on barges and placed in the gallery floor at the 30-foot blocks by a derrick boat. They were then rolled into the 15-foot block, clear of all form erection, where they were stored until the remainder of the concrete above gallery floor level was poured and forms stripped. The units were then placed in position on the anchor bolts provided, and after being set to line and grade the bases were grouted in.

To facilitate setting of the assembled tainter gate, a special traveler was designed and constructed of beams mounted on four sets of double swivel trucks. The trucks were spaced to operate on a 9-foot 5-inch gage track, laid on wood ties on the roadway bridge. The machine was equipped with two 2-drum, 80-horsepower, 1,650-pound line load, Lidgerwood hoists.

The tainter gates were delivered by barge from Wilson Dam. On the decks of the two barges used for transporting the gates, track was laid on a 20-foot gage to expedite loading and unloading. Each gate was delivered to the job in three parts, consisting of two arms and the completed gate face.

The arms were first hoisted into place and connected to the pin bearings which had previously been bolted to the grillages. The arms were then suspended in the proper relative position by means of chain hoists hung from the roadway beams above. The gate was finally hoisted into position by the traveler and connected to the arms by the permanent steel bolts. The various steps in the gate erection operation, using the erector, are shown in figure 113. The 60 gates were transported from Wilson Dam, hoisted into place, and connected to the pin bearings in 18 working days, one 8-hour shift per day. During 1 day, 11 gates were erected in 10 hours.

Electrically operated solenoid brakes were installed in the ends of all the gate hoist motors, except those on the trashway gates, to relieve overload on the gearing of the gate operating mechanism.

Specifications for the painting of the radial and trashway gates called for one coat of bituminous primer and one coat of hot bituminous enamel. The gates had been given one shop coat of bitumastic primer, but because they had been stored out of doors for some time at Wilson Dam, this paint had deteriorated. The poor condition of the shop coat of primer and the formation of a thin film of rust during storage made it necessary to sandblast each gate before applying the field coat of primer.

To serve the painters on the gates, a plant consisting of a portable compressor, two oil-fired melting pots, and a sandblasting machine was set up on a 25- by 100-foot steel barge. This barge was moved

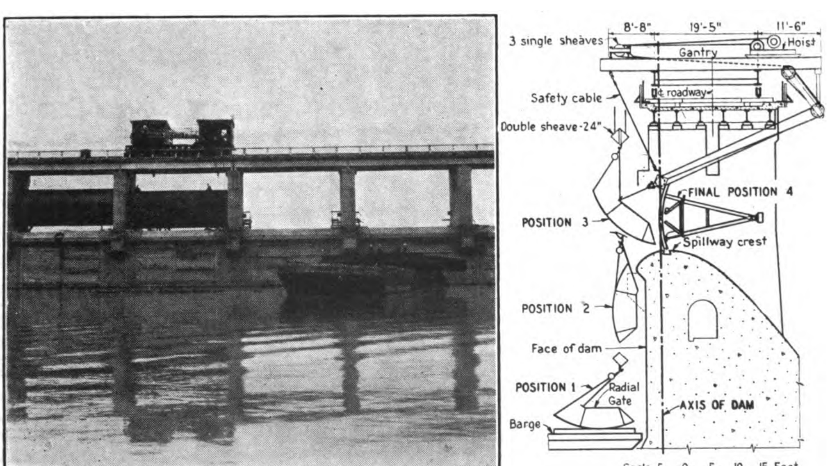

FIGURE 113.—*Spillway gate erection.*

along the dam on the downstream side from gate to gate as the painting progressed.

The proper interval between the application of the primer and enamel was found to be about 18 hours. An afternoon shift sand-blasted and primed as much surface as the morning shift could cover with enamel, which amounted to approximately two gates a day after the men became familiar with the work. Proper temperature of enamel for application was found to be between 400° and 450° Fahrenheit. Enamel heated above the maximum temperature became very brittle after cooling.

Immediately after the painters finished work on a gate, the inspectors tested the coat of enamel for flaws by means of the detector shown in figure 114. This machine proved to be very effective in locating pinholes and other imperfections in the finished coat—the enamel acting as an insulator where properly applied, flaws being located by the arcing between the detector and the steel of the gate. Bare spots were marked by inspectors and covered by the painters in the same manner as used to apply the original coat.

Inertol, a waterproof paint applied without heating, was used where limited space made the proper application of the bitumastic enamel impossible. Principal points of use were around hinges and gate arms.

The fumes of the hot enamel proved very irritating to most of the men until they became accustomed to them. The Authority provided all men on this work with respirators, goggles, and heavy gloves. Cocoa butter smeared on the men's faces proved to be the best means of counteracting the blistering effect of the fumes. A total of 450 gallons of primer, 26,450 gallons of enamel, and 100 gallons of Inertol were used in painting these gates.

The final operation on the radial and trashway gates was calibrating the gate position indicator dial on the hoist. These dials

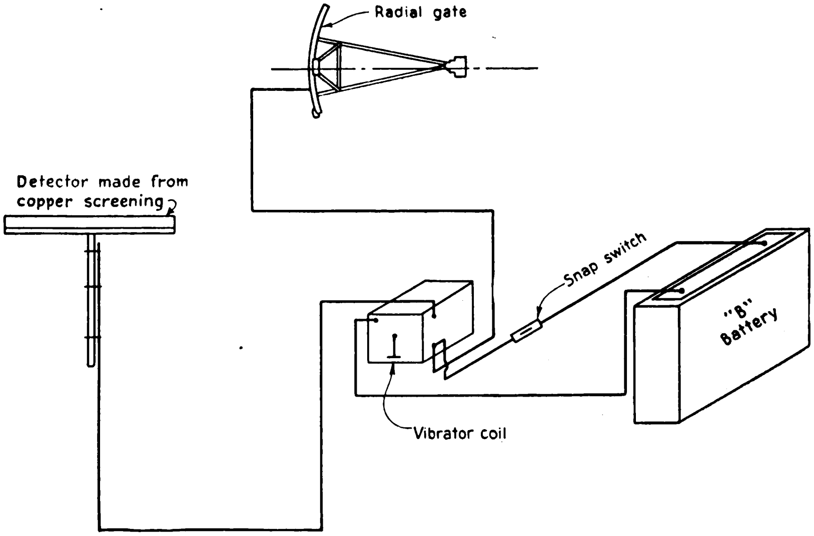

FIGURE 114.—*Paint flaw detector.*

were marked by the manufacturer to show the relative position of the gates only. As it is desired to control and determine the flow over Wheeler Dam very closely, the dials were graduated accurately to indicate the gate opening in feet.

The limit switches were then adjusted to stop the motors when the gate reached its limit of travel in either direction. Tainter gates have a travel of 15 feet 6 inches, while the trashway gates have a travel of 6 feet 1 inch. An average time of 13 minutes and 40 seconds is required to open the gates fully and an average time of 13 minutes and 20 seconds is necessary to close them fully.

NAVIGATION LOCK

A contract was let on November 12, 1932, by the United States Army Engineers to the Stevens Bros. and The Miller-Hutchinson Co. for the construction of the Wheeler lock, and work was started in January 1933. This lock was to be a single 37-foot lift structure; the pool was to have been at elevation 542. Following the passage of the TVA Act, the Authority made arrangements with the United States Army Engineers to change the design to conform to the development of a system of high dams. At that time the walls were redesigned for a pool elevation of 561; however, before construction was completed, the pool level was fixed at elevation 555 and the lock was finished to accommodate the 50-foot lift required. The lock was substantially completed early in 1935.

Additional work was done by the United States Army Engineers for the Authority to facilitate the construction of the dam and to utilize existing facilities of the Army Engineers and their contractor then engaged in building the lock. A contract was entered into on May 1, 1934, between the Army Engineers and the Authority for the Army Engineers to construct the nonoverflow dam section between the lock and the north abutment; to remove any existing obstacles to navigation at the entrance and exit of the lock and to do any required dredging upstream from the lock to provide a navigable channel; and to perform other construction work required for the protection and maintenance of navigation during the construction of the dam.

The lock, with temporary gates, was used for navigation during the later construction stages. However, before the pool was raised, it was necessary to construct the upper miter sill and install the permanent gates. Construction of the upper miter sill in the navigation lock was started in September 1936. Some of this work was done by the Authority's forces and equipment.

Tennessee Valley Authority equipment consisted of a floating mixing plant, a 20-ton guy derrick, concrete buckets, concrete vibrators, a portable compressor, several pumps, and the necessary towboats, barges, and derrick boat services. The United States Army Engineers furnished two derrick boats, unwatering pumps, pump barge, and towboats. They also furnished the necessary formwork, needle dam, and steel bulkhead, and most of the labor. Concrete aggregate was procured from storage piles at the dam site.

The needle dam was first erected across the lock chamber near the lower gate. After this work had been completed, the temporary upper gate was removed, and the steel bulkhead floated into position against the concrete blocks projecting into the lock chamber, and seated on the floor of the lock. The lock chamber between the needle dam and steel bulkhead was then dewatered until the concrete of the temporary upper miter sill was exposed.

At the downstream side of the sill, ordinary panel forms were used. Upstream, the forms were laid directly against the steel bulkhead. A total of 4,000 cubic yards of concrete was required. It was poured in six lifts with each lift divided into two pours—the dividing line between pours being the center line of the lock.

Concrete was mixed by one of the floating mixing plants which for this work was moored between the land wall and riverbank just downstream from the dam. The 20-ton guy derrick was on top of the land wall and transferred concrete buckets from the mixing plant to the miter sill. Buckets were lowered to the miter sill, and the guy derrick fall block released. From this point one of the derrick boats in the lock chamber moved the bucket to the desired placement point.

Setting of the gate seals was done by the United States Army Engineers, and the lock was reopened to navigation on December 1, 1936.

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CHAPTER 6

RESERVOIR ACTIVITIES

The Wheeler reservation area with approximately a 900-mile boundary extends into 6 counties. The preparation of the reservoir necessitated considerable diversified work such as the mapping and purchase of more than 103,400 acres of land, the moving of 840 families and 176 graves, the clearing of forests and structures from the draw-down area, the relocating or protecting of affected highways, railroads, and utility lines, and the providing of adjustments for other backwater damages. Labor from Civilian Conservation Corps camps stationed in the valley was not utilized for any of that work strictly necessary for the completion of the project to serve purposes of navigation, flood control, and power. Such labor was used, however, for related work in the fields of erosion control, park development, recreation, and general beautification of the area.

SURVEYS AND MAPPING

The preliminary surveys required for this project were made by the United States Army Engineers before the passage of the TVA Act. They included topography of most of the reservoir area to a scale of 1:6000 and a 5-foot contour interval; hydrographic surveys, including soundings of river bottom and river profiles, and certain section line surveys (as a basis for property surveys) by transit-tape and transit-stadia methods.

The TVA made additional surveys and maps for reservoir land purchases; highway, railroad, utility, and cemetery adjustments; contour marking for reservoir clearance; permanent marking of the TVA property boundary lines; and for special areas affected by backwater. Approximately 700 square miles were surveyed and mapped, including the establishment of horizontal and vertical control; contour markers were painted along the rim of the pool, measuring 1,063 miles; and permanent concrete boundary monuments were established along the TVA property boundary line at intervals not exceeding ½ mile. In connection with the land purchases, reconnaissance ownership surveys were made of 224,000 acres and approvals for purchase were issued for 103,400 acres.

The surveys were started in November 1933 and were completed in December 1936. A maximum field force of 70 men was reached in the summer of 1934, and altogether a total of about 420 party months, each field party averaging 3 men, was required to complete the mapping and surveying work. The field surveys were directed from an area office at Wilson Dam and from 3 unit field offices at Wilson Dam, Decatur, and Huntsville. Three to eight survey parties worked from each of these field offices.

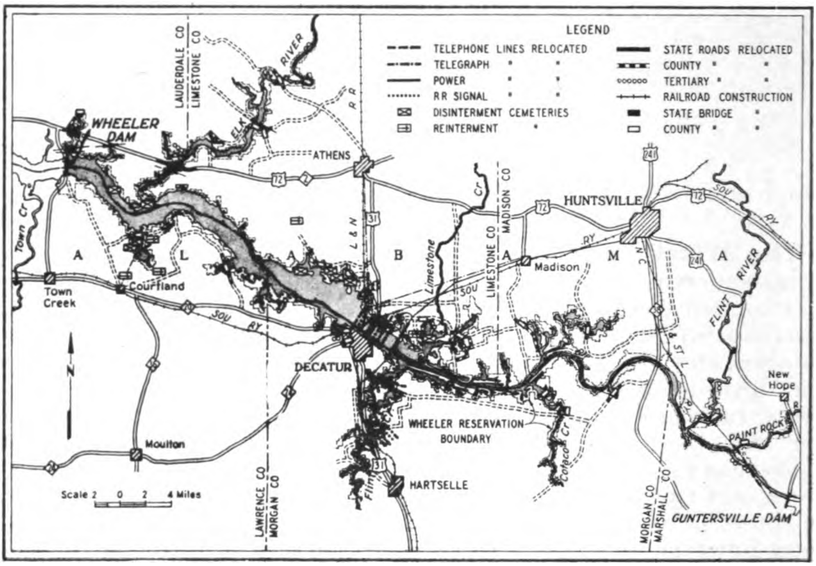

FIGURE 115.—*Adjustments in the reservoir area.*

Silt ranges were established at 126 locations across the reservoir for use in measuring the depth of future silting. The ranges were spaced from ½ mile to 1 mile apart. These ranges, established for future measurement of siltation in the reservoir, are closely connected to and utilized as the permanent basic control for the reservoir. Since neither the traverse nor the levels were marked by permanent monuments, the monuments marking the terminals of the silt ranges were utilized as the permanent basic control monuments of the reservoir.

Aerial photographs were secured by contract with a commercial aviation company as one of the first steps in the surveying and mapping of the reservoir area. These photographs furnish material for a comprehensive examination of natural features and of proposed improvements and their relations to property details. They reveal a wealth of helpful detail. Such features as topography, cleared fields and wooded areas, streams, roads and buildings, in relation to superimposed shore lines and property lines, stand out sharply under a magnifying stereoscope. The original photographs were taken from an altitude of about 10,000 feet, using a single-lens camera which furnished 7- by 9-inch negatives at a scale of about 1 inch = 1,200 feet. Enlargements of these photographs to a scale of 1 inch = 500 feet were used as a base for the property surveys; and these prints, with data added in the field, were later converted by the drafting office into permanent land maps. Field checking of the photographs was essential to determine the exact scale. The aerial-photograph survey method was not feasible for every tract. It was used, however, for practically all farms ranging from 15 or 20 acres to the largest. It is estimated that for these farm surveys the cost by the aerial-photograph method was about one-half of what the cost would have been by the tape-and-transit method.

Numerous tests have proved this method to be very accurate and that the variation in acreage per tract from the test surveys is less than one percent. The aerial photographs were also very useful to the land appraisers and for other purposes such as studying forested areas, estimating types of timber to be cleared, and studies of land use.

Horizontal control.

The horizontal control established consisted of transit-tape traverse lines of third-order accuracy (1:5000), originating and closing on triangulation stations of the United States Coast and Geodetic Survey, or closed in loops. The traverses were routed as much as possible along roads or other feasible locations near and above the proposed shore line. Connecting cross ties were made at frequent intervals, so that the traverse circuits would be nearly square or round rather than long and narrow. River crossings of the traverse were made approximately every 10 miles. The instrument stations were marked by hubs, iron pins, or other such semipermanent markers.

Vertical control datum.

Vertical control surveys of third-order accuracy [1] were run along the same routes selected for the traverse lines. The levels originated and closed upon the permanent first-order and second-order benchmarks of the United States Coast and Geodetic Survey. The third-order benchmark usually consisted of a copper washer held by a copper nail, driven into the root of a substantial tree or building. These semipermanent benchmarks were set at intervals, averaging three-fourths mile, along each level line. Permanent concrete control monuments were established which also served as terminals for 126 silt ranges.

Elevations supplied by the United States Coast and Geodetic Survey are computed on the 1929 General Adjustment Level Datum. The level datum for the construction of the Wheeler Dam and lock, which was well under way before the United States Coast and Geodetic Survey had completed and published its 1929 datum elevations, is known as the Fourth General Adjustment of 1912. To convert elevations on the 1912 datum to the 1929 datum at Wheeler Dam, 0.28 foot must be added to the 1912 elevations.

ENGINEERING FOR LAND ACQUISITION

Engineering work for land acquisition included comprehensive studies for establishing the location of the Authority's purchase boundaries; preparation of legal deed descriptions for reservoir projects and rights-of-way; preparation of approvals for land purchases; and studies relating to property damage claims. Field engineering work included surveys for scale checking aerial photographs; property ownership reconnaissance surveys; deed copying; surveys to establish property boundaries, topography, and cultural features; and special surveys required for establishing the purchase boundaries. Office

[1] Closures of 0.05 foot $\sqrt{\text{distance of line in miles}}$, or better.

drafting and computing work included computations for scale determination of aerial photographs; preparation of property reconnaissance maps; construction of final property maps showing boundaries, topography, and cultural features; preparation of special maps required for establishing the Authority's purchase boundaries; and preparation of maps relating to property damage claims.

The property survey program consisted of two principal parts. The first included the land acquisition surveys, or surveys which are sufficiently complete and accurate to permit purchase of the necessary lands, but not necessarily accurate enough to serve as a basis for later retracement surveys. Since most of the property boundary lines between individual property tracts were to be inundated, the ability to retrace or restore accurately these lines at some later date was considered unimportant. It was only necessary in the beginning to provide surveys and plats which would permit acquisition of the tracts to be accomplished legally, fairly, and efficiently. The general term, "acquisition," as used in this report, includes appraisal, abstracting and title examination, and procurement by purchase or condemnation proceedings.

The second part of the property-surveying program included the monumenting and measuring of the final outside boundary line of the Government reservation. After all of the land in the reservoir area is acquired, the boundary line between the Government-owned and privately owned land becomes important. The ability to relocate it precisely at some later date is essential. Also, its location on the ground must be known with certainty at all times so that policing, fencing, clearing, cultivating, and other administering may be with regard to the proper boundary line.

Land acquisition surveys.

It was necessary, first, to make a preliminary property reconnaissance survey to furnish essential information for the convenience of the deed copiers and to facilitate the property surveys coming later. Each property owner or tenant was interviewed and asked to point out the boundaries of his tract. These boundaries were identified and sketched on contact-print aerial photographs (1 inch=1,250 feet). The surveyor also obtained the owner's name, the date of purchase or inheritance, and other such important data, so that deed-copying, abstracting, and title examination would be aided. These reconnaissance photographs were later furnished to the field property surveyor for his use in making the more refined property surveys.

Immediately following the reconnaissance, deed copiers in each county seat copied the recorded deed or deeds for each tract. Many unrecorded deeds were also borrowed from the different owners. After copying, each deed was examined, the descriptions of the boundaries being compared with the approximate boundaries shown on the reconnaissance photographs. Major discrepancies between deed descriptions and field reconnaissance sketches were reconciled before the property surveys were started.

The property tracts were classified according to size into three general groups. The first group, comprising farm tracts ranging in size from 10 to several hundred acres, was surveyed almost entirely by the use of aerial photographs. The second group, including the small tracts, ranging in size generally from ½ acre to 10 acres, were usually surveyed to scales ranging from 1:2,400 to 1:600 by the plane-table method. The third class of properties, comprising the city lots or subdivision lots, was surveyed by the usual transit-tape method, or by using the recorded plats if these were available and proved to be sufficiently accurate.

In the aerial photographic survey method, enlargements of the aerial photographs to the scale of 1:6,000 were used. The scaling accuracy of the enlargements was checked frequently by measuring new scale lines on the ground, then comparing these lengths with the same lengths scaled from the enlarged photographs. Such tests indicated that 95 percent of the enlargements were correct for scale at the adopted datum plane within one-half of 1 percent, and that the scaling error rarely exceeded 1 per cent. The photographs were prepared for the field by marking in ink the "net center area," this area being defined by the midoverlap lines. Since end overlap of the aerial photographs averaged 60 percent and side overlap averaged more than 50 percent, only a small central portion of each photograph would come within the "net center area." The field surveyors were instructed to keep their photographic plotting as much as possible within this net center area, to minimize the effects of relief displacement or tilt.

The field surveyor was furnished with the 1:6,000 scale photographic enlargements, the property-reconnaissance photographs, copies of the deeds, and plane-table outfit. The owner of each tract was contacted; and, if possible, several adjoining owners were called together at one time to determine from them jointly their common boundary corners and the nearby section corners. If all interested owners agreed, and search on the ground did not reveal any conflicting evidence, these corners were plotted as precisely as possible on the enlarged photographs. If the property corners and lines were directly identifiable on the photographs, such as fence lines in cleared areas, roads or lanes, or individual trees, the plotting was merely a matter of precisely pricking the proper images on the photographs. If they were not directly identifiable, then the property corners were located on the photograph by plane-table traverse.

Such traverse was accomplished by using the photographs as plane-table sheets and using conventional plane-table traverse methods. Each traverse was originated and closed on identifiable image points, as near as possible to the corner to be plotted. In the hilly terrain in the eastern part of the reservoir, approximate elevations were determined for the starting and closing points from barometer readings. These points were then replotted toward or away from the center of the picture by the proper amount for relief displacement. The plane-table traverse was then begun at the replotted originating point, was run through the desired property corner, and on to the replotted closing point. The closure, if small, was distributed graphically. The property corner then appeared on the photograph in correct map location, but this location did not always agree exactly with correct photographic location because of relief displacement. To make the property corners on the photographs represent the photographic position, approximate elevations were determined for the property corners and the points replotted toward or away from the center of the photograph by the proper amount for relief displacement. Barometer elevations, correct probably to the nearest 25 feet, were secured on each property corner in the eastern part of the reservoir, whether traversed or merely identified, so that the corners could be corrected later in the office for relief displacement.

In the western, or downstream, part of the reservoir area, where the terrain is fairly flat or gently rolling, approximate elevations were not secured and relief displacement of photographic images was ignored.

Where the property owners were not able to point out or agree upon property corners, the surveyor usually was required to make a conventional transit-tape resurvey from the deed records or other evidences, and establish the corners on the ground before plotting on the photographs. Also, since this reservoir, located entirely within the State of Alabama, had been surveyed and subdivided some 100 years earlier by the General Land Office into townships, ranges, and sections, many of the property lines followed these subdivision lines. One of the major items of work was, therefore, to find or reestablish on the ground the original location of each section corner. In this operation, the enlarged aerial photographs proved to be of great help by indicating the approximate positions where missing section corners should be located. Using the photographs as a scale model of the ground and substituting an engineer's scale for a tape and straightedge for transit, the probable original alignment of the old official survey was drawn on the photographs. Then, taking these photographs to the field, a thorough search was made in the vicinity of each corner location as indicated on the photographs. In many instances, the original corner was found at, or within a short distance of, the location indicated on the photographs. If the corners or lines, or evidences of the corners or lines, were not found, it was necessary to resort to conventional transit-tape retracement surveys and, by single or double proportion, to establish new corners in the proper locations, all according to the published rules of the General Land Office.

While on the location, the surveyor outlined in pencil on the enlarged photograph all of the features to be shown on the final plat, such as roads, lanes, trails, buildings, orchards, woods, bridges, culverts, streams, canals, millraces, railroads, power and telephone lines, and other features. He also lettered on the photographs all necessary explanatory notes, describing each property and subdivision corner, whether stone, post, fence corner, or tree; also describing each property and subdivision line, whether fence, tree row, timber line, road, or stream. For assistance to the draftsmen who would later compile the property map, an inventory of structures and improvements on each property was prepared.

The elevation 556 flow line and clearing contour had been marked previously on the ground. This contour was drawn on the enlarged photographs in correct photographic location identified directly by images where practicable or plotted by means of plane-table traverses. The elevation 560 contour, being the general guide for the property purchase line, was plotted similarly. Many of the photographs, however, had to be returned to the field later for additional elevations in critical areas, sometimes adding elevations of certain spots and sometimes additional 1- or 2-foot contours.

For the downstream half of the reservoir, the 1:6,000 scale topographic sheets of the United States Army Engineers, which showed section lines, were used as the base sheets for the land maps. A new skeleton tracing, showing only the section lines and called control tracing, was made from these sheets. The control tracing was then placed over the enlarged property photographs, the photographs fitted to the tracing by means of the section lines, and remaining property lines and other details then traced directly from the photographs. In the upstream half of the reservoir, where no such topographic base sheets existed, the land maps were prepared to a scale of 1:6,000 by the radial-line procedure, using the third-order reservoir traverse stations as ground control and compiling a skeleton control sheet, establishing section corners and other critical points by radial-line triangulations, or standard principal-point photographic traverses. After these key points and the picture centers had been located on the tracing, the remaining property corners and other map details were traced directly from the photographs. The acreage of each tract was then planimetered on the map tracing, and distances and bearings scaled where necessary for metes-and-bounds descriptions of irregular tracts.

Whenever a small tract (all tracts below 10 acres and many tracts of between 10 and 20 acres) was encountered, the survey was made by plane table on paper sheets, the plotting scale being from 1:2400 to 1:600, depending upon the size of the tract. Each such plat showed the same information shown on the photographic plats. In the office, the small tract was shown in correct position on the 1:6000-scale land maps, then traced in the marginal area of the map sheet to the same scale plotted in the field.

For city building lots, the original record plats were obtained, the street and lot locations found or established by resurveys on the ground, and mapped in accurate position on large scale plane-table sheets.

Monumenting and surveying the reservation boundary line.

As rapidly as land purchases were completed within certain areas, the reservation boundary line was monumented and surveyed. A momument was placed at each angle point in the boundary, at all section and quarter-section corners along the boundary, and at each intersection of the boundary with a section or quarter-section line. In no case was the distance between monuments greater than one-half mile. Where the actual corner was occupied by a natural mark, as a tree, a monument was set nearby on the Government land as a witness or reference corner. Where the reservation boundary corner fell in a stream, a witness corner was set on the bank. Where a corner fell in a roadway, the monument was buried about 6 inches below the surface (except in the case of hard-surfaced roads) and a witness monument set in the right-of-way line of the Authority's property.

The first step in this survey was to coordinate, by means of transit-tape fourth-order traverses,[2] the section or quarter-section corners of each square-mile section within which the purchase line fell. From these coordinates of the section and quarter-section corners,

[2] Accuracy of 1 : 1000 or better.

the coordinates of the interior unmarked boundary-line corners, which usually followed along the quarter, eighth, or sixteenth subdivision lines of the section, were computed. These missing corners were then established on the ground by traverse surveys from the nearest coordinated stations. If the theoretical corner or line thus established fell within reasonable distance of a marked line on the ground, the marked line was ordinarily taken as the subdivision line.

Where the purchase line followed existing well-marked lines, such as a fence, road, lane, or hedge, or ran between existing corners, the coordinate method of subdivision was, of course, not necessary.

By the coordinate procedure, it was not necessary to traverse in its entirety each course of the boundary line or the perimeter of each square-mile section. In many instances it was necessary only to occupy a traverse station, or a previously coordinated section or quarter-section corner, and establish the desired missing corner by means of a short spur traverse.

Since the coordinates used in this survey are on the Alabama State Plane Coordinate System, the bearings on the boundary courses may not agree exactly with either true astronomic bearings or local compass bearings. This should be no cause for confusion to local surveyors, especially since it appears that the Alabama State plane coordinate system is gradually coming into general use. In any event, each reservation boundary plat carries explanations, giving the essential information required for using the State coordinate systems.

Index plats of the boundary-line survey, showing the number and coordinates of each monument and the distance and bearing of each course of the boundary, have been prepared to a scale of 1:24,000, using the planimetric map sheets as a base. It is expected that, sooner or later, these will be superseded by larger scale land-record plats which will give more complete information on the boundary-line monuments and survey, as well as ownership data on each of the tracts acquired (notations of unacquired interests, rights excepted, access rights granted, and other information). It is further expected that copies of each boundary-line plat will be filed in local courthouses for the use of local surveyors and engineers, abstractors, title examiners, and others needing this information.

LAND ACQUISITION CONTROL

A central organization controlled land purchase for all projects, and was alone authorized to determine, within prescribed limits, the exact location of the project boundaries. These limits provided for the acquisition of a marginal strip in addition to the lands actually flooded. General authority was granted for: acquiring all land lying below a line extending ⅛ mile beyond the 560-foot contour; purchasing any tract of land containing 20 acres or less lying beyond the ⅛-mile line that was attached to and a part of land below this line; and omitting from purchase lands within the ⅛-mile limit considered unnecessary for the project in order that valuable agricultural land could be left in private ownership.

The land purchase control organization made the investigations required to determine the extent of the land purchases, to prepare

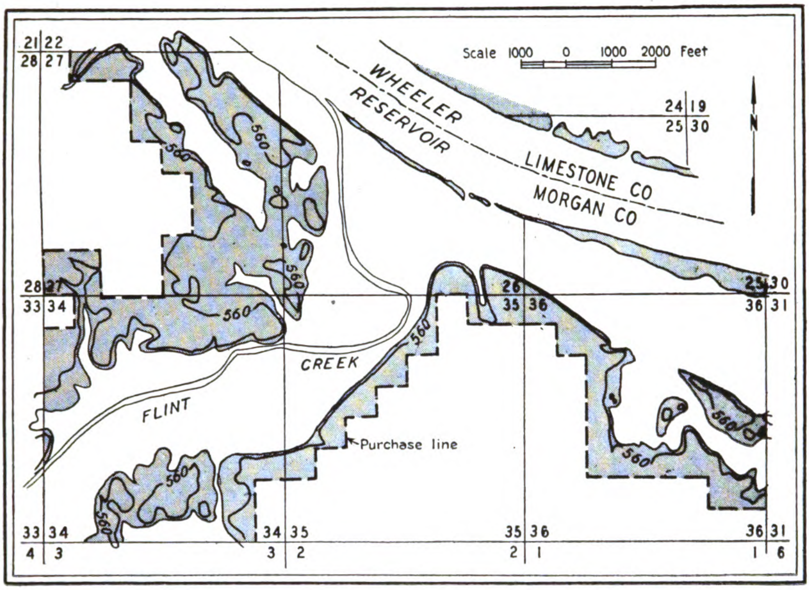

FIGURE 116.—*Typical boundaries of land acquired in even subdivisions and sections.*

legal descriptions, and to issue formal approvals for land acquisition. A study was made of each property, not subject to inundation in its entirety, in which the owner's welfare was considered as well as the requirements of the Authority.

The reservoir area, a region of broad rolling plains, was favorable to large plantation-type farms with boundaries conforming in general to the Government sectionalized subdivision. In many cases a farm extended into several sections. Therefore, most farms could sustain the severance of considerable areas without serious disruption of the farm unit. As a result of the flat topography some areas were encountered where there was no conclusive evidence regarding the extent of future damages and, since substantial costs were involved, these were not purchased. An outstanding example is the sinkhole area around Spring Creek discussed later in this chapter. Although the bulk of problems handled concerned farm tracts, considerable time and study were expended on special problems involving water-front and industrial development, salvage of lands by means of protective structures, provision of new sources of raw material to replace those flooded, and the preservation of recreational areas and lands of unusual scenic value. Some of the special problems will be discussed.

Along the Decatur water front, it was decided, in general, to acquire flowage easements instead of fee simple titles. The decision was contrary to the policy adopted for lands elsewhere and was made more to permit the Decatur industries to retain the raw water supply and navigation rights and to overcome the objection to retiring the area from private ownership than for the economies that were effected. The land which seemed most suitable for navigation terminal development, however, was approved for purchase in fee simple title.

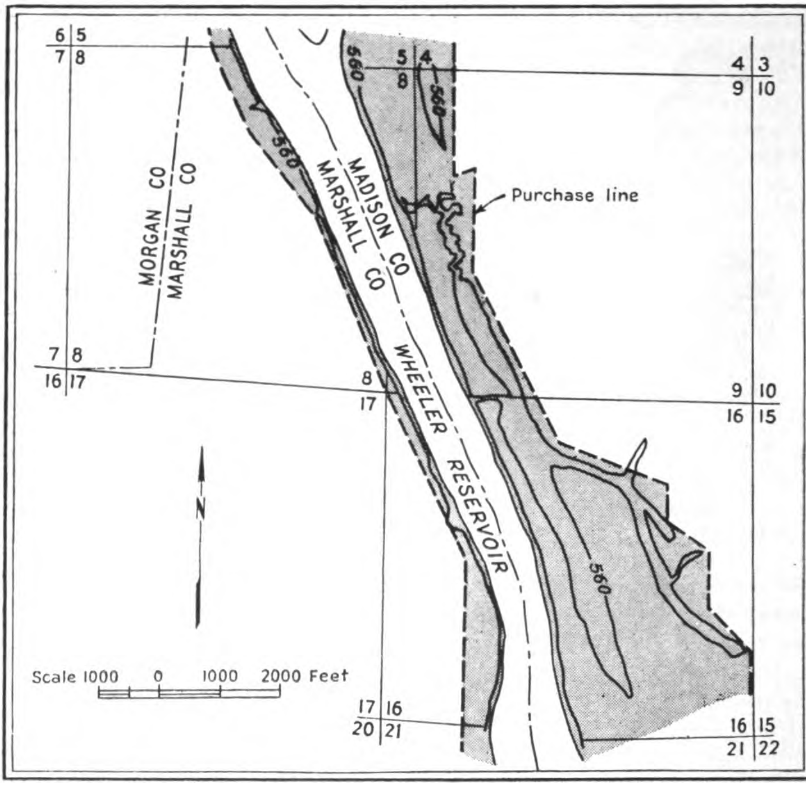

Figure 117.—*Typical boundaries of land acquired without regard to sectional lines.*

The area adjacent to the Dry Creek embayment of the Wheeler Reservoir, although ill-suited for the purpose, had been subdivided into residential lots. Shallow flooding by the reservoir would make this area still less desirable for residential purposes and in addition would create a swampy area within the city limits of Decatur. This property could, however, be left in private ownership by improving the channel of Dry Creek and by a small amount of filling and grading work. Accordingly, it was decided to acquire flowage easements and construction rights for a nominal monetary consideration (usually $5) with the understanding that the necessary construction work would be done.

Consideration was given to acquiring the lands in the vicinity of the reservoir along Spring Creek where it was known that numerous sinkholes existed. Action was deferred, however, until the reservoir had been filled in order that more definite information would be available regarding the effect of the reservoir upon these sinkholes. From gages installed at several locations it was learned that the elevation of water in various ponds was increased and that the water remained at higher elevations for longer periods. In addition greater difficulty was experienced with mosquito control than was anticipated. Accordingly, the acquisition of easements on approximately 500 acres and the draining of the main ponds involved were authorized. The construction of the drainage ditches required the excavation of about 200,000 cubic yards of earth.

The purchase of a small number of parcels of land for protection, recreational use, and preservation of special areas of scenic beauty was approved. This land lies near the mouth of Paint Rock River in the upper reaches of the reservoir.

The following tabulation shows the extent of the land approved for purchase for the final reservoir at elevation 556.3 [3]:

Shore line	miles	899
Land acquired for pool to elevation 556.3:		
Woods	acres	30,948
Cleared	do	18,440
Total	do	49,388
Public lands in stream beds	do	17,441
Total land acquired (Sept. 30, 1938)	do	[1] 95,097
Total reservation area	do	120,912

[1] Excluding 81 tracts in process of condemnation on Sept. 30, 1938.

LAND ACQUISITION

Reservoir land acquired for flooding, erosion control, and other purposes consisted of 1,296 tracts [4] with a total of 95,097.08 acres (see table 27). Of the property purchased, 78.2 percent was acquired by voluntary transfer; 8.6 percent had to be condemned for title reasons; and 13.2 percent had to be condemned because of refusal to sell at the appraised value. The total cost of appraising, buying, and title closing for land acquisition plus the cost of acquiring all miscellaneous property rights in the area such as right-of-way ease-

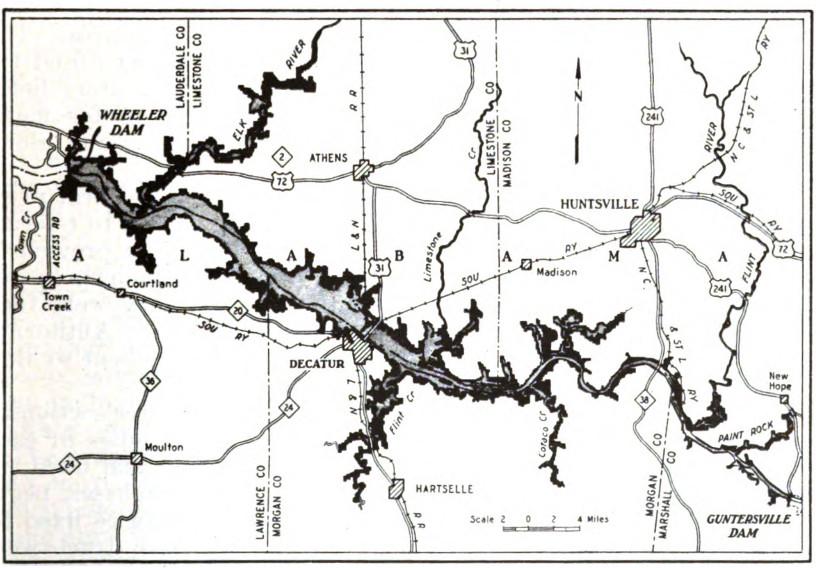

FIGURE 118.—*Land acquired for the reservoir and for reservoir protection.*

[3] 1929 adjustment; see p. 221.
[4] Excluding 81 tracts in process of condemnation on September 30, 1938.

ments for relocation of highways, railroads, telephone and transmission lines amounted to $538,457.34.

The agricultural land in the Wheeler area was more productive than that in the Norris area. Land was held in larger units farmed under a tenant system. It was far less intensively improved, and such improvements as were maintained on the land were far less valuable than those found in the Norris area. The average reservoir tract in the Wheeler area was 81.76 acres and cost $44.11 per acre, $40.11 for land and $4 for improvements. The average tract in the Norris area cost $31.50 per acre, with $20.17 per acre for improvements.

TABLE 27.—*Land acquired for Wheeler Project as of September 30, 1938*

	Tracts	Acres	Land cost	Improvement	Total cost
Distribution of land purchases in fee: Reservoir [1]	1,142	93,373.12	$3,744,711.73	$373,749.95	$4,118,461.68
Distribution of land easement purchases:					
Flowage	118	1,680.89	109,746.62	12,928.50	122,675.12
Highways	33	42.53	4,046.00	206.00	4,252.00
Railroads	3	.54	499.00	50.00	549.00
Total land and land rights [2]	1,296	95,097.08	3,859,003.35	386,934.45	4,245,937.80

[1] Average cost, $44.11 per acre.
[2] These totals exclude 81 tracts in process of condemnation, and 14 tracts totaling 523.2 acres, 6 of which were acquired by transfer from the U. S. Army and 8 of which have been approved for purchase but not yet acquired.

The land was acquired by unencumbered fee purchases except where there were outstanding mineral rights in properties lying above the flood level of the reservoir and where it was found more desirable to purchase easements. No attempt was made to acquire such mineral interests; this resulted in lower acquisition costs and avoided retarding possible mineral development in the area. The greater part of the land acquired for the reservoir had been used for agriculture, corn and hay having been grown on the bottom fields and cotton and truck on the uplands. Flowage rights for backwater adjustments were acquired in the Dry Creek area and along the Decatur water front.

The standards for appraisals were such as to fix a uniform price for similar property at values which would enable owners to relocate with surrounding conditions similar to those which they previously enjoyed. Landowners were permitted to remain on their property until possession was needed by the Authority, and they were then permitted to remove any improvements not needed by the Authority. Surrender of possession was fixed during the winter season so that there would be no liability for ruin of crops.

The land acquisition organization was divided into three sections—appraisal, buying, and titles. The appraisers made studies of each tract to be acquired, evaluating soil and improvements, and fixed the final price to be offered for the property. For fee purchases, negotiations were made on a nonprice-trading basis at the price fixed by a board of appraisal and review. Contracts for each tract were examined by a title closer, who, after clearing all defects in titles, would initiate payment of the purchase price or recommend other action.

CLEARING OF RESERVOIR LAND

The preparation of the reservoir required the clearing of 31,228 acres of woodland. This work began in February 1934 and was completed in August 1936. The area lay in vast swamps on each side of the Tennessee River and its tributaries and extended nearly 80 miles up the river from the dam site. Interlacing the swamp were many sloughs filled with water and lakes in which were thick growths of tupelo and other semiaquatic timber. The entire reservoir area was cleared of all trees and underbrush below elevation 556.3. This is 1 foot above the maximum pool level required for the malaria control fluctuations. A small amount of clearing was done above elevation 556.3 where special malaria control problems existed. The work was conducted in accordance with the requirements of the Alabama State Board of Health governing the impoundment of waters.

The cost of reservoir clearing activities per acre was as follows:

Cutting and piling	$85.74
Burning	6.12
Malaria control	.46
Reshrubbing and modified clearing	1.41
Control of flotage	.10
Damage payments	.45
Sale of timber and buildings	[1] .42
Total reservoir clearing activities	[2] 93.86

[1] Credit.
[2] Base cost.

An average of 22.8 man-days per acre was used in the clearing operations. All labor personnel worked on an 8-hour shift, 5 days a week.

The field superintendent in charge of the clearance work was assisted by a clerical staff and assistants who directed the various phases of the work. Time checkers in light trucks checked the working forces in widely scattered areas, reporting their time each day to the project office at Decatur, Ala. The time checkers also delivered tools and supplies needed by clearing crews.

The operating forces were divided into independent units of 70 men directed by a unit foreman. Under him were three subforemen, a saw filer to care for small tools, and a trained first-aid man who rendered emergency first-aid treatment, dressed minor injuries, and served as a clerical aide to the foreman. The three subforemen directly supervised the three phases of the work—the bush hook and axe crew, the saw crew, and the piling crew. In order to take full advantage of the dry season, when swamps had dried out, the clearance organization was expanded in 1934 to 50 units employing 3,600 men, including 50 unit foremen and 150 subforemen, practically all of whom were recruited from the labor forces.

The order of clearing operations was: first, the bush hook and axe crew moved into the area and cut and piled underbrush and small timber; second, the saw crew felled the large timber; and third, the piling crew with teams piled all waste timber and debris for burning. Thus, as the unit moved along the riverbank and through a swamp, it left behind a completely cleared area ready for burning except for

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merchantable timber. Any merchantable timber was later removed by contractors and sawed into lumber for construction purposes or sold as logs for pulpwood, veneer, handle stock, and other purposes. Many thousands of cords of waste timber suitable for firewood was given to those living in the neighborhood who hauled it away.

After waste timber and other debris had seasoned for several weeks, it was burned. Most of the burning was done by farmers and others under contract. The burning at some of the most difficult places, such as along the riverbank and creekbanks subject to sudden overflow, was done by the Authority's own forces.

FIGURE 119.—*Bank-pulling machine.*

All merchantable timber that could be salvaged economically was sawed by small contract mills for use on the Authority's construction projects. Logs not suitable for this purpose were sold. The timber utilization thus amounted to:

	Feet board measure
Lumber used by the Authority	7,450,872
Logs sold to outsiders	2,578,755
Total	10,029,637

Labor was recruited from the surrounding region and consisted of men experienced in the use of the axe and crosscut saw. Of the 3,600 men at the peak of the organization, between 700 and 800 were Negroes—the number employed being governed by the ratio which they bore to the population of the area. No camps were necessary because the men were employed from surrounding regions.

Operations were begun on February 16, 1934, and by December 1934 the organization had grown to approximately 3,000 men. At that time, because of bad weather, the organization had to be reduced by half. The men voted for alternate weeks' work for all rather than complete lay-off of half of the force. Thus half-time work continued until the early summer of 1935. The Authority encouraged the men to utilize their off weeks in planting gardens and small crops.

Clearing was hampered by the presence of many swamps. In addition, numerous depressions or pools existed near the pool level of the reservoir which would become stagnant ponds when the pool level dropped. It was found that drainage would be practical and economical in clearance work and would materially aid in the control of mosquito breeding and reduce malaria-control expense. Light-weight crawler draglines with ½-cubic-yard buckets were used for digging the ditches for draining these areas. When not needed for excavation work, they were also used for piling logs. In preparing these drainage canals, 117,000 cubic yards of earth were excavated to facilitate clearing, while 23,700 linear feet of lateral ditches were dynamited to promote drainage. (See figure 120.)

FIGURE 120.—*Light-weight draglines.*

The two outstanding pieces of machinery used in the clearance work were the bank-pulling machines and the lightweight crawler draglines for draining swamps. These machines were selected after a thorough investigation of available equipment. (See fig. 119 and 120.) Power saws were investigated but were not used because it was felt they had not been developed sufficiently.

A proposal was made to use skidders in some of the flatter regions. This machinery was expected to draw untrimmed felled trees from a 2- or 3-acre tract into a centrally located pile for burning. Tests, conducted and run by a manufacturer's representative, revealed that clearing cost for such equipment was about double the cost of unit-crew methods being used by the Authority. The scheme failed because of unpredictable difficulties and delays experienced in handling cable and in getting the trees to "ride" up the pile without gouging into it.

Bank-pulling machines, developed by the Authority, were used for dragging timber out of the water, up the bank, and then to suitable ground for the saw crew. The importance of these machines is recognized when it is understood that all along the Tennessee River and its tributaries there grew a thick fringe of timber which overhung the water and that the most economical manner of disposition was to cut the timber, let it fall into the water, and drag it to dry land. It is estimated that the use of the machines resulted in a saving of $250,000 compared to the cost of doing this work with any other equipment then on the market. This machine consists of a Caterpillar 35 tractor equipped with a boom, stiff-leg, and winch. Each machine cost approximately $5,000 and weighed about 20,000 pounds.

Lightweight crawler draglines with one-half-cubic-yard buckets were used for excavating the canals for the drainage system. At other times, this equipment was used for piling logs. Speeder draglines, which cost $5,600 each, were used in this work.

The clearing organization also did some special work at various places to facilitate malaria control, including: the construction of drainage ditches involving 48,000 cubic yards of excavation; poisoning of willow trees over an area of 576 acres; and cutting second growth timber over an area of 3,700 acres. The malaria control activities are discussed in chapter 7.

Pool contour marking.

Fourth-order levels, originating and closing on the third-order benchmarks of the reservoir control system, were utilized in marking the elevation 556.3 [5] contour line on the ground. In wooded areas, a band about 2 inches wide was painted in white on the trunks of trees at the contour elevation. The trees were selected at such elevation that the marks would fall about one foot or less above the ground. The marks in wooded areas were spaced so that the next mark was always visible, the distances between marks ranging between 25 and 75 feet. Through cleared areas, the line was marked by white stakes and by white marks on fence posts, buildings, and other structures.

The fourth-order contour levels were tied to the third-order control benchmarks at intervals of 1 or 2 miles, the accumulated closing error seldom exceeding 0.3 foot. A partial inspection of the pool contour in the downstream part of the reservoir, made when the water reached elevation 556.3, disclosed no errors in the elevation of this contour greater than 0.3 foot. The original contour-marking program included 1,150 miles of contour and extended upstream several miles above the present Guntersville Dam. After the contour mileage above Guntersville Dam is subtracted, there remain 1,063 miles of Wheeler Reservoir shore line at the 556 elevation, including islands.

Navigation contour marking.

Within certain areas designated by the United States Army Engineers, all stumps were sawed off flush with the ground between the 556.3 and the 538.3 contours. The 538.3 contour, called "navigation contour," was marked with red paint, using the same procedure as in preparing the 556.3 shore-line contour.

ADJUSTMENTS TO COMMUNITIES AND INDUSTRIAL PLANTS

Adjustments made necessary by the backwater from the reservoir were confined to the vicinity of the city of Decatur, Alabama, and affected the city's water supply system, the sewerage system, and eight industrial plants.

[5] 1929 datum; see p. 221.

At the west end of the city, flowage rights were required over a large area along Dry Creek which was regraded and improved providing a satisfactory shore line. This area included land belonging to the city, and the adjustment included reconstruction of streets and relocation of the Dry Creek main sewer. About 10,000 acres of river bottom land across the river from Decatur were to be inundated by the reservoir to a depth of from one to eight feet. Originally about half of this land was in cultivation, the remainder being a heavily wooded swamp. An extensive investigation was made to determine the feasibility of protecting this land from overflowing and draining it so that it could be returned to agricultural use. Such a plan involved a levee of considerable length, interior drainage ditches, and a pumping plant for discharging drainage water over the levee into the river, and would require perpetual operation and maintenance. Also, the protection of this area from floods would materially reduce the surcharge capacity of the reservoir flood control. After consideration of its relation to the whole project of river control, it was decided to inundate this property.

Water for the city of Decatur is supplied by the Alabama Water Service Co. which obtains it from the river near the easterly boundary line of the city and filters it in a purification plant adjacent to the riverbank. The original plant consisted of a pumping station with steam pumps and an intake line running into the river a substantial distance from the bank. The pumps were located on a floor at elevation 546, which elevation was necessary in order that a supply could be obtained at periods of extreme low water. The floor, however, was about 10 feet lower than the proposed pool level and substantially lower than the water surface of great floods of the past. The steam pumps were superseded by motor-driven, centrifugal pumps, but a portion of the steam equipment was left in place to care for an emergency occasioned by the possible loss of electric power during a flood period. Although the intake was provided with gates, inflow into the station was possible through springs over which the station was built and such an inflow might submerge the pumps if pumping were not continued throughout the flood period.

The company claimed that a constant reservoir level at elevation 554 to 556 caused radically changed conditions and made it necessary to provide for dependable continuous pumping rather than emergency pumping at flood periods only. Under normal operation the city supply could be furnished with about 10 hours' pumping per day. It was also claimed that the construction of Wheeler Dam might result in an undesirable quality of water at the bed of the river where the intake was placed and, further, that the filtered water in the existing clear water well might be contaminated by infiltration of river water as a result of the higher level of Wheeler pool. Another consequence of the creation of the reservoir was the flooding of land in which certain water mains were located. Since the original plant was built about 1889 and some portions were obsolete or inefficient compared with recent practice in water treatment, the company desired to make substantial improvements.

A settlement was made with the Alabama Water Service Co. under the terms of which the Authority obtained full flowage rights over land belonging to the company and a release from all obligations

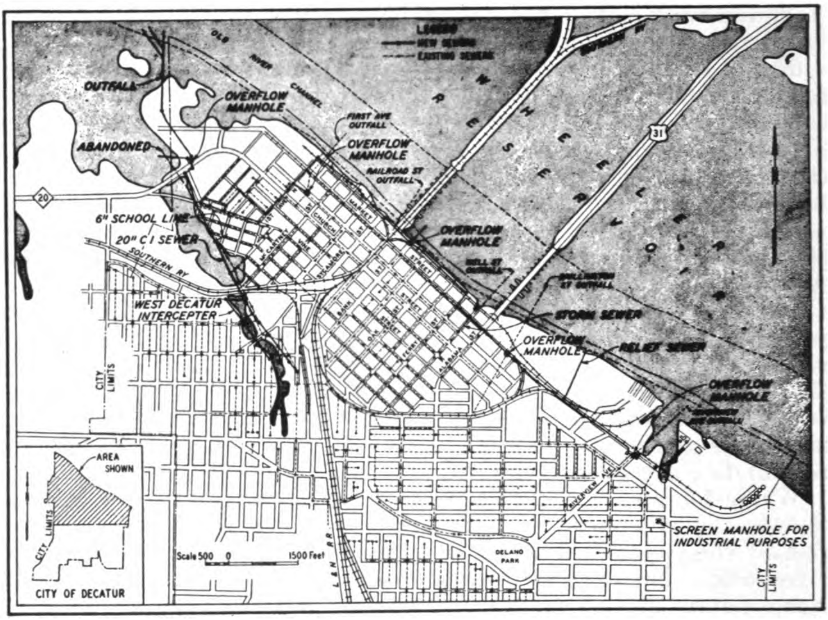

FIGURE 121.—*Adjustments to sewerage system at Decatur, Ala.*

arising from the effect of the reservoir on the waterworks plant. By the terms of this settlement, the company was paid approximately $60,000. To this amount the company added an amount sufficient to build a new intake, new pumping station, filters, and clear well, and for the reconstruction of the entire plant with the exception of the recently constructed coagulating basin which was on comparatively high ground.

The sanitary sewerage system of the city of Decatur had been developed sufficiently to serve nearly the entire city. The sewage was discharged into the river through six outlets, one of which was located only about 500 feet downstream from the water supply intake. This was an unsatisfactory condition because of the probability of sewage being carried to the intake by wind action or eddy currents during low water periods. The principal effect of the reservoir was to submerge some of the sewers, thus preventing free discharge. These sewers were subject to river backwater during flood periods, but such a condition was temporary and not a serious disadvantage.

When adjustments were considered by the Authority, three principal features were studied: first, the collection of the sewage and its conveyance through an intercepting sewer to the westerly or downstream boundary of the city; second, the installation of pumping equipment for raising the sewage to the pool level; and third, treatment of the sewage. Treatment was first considered on the ground that it would be objectionable to discharge the sewage into comparatively still water instead of to the flowing river. It was found, however, that the increase of low water flow owing to the construction of Norris and other reservoirs would provide a much greater dilution of the sewage than under natural conditions, and a minimum velocity in the reservoir not much less than during low water seasons of the past. Up until the present time, 1939, the State of Alabama has not required the treatment of sewage by the municipalities located on the river and discharging sewage to the main stream. After consideration of conditions by the Authority and the city officials, an agreement was reached whereby the Authority agreed to construct intercepting sewers to convey the sewage to the river immediately downstream from the western city limit and to construct two pumping stations to lift the sewage to the outfall. The total cost of the sewerage system adjustment was $207,500. This amount includes $25,000 paid to the city, chosen by the city council in preference to the construction of a sewage treatment plant. The city has assumed the operation of the pumping stations and the care of the new sewers and pump maintenance of the existing system, the Authority supplying electric power for the operation of the pumps.

Adjustments necessary to the plants and facilities of 8 industrial concerns along the water front at Decatur were so planned that regular manufacturing operations were hampered as little as possible. For this purpose, it was necessary to purchase 35 flowage easements and 9 tracts in fee simple. Condemnation proceedings were instituted against 11 owners. In every instance a final settlement was made releasing the Authority from any further liability. The investment in water front property and the settlement of damages in or near Decatur cost the Authority $350,000. At 3 plants, the Decatur Ice & Coal Co., the Decatur Box & Basket Co., and the Walcott & Smith Sawmill, flowage easements were acquired and slight relocations and adjustments were made to their pumping plants and water intake facilities. The 5 major adjustments to industrial plants are discussed in the following sections.

Alabama Brick & Tile Co.

The Alabama Brick & Tile Co. plant is within the city near the western city limit. The plant, originally built in 1921, and the clay pits covered about 30 acres. The rated capacity of the 8 kilns was 1 million bricks per month. Two independent appraisals of the physical property and present condition of the plant were made in 1936 by reputable consulting ceramic engineers.

The plant itself is located on high ground and was not subject to inundation. The clay pits, however, lie in a long, narrow strip adjoining the plant along the riverbank within the flood plain. These were the only pits operated by the company and had been mined for almost 15 years. A careful exploration revealed that approximately 78,180 cubic yards, or 35.5 percent of the original clay deposits, were available for commercial use. This amount represented about a 4-year supply for the plant at full capacity. About one-half of a mile to the west and also in the flood plain along the riverbank there is an additional supply of clay sufficient for the normal life and future operation of the plant but which the company neither owned nor directly controlled.

Various studies were made to determine the most equitable adjustment. It was uneconomical to dike the pits and drain the area by pumping. Tests for making brick from dredged clay proved that the excess moisture content would slow production and lower the quality of the product sufficiently to make the operation uneconomical with the existing equipment and facilities. Removing and storing a 10-year supply of clay before flooding the pits involved too high a cost and attendant uncertainty of commercial operation. A small supply of clay was stored and used for making brick during the first summer the pit was submerged, however, which enabled the plant to continue operations until a new source of satisfactory clay was found.

An extensive search was conducted for other suitable clay deposits. Clay from a deposit immediately across the river from the plant produced a brick of different color and inferior quality. Ample deposits, from which brick of uniform and satisfactory quality could be produced, were finally found at Whitesburg on the Tennessee River about 30 miles upstream from the plant.

To keep the industry at Decatur where a good market was already established, it was decided to transport the clay by barge from Whitesburg to the plant. This required the building of loading facilities at the pit, a towboat, two 350-cubic-yard barges, and unloading facilities and storage yard at the plant. A voluntary settlement based on this plan was made with the Alabama Brick & Tile Co. and a flowage easement and release from further damage was acquired by contract.

The Stephenson Brick Co.

The Stephenson Brick Co. plant and clay pits are located at the east corporation line of the city of Decatur. The clay deposits, plant layout, and product and daily output were practically identical with those of the Alabama Brick & Tile Co. The plant itself is on high ground well above the proposed pool level and only the clay pits were submerged. All nearby clay supply is in the bottom land along the riverbanks below the normal pool level.

The plant was built about 1903 and purchased by the present owners in 1926 at a forced sale. Since then numerous improvements and additions were made. The plant and pits cover about 33 acres. When operated at full capacity, the 9 kilns produced approximately 10,000,000 bricks per year. At the time negotiations were started the plant had been operating at less than 50 percent capacity and had been voluntarily shut down for some time. During the previous 10 years the company had operated the plant intermittently and at about the same capacity or less than similar building brick plants in that locality.

The company was not willing to accept a settlement on a similar basis to that made with the Alabama Brick & Tile Co. or to conduct any experiments leading to the adaptation of new material and continued operation of the plant. The case, therefore, is still in process of condemnation, the award of the lower court having been appealed by the Authority. Independent appraisals of the property and present condition of the plant and machinery were made by the same consulting ceramic engineers, and at the same time as those for the Alabama Brick & Tile Co. plant.

The American Oak Leather Co.

This company manufactures tanning extract from chestnut wood. The raw water supply for the mill is pumped directly from the river. The raising of the water to the normal reservoir level necessitated the raising of the floor of the pumping plant, resetting the pumps and motors, and changing the intake and discharge lines. The company also operates a gravel screening and washing plant. The gravel is dredged from bars in the river bed and transported to the plant by barge. It was necessary to relocate and raise the unloading trestle and hoist, and to rebuild the lower end of the belt conveyor carrying the screened aggregate to the storage bins.

A small shipyard or boat ways was maintained by the company for repairing its own barges and towboats and was also leased at various times to local towboat and barge outfits for similar boat repairs. The yard and ways were at the time under lease to the Ingalls Iron Works of Birmingham who were engaged in the manufacture of welded steel barges. The shipyard would be completely submerged at the normal reservoir level and would have to be abandoned or rebuilt.

In order to effect a settlement, hearings before a commission were required. This commission issued a decree under which the Authority reimbursed the company for all necessary repairs, adjustments, and improvements; and the company released the Authority from all further claims.

The Ingalls Iron Works.

The Ingalls Iron Works, in anticipation of the expiration of its lease of the shipyard and boat ways owned by the American Oak Leather Co., purchased a site at the west end of the city from the Alabama Brick & Tile Co. and erected a new shipyard with permanent concrete ways and a large plant for building welded steel barges. Although the new plant was designed to operate under future reservoir levels, it was thought necessary by the Authority to obtain flowage easements on two tracts of land totaling 4.13 acres, and a voluntary settlement was made.

The Holland-Blow Stave Co.

This company operates a small hardwood stave mill on the bank of the river west of the railroad bridge. The mill had been run at reduced capacity for a number of years, but because of a recent steady demand for barrel staves, was running at full capacity at the time negotiations were conducted. Water for the plant was pumped from the river, and it was necessary to move and relocate the pump, pump house, intake line, and the storage well on top of the bank inasmuch as that part of the property on which these facilities were located was needed for reservoir purposes and the proposed navigation terminal. The tract was condemned in fee, a hearing held, and a final decree entered into under which the Authority obtained full rights to the property condemned.

HIGHWAY ADJUSTMENTS

The project involved the adjustment of 30.4 miles of highway—4.5 miles of State highways, 21.4 miles of county highways, and 4.5 miles of farm roads—and the replacement of three major bridges. When improved standards of construction were desired beyond that for which the Authority was liable, the Governmental unit making the request participated to the extent of the betterment. The total cost of the highway and bridge relocations was $842,735.71.

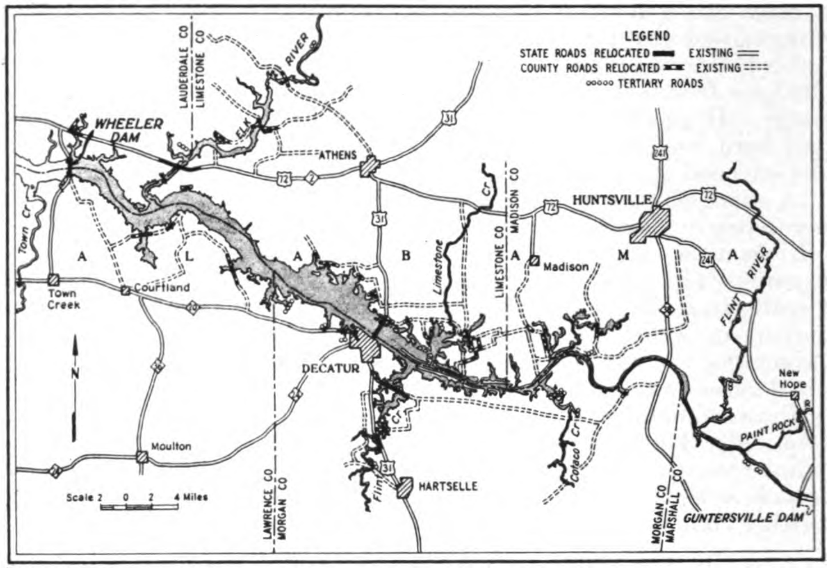

FIGURE 122.—*Highway adjustments.*

Highway adjustment contracts were made with the State of Alabama and the counties of Lauderdale, Lawrence, Limestone, Madison, Marshall, and Morgan. The contract with Marshall County was not fulfilled because the final location of Guntersville Dam changed the conditions of the contract. The majority of highways affected were either raised on their existing locations or relocated. In certain instances the purchase of isolated lands rather than the construction of new roads to them was found to be more economical. To compensate partially for the increased distance required in traveling to and from various localities, some of the new roads were constructed to better standards than those they replaced. The contracts included clauses relieving the Authority from further maintenance and all liability in connection with the inundated highways after the completion of the specified terms.

In addition to the work indicated above, the agreement made with the city of Decatur, Alabama, stipulated that Danville Road and Vine and Washington Streets be adjusted to meet the new conditions. This construction was done in connection with the Dry Creek Channel development.

A summary of the State and county highways affected and relocated in the reservoir area is given in table 28.

TABLE 28.—*State and county highways affected and relocated*

	Miles	Cost to TVA
State highways: Reconstructed by TVA	4.51	$377,150.58
County highways:		
Reconstructed:		
Principal county roads	21.36	433,934.38
Tertiary roads	4.53	29,215.40
Removing 13 bridges		2,455.35
Total State and county highways reconstructed	30.40	842,735.71

State highways.

In the rebuilding of the roads that had to be relocated, it was agreed by the Authority and the State that it would be desirable in some instances to reconstruct highways to more modern and up-to-date standards. The contract with the State, therefore, provided that on certain roads the Authority would construct roads of greater width and better alignment, and that in compensation for the extra expense the State would reconstruct certain other sections of flooded roads at its own expense and would release the Authority from further responsibility for all of the highways that were flooded. In accordance with the terms of the contract, the work was divided as follows:

1. The Authority procured all rights-of-way and all such other rights and permits that were necessary for reconstruction of highways, either by the Authority or by the State.
2. The Authority reconstructed State Highway No. 3 both north and south of Decatur, No. 67 between Decatur and Somerville, No. 20 west of Decatur at Dry Creek, and No. 2 at Elk River, with the exception of the necessary surfacing.
3. The Alabama State Highway Department reconstructed State Highway No. 2 at Second Creek, State Highway No. 20 west of Decatur, and applied the surfacing of State Highway No. 2 at Elk River.

The relocated State highways were all built above elevation 565.

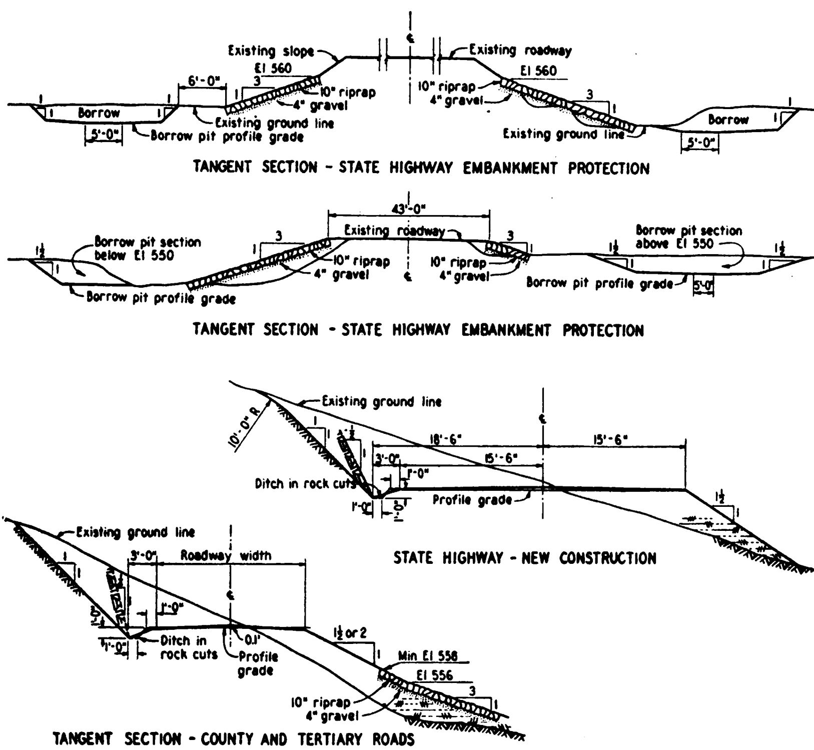

FIGURE 123.—*Typical cross sections for road protection and relocation.*

Rights-of-way, generally, are 100 feet in width, the width between shoulders 31 feet, and 37 feet between center lines of ditches. The minimum roadway width on bridges is 24 feet. The slope on fills is 3:1 below elevation 560 and 1½:1 above this elevation. Figure 123 shows typical cross sections of the relocated State highways. Where necessary, the fills are protected with riprap not less than 10 inches thick laid on a 4-inch layer of gravel. Approximately 92,000 square yards of riprap were used.

Two large bridges were built on the State highway projects, one at Elk River and the other at Second Creek. The Elk River bridge is 1,562 feet long and consists of twenty 69-foot viaduct spans and one 200-foot steel truss span. The bridge originally across the Elk River was dismantled and re-erected at the Buck Island bridge site. The base cost of the new bridge totaled $206,430.30. Figure 125

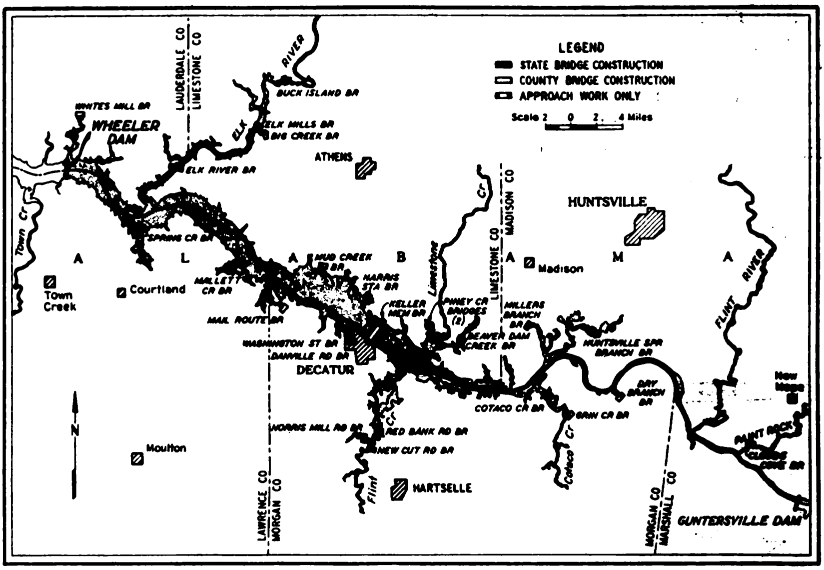

FIGURE 124.—*Highway bridge adjustments.*

shows the completed bridge. The bridge over Second Creek is 610 feet long and supports a roadway 24 feet wide. Both bridges have a concrete substructure and a steel superstructure with a concrete floor.

County highways.

It was necessary to relocate portions of certain county highways and bridges because they would be inundated permanently or at frequent intervals by the reservoir. Contracts were executed with each of the six reservoir counties, Lauderdale, Lawrence, Limestone, Madison, Marshall, and Morgan, whereby the Authority agreed to replace or to reconstruct certain sections of highways affected by the creation of the reservoir. A total of 21.18 miles of principal roads and 4.52 miles of tertiary roads was relocated or protected.

The Authority made all surveys and plans, supervised all the reconstruction work, secured all rights of way and such other rights and permits as were necessary, and removed bridges that could not be used in their existing locations. The counties took over the highways for maintenance after the completion of the construction work.

Usually the affected sections of the highways were reconstructed by raising them to the required elevation on the existing general alignment. Relocations were made on higher ground where it was found more feasible and economical, however. In a few instances substitute highways were built in lieu of highways that were inundated but not reconstructed.

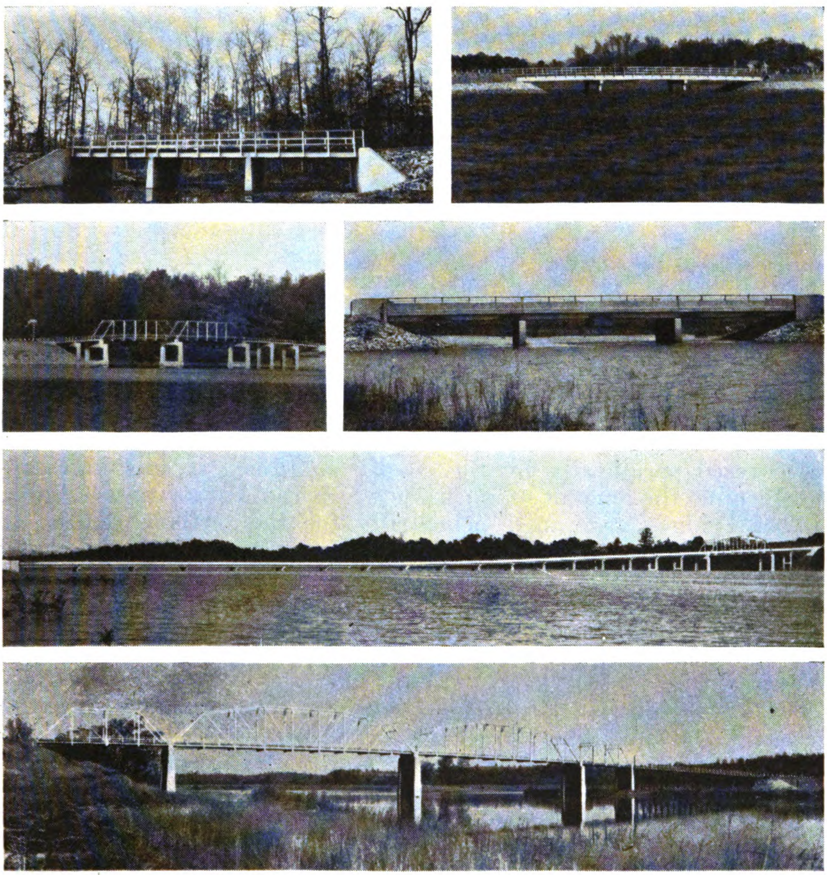

FIGURE 125.—*Typical new bridges built in the reservoir. Beginning at top; left to right, Mud Creek Bridge, Mallett Creek Bridge, Elk River Mills Bridge, Beaver Dam Creek Bridge, Elk River Bridge, and Buck Island Bridge.*

The rights of way for new county road locations were in general 50 feet wide, and the principal new roads were reconstructed 24 feet wide with a gravel surfacing spread at a rate of 1,000 cubic yards to the mile. These roads were all constructed above elevation 560. Cut slopes are 1:1, and fill slopes are 3:1 below the normal pool level. Above the normal pool level the fill slopes are 1½:1 where riprap was considered necessary and 2:1 where there is no riprap. Approximatey 50,000 square yards of riprap not less than 10 inches thick was placed on a four-inch gravel layer base. Figure 123 shows typical county highway sections.

Fifteen steel and concrete bridges were built for the county highways. Several of these were built from parts of the 13 abandoned bridges removed by the Authority. Ten timber bridges were constructed, aggregating 521 feet in length, and 14 concrete culverts were constructed, ranging from single 4- by 4-foot to double 8- by 8-foot culverts. Figure 125 shows a typical concrete and I-beam type of bridge and a typical I-beam bridge with a wood deck flooring. The steel and concrete bridges constructed are listed below:

Location	County	Type of bridge	Number spans	Length, feet
Second Creek	Lauderdale	Steel truss	1	150
Mallett Creek	Lawrence	Wood deck I-beams	3	147
Spring Creek	do	do	1	90
Elk Mill	do	2 steel trusses and wood deck I-beams.	7	506
Big Creek	Limestone	Wood deck I-beams	2	124
Mud Creek	do	do	3	60
Harris Station	do	do	1	25
Beaver Dam Creek	do	Concrete deck girder	5	222
Buck Island	do	4 steel trusses and wood deck I-beams.	16	718
Huntsville Spring Branch	Madison	Concrete deck girder	5	222
West Fork Flint Creek	Morgan	Wood deck I-beams	1	87
East Fork Flint Creek	do	Additional span	1	20
Dry Branch	do	Additional spans	2	38
Cotaco Creek	do	Steel trusses	2	267
Flint Creek	do	Additional span	1	36

The necessity for replacing minor county and private access roads entailed considerable study. Since these tertiary roads were not covered specifically by the county contracts, it was necessary to determine the amount of work for which the Authority was liable. Timber bridges were used principally for the tertiary roads. In some cases it was found more economical to buy parcels of land rather than to construct roads to them. Careful studies were made to determine the economic justification of such action.

RAILROAD ADJUSTMENTS

The major railroad adjustments required were in the vicinity of Decatur, Alabama, where trackage owned by the Southern Railway Co. and the Louisville & Nashville Railroad Co. was affected, as shown in figure 126. Several minor adjustments to the bridges and other structures were made. Embankments carried the tracks through the river-bottom area at an elevation higher than the crest of any expected flood and accordingly it was unnecessary to change alignments or raise grades. The problem consisted principally of protecting the existing embankments and structures and building a few small bridges. This protective program required the placing of 638,000 cubic yards of material. The changes affected 7 miles of track at a total base cost of $177,150.77, divided as follows:

Southern Ry. reconstruction	$122,071.23
Louisville & Nashville Railroad reconstruction	51,079.54
Nashville, Chattanooga & St. Louis Ry. settlement	4,000.00
Total relocation base cost	$177,150.77

Negotiations with the railroads developed that either of two embankment protection schemes would be acceptable: first, to increase the existing fills to 3:1 slope and riprap the surface, or, second, to increase existing fills to 8:1 slope without riprap. The Authority elected to do the latter and bids were received on that basis. The lowest bid for grading being $0.30 per cubic yard, it was decided that the Authority's forces using modern earth-moving equipment could effect a considerable saving over the low bid. The average base cost of making the fill was $0.126 per cubic yard, which is $0.174 per cubic yard less than the low bid. A total saving of approximately $90,000 was effected. It has been estimated that the 8:1 slope scheme was approximately $260,000 cheaper than the

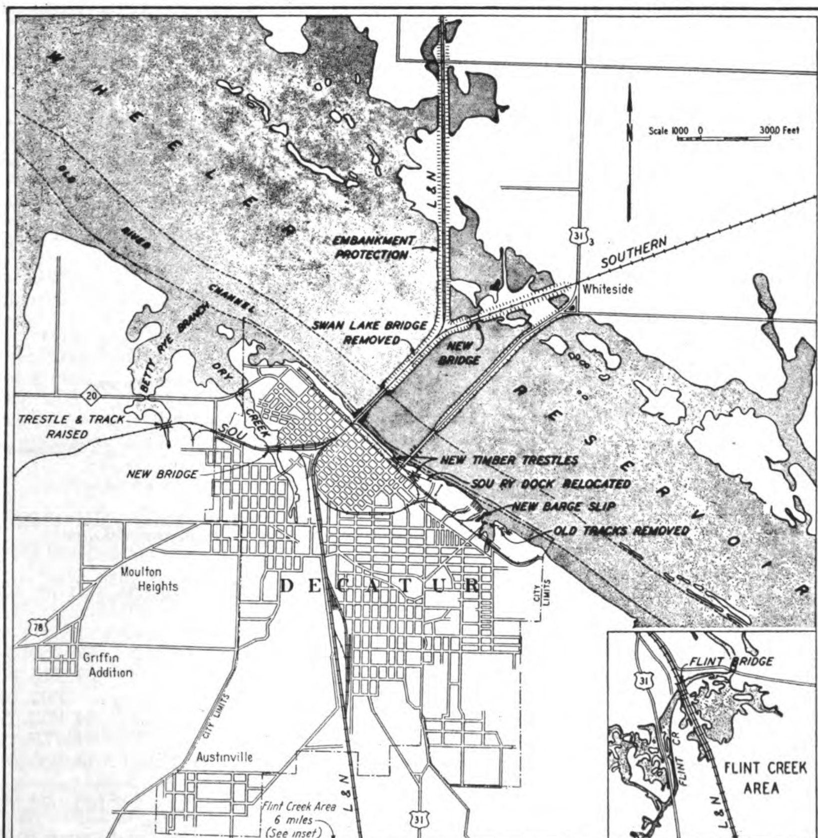

FIGURE 126.—*Railroad adjustments—Decatur, Ala.*

scheme of 3:1 slope with riprap; and had the railroads been agreeable only to placing riprap on the existing embankment, it might have cost the Authority $107,000 more than the 8:1 fill actually constructed.

All track work was done by the railroads involved, and all necessary ballast was furnished by the Authority.

The earth fills were protected from erosion by use of riprap, sodding, and additional rock fills. Riprap approximately 18 inches in thickness was placed around the abutments and culverts of existing or newly built structures within the reservoir. Bermuda grass was transplanted on the slopes of all 8:1 fills above the pool stage. This type of sodding proved very successful except at the water's edge, where additional protection was found necessary. There it was necessary to place approximately 24 cubic feet of loose rock per linear foot of roadbed at the water's edge to prevent erosion by wave action. This was due principally to the texture of the filling material.

Southern Railway.

The greatest expense, as shown by the following tabulation of cost for the Southern Railway, was for earth fills and earth-fill protection. A total of 363,296 cubic yards of earth for grading, 6,915

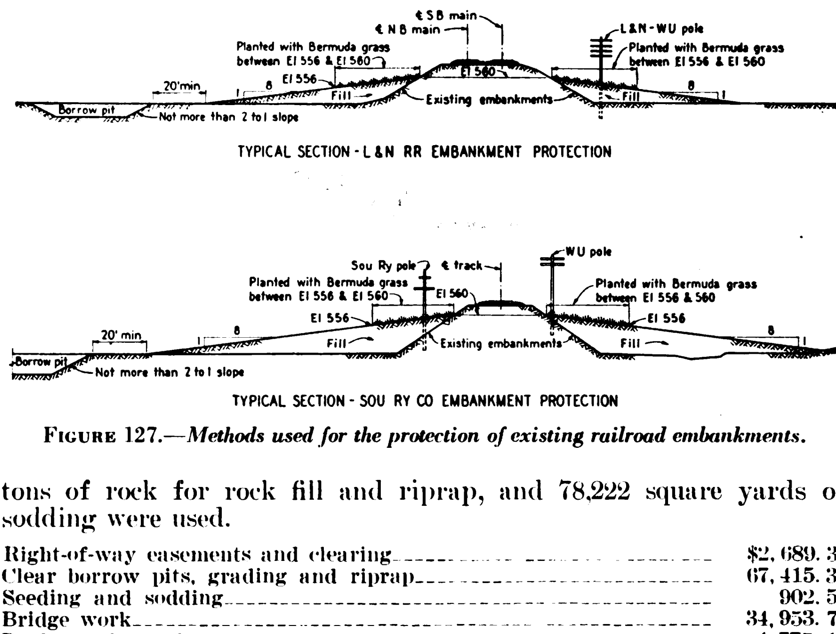

FIGURE 127.—*Methods used for the protection of existing railroad embankments.*

tons of rock for rock fill and riprap, and 78,222 square yards of sodding were used.

Right-of-way easements and clearing	$2,689.39
Clear borrow pits, grading and riprap	67,415.36
Seeding and sodding	902.51
Bridge work	34,953.77
Dock track work	4,775.19
Ballast and other charges	11,335.01
Total base cost of reconstruction	122,071.23

The largest single earth fill was at the Swan Lake opening, 0.3 mile northeast of the Tennessee River, where there was a 400-foot bridge of deck-plate girders, the two end spans of which were below maximum surcharge pool stage. This necessitated the removal or revision of the bridge, and economic studies showed that the former plan would be less expensive. Negotiations with the railroad and the United States Army Engineers resulted in permission to remove the bridge, make the fill with 8:1 slope, and provide necessary waterway openings by the construction of a less expensive bridge. This bridge was a concrete pile bent trestle design, 0.6 miles east of the original Swan Lake bridge. The plan adopted was $12,000 cheaper than the alternate plan of making revisions to the existing Swan Lake bridge.

Two concrete pile bent bridges and four timber pile bent bridges were constructed. The superstructure of the concrete trestles was concrete slab except for the center span of Dry Creek bridge, which was a steel beam span. The superstructure of the timber trestles was open-deck timber except the Betty Rye trestle, which was timber ballast deck.

The over-all length of the Dry Creek bridge was 90 feet 7 inches. It consisted of six concrete pile bents; four bents of five 16-inch piles each, and two bents of eight 16-inch piles each. The superstructure was four spans 13 feet 6 inches each of concrete slabs and one 30-foot span of 36-inch I-beam open-deck construction.

The only bridge built on the north side of the river was at milepost 361.6A. This was an 81-foot 6½-inch bridge consisting of six concrete pile bents of six 16-inch piles each. The superstructure consisted of 16-foot concrete spans.

Louisville & Nashville Railroad.

Work for the Louisville & Nashville Railroad consisted of earth fills or earth fill protection and included 264,882 cubic yards of earth for grading, 3,639 tons of rock for rock fill or riprap, and 135,467 square yards of seeding and sodding. The following tabulation shows the cost of work done:

Clearing right-of-way	$1,601.31
Clearing borrow pits	8,903.29
Grading	30,253.68
Riprap	5,049.45
Seeding and sodding	1,548.96
Dock track and other work	8,722.85
Total Base Cost of Reconstruction	51,079.54

Nashville, Chattanooga & St. Louis Railway.

The Nashville, Chattanooga & St. Louis Railway accepted $4,000 as full compensation for anticipated damages to their roadbed at Hobbs Island, Ala. They did all the work and furnished all necessary materials for the protection of their property.

UTILITY RELOCATIONS

Properties of four major utility companies and several private telephone companies were in the proposed reservoir area. Settlement of claims for damages was complicated by questions of liability, and extended negotiations over a period of several years were required. All matters were finally adjusted by compromise agreements, without litigation, at a total base cost of $102,162.72.

The general procedure adopted for settling all utility relocation problems was first to inform the companies of the Authority's plans for creating the reservoir, and then to have the private companies submit their claims for negotiation and settlement and to place the burden of establishing the validity of such claims upon the companies. Plans for protection or relocation were developed and agreements reached on the extent of the Authority's liability for the work required. Most of the construction work was done by the utility companies; but in the case of a few privately owned rural telephone lines, the work was performed by the Authority.

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Alabama Power Co.

Negotiations were started first with the Alabama Power Co., the owners of the most extensive utility lines affected. This company submitted plans and cost estimates of the relocations it considered necessary. Owing to the uncertainty of the extent of the Authority's liability, the negotiations lasted a relatively long time. Although it seemed that litigation would be necessary, a settlement was reached for the major portion of the adjustments June 11, 1936, and final settlement of all claims was reached and incorporated in a contract dated September 20, 1938. The total cost of these settlements was $90,000.

The most important transmission line of the Alabama Power Co. affected was the Decatur-Huntsville transmission line at the Tennessee River crossing just above Decatur, Ala. This line spanned the

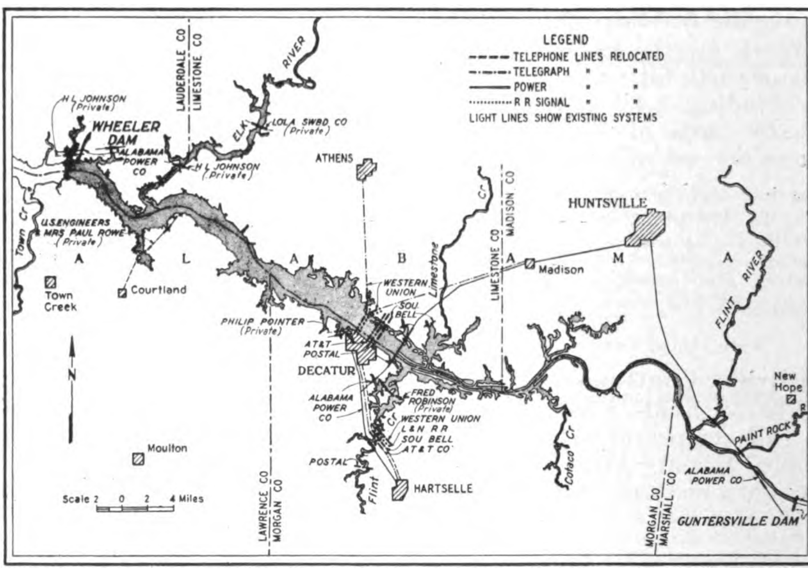

FIGURE 128.—*Relocation of electric power, telephone, and telegraph lines.*

river by means of two steel towers and consisted of a three-phase, 110-kilovolt line, a three-phase, 44-kilovolt line, two spare conductors, two ground wires, and two telephone wires. The river span was 1,850 feet, and the clearance above the proposed maximum pool level was 72 feet. The clearance of this crossing being sufficient, the only adjustment needed was some protective work around the base of the towers. From the tower on the north bank of the river to the reservoir shore line, approximately 7,800 feet, the conductors were supported on H-frame pole structures. After filling of the reservoir, the water would stand for an average height of 5 to 6 feet on these poles, reducing the clearance by the same amount. The power company's plans required the protection of their pole structures, and the United States Army Engineers advised that it would be necessary to provide a minimum clearance of 35 feet over this part of the pool. The power company submitted several different estimates ranging from $293,527 to $525,642 to cover their entire relocation work, consisting of the main river crossing and 12 other smaller adjustments. The Authority considered the estimates extremely high as they provided for a considerable betterment of the present system at the expense of the Authority and increased the company's facilities for future expansion. An offer for settlement for this main river crossing was made by the Authority based on a replacement in kind of facilities. After a series of negotiations, a settlement was mutually agreed upon on April 9, 1936, providing for payment by the Authority of $75,000.

Owing to purchase of other lines by the Authority the number of claims by the Alabama Power Co. was reduced to five as follows:

Location	Alabama Power Co., original estimate	TVA and Alabama Power Co., revised estimate
Decatur Highway Bridge	$284.00	$284.00
Decatur-Huntsville 110-kilovolt line: Bush Creek on Somerville Road	818.00	300.00
Gadsden-Huntsville 110-kilovolt line:		
Flint River	16,324.00	7,386.00
Paint Rock River	16,331.00	7,393.00
Shoal Creek	23.00	23.00
Total	$33,780.00	$15,386.00

These claims were settled upon payment by the Authority of $15,000 as stipulated in a contract dated September 20, 1938.

Figure 129 shows the protection work placed around the footings of the steel tower of the Alabama Power Co.'s Tennessee River crossing east of Decatur. This figure also shows earth protection around transmission line poles. This protection is built to elevation 560.

Southern Bell Telephone & Telegraph Co.

A 22-wire trunk line of the Southern Bell Telephone & Telegraph Co. crossed the Tennessee River one-quarter of a mile east of the Decatur highway bridge. This aerial crossing was suspended between two 136-foot steel towers with a 2,000-foot span. The elevation of the base of the south tower was 573 and of the north tower 551. In addition, at the same location there was a 22-wire submarine cable 2,000 feet long, the north end of which was at elevation 551. From the north bank of the river, the wires from each of these crossings were carried across the flood plain to the north shore line of the reservoir on joint poles. There were 80 wood poles carrying the 44 wires over a distance of approximately 2 miles that would have been inundated by the reservoir.

Several plans of relocation were proposed before a mutually satisfactory agreement was reached which provided for a compromise settlement of $4,500. The basis of this compromise was an estimate of the construction cost of the new facilities together with the removal cost of the abandoned lines, with allowances for depreciation and for salvage of the original lines. This came to approximately $5,400. The settlement of $4,500 was made in compromise because it was considered that approximately $900 of the damage was due to adjustments necessary to improve navigation for which the Authority was not liable.

The jointly owned line of the American Telephone & Telegraph Co. and Southern Bell Telephone & Telegraph Co. paralleling the Louisville & Nashville R. R. at Flint Creek was also affected by the reservoir. Some of the poles in this line would be standing in a few feet of water and access to the poles would be cut off. The telephone companies agreed to a protective measure whereby the Authority would construct islands around the poles with a walkway or embankment built to each from the railroad fill. In consideration of this work the telephone companies waived all other claims for damages on this line.

Postal Telegraph Co.

The Postal Telegraph Co. owned a trunk line crossing the Tennessee River one-quarter of a mile east of the Decatur highway bridge. It was a 20-wire submarine cable 2,000 feet long, the south bank entrance of which was at elevation 572, and the north bank entrance at elevation 552. From the north terminal the wires were carried on

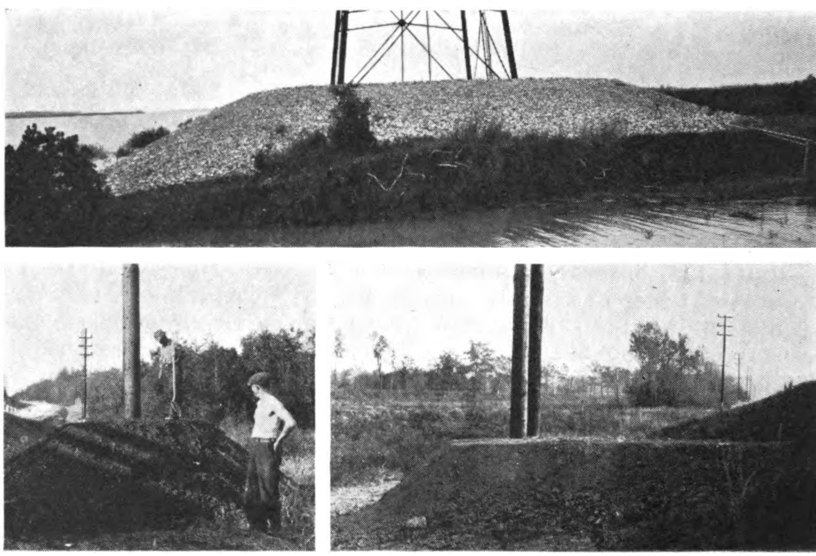

FIGURE 129.—*Protection work for poles and towers.*

a pole line to the north shore line of the reservoir. There were 80 poles over a distance of about 2 miles that were below the pool level.

Two routes were proposed for relocation. One was along the original location, using a different type of pole structure and utilizing the existing river cable crossing; the other was for relocation along the State highway, using a new river cable crossing at the highway bridge. The first plan was estimated to cost approximately $4,200 but was not acceptable to the telegraph company because of the long route through the reservoir. The estimate for the second plan was approximately $4,700, of which about $2,500 was for the pole line and $2,200 was for the submarine cable river crossing. The Authority could not accept liability for the cost of the river crossing involved, but a compromise settlement was finally reached for a payment of $3,000, which was less than the cost of the first plan, and was acceptable to the company.

Western Union Telegraph Co.

The Western Union Telegraph Co. line was on the company's own poles paralleling the railroad track from the north bank of the Tennessee River to the junction of the two railroads. The junction is located about 4,000 feet along the railroad northeast of the Tennessee River railroad crossing at Decatur. From this junction 15 of the Western Union wires branched off and continued to Huntsville on their own poles located along the Southern Ry. tracks, and the remainder continued along the Louisville & Nashville R. R. attached to poles of the Louisville & Nashville R. R. Co. Many of the poles were as low as elevation 551, some as low as 546, and the entire line was affected by the protective embankment work planned for the railroad companies.

The estimated cost of adjustments of the Western Union Telegraph Co. amounted to $2,682, and payment of this amount was agreed to by the Authority. A contract was drawn, releasing the Authority from all further damages due to the flooding of the reservoir. The cost of relocating the signal lines for both railroads was included in agreements with the railroads for other relocation and protective work.

Private telephone lines.

Five privately owned telephone lines in the Wheeler Reservoir required relocation. Where possible, settlements were made by paying a lump sum to the company or party involved. Contracts were made whereby the Authority was released from all further claims on payment of a stipulated amount.

The Farmers' Telephone Line parallelling the old Somerville Highway location across Flint Creek served 18 stations. It was necessary to relocate 2½ miles of the line along the new highway fills. Estimates for the relocation work amounted to $575, and an agreement was prepared for this amount.

The H. L. Johnson Line was affected in two locations: first, a four-wire telephone line on the Lee Highway at Second Creek and, second, a two-wire line on the Lee Highway at Elk River. It was necessary to relocate about 2 miles of the first line and a little over 1 mile of the second, moving them from the old road location to the new relocated highway. Because it was possible to salvage old poles and wire, this relocation work was settled upon payment by the Authority of $100.

The Lola Switchboard Co. had approximately one-half mile of two-wire telephone line that had to be relocated on the new highway fill and bridge crossing Elk River. Settlement was made for all damages by payment of $75.

The United States Army Engineers and Mrs. H. D. Lane had a jointly owned telephone line between Courtland, Ala., and the navigation canal on the Tennessee River. About 5 miles of this line required relocation, but agreement could not be reached on the amount of settlement. Owing to the joint ownership of the line it was impossible to conclude any lump sum agreement, and inasmuch as the United States Army Engineers' interest in it was being taken over by the Authority, it was decided to relocate this line with the Authority's forces. It was satisfactorily completed at a cost of $282.98.

The Philip Pointer Line, a two-wire telephone line on the north side of Alabama Highway No. 20 crossing Betty Rye Branch at Decatur, also needed relocating. The relocation of this line was complicated by the work of the Authority along Dry Creek which involved the structure of the Southern Bell Telephone & Telegraph Co. line connecting the Philip Pointer Line. After considerable negotiation, an agreement was finally reached releasing the Authority from all damages upon the payment of $150.

RELOCATION OF CEMETERIES

The flooding of the reservoir necessitated the removal of 176 graves from 4 cemeteries. Although 249 graves were located in 9 cemeteries actually affected, permits were obtained to leave many of them in their original locations. The removal and reinterment work done by the Authority's forces cost an average of $9.79 per grave. To determine the necessity for removal, it was necessary to survey 3,100 graves; and the cost of these surveys together with the cost of identification, securing permits, and the preparation of records brought the total cost of the grave-removal program to $6,842.10. All work was carried out amicably with persons concerned, and it was completed August 1, 1935, more than a year before the reservoir was flooded.

Surveys were made of all cemeteries within the reservoir area, showing boundary lines, elevations, and grave locations. It was recognized that the Authority should remove all graves as requested which were located below elevation 556. Legal contracts for each grave to remain or to be removed were obtained from the nearest of kin. Special permission was obtained from the Alabama State Board of Health authorizing the TVA to relocate graves affected by the reservoir.

Plane-table surveys were made of the 3,100 graves contained in 42 cemeteries located on land purchased by the Authority and in 20 additional cemeteries immediately adjoining the land finally acquired. These surveys were made in order to obtain an identification record of all cemeteries on Authority land and in which the Authority made removals or reinterments. The surveys included the following information: location and property lines of cemeteries, with fences or enclosures around individual graves, or lots, indicated; location of all identifiable graves, with monuments and inscriptions noted; elevations of cemeteries indicated with 2-foot contours; all isolated trees and woods lines; and names of adjoining property owners.

Of the 42 cemeteries on TVA land only 9 were entirely or partially below elevation 556. Maps were made of these 9 cemeteries and were used as work sheets for identification and removal and for permanent record of relocations.

Two types of contracts were prepared for the nearest of kin in each case where the grave was identified—one for the deceased to remain in their original locations, and the other where requests were made to relocate the graves. Relocations were made as requested by the nearest of kin, but in all cases they were made as close as possible to the original site. Contracts were executed for 57 graves to remain in their original location and for 185 to be relocated. It was found by examination after digging that there were no remains in 9 of the graves previously identified, hence there was actual removal of only 176 graves. There were 7 graves in other cemeteries that could not be identified, and no one was found who could sign the contracts. Therefore, no contracts were executed for them and they were not relocated. In the Campbell Church Cemetery (Negro), it was not possible to identify all of the 173 graves. As a result, the board of deacons of the church accepted responsibility for the relocations and requested the removal of all graves, authorizing the chairman of the board of deacons to sign permits for the removal.

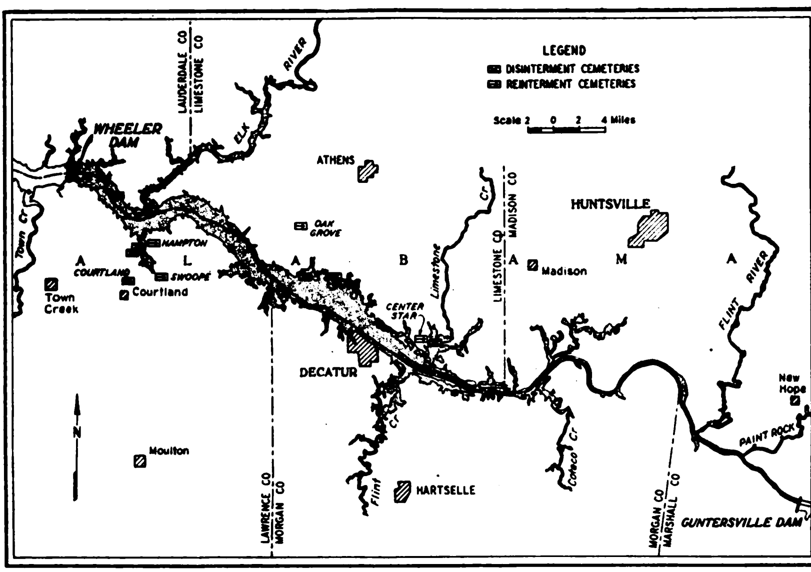

FIGURE 130.—*Cemetery relocations.*

Removal and reinterment work was done by a crew of men who had been trained in this work in the Norris Reservoir region, where some 5,000 graves were relocated. The work was conducted in a simple yet dignified manner with proper reverence shown for the dead. Where possible, caskets were removed from the graves intact and placed in new boxes. Where this was impossible, the remains, including all bones, dust, articles, and any remaining parts of the original casket were placed in the new casket in the same position as found in the grave. An inventory was made of the remains found in each grave. Inexpensive wooden boxes were used as reinterment caskets and a simple grave marker with card insert was placed at the foot of each relocated grave for temporary identification. All markers or monuments and fences on the original graves were moved to the new grave sites. Sixteen monuments were relocated. The following table indicates the action taken in the 9 cemeteries below elevation 556:

Cemetery	Total graves	Removal contracts	Remain contracts	Graves moved	Monuments moved
McGuire	1	0	1	0	0
Dabney	[1] 1	0	0	0	0
Lucas	11	0	11	0	0
Lucas	[2] 24	0	23	0	0
Bridgeforth	[3] 23	0	18	0	0
Campbell	173	173	0	[4] 164	9
Sherrod	11	7	4	7	6
Center Star	[5] 4	4	0	4	1
Jones	1	1	0	1	0
Total	249	185	57	176	16

[1] Unidentified.
[2] One grave unidentified.
[3] Five graves unidentified.
[4] No remains found in 9 graves.
[5] Moved from below elevation 556 to location above pool level in same cemetery.

The reinterment of the 176 graves was made in 5 existing cemeteries as follows:

Hampton Cemetery	164
Courtland Cemetery	4
Swope Cemetery	3
Oak Grove Church Cemetery	1
Center Star Cemetery	4
Total	176

All grave removal records were prepared in duplicate, one set retained in the Wilson Dam office and one set placed in the Tennessee Valley Authority library. These records, in addition to the plane-table sheets and maps of the disinterment and reinterment cemeteries, include: identification information and history of each grave, agreements for removal or to remain, and reinterment data including identification and inventory of the remains.

In addition to the above information retained in the Authority's file, a report was prepared and delivered as required by law to the Alabama State Board of Health listing the names of all deceased whose remains were relocated with the date of death and the location of disinterment and reinterment.

Cost.

The following tabulation shows the cost of all grave removal work including overhead and administrative charges:

Total cost of identification and surveys	$5, 118. 92
Grave removals and reinterments (base cost)	1, 723. 18
Total	6, 842. 10

FAMILY READJUSTMENT

The readjustment of families moving from the reservoir area presented a serious problem, principally because only 61 of the 840 families living on land purchased by the Authority were landowners. Thus, 93 percent of the families to be relocated were tenants. The problem was accentuated by two additional factors. First, an industrial and business depression had caused a "back-to-the-farm" movement and had increased the population dependent for its support upon the land. Second, the amount of cultivable land had decreased because of soil erosion and lack of conservation practices. Also, the forming of the reservoir was removing several thousand acres of fertile land from cultivation.

The 1935 amendment to the TVA Act empowered the Authority to advise and cooperate in the readjustment of families. An organization was set up similar to that in the Norris Reservoir area [6] for the purpose of planning and executing a family readjustment program. Surveys of the problem and of already established agencies equipped to give assistance to affected families resulted in a program of cooperation between the Authority and such agencies as the Resettlement Administration (now incorporated into the Farm Security Administration), the Civilian Conservation Corps, the Works Progress Administration, the Alabama State Department of Public Welfare, and many local agencies. The Alabama Polytechnic Institute was already under contract with the Authority to assist farm families, especially the land-owning group, in selecting new farms. The hardships which might have resulted from speculation in land values were thus avoided.

The family readjustment work was started in October 1935, and a field office was opened in Decatur, Ala. The employees for this work were selected because of their ease in meeting people, their familiarity with and understanding of problems of the valley, and their ability to analyze individual family problems. Social work training and experience and some knowledge of agriculture were also desirable.

The reservoir area was divided into seven districts according to the number of families, distances involved, and accessibility; and a field worker was assigned to each district. Families were visited to secure information needed to determine their ability to readjust. The families were considered individually and visited as often as thought necessary and feasible until removal was completed. Special attention was given to economic, health, social, educational, and religious factors, and every effort was made to have families relocated in places as good as if not better than their previous ones. Some families required little attention and service, but others needed many visits and a great deal of consideration.

There was close cooperation between the family removal organization and other departments within TVA to avoid duplication of work and any confusion which might arise. The construction forces at the dam held in readiness boats, trucks, and other necessary equipment to be available for emergency removal in the event of high water due to sudden heavy rains. In January 1936, several families were moved by boat and barge from an island when high water threatened their safety. Tents were used as emergency shelter for 11 families who were forced to leave their homes because access roads were flooded.

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[6] See ch. 7, p. 505, *The Norris Project*, Tennessee Valley Authority, Technical Report No. 1.

The removal was effected without recourse to legal eviction. The following table gives information regarding the families affected:

	Land-owners	Tenants	Total
Above elevation 556	52	621	673
Below elevation 556	9	158	167
Total families displaced	61	779	840
Relocated within reservoir counties	66	682	748
Moved to other Alabama counties	2	12	14
Moved out of State	2	8	10
Relocation unknown	0	68	68
Total	[1] 70	770	840
Families moved before contact	11	101	112

[1] 9 tenants became landowners upon relocation, making a total of 70 landowners

A spot study of 100 families before and after removal may be summarized as follows:

Location.—Thirty-nine percent of the families moved from the reservoir to locations providing greater community opportunities and advantages, including schools, access to market in case of farmers, and to work in the case of industrial employees. Fifty-four percent show equally as desirable locations as they formerly had. Seven percent show less desirable locations.

Housing.—Thirty-seven percent of the families have better housing conditions, including physical surroundings, although 10 percent are still inadequate; 29 percent are equally as satisfactory as before removal; and 34 percent are classified as having unsatisfactory housing.

Health and Sanitation.—Fifty percent of the new locations show improved health and sanitary conditions and 41 percent were approximately the same as before removal, which conditions were in many instances inadequate. Nine percent reported less satisfactory health and sanitary conditions than before removal.

Land.—Eight percent of the new farms had better land and lay of soil; 23 percent had land that is approximately as good as before removal; 69 percent were located on land adjudged "generally poor and less satisfactory than land on which families formerly lived."

Attitude.—Sixty-seven percent of the former reservoir residents showed favorable attitude toward new locations and to the Authority. The principal reasons for unfavorable attitude on the part of the 33 percent may be explained by the figure showing only 31 percent relocated on land as good or better and with general improved conditions.

Cost of relocation work.

The cost of the family readjustment activities amounted to $62,325.92. This is an average of $74.20 for each of the 840 families moving from the Wheeler area.

ARCHAEOLOGICAL INVESTIGATIONS

Archaeological surveys and excavations were made in the reservoir area to bring to light before flooding detailed evidence of prehistoric occupations which might be valuable in solving some of the many problems of the archaeology of the southeastern United States. Reservoir areas constitute an important source of archaeological investigation since prehistoric man used the rivers as highways and as a source of food. For these people, practicing rudimentary agriculture, the fertile bottom lands were ideal sites for the location of villages.

In December 1933 representatives of the University of Alabama, the University of Tennessee, and the Tennessee Valley Authority discussed steps to be taken to uncover and preserve the archaeological records in the Norris and Wheeler Reservoir areas prior to the filling of the reservoirs. The Curator of Archaeology of the United States National Museum was invited to act as consultant for this movement, and tentative plans were made for the survey of the basins. The work was placed under the direction of the head of the Department of Anthropology and Archaeology at the University of Kentucky.

The Alabama Museum had previously made an archaeological survey of northern Alabama including the Wheeler area. With this survey as a basis, it was possible to locate 237 sites, of which 19 were considered the most important representative samples. These were excavated with the help of Civil Works Authority labor. The

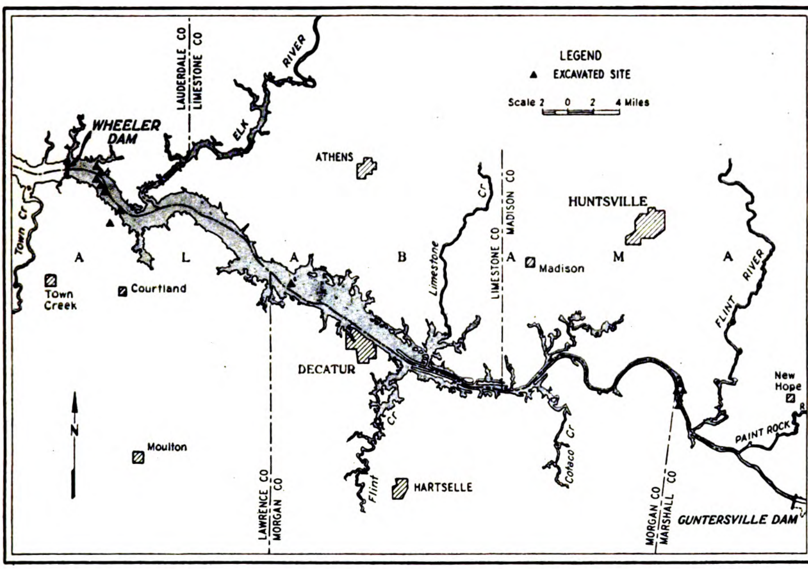

FIGURE 131.—*Location of archaeologic excavations.*

skeletal material obtained was sent to the University of Kentucky for restoration and study, and the artifacts were placed in the museum of the University of Alabama. Samples of pottery were sent to the University of Michigan to become the basis of a report on the significance of the pottery development in the area.

The sites investigated were of two types, one of earth and the other of accumulated shell. Investigations of these mounds and village sites have led to significant additions to the prehistory of southeastern United States. One of the most important was the discovery of a people not previously known in the lower region of the shell mounds, who not only antedated the use of pottery but were even of a preflint culture. These people used implements fashioned of bone and had for their chief hunting tool the dart thrown with an atlatl, or throwing stick.

The earth mounds yielded another new complex, which has been termed the "Copena." The people who built these mounds worked native copper, often in the form of curious reel-shaped breast ornaments. They also buried galena with their dead, possessed many ocean shells, practiced ceremonial destruction of artifacts, and produced woven fabrics.

The significance of these findings, the separate studies of ceramics and osteology, and the detailed description of each site, fully illustrated by photographs and charts, has been published as a bulletin of the Bureau of American Ethnology.[7] The Bureau Bulletin 118, recently issued, covers similar work in the Norris Reservoir area.

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Chapter 7

INITIAL OPERATIONS

Project operations, while relating primarily to the general objectives of navigation, flood control, and power, also concern related problems of sanitation and management of the land and water resources inherent to the project properties in such a manner as to provide the maximum local benefits to the greatest number. They include such activities as malaria control; sanitation; planned use of agricultural land by local citizens; the management of the forest resources, including reforestation and erosion control, protection, production of forest products, and the biological adjustment of fish and game; and recreation, including the cooperative development of parks, boating, swimming, and the like. These phases, while incident to the actual operation of the dam and reservoir, are of real importance in furthering the soil and water conservation program of the Authority.

RESERVOIR OPERATION

The operation of a single project such as Wheeler is governed to a large extent by considerations relating to the other parts of the system. As additional projects are completed and added to the system, the operation of any single project may be somewhat altered to permit the most effective use of the entire system. For this reason the general method of operation of the Wheeler plant cannot be considered as finally established until the whole Tennessee Valley Authority system of projects is substantially complete. Furthermore, as the stream-flow conditions are never alike in any two years, the details of operation can never be fixed but must be varied to meet the situation existing from time to time. Therefore operating rules are subject to revision as necessary in order to profit by experience and to meet operating conditions.

As long as construction work continues on the Tennessee River below Wheeler Dam, it will be desirable to operate Wheeler Reservoir to afford flood protection to these works as well as to the lower Tennessee, Ohio, and Mississippi Rivers. During the progress of construction critical flows will probably change from time to time, and the details of operation will likewise change to meet these changing requirements.

For river control operations, use is made of the storage space in the reservoir between elevations 548 and 556.3 at the dam. Between about December 15 and April 1, the reservoir is held at about elevation 550 to 552. On the approach of a flood it is drawn to elevation 548 if practicable, and the storage space between elevations 548 and 556.3 is used for flood flow regulation. In this operation an effort is made to limit the reservoir stage to 24.7 on the Decatur gage, which corresponds

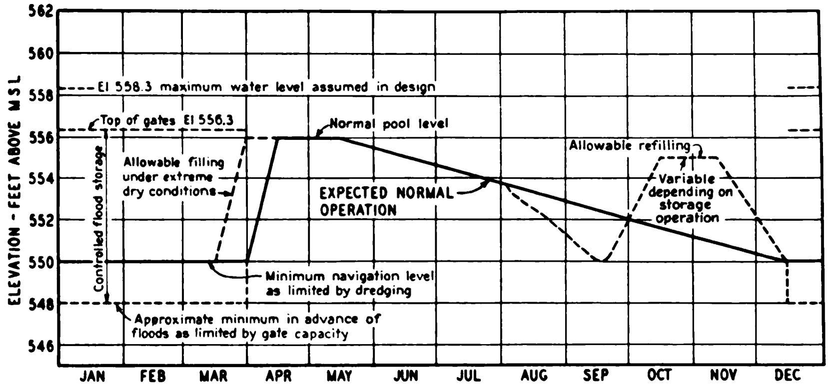

FIGURE 132.—*Reservoir operation curve.*

approximately to elevation 559, above which certain railroads, highways, and other structures would be affected. While natural flows may considerably exceed elevation 559 at Decatur, it is desirable that the operation of Wheeler Dam be such as not to produce backwater effects above these stages. As each flood recedes, the reservoir is again drawn down to elevation 550 as quickly as practicable, thus providing storage space for the next flood.

For malaria-control purposes, the reservoir is raised to about elevation 556 during early April and held at that elevation until about May 1, except that in the event of late spring floods it is drawn down as much as practicable in advance, and the storage space thus made available is used for regulating the flood flow. During the first half of May, weekly fluctuations of about 1 foot are made between elevations 556 and 555. About May 15, a gradual draw-down of the reservoir is begun and continued at a rate so as to reach about elevation 552 on October 1 and elevation 550 by the end of the low-flow season. The gradual draw-down is accompanied by weekly fluctuations of varying height depending largely on stream flow. This schedule has been followed in general up to the present time (1939); however, the details of this schedule may vary with experience and are affected by other activities.

As soon as sufficient water is available, the reservoir may be refilled and used for stream flow regulation until about the middle of December, when it is again drawn down to elevation 550 to provide for flood control.

Throughout the low-flow season, Wheeler Reservoir is used to re-regulate releases made at Norris Dam and to supply stored water for maintenance of flow down stream. Until Kentucky Reservoir is completed it is desirable to maintain as high a minimum flow as practicable below Pickwick Landing for benefit of navigation. In years of normal water supply this minimum will be around 18,000 cubic feet per second, but in dry years before the completion of the Hiwassee and other upstream projects it may not exceed 13,000 cubic feet per second.

NAVIGATION ON THE RESERVOIR

The completion of the Wheeler project opened a navigable stretch of river extending 69 sailing miles to the Guntersville Dam. A potential 6-foot navigable channel also extends about 30 miles up the Elk River. Navigation on the Wheeler pool has steadily increased since the opening of the lock December 1, 1936. The most important movement has been in the transportation of gasoline from Wood River, Ill., to Decatur and Guntersville, Ala., distributing points for northern Alabama and south central Tennessee. The Tennessee River, when developed, will not only be used for long distance or through traffic, but will also be used for the transportation of certain raw materials for processing locally. The assembly of these raw materials could not be done economically without the low cost means of transportation provided by the reservoir. An important local movement of this type which has developed in Wheeler Lake is the movement of brick clay, about 800 tons per week, from a clay pit at Whitesburg Landing to a plant at Decatur, Ala. Other commodities move more or less irregularly. Navigation in Wheeler Lake will increase steadily, but will not reach its maximum proportions until downstream and upstream improvements have opened the 9-foot channel from Paducah to Chattanooga and Knoxville.

The Authority has made a study of possible navigation terminals. It has erected a service building at Decatur, Ala., to base its marine operations in that vicinity. This base is designed for possible expansion into a complete terminal. Figure 133 is a view and a plan of this base. The shaded portion represents that part of the terminal already installed, and the unshaded portion illustrates the possible complete development. A number of minor landings have been located and partially developed, principally for the convenience of craft engaged in mosquito control measures.

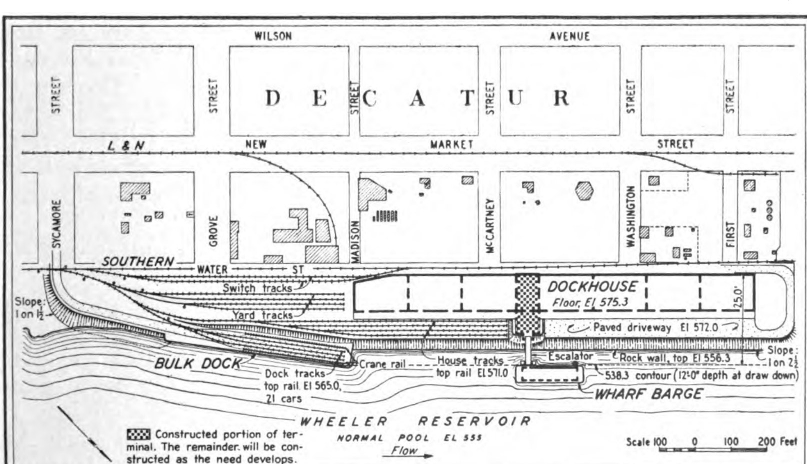

FIGURE 133.—*Plan of Decatur terminal and possible future development.*

FLOOD-CONTROL OPERATIONS

The Wheeler Reservoir was first used for flood control in March and early April 1936 before the spillway gates were installed at the dam. In conjunction with the operation of Norris Reservoir the storage space in Wheeler Reservoir below the concrete spillway crest was filled during the time the inflow exceeded the capacity of the turbine intake openings, and the excess flow passed over the spillway. This filling materially retarded flows and appreciably aided in reducing the flood.

Ever since the installation of the gates, the reservoir has been operated as a unit in the growing system of reservoirs in conformity with the general plan of flood-control operation previously described. It aided in regulating high flows which occurred during the flood seasons of 1937, 1938, and 1939, as well as during the summer of 1938. In this period of operation, Wheeler Reservoir participated on several occasions in the reduction of flood heights and resulting damages on the lower Tennessee, Ohio, and Mississippi Rivers.

PLANT OPERATION

Two generating units were installed initially in the powerhouse, which was designed to provide space for a possible installation of eight units. The first of these units was placed in commercial operation on November 9, 1936, and the second on April 14, 1937. Both generators are rated at 36,000 kilovolt-amperes, and are capable of operating under a continuous overload of 15 percent without exceeding maximum safe temperature. As determined by field tests, the units are capable of producing power in excess of the guarantees amounting to about 11 percent for the 48-foot head and about 15 percent for the 43-foot head.

Mainly because of bearing and shaft alignment difficulties, the first unit was out of service from September 30, 1938, until April 3, 1939, and the second unit during the periods from July 3 to July 10, 1937; September 4 to September 17, 1937; December 5, 1937, to February 18, 1938; and March 10, 1938, to September 2, 1938. During the first 2½ years of operation, unit No. 1 was available for service about 80 percent of the time and unit No. 2 about 70 percent of the time. The difficulties experienced during the period of initial operation have been corrected and since that time both units have operated satisfactorily.

The installation of two additional units has been authorized and should be completed in 1941.

At the beginning the Wheeler plant was operated as an addition to the then existing Wilson and Norris plants. Subsequently the Pickwick Landing plant, the Memphis steam plant, and the properties acquired from the Tennessee Electric Power Co. were added to the system. The service area of the system includes mainly the major portion of Tennessee, northeastern Mississippi, northern Alabama, northwestern Georgia, and southwestern North Carolina. Large industries served include the Aluminum Co. of America, the Monsanto Chemical Works and the Victor Chemical Works.

The amounts of energy and maximum peak load supplied by the Wheeler plant as a part of the system from the beginning of operations through December 31, 1939, are shown in table 29.

TABLE 29.—*Power generation*

	Nov. 9 to Dec. 31, 1936	Jan. 1 to Dec. 31, 1937	Jan. 1 to Dec. 31, 1938	Jan. 1 to Dec. 31, 1939
	Kilowatt-hours	*Kilowatt-hours*	*Kilowatt-hours*	*Kilowatt-hours*
Gross energy generation	19,975,000	118,227,510	258,484,000	424,815,000
	Kilowatts	*Kilowatts*	*Kilowatts*	*Kilowatts*
Average energy generation (per unit)	±15,900	±7,880	±14,750	±24,250
Maximum hourly demand	35,000	52,000	68,000	72,000

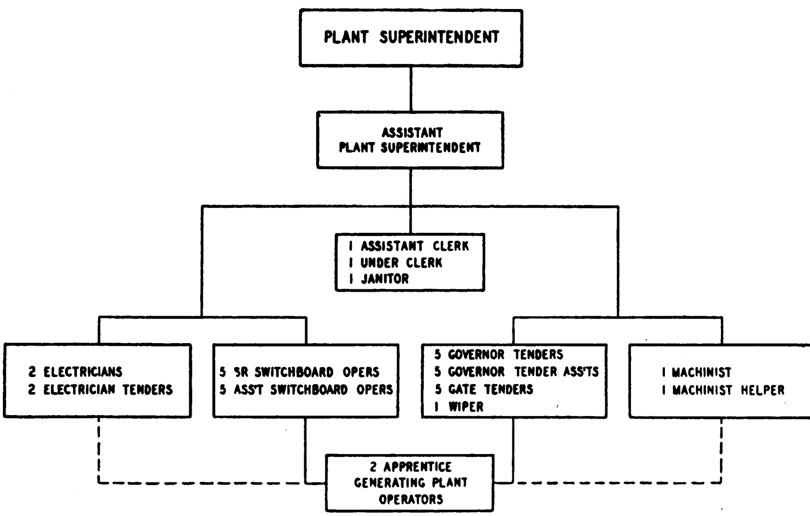

FIGURE 134.—*Organization chart—Operating staff.*

The personnel required to operate the Wheeler plant consists of 39 employees (see figure 134) classified as follows:

Supervision and engineering	2
Clerical	2
Operating	27
Maintenance	6
Janitors and wipers	2
Total	39

MALARIA CONTROL

Control of malaria mosquitoes in the reservoir is accomplished by fluctuation of the water level, removing vegetation which tends to favor the propagation of mosquitoes, and applying larvicides from boats and airplanes.

The first season of impoundage in the reservoir was the summer of 1937. Extensive sprouting of second growth had occurred after the initial clearing, and excessive rains and floods during the fall and winter of 1936–37 prevented the proposed rebrushing, resulting in vegetation protruding above the water surface in flat areas of shallow water. Application of Paris green from airplanes was the only practical means of control, and this was done by one plane under contract and one plane owned by the Authority. In July the reservoir was lowered about 8 feet to facilitate certain work at Wheeler and to aid in making closure of the third-stage cofferdam at Pickwick Landing Dam; and because of the sprouting which occurred during the period, it was not refilled during the mosquito-breeding season. During this time extensive rebrushing operations were carried on so that conditions were much improved at the beginning of the 1938 season, though further shore-line improvement is still necessary. In general, satisfactory results were obtained, but in some areas the emergent vegetation provided favorable conditions for mosquito breeding and made the effective application of larvicides quite difficult. Since it was not possible to obtain 100 percent kill under these conditions, the light breeding which occurred over extensive areas resulted in high mosquito densities at the nearby catching stations. The apparent results were also affected to a considerable degree by mosquito breeding in ponds and other natural breeding areas which were near the reservoir but on which control methods were not being applied because the areas were mainly outside the Authority's property.

Mosquito-control operations for the 1938 season were begun the first week in May. Although favorable water level fluctuations were maintained in the lower part of the reservoir until July, the excessive rainfall and resulting high stream flows produced high water elevations in the upper part of the reservoir and greatly increased the amounts of larvicides required. It also resulted in increasing the extent of natural breeding areas adjacent to the reservoir, with extensive production of mosquitoes.

The cost of mosquito-control operations, together with technical supervision in the Wheeler Reservoir for the fiscal year 1937–38, was $151 per mile of shore line, which compares favorably with costs of approximately $175 to $225 per mile of shore line during the first years of impoundage of other reservoirs for which records are available.

Malaria-control activities are divided into four stages: pre-impoundage procedures, reservoir maintenance procedures, investigations, and cooperative relationships.

Preimpoundage procedures.

Before undertaking the reservoir-clearing operations, the malaria endemicity in the population groups contiguous to the reservoir was determined by making a house-to-house family survey during which blood smears were obtained and examined for malaria parasites. Likewise, spleen and blood surveys were made of school children. The school surveys were repeated annually to determine fluctuations in the amount and type of malaria in the population groups and the extent to which preimpoundage operations influenced such fluctuations. Collecting stations were located at appropriate points, and collections of adult anophelines made twice a month. The extent of breeding in the area was determined by the collection of larvae within the limits of the reservoir and by investigation of natural breeding places within the 1- and 2-mile limits of the reservoir. This procedure is based upon the belief that the dispersion of *Anopheles quadrimaculatus* (the mosquito associated with malaria) occurs for a distance of approximately 1 mile from a given breeding place. These activities determined the potential transmission of malaria in and around the reservoir.

The thorough preparation of a reservoir in advance of impoundage is essential for efficient control of mosquito breeding during impounding. The regulations of the Alabama Health Department for filling the reservoir were observed. In clearing the reservoir, provision was made for the drainage of marginal areas in order that draw-downs of the main lake would not leave isolated pools of water suitable for mosquito propagation. Willow stumps are particularly obnoxious at times of draw-down, and procedures were developed for poisoning these stumps by the application of alkaline solutions of arsenites during the last growing season before impoundage. Cut-over areas resprout and within the fluctuation zone these shrubs, which may reach a height of 10 to 20 feet, create a considerable mosquito hazard. The shrubs as well as coppice, weeds, vines, and other herbaceous growth should be removed from the fluctuation zone after the close of the last growing season prior to impoundage.

The date of closure is of great significance in a mosquito-control program. It is desirable to fill reservoirs between growing seasons, and for this reason the reservoir was filled between March 27 and April 10, 1937. (The reservoir was temporarily filled in two periods between January 1 and February 5, 1937, by flood-waters.)

Maintenance procedures.

The season of active breeding of *Anopheles quadrimaculatus* is short, occupying that period of the year when the average daily temperature is 70 degrees Fahrenheit or higher, that is, from approximately May 15 to October 15. Mosquito control operations on impounded waters are therefore largely seasonal; rather, there is a seasonal intensification of these operations, characterized principally by the establishment of a summertime schedule for pool level fluctuation and the use of larvicides. Mosquito control is sought principally through maintaining an environment unsuited for growth, and much can be accomplished through water level fluctuation and shore line improvement. Since these measures cannot be expected to give complete control of *Anopheles quadrimaculatus* mosquitoes under all conditions encountered, resort is had to the application of larvicides as an emergency or temporary measure. Larvicidal operations consist of applying oil or Paris green dust from specially constructed boats and airplanes.

The most important of these control measures is water level fluctuation, which is the basis of biological control of mosquito breeding. Its effectiveness as a control measure is due to the fact that the variation of the pool level strands flotage along the shore line and dewaters vegetation which may be harboring mosquito larvae. In the past the fluctuation has been chiefly a slow, progressive dewatering of the shore line of storage reservoirs and cyclical fluctuation of the pool level in main river reservoirs. In the latter instance, the lake is periodically drawn down and refilled.

Experience in reservoir maintenance has shown that expenditures for shore-line improvement materially reduce the cost of applying larvicides. Such operations include the removal of drift, control of vegetation, and maintenance of marginal drainage. Accumulations of drift and flotage tend to occur at the heads of bights and indentations on the reservoirs and constitute a favorable condition for mosquito production. They likewise interfere with the larvicidal activities and with the benefits derived from pool level fluctuation. With the aging of reservoirs, aquatic and semiaquatic vegetation tend to invade the shore line. This produces a favorable environment for the production of *Anopheles quadrimaculatus*. Attempts have been made to control this vegetation both by mechanical removal and by the use of herbicides. To derive the maximum benefits from pool level fluctuation, it is essential that all flat areas drain out at times of draw-down, and this in turn necessitates maintenance of the marginal drainage projects.

As an indication for larvicidal procedures and as an index of the efficiency of all measures, indices of anopheline breeding are constantly maintained through dipping for larvae and the collection of adult anopheline mosquitoes at weekly intervals from established collection stations. Annual malaria infection surveys of the population of the mile zone are begun before impoundage and continued annually as a progressive measurement of control efficiency. It is thus possible to determine the influence of impounding on endemic malaria, except in those instances where transmission of malaria in the mile zone is affected by mosquitoes from natural breeding places.

Investigations.

Investigations of malaria endemicity were begun in 1934 and include studies of the treatment of malaria under field conditions; the development of a serodiagnostic test for malaria; studies on the season of malaria transmission; and studies of the epidemiology, parasitology, and experimental chemotherapy of malaria. Engineering studies are being made, including preimpoundage studies, and planning of reservoir preparation, engineering studies of the control of mosquitoes by larvicidal measures, drainage, pool level fluctuation, and improvement of shore lines; special studies of the improvement of equipment and methods of larval control. Attempts are constantly being made not only to improve the efficiency of these operations but also to reduce the cost.

Cooperative program.

Malaria control in the reservoir is part of the larger problem of malaria control in the South. These activities involve considerable cooperation with the Alabama State Health Department, the Tennessee State Health Department, the University of Tennessee, and other governmental organizations.

GROUND-WATER STUDIES

To study the ground-water conditions on lands near the reservoir, 75 wells of which 10 were recorder-equipped were placed at selected locations adjacent to the reservoir, and systematic measurements were taken over a period of time 1 year prior to and 2 years subsequent to the filling of the reservoir. A study of the fluctuations of the water table during this time has shown that only a few low areas adjacent to the lake were affected. The water table has been raised slightly at about 15 well sites and as much as 10 feet at a few of the wells.

After filling the reservoir it was observed that a small stream of water issued from the side of a small ravine and discharged into a branch just below the south abutment of the dam. Flow records of this leak have been kept since 1935 in order that any increase may be detected without delay. Samples of water have been collected both from the point of inflow and the outlet of the leak, and tests on these samples show that there has been no enlargement of the passage by solution. In fact there has been a slight deposition of silt in the channel. No attempt will be made to stop this leak unless there is a marked increase in discharge.

The geologic conditions in the reservoir are favorable to the formation of sinks, large numbers of which exist in that area. About 120 of these have been examined in the reach of the river between the dam and a point a short distance above Decatur, Ala. The condition of each sink at the time of inspection was determined, and persons living in the area were interviewed for information relating to past conditions in these sinks. Staff gages have been placed in the more important sinks, and readings taken since April 1937. These investigations have been necessary to determine whether the raising of the normal river level has changed conditions in these sinks. In the Spring Creek area conditions became worse after the filling of the reservoir and the Authority constructed drainage ditches to alleviate the conditions.

SILT INVESTIGATIONS

The deposition of silt above a dam may be one of the chief factors in shortening the economic life of the reservoir. In recognition of this fact, the Authority has conducted an extensive study of the movement of silt by the streams in the watershed and is actively engaged in stimulating and assisting in a program of soil and water conservation through improved agricultural and forestry practices throughout the Tennessee Valley. Since the latter part of 1934, silt sampling stations have been maintained on the main river at Decatur and at one gaging station on each of the lower reaches of the Elk, Flint, and Paint Rock Rivers. The data obtained from these stations are useful in estimating the probable life of the reservoir, in studies of the clarity of the reservoir for recreational purposes, and in determining the amount of dredging, if any, which would be required to maintain navigable depths.

When the silt investigations were first undertaken, studies were made to correlate the silt load of a stream with either the velocity or the discharge. Since a stream can transport no more silt than is available, a river at flood stage might carry but little more silt than at ordinary stages, depending upon the amount contributed by its tributaries or available for entrainment from the river bed and banks. Consequently, the only way in which this amount can be determined is by periodic sampling.

Present sampling practice utilizes a streamlined horizontal trap, developed by Authority engineers, with openings at both ends that may be closed simultaneously to entrap a sample of water at any desired depth. Samples are collected at top, mid-depth, and bottom from several vertical sections, the number depending upon the width and depth of the stream. The samples are transported to the silt laboratory at Norris for determination of the suspended silt content by weighing the dried residue from filtration.

The results of laboratory analyses are correlated with stream discharge to determine the weight and volume of silt transported annually past the sampling station by the stream. These values are then coordinated with similar figures for other locations along the river to determine the amount of deposition in any intervening reservoirs or the increase in silt load due to local tributaries.

Prior to the filling of the reservoir, a number of ranges were established across the main river and tributaries within the pool area. Cross sections were prepared for comparison with results of future surveys along the ranges to determine the actual amount of silting that may occur during the intervening period of time.

Wheeler Reservoir occupies a sheltered position, from a silting standpoint, since most of the water entering this pool will be practically desilted during its slow passage through the several reservoirs being constructed upstream. The streams directly tributary to Wheeler Reservoir contribute more silt than will be carried into it by the Tennessee River after completion of the proposed reservoirs. About one-third of all the silt entering the pool will pass through it and be discharged over Wheeler Dam. Taking all such factors into consideration, it is estimated from the silt data available that not more than 20 or 25 percent of the reservoir volume will be filled with silt in the next 100 years.

FORESTRY AND EROSION CONTROL

Soil erosion was recognized as a definite problem on many of the open areas acquired as part of the protective strips surrounding the reservoir. On many of the other open areas the soil had been so depleted by row cropping that it was no longer useful for agriculture. The program of reforestation and soil erosion control was started during the fiscal year 1935–36. On some areas erosion had been so severe that engineering treatment was necessary prior to reforestation. Table 30 shows the accomplishments in engineering erosion control since this work was initiated in 1936–37. Table 31 shows the progress which has been made in reforestation from fiscal year 1935–36 to December 31, 1938.

Planting stock for the reservoir areas is supplied from the Authority's forest tree nursery located at Muscle Shoals. The areas to be planted are selected by the Authority and labor for engineering treatment and reforestation is supplied by CCC camps. During the fiscal year 1938–39, camps located at Wilson Dam, Athens, and Huntsville, with a side camp at Hartselle, were available for this activity. The TVA-CCC camps are operated jointly by the Authority and the United States Forest Service. A representative of the Authority serves as the technical supervisor of the program; actual supervision of work on the job and the actual operations of the camps are handled by the United States Forest Service. The camps are available for general watershed protection work, but special emphasis is placed on the control of erosion and reforestation of the reservoir lands.

TABLE 30.—*Summary of engineering work for erosion control*

Year	Number of tracts	Tract area (acres)	Project area (acres)	Number permanent dams constructed	Number temporary dams constructed	Bank protection (square yards)	C. C. C. man-days
1936-37	5	309	109	1	374	60	422
1937-38	28	1,479	455	0	2,110	7,788	4,803
1938-39 (July 1 to Dec. 31)	18	1,995	431	0	391	9,301	2,743
Total	51	3,783	995	1	2,875	17,149	7,968

TABLE 31.—*Summary of reforestation work*

Year	Number of tracts	Number of trees planted				Total area planted (acres)
		Black locust	Pine	Others	Total trees	
1935-36	10	1,000	500,800	----------	501,800	407
1936-37	205	9,990	3,412,800	139,072	3,561,862	3,081
1937-38	184	68,100	3,536,050	134,740	3,738,890	2,992
1938-39 (July 1 to Dec. 31)	7	700	159,400	8,400	168,500	140
Total [1]	406	79,790	7,609,050	282,212	7,971,052	6,620

[1] Includes 21 tracts and 990,000 seedlings, involving 830 acres of replanting.

The Authority is preparing technical plans for the development, protection, and utilization of the timber resource. The object of such work is to bring the depleted and burned-over forest land to its maximum productivity at the earliest date in order that its full effectiveness in water and soil conservation will be realized and the greatest local benefit achieved.

FISH AND GAME

Impounded lakes may become, after the first few years of their existence, decidedly lacking in fish productivity. Such lakes have sometimes been referred to as "biological deserts." In an effort to counteract these possible effects on the biological resources of the involved areas, the Authority undertook the specific functions of readjusting the biological factors of the territory to the newly created physical conditions. A survey of the Tennessee River in the immediate vicinity of Decatur, Ala., showed that in 1936 there were approximately 7,550 fishermen dependent annually upon that river for either their food or their livelihood, and that these people were removing about 300,000 pounds of food fish per year having a market value of around $37,500. These were river fish. With the impounding of the reservoir, the river became a body of slack water having entirely different physical characteristics. With the large population of the area dependent upon the water for food, the Authority recognizes its moral responsibility to make every effort to maintain and to build up the fishing industry.

A fish hatchery has been constructed on the Elk River in cooperation with the United States Bureau of Fisheries. Here 150 acres are developed as a hatchery to stock this particular chain of lakes. A floating laboratory is being utilized to make a series of studies of problems involving the fisheries' aspects of shallow lakes.

In managing the marginal property along the lake the Authority is not overlooking the possibilities of game development and protection. Game management activities include the establishment of game management areas, refuge areas, and other restricted localities where quail, wild turkey, and economically important fur-bearing forms may be increased for the benefit of the people of the valley. With the advance of civilization, the wildlife resources of the valley have been so far reduced that they are now a fraction of their former importance, and every effort is being directed toward the rehabilitation of the original native species without the complications resulting from the introduction of exotic forms. There is constantly being kept in mind the vital fact that wildlife is a source of food for the native population.

Much of the Authority's land in the Wheeler area will be covered by a shallow body of water admirably suited to migratory fowls such as ducks and geese. Wheeler Lake is part of the series of broad shallow lakes in the lower river area. The lakes are expected to form a great migratory waterfowl area. Some 41,000 acres have been set aside by Executive order, for a migratory waterfowl and upland game refuge which may be planted with adequate food and shelter plants, and which, under proper protection, may afford resting and breeding areas suitable to the multitude of birds using them. The establishment of such areas will not stop the hunting possibilities of the areas. In all probability the establishment of such areas will assure the region of fine hunting both for food and for sport; and since the sporting angle brings with it a revenue in the form of wages to guides, as well as money spent for food, lodging, ammunition, and other incidentals, this phase of the work should prove a considerable source of income to the local population over and above their own shooting for food purposes.

AGRICULTURE

The Authority acquired approximately 53,500 acres of land in the Wheeler area which is not actually flooded by the impounded water. Such portions of these lands as have not been required up to now in carrying out major objectives of the Authority's program have been available for agricultural and other uses.

In accord with its objectives of serving the valley people to their best advantage, the Authority has initiated an agricultural land-use program through organized local groups of farmers in an effort to protect and utilize Wheeler Reservoir lands most efficiently and economically and to promote better agricultural practices in the communities by means of education and demonstration. As a basis for determining proper land use, organized groups of farmers asked for and were supplied, as soon as it could be completed, a modern soil survey. Pending completion of the soil survey, soil conservation associations in the counties adjacent to the reservoir asked permission to assume certain responsibilities in the conservation of these lands in fitting them back into the local economy under readjusted systems of farm management. This request was granted, and subsequent land use was in accord with their recommendations.

Continuous cultivation had reduced much of this land to a low state of fertility and subjected it to erosion. Pending the completion of the soil survey it was decided to seed some of the land to lespedeza. In the spring of 1936, 17,278 acres of the reservoir land was seeded at an average cost of $1.50 per acre; this was accomplished in cooperation with the county soil-conservation associations and the Alabama Agricultural Extension Service. Arrangements were made, under similar auspices, to harvest some of the seed crop that fall. The Authority received 66,000 pounds of seed harvested by the farmers on a share basis. Most of this seed was used in the spring of 1937 to seed additional land around Wheeler and other reservoirs.

The success of this project aroused greater local interest in the use of the publicly owned land, and in 1937, under cooperative agreements varying somewhat as to terms of seeding and harvesting, the Authority acquired approximately 240,000 pounds of lespedeza seed in which four varieties were represented. Most of this seed was also used on reservoir lands, only 50,168 pounds being sold after the 1938 seeding season was well advanced and the seed available commercially was about exhausted. A continuation of this cooperative program during the year 1938 produced a cash income of $3,319.30 from rental permits in which 6,613 acres of land was involved. In addition, the Authority received approximately 187,000 pounds of lespedeza seed as its share of the crop harvested.

The soil survey having been completed and experience gained in its interpretation, it appeared that for the year 1939 local county soil-conservation associations were in position to assume responsibility for allotment and management of these lands. Based on a study of this completed survey and other important factors, these farmers judged at least half of the land to be adapted to the production of crops wtihout materially affecting the efficiency of the reservoir, an additional one-third to the production of permanent pasture, and the remainder to farm timber production. Consequently, at the request of the associations, contracts were drawn up to develop broader and more diversified land-use practices on those lands suited to agricultural use and not needed for the Authority's primary functions. The soil-conservation associations in all counties affected by the reservoir assumed the responsibility of leasing the land to farmers, preferably to those farmers adjacent to the reservoir property in need of land to complete an efficient farming unit. This program is developing an intense interest in the proper use and care of the land, and is producing a reasonable income for the Authority. The income in 1939 from leases on 10,457 acres of agricultural lands amounted to $26,967. Ten percent of this amount went to the soil-conservation association for administrative work done by them.

Before this locally sponsored program, practically no winter legumes were grown on this land. Now, however, the current year's contracts provide for the seeding of 1,796 acres of winter legumes and the construction of terraces on 140 acres of land. This demonstration in diversification and good land-use practices is producing desirable effects on privately owned lands in the area as well as on the reservoir lands. This increasing interest in land-use and the actual need which exists in this area for agricultural land should bring about a more efficient management of the land and an increase in its productive capacity, contributing to readjustments in the local economy.

RECREATION

Within the reservation immediately south of the dam, the Authority, with the cooperation of the National Park Service and the Emergency Conservation Work program, developed two small areas for intensive recreational use. The smaller of these two areas, comprising about 10 acres, lies immediately east of the south abutment of the dam on a wooded hill overlooking the lake. It includes a cherted access road, a parking area, a picnic shelter with stone fireplaces, sanitary facilities, a room for storage of landscaping implements, and a stone overlook. Winding trails lead along the brow of the cliff overlooking Wheeler Lake. The larger of the two areas is located along the shoreline of Big Nance Creek and its junction with Wilson Lake, and consists of approximately 50 acres of heavily wooded land. Facilities include a cherted access road, a parking area, a frame picnic shelter with twin fireplaces, a rustic overlook building, a latrine building, drinking fountains, tables, benches, and outdoor ovens, together with foot trails leading to various points of interest. A National Park Service CCC camp constructed the facilities in these two recreational areas between April 1934 and November 1935. The areas are used extensively by individual and local groups from the nearby towns and cities within a radius of 75 miles.

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Chapter 8

COSTS

Generally, all actual construction and reservoir activity accounts were kept by the dam accounting office, and all engineering planning and design accounts and miscellaneous administration and overhead accounts were kept by the accounting office in Knoxville. A cost engineering staff at the dam analyzed and interpreted the construction costs of the dam, powerhouse, and immediately surrounding works but was not concerned with miscellaneous reservoir activity work. As the job progressed, changes and refinements were made in the accounting and cost-keeping methods and only the methods finally adopted are set forth in this chapter. Many of these methods have been adopted by subsequent TVA construction projects while others have undergone further improvements. The accounting and cost engineering methods used for Wheeler Dam were essentially the same as those used for Norris Dam and are discussed in greater detail in the report on Norris Dam.[1]

FIELD ORGANIZATION AND RESPONSIBILITIES

The accounting organization at the dam was responsible for general accounting and bookkeeping, while the cost engineering staff was responsible for allocation of charges to the proper accounts and the interpretation and application of cost information.

The accounting organization reported administratively to the construction superintendent and functionally to the construction accountant at Knoxville. In addition to the usual accounting work, this staff handled timekeeping, pay rolls, and warehouse and shop order accounting. Accounting work for the construction and operation of the camp was done by independent offices not directly connected with the dam organization. Employee's statements were rendered each pay period to the dam accounting office, from which charges were made for camp services such as meals, room rent, and other charges. Accounting for the various reservoir activities was done by separate organizations, not responsible to the dam construction organization. Details and supporting data for these activities were kept by the Knoxville offices of the respective organizations or by the temporary field offices, while the control accounts were maintained at the dam.

The cost engineering organization reported administratively to the construction engineer and functionally to the general cost engineer at Knoxville. It prepared the bases for the distribution of clearing accounts, allocated all charges, applied plant and equipment depreciation schedules, and worked in cooperation with the accounting staff,

[1] Ch. 9, *The Norris Project,* Tennessee Valley Authority Technical Report No. 1.

handling matters where technical questions entered into accounting operation. The cost engineering staff based its analyses and interpretation of cost data on the cost books and records kept by the accountants. It was their duty also to gather data for various reports and publish current cost data for historical and job control purposes, prepare record analyses of completed costs, and prepare cost and budget estimates.

CONSTRUCTION ACCOUNTING

The practices and procedures developed as the work progressed closely followed methods and principles previously developed for other similar construction work. The field books of account consisted of a general ledger, voucher register, invoice register, stores ledger, cost ledger, equipment ledger, and warehouse, receiving, and issuing reports. Control accounts were maintained by the accounting office in Knoxville. Standard purchase vouchers, prepared in Knoxville, were certified by the field office before payment. All disbursements except emergency payments were made from the Knoxville office.

Classification of accounts.

In addition to the clearing equipment and plant accounts, key accounts were maintained for principal construction features. Each of these key accounts was further subdivided as circumstances warranted.

Accounting for personal services.

Accounting for personal services provided pay roll records and enabled distribution of labor to proper cost accounts. The method of accounting for annual and hourly employees was similar, except that separate pay rolls were maintained and methods of distribution differed. Foremen's time cards, prepared for each group of men, gave a full description of the work done and the hours worked by each man. After being classified by the cost engineers, these were posted by the time office. Twice each shift a time checker made a tour of the job, spotting the men at work as a check against the pay roll. Where possible, pay roll charges for foremen were applied directly to features or to clearing accounts. Other annual pay roll charges were allocated by the cost engineering staff on an appropriate proportional basis. Pay rolls were prepared twice monthly from employees' ledger cards with deductions being made for camp services, lost tools, and other charges. They were sent to Knoxville for auditing and the issuing of checks by the treasurer which were delivered to the employees through the time office at the dam.

Accounting for material.

Materials accounting was logically divided into three steps: requisitioning and purchasing, receiving, and distributing and charging to accounts.

All special and regular stock material was requisitioned on a standard purchase requisition (copies of which were sent to the budget office) and ordered on standard purchase orders. Information concerning the ordering, shipping, and receiving of the material was noted on the storeroom copy of the requisition.

The receiving report showed the articles purchased, the order or contract number, invoice price, handling charges (for stock material), and account number for materials charged directly to features. Notation was made as to when and how the material was received. Special reports were prepared for partial shipment of material and over, short, and defective materials. When material was received which had been damaged in transit, proper notation was made on the transportation expense bill and information sent to the central office for the filing of a claim. Stock materials were entered on the stock ledger directly. Special material was charged to the feature on which it was used.

The field voucher register was the basic document for distributing direct material charges. At the close of a month the total amount chargeable to each account was posted in the cost ledger through the journal voucher. The cost engineering staff coded and classified all direct charges. A materials requisition signed by the section foreman was necessary for warehouse issues. This, with the warehouse "Materials Requisitioned and Issued" form, was coded by the cost engineering section and charged by the accounting office to the proper accounts.

Equipment and plant capital accounts.

A separate group of accounts was assigned for equipment charges and another similar group for plant charges. Equipment consisted in general of all tools and movable construction machinery, while plant consisted of units of operation necessary or incidental to construction work. Supporting the capital accounts there was a detailed engineering data and physical description sheet for each piece of major equipment and each assembled plant. These sheets were prepared by the warehouse when the equipment was received on the job.

The original TVA depreciation policy provided that 100 percent depreciation of both plant and equipment should be absorbed in feature costs during the construction period, and accordingly depreciation schedules were developed from which a monthly charge was made to the work being performed and a credit given to an equipment and plant reserve account. The "straight line" method of depreciation was used on a monthly basis for most equipment in order that a chosen depreciation date would be met. Equipment assigned to the mixing plants and cranes was depreciated on a production basis. Plant charges for power, water, and air systems and for plant buildings were depreciated on a uniform monthly rate in order to absorb their costs by the end of the heavy construction operations. Plant roads and bridges were written off completely at the close of construction, their costs being distributed over the entire job on the basis of total feature cost.

As the work drew to a close, much of the equipment plant was still useful and was transferred to other projects. Values were determined by the construction plant organization in Knoxville and were approved by both the receiving and issuing projects. Transfer values for large assembled plants and floating equipment were generally established by executive decisions. Credit for equipment transferred or sold was placed on the project's books in special accounts in the equipment group. In this way credits were applied to reduce the final project investment in plant and equipment. At the close of the job credits for the principal pieces of construction equipment and plant were distributed over the features on which the equipment and plant were operated. For small and minor equipment the credits were accumulated and distributed to all features on the basis of total costs.

Much study was given to the valuation of construction plant equipment. For the purpose of distributing ownership investment, equipment was divided into two classes: first, hourly or major, including equipment adapted to recording of hours, service, and repairs; and second, monthly or minor, consisting of equipment not well adapted to such records. It was also necessary to determine both the economical life and the probable required length of service for TVA needs of individual types of equipment in both classes. On equipment whose utility was not completely exhausted at Wheeler, service and repair records fairly well established the economical service life for the next project. Some units, mostly major, were found to have an economical life exceeding the period of the Authority's scheduled construction. In such cases an expected TVA life less than the economic life was established for depreciation purposes in order that the ownership investment might be liquidated at the close of the scheduled service without unfavorably altering cost records. Straight-line monthly depreciation rates were used for minor equipment which represented the greatest number of items and involved relatively small ownership investment. The most suitable unit of service for valuation of major units of equipment was found to be "net operating hours." The conditioning investment on major equipment was found to increase with an upward curved trend as the service increased.

Equipment use-rates were thus set up to cover cost of owning and conditioning major equipment. These rates enabled construction plant investment accounts to show book values and accord with intrinsic values of the equipment and provided sound transfer values. They also enabled distribution of ownership investment as work progressed, and each project was able to apply a uniform method of clearing equipment repair and depreciation expense.

Equipment and plant operation.

Clearing accounts were maintained for each major unit or group of similar units of major construction equipment and other plant operations such as electric and water systems and maintenance of general construction facilities. At the close of each period the charges shown in the accounts were distributed, usually on the basis of use, but in some instances on the basis of labor or feature cost. The debits for clearing accounts were accumulated and totaled, and a separate account for credits was assigned so that the cost ledgers would always show the accumulative total cost as well as accumulative credits to any one operation. Daily shift reports were made by machine operators indicating their estimate of the time spent on each operation. These facilitated equitable distribution of each machine's operating time.

Shop order systems.

Shop order systems were maintained for the machine shop, garage shop, and carpenter shop. Electric shop charges were made directly to the feature on which the work was done. The general method of handling shop operating charges was to allocate to the proper shop operating account (set up in the clearing account group) all charges for direct or supervisory labor, materials, and shop expense. The shop account was cleared by means of shop orders. Separate shop orders were prepared for each major job. On these orders were posted quantities of materials used, time required, and labor rates. All orders were cleared each month whether the work they covered was completed or not. To facilitate record keeping and where possible, principal items of material, such as repair parts for trucks and tractors, were charged directly to machine rather than through shop orders. Blanket shop orders were used for all small jobs performed in the shops rather than preparing a different order for each one. Each job was numbered on such shop orders so that labor and materials for them could be segregated and properly allocated.

Work was frequently performed by the TVA forces for manufacturers and subcontractors occasioned either by correcting errors of fabrication or as a facility for contractors working on the job upon written authorization of the contractor or manufacturer. Separate work orders were maintained from which monthly statements were prepared.

Hired trucks and teams.

Trucks and teams hired to supplement equipment of the Authority were paid for by the hour. These charges were handled in the accounts similarly to material charges through the invoice register.

General expense and miscellaneous accounts.

General expenses consisted of administrative cost, engineering and superintendence, maintenance of headquarters facilities, project investigations, designing, and other items of administration which could not be equitably distributed as work progressed. This general expense is similar in nature to the clearing account in that it was distributed to the features upon completion but was carried intact until the completion of the work. The central office overhead applicable to all construction projects was distributed to the projects on the basis of labor. The project overhead, together with the central office overhead, was distributed to the feature accounts at the close of the job on the basis of total feature cost.

Engineering planning and design costs were kept in separate project accounts in the central accounting office in Knoxville. The direct design charges of the Bureau of Reclamation were kept at the construction accounting office.

Land acquisition costs including costs of land, as well as the cost of engineering and acquisition, were carried in a separate project account in the central accounting office records in Knoxville.

Camp and village operations costs were not distributed to the dam construction during the process of the work, but an arbitrary amount based on total probable charge for this service was entered monthly in the project books as an item of general expense with a corresponding credit to an account designated as "Reserve for Camp Cost."

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CONSTRUCTION COST ENGINEERING

The cost engineer sought to develop significant cost information on controllable and salient construction features, and to bring it quickly to the attention of the superintendent and his staff. This information was developed in many ways.

Cost analyses.

Cost analyses were made of specific accounts involving the separation of the various components of cost found in the records and the application of quantities to obtain unit costs. These showed the total and unit costs of the different components, trends and important notes that were valuable to the construction superintendent. Detailed analyses were prepared for all principal features, and as conditions warranted, summarized analyses of more important construction features were prepared. Study and interpretation gave great significance to the finished analyses. It was usually necessary to make some adjustments with appropriate explanations in order that the analyses would not be misleading. In several instances charts were kept showing the monthly variation of component unit costs within a given total unit cost.

Equipment plant and cost records.

Equipment plant and cost records were kept for several uses, such as: establishing probable costs for future projects or in determining their most economical design, comparing unit costs at Wheeler Dam with other projects of the Authority, setting up a permanent plant inventory record, and preparing reports to the Federal Power Commission.

Daily reports for job cost control.

Daily reports for job cost control were prepared with the view of reducing and controlling costs. These included force reports and inspectors' shift reports prepared by the time office and field engineering section, respectively. The cost engineering staff prepared an hourly labor distribution showing the day's labor charges to features compared to the previous days' work.

Time studies.

Time studies were made by a specially trained time study group transferred from Norris Dam to make studies on certain construction operations where opportunities for considerable savings appeared to to exist. The more important of such assignments were formwork erection and intake gate assembly. Such time studies developed information which was not only valuable from the standpoint of assisting in reducing job costs, but was useful in developing more accurate information for selection of equipment for other projects of the Authority.

Machine operation and delays.

Machine operating time and delays records were kept on principal features of equipment and assembled plants. In addition to indicating weaknesses, such summarized information was valuable in making future purchases of plant and equipment for other projects.

CONSTRUCTION BUDGETS

Budget requests were generally submitted semiannually by the construction forces and consisted essentially of a brief statement of the scope of the work and the estimated cost of the work to be done under the requested allotment, together with a summarized cost statement of construction features, showing the status of current and estimated obligations. These were submitted to the Knoxville engineering office where they were assembled with similar estimates from other projects into a final statement showing needs for all engineering and construction operations. When funds were appropriated by Congress, the amounts as shown in the approved budget were allocated to the several organizations.

Organization budgets were watched carefully to see that expenditures and commitments did not exceed approved allotments and also to see that unobligated balances were promptly returned to the general fund of the Authority when expenditures were less than allotments. The central office in Knoxville issued a monthly report comparing the status of obligations against individual allotments. In order to check the budgetary status for the job, a more detailed monthly report was used in which it was possible to use the more recently developed unit costs and estimates of uncompleted quantities. With this it was possible to forecast probable modification of the previously estimated obligations.

SUMMARY OF COSTS

The final cost of the original Wheeler project as reported herein at $30,378,889.18 (General Summary, p. 283), includes hydraulic multiple purpose plant consisting of land and land rights for which title rests in the United States Government, structures, improvements and equipment together with transmission plant and general plant constructed concurrently with the dam.

The final cost of the project by character of expenditures is as follows (Detailed Summary, p. 284):

Land costs		$4, 784, 395. 14
Construction costs:		
Direct construction costs	$20, 891, 001. 59	
Indirect construction costs	1, 616, 207. 88	
		22, 507, 209. 47
Distributive general expenses:		
Design and construction engineering	1, 452, 902. 15	
Executive and administrative costs	1, 366, 781. 53	
Other general costs	267, 600. 89	
		3, 087, 284. 57
Total cost		30, 378,889.18

Land costs of $4,784,395.14 cover reservoir land purchased in fee, flowage easements, and highway and railroad relocation easements, for which title rested in the United States Government as of September 30, 1938.

Direct construction costs, totaling $20,891,001.59, are for labor, material, construction plant, equipment, tools, shop expense, warehouse charges, transportation, and other similar expenditures.

Indirect construction costs total $1,616,207.88 and include field superintendence, field office expense, provision for medical service, police and guide service, and the cost of camp operation including the entire first cost of temporary camp facilities and normal depreciation during the construction period on the permanent village facilities (pp. 299 and 300). These costs have been distributed on the basis of the direct construction costs of those organizations incurring them.

Design and construction engineering costs amounting to $1,452,902.15 include the Wheeler project's proportion of the chief engineer's general administrative office, the salaries and expenses of executive and supervisory engineers, cost engineers, concrete technicians, inspectors, engineering for control lines, geologic studies, design of structures including the United States Bureau of Reclamation design charges and a small amount of consultation fees (p. 300). These engineering costs have been distributed on the basis of related construction costs.

Executive and administrative costs, totaling $1,366,781.53, include the Wheeler project's proportion of the salaries and expense of the general administrative offices of the Authority and those division administrative offices, such as the construction and maintenance and the highway and railroad organizations that were concerned with the construction of specific portions of the project (p. 300). These costs have been distributed likewise on the basis of related construction costs.

Other general costs of $267,600.89 include such charges as reservoir and dam site surveying and mapping, hydraulic studies, and general project planning (pp. 300 and 301). These have been distributed over all accounts exclusive of land costs.

In the three-plant allocation report[2] approved on June 6, 1938, a preliminary figure was used for Wheeler hydraulic multiple purpose plant of $32,473,542. This was based upon the books of account as of February 28, 1938, adjusted to exclude those items of cost not subject to allocation and to include estimates of allocable expenditures yet to be made.

The balance sheet of the Authority as of June 30, 1938,[3] shows the hydraulic multiple purpose plant of the Wheeler project at $31,586,564.45, a reduction of $886,977, which is accounted for by elimination of such items as estimates to complete, operators' villages, and nonproject charges.

The cost of the hydraulic multiple purpose plant as reported herein, totaling $29,294,720.07 (p. 283) differs from the June 30, 1938, balance sheet figure principally because of the elimination of land costs, where clear title has not yet passed to the United States, and the exclusion of the cost of the navigation lock ($1,734,038.34) constructed by the War Department.

The Authority has deposited $550,280.10 with the courts for land in condemnation. As the cases are settled and clear title has passed to the United States, the final costs thereof including acquisition and condemnation costs will be added to the project cost.

The schedules following represent summary statements of detailed analyses of cost.

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[2] H. Doc. No. 709, 75th Cong., 3d Session.

[3] See Annual Report of the Tennessee Valley Authority for the fiscal year ended June 30, 1938.

TABLE 32a.—*Final project cost—General summary*

Ac-count	Description	Amount
	HYDRAULIC MULTIPLE PURPOSE PLANT	
20	Land and Land Rights	$6, 819, 343. 77
21	Structures and Improvements	3, 496, 706. 20
22	Reservoirs, Dams, and Waterways	14, 848, 116. 53
23	Water Wheels, Turbines, and Generators	2, 236, 405. 42
24	Accessory Electric Equipment	563, 756. 66
25	Miscellaneous Power Plant Equipment	385, 118. 51
26	Roads, Railroads, and Bridges	945, 272. 98
		29, 294, 720. 07
	TRANSMISSION PLANT	
42	Structures and Improvements	7, 393. 42
43	Station Equipment	691, 054. 18
		698, 447. 60
	GENERAL PLANT	
	Operators' Village	376, 570. 20
	Communication Equipment	9, 151. 31
		385, 721. 51
	Total	30, 378, 889. 18

TABLE 32b.—*Final project cost—Detailed summary*

		Land costs	Direct construction costs: labor, material and other charges	Camp and other indirect costs	Total direct and indirect construction costs	Distributive general expenses: Design and construction engineering	Distributive general expenses: Executive and administrative costs	Distributive general expenses: Other general costs	Tota
	HYDRAULIC MULTIPLE PURPOSE PLANT								
	Land and Land Rights:								
200	Purchase price of land	$4,241,136.80			$4,241,136.80				$4,241,136.80
201	Expense of land and privilege acquisition	523,500.19			523,500.19				523,500.19
204	Relocating highways and highway bridges	17,962.72	$824,772.99	$66,222.50	908,958.21	$153,433.08	$54,041.10	$11,606.30	1,128,038.69
205	Relocating railways and railway bridges	1,795.43	175,355.34	11,973.81	189,124.58	30,034.22	10,303.20	2,405.50	231,867.50
207	Relocating other structures and improvements		169,167.49	440.47	169,607.96	7,374.12	283.23	1,872.96	179,138.27
209	Protecting existing structures and improvements		404,494.28	18,733.82	423,228.10	70,343.70	16,699.06	5,391.46	515,662.32
		4,784,395.14	1,573,790.10	97,370.60	6,455,555.84	261,185.12	81,326.59	21,276.22	6,819,343.77
	Structures and improvements:								
211	General preparation of site		7,995.20	653.39	8,648.59	531.64	447.77	101.73	9,729.73
212	General yard improvements		62,781.12	5,130.72	67,911.84	4,174.66	3,516.02	798.81	76,401.33
213	Powerhouse		2,038,258.27	166,574.32	2,204,832.59	135,535.29	114,151.36	25,934.15	2,480,453.39
214	Control building		689,299.91	56,332.25	745,632.16	45,835.44	38,603.80	8,770.43	838,841.83
215	Station filter plant		40,434.29	3,304.45	43,738.74	2,688.70	2,264.50	514.47	49,206.41
216	Miscellaneous buildings		22,654.48	1,851.42	24,505.90	1,506.42	1,268.75	288.25	27,569.32
217	Air, water, and automotive terminals		11,918.50	974.02	12,892.52	792.53	667.49	151.65	14,504.19
			2,873,341.77	234,820.57	3,108,162.34	191,064.68	160,919.69	36,559.49	3,496,706.20
	Reservoir, dams, and waterways:								
220	Reservoir		2,931,127.21	180,052.00	3,111,179.21	104,827.48	359,969.02	37,783.33	3,613,759.04
221	Concrete dam and spillway		9,126,951.47	745,889.65	9,872,841.12	606,902.47	511,149.14	116,128.44	11,107,021.17
226	Water conductors		62,583.91	5,114.60	67,698.51	4,161.56	3,504.97	806.86	76,171.90
228	Channel improvements and navigation facilities		42,043.24	3,435.94	45,479.18	2,795.69	2,354.60	534.95	51,164.42
			12,162,705.83	934,492.19	13,097,198.02	718,687.20	876,977.73	155,253.58	14,848,116.53
	Water wheels, turbines, and generators:								
230	Foundations and miscellaneous steel and iron		17,303.42	1,414.10	18,717.52	1,150.60	969.07	220.16	21,057.35
231	Turbines, including scroll case, speed ring, and draft tube liners		760,919.27	62,185.26	823,104.53	50,597.81	42,614.80	9,681.70	925,998.84
232	Auxiliary equipment for turbines		111,275.82	9,093.89	120,369.71	7,399.36	6,231.93	1,415.84	135,416.84
235	Generators		917,241.24	74,960.49	992,201.73	60,992.54	51,369.52	11,660.12	1,116,223.91

236	Auxiliary equipment for generators		4,745.02	387.78	5,132.80	315.53	265.74	60.37	5,774.44
239	Miscellaneous equipment for turbines and generators		26,241.10	2,144.52	28,385.62	1,744.92	1,469.62	333.88	31,934.04
			1,837,725.87	150,186.04	1,987,911.91	122,200.76	102,920.68	23,372.07	2,236,405.42
	Accessory electric equipment:								
241	Switchgear		80,568.11	6,584.34	87,152.45	5,357.43	4,512.16	1,025.12	98,047.16
242	Switchboards		190,462.99	15,565.37	206,028.36	12,664.97	10,666.75	2,423.39	231,783.47
243	Protective equipment		28,370.49	2,318.55	30,689.04	1,886.51	1,588.87	360.98	34,525.40
244	Electrical structures		12,317.80	1,006.66	13,324.46	819.08	689.85	156.73	14,990.12
245	Conduit work		44,248.72	3,616.18	47,864.90	2,942.35	2,478.12	563.01	53,848.38
246	Power wiring		15,288.42	1,249.43	16,537.85	1,016.61	856.22	194.53	18,605.21
247	Control wiring		9,876.31	807.13	10,683.44	656.73	553.12	125.66	12,018.95
249	Station service equipment		82,121.84	6,711.31	88,833.15	5,460.75	4,599.18	1,044.89	99,937.97
			463,254.68	37,858.97	501,113.65	30,804.43	25,944.27	5,894.31	563,756.66
	Miscellaneous power plant equipment:								
252	Station maintenance equipment		9,320.40	761.70	10,082.10	619.77	521.98	118.59	11,342.44
255	Cranes and hoisting equipment		168,778.48	13,793.23	182,571.71	11,223.04	9,452.33	2,147.48	205,394.56
256	Compressed air and vacuum cleaning system		16,689.04	1,363.89	18,052.93	1,109.75	934.66	212.35	20,309.69
258	Station service water system		9,664.82	789.85	10,454.67	642.67	541.27	122.97	11,761.58
259	Other miscellaneous equipment		112,009.95	9,153.88	121,163.83	7,448.18	6,273.05	1,425.18	136,310.24
			316,462.69	25,862.55	342,325.24	21,043.41	17,723.29	4,026.57	385,118.51
	Roads, railroads, and bridges:								
260	Access roads for permanent use		49,050.95	4,008.63	53,059.58	3,261.68	2,747.07	624.11	59,692.44
262	Roadway on concrete dam		727,706.42	59,470.97	787,177.39	48,389.30	40,754.74	9,259.11	885,580.54
			776,757.37	63,479.60	840,236.97	51,650.98	43,501.81	9,883.22	945,272.98
	Total hydraulic multiple purpose plant	4,784,395.14	20,004,038.31	1,544,070.52	26,332,503.97	1,396,636.58	1,309,314.06	256,265.46	29,294,720.07
	TRANSMISSION PLANT								
	Structures and improvements:								
422	General yard improvements		6,075.38	496.50	6,571.88	403.99	340.25	77.30	7,393.42
			6,075.38	496.50	6,571.88	403.99	340.25	77.30	7,393.42
	Station equipment:								
430	Outdoor substation structure		110,220.28	9,007.62	119,227.90	7,329.17	6,172.82	1,402.41	134,132.30
431	Switchgear		233,769.12	19,104.52	252,873.64	15,544.62	13,092.09	2,974.40	284,484.75
433	Protective equipment		10,307.33	842.36	11,149.69	685.39	577.26	131.15	12,543.49
435	Conduit work		6,636.45	542.35	7,178.80	441.30	371.67	84.44	8,076.21
436	Power wiring		23,647.55	1,932.57	25,580.12	1,572.46	1,324.37	300.88	28,777.83
437	Control wiring		12,830.37	1,048.55	13,878.92	853.16	718.56	163.25	15,613.89

TABLE 32b.—*Final project cost—Detailed summary*—Continued

		Land costs	Direct construction costs: labor, material and other charges	Camp and other indirect costs	Total direct and indirect construction costs	Distributive general expenses			Tota
						Design and construction engineering	Executive and administrative costs	Other general costs	
	TRANSMISSION—continued								
	Station equipment—Continued.								
438	Main conversion equipment		$164,002.60	$13,402.94	$177,405.54	$10,905.46	$9,184.86	$2,086.72	$199,582.58
439	Station service equipment		6,444.92	526.70	6,971.62	428.57	360.94	82.00	7,843.13
			567,858.62	46,407.61	614,266.23	37,760.13	31,802.57	7,225.25	691,054.18
	Total transmission plant		573,934.00	46,904.11	620,838.11	38,164.12	32,142.82	7,302.55	698,447.60
	GENERAL PLANT								
	Village:								
	Land improvements		28,999.01	2,324.01	31,323.02	1,562.47	2,678.52	375.77	35,939.78
	Dwellings		166,313.92	13,377.80	179,691.72	9,377.29	14,151.77	2,147.21	205,367.99
	Other buildings and structures		24,228.77	1,974.95	26,203.72	1,586.59	1,420.71	308.64	29,519.66
	Sewer system		27,536.53	2,206.81	29,743.34	1,483.68	2,543.44	356.82	34,127.28
	Water system		23,076.61	1,849.39	24,926.00	1,243.37	2,131.50	299.02	28,599.89
	Electric distribution system		29,078.13	2,373.44	31,451.57	1,931.17	1,626.49	369.90	35,379.13
	Miscellaneous equipment		6,276.42	512.30	6,788.72	416.84	351.07	79.84	7,636.47
			305,509.39	24,618.70	330,128.09	17,601.41	24,903.50	3,937.20	376,570.20
	Other:								
	Communication system		7,519.89	614.55	8,134.44	500.04	421.15	95.68	9,151.31
			7,519.89	614.55	8,134.44	500.04	421.15	95.68	9,151.31
	Total general plant		313,029.28	25,233.25	338,262.53	18,101.45	25,324.65	4,032.88	385,721.51
	Totals	$4,784,395.14	20,891,001.59	1,616,207.88	27,291,604.61	1,452,902.15	1,366,781.53	267,600.89	30,378,889.18

TABLE 32c.—*Final project cost—Details*

LAND COSTS AND DIRECT CONSTRUCTION COSTS

HYDRAULIC MULTIPLE PURPOSE PLANT

LAND AND LAND RIGHTS

Account	Description	Quantity	Unit	Rate	Amount
200	Purchase price of land:				
-1	Reservoir land—fee purchases; excludes tracts in process of condemnation where title had not been transferred as of Sept. 30, 1938:				
-10	Purchased under resolution of board of directors, dated May 1, 1934, authorizing acquisition of all necessary tracts lying within ⅛ mile of reservoir elevation 560 (1,121 tracts).	92, 769. 54	Acre	$43. 87	$4, 069, 680. 18
-11	Purchased under separate board resolutions covering recreational lands, city of Decatur navigation terminal, sinkhole drainage, and other lands within the reservation limits (21 tracts).	603. 58	Acre	80. 82	48, 781. 50
-2	Reservoir land rights—flowage easement purchases (118 tracts).	1, 680. 89	Acre	72. 98	122, 675. 12
	Total account No. 200	95, 054. 01	Acre	44. 62	4, 241, 136. 80
201	Expense of land and privilege acquisition: Costs of acquiring land and land rights set forth in account 200 (95,054.01 acres).	1, 260	Tract	415. 48	523, 500. 19
	Total account No. 201				523, 500. 19
204	Relocating highways and highway bridges:				
-0	State highways:				
-01	Elk River Bridge				206, 430. 30
-02	Highway No. 2, Elk River, including drainage structure				29, 484. 91
-03	Highway No. 3 north of Decatur				104, 244. 78
-04	Highway No. 3 south of Decatur				1, 205. 18
-05	Decatur to Somerville				35, 765. 41
	Total State highways	4. 51	Mile	83, 620. 97	377, 130. 58
-1	County Highways:				
-100	Buck Island Bridge				43, 616. 67
-101	Buck Island Bridge approaches				4, 822. 20
-102	Elk Mills Bridge				52, 127. 79
-103	Elk Mills Rd., including drainage structures.				60, 807. 95
-104	Norris Mill Rd., including bridges and drainage structures.				22, 341. 77
-105	Decatur-Mooresville-Triana Rd., including bridges and drainage structures.				34, 144, 44
-106	Rock House Landing Rd				12, 491. 86
-107	Huntsville-Triana Rd., including bridges and drainage structures.				9, 387. 77
-108	Huntsville-Green Grove Rd., including bridges, and drainage structures.				28, 822. 03
-109	Harris Station-Brown's Ferry Rd., including bridges and drainage structures.				11, 832. 53
-110	Hendrick's County roads, including bridges and drainage structure.				18, 058. 82
-111	Snake Rd. at Lucy Branch, including drainage structures.				12, 644. 26
-112	Second Creek Rd., including bridges and drainage structures.				14, 900. 25
-113	Old Somerville-Decatur Rd., including drainage structures.				6, 024. 47
-114	Lacy Spring Rd., including bridges and drainage structures.				5, 047. 25
-115	Lock A & B Rd., including bridges and drainage structures.				81, 060. 03
-116	Rogersville-Elk Mills Rd., including drainage structures.				10, 804. 29
-117	Anderson Creek Bridge				5, 000. 00
	Total county highways	21. 36	Mile	20, 315. 28	433, 934. 38
-2	Tertiary roads, including bridges, and drainage structures.	4. 53	Mile	6, 449. 32	29, 215. 40
-3	Bridge removals	13	Bridge	188. 87	2, 455. 35
	Total account No. 204				842, 735. 71

Table 32c.—*Final project cost—Details*—Continued

LAND AND LAND RIGHTS—Continued

204 **Relocating highways and highway bridges.**—Detail by components as indicated—supporting page 287.

Account	Land rights highway easements	Land privilege acquisition expense	Contract work	TVA force account	Total
204-01			$107,508.88	$98,921.42	$206,430.30
-02	$3,116.50	$5,816.67		20,551.74	29,484.91
-03			103,516.45	728.33	104,244.78
-04				1,205.18	1,205.18
-05			35,749.10	16.31	35,765.41
-100				43,616.67	43,616.67
-101				4,822.20	4,822.20
-102				52,127.79	52,127.79
-103				60,807.95	60,807.95
-104	75.00	415.48		21,851.29	22,341.77
-105	128.00	830.95	6,390.91	26,794.58	34,144.44
-106	628.50	3,739.29		8,124.07	12,491.86
-107				9,387.77	9,387.77
-108				28,822.03	28,822.03
-109				11,832.53	11,832.53
-110				18,058.82	18,058.82
-111				12,644.26	12,644.26
-112	35.00	415.47		14,449.78	14,900.25
-113				6,024.47	6,024.47
-114				5,047.25	5,047.25
-115	254.00	2,077.38		78,728.65	81,060.03
-116				10,804.29	10,804.29
-117			5,000.00		5,000.00
-2	15.00	415.48		28,784.92	29,215.40
-3				2,455.35	2,455.35
	4,252.00	13,710.72	258,165.34	566,607.65	842,735.71

Account	Description	Quantity	Unit	Rate	Amount
205	Relocating railways and railway bridges:				
-1	Southern Ry. protection (including land costs, $1,795.43).				$122,071.23
-2	L. & N. Ry. embankment protection				51,079.54
-3	N. C. & St. L. Ry				4,000.00
	Total account No. 205				177,150.77
207	Relocating other structures and improvements:				
-0	Relocating utilities:				
-00	Power lines				90,943.73
-01	Telephone lines				5,532.98
-02	Telegraph lines				5,686.01
-1	Relocating cemeteries:				
-10	Grave removal	176	Grave	$9.79	1,723.18
-3	Relocating families	840	Family	74.20	62,325.92
-7	Preservation of prehistoric material (Archaeological).				2,955.67
	Total account No. 207				169,167.49
209	Protecting existing structures and improvements:				
-7	Protecting Decatur, Ala., Municipal Facilities:				
-700	Improvement of State Highway No. 20.				28,600.22
-701	Vine St. relocation				3,721.02
-702	Washington St. bridge and road work.				8,098.46
-703	Danville Rd.—Bridge and road work.				19,782.96
-704	Surfacing Vine St				5,669.78
-705	Dry Creek improvement				62,458.69
-706	Dry Creek sewer				26,373.99
-707	Changes to water main				363.01
-708	Payment in lieu of water plant construction.				55,050.00
-709	Raising houses—Dry Creek area				6,275.65
-710	Draining colored school basement.				709.42

TABLE 32c.—*Final project cost—Details*—Continued

LAND AND LAND RIGHTS—Continued

Account	Description	Quantity	Unit	Rate	Amount
209	Protecting existing structures and improvements—Continued.				
	Protecting Decatur, Ala., Municipal Facilities—Continued.				
-711	McCartney Rd				$268.93
-712	Raising Railroad Ave				6,164.42
-713	Sewer construction	4	Mile	$26,438.16	105,752.64
-714	Main pumping station				29,636.36
-715	Riverview pumping station				12,247.12
-716	Initial operation of pumping stations.				3,413.25
-717	Relief outfall sewer				4,908.36
-718	Release from damage after sewer construction.				25,000.00
	Total account No. 209				404,494.28
	Total land and land rights				6,358,185.24

STRUCTURES AND IMPROVEMENTS

Account	Description	Quantity	Unit	Rate	Amount
211	General preparation of site:				
-1	Remove rubbish, clear, and grub	40	Acre	$199.88	$7,995.20
	Total account No. 211				7,995.20
212	General yard improvements:				
-0	Grading and landscaping				10,409.96
-1	Roads, sidewalks, bridges, trestles:				
-11	Sidewalks and steps (parking area to control building).				8,647.23
-13	Culverts				4,894.94
-15	Trestles (crane track)				9,783.15
-16	Locomotive crane track	2,528	Linear foot	5.74	14,502.58
-2	Retaining walls, fences, gates, and railings:				
-20	Retaining or property walls	106	Cubic yard	39.27	4,162.58
-21	Fences, gates, and railings				1,928.06
-8	Communication system (intrasite)				4,150.99
-9	Power and lighting (not part of any structure):				
-96	Lighting standards, fixtures and lamps (station area and tennis courts).				4,301.63
	Total account No. 212				62,781.12
213	Powerhouse:				
-0	Exploration of foundation				4,775.42
-1	Diversion and care of water				198,871.12
-2	Excavation and backfill:				
-23	Rock excavation	256,317	Cubic yard	1.81	465,074.92
-3	Foundation preparation and treatment:				
-30	Drilling grout holes				6,889.64
-37	Pressure grouting	2,180	Cubic foot	5.56	12,125.72
-39	Gravel fill under service bay and control building.	1,029	Ton	3.58	3,687.85
-4	Concrete:				
-40	Substructure concrete:				
-400	Concreting	58,704	Cubic yard	9.39	551,287.42
-401	Forms	439,753	Square foot	1.23	542,874.95
-402	Reinforcing steel	4,576,669	Pound	.046	209,104.74
	Total substructure concrete.	58,704	Cubic yard	22.20	1,303,267.11
-42	Gantry deck parapet and governor gallery temporary wall.				4,044.05
-5	Joints, stops, waterproofing and drains:				
-52	Metal water stops				12,657.33
-53	Asphalt water stops (joint filler)				5,321.45
-55	Waterproofing				8,981.13
-58	Drains (formed or porous tile)				10,813.68
-65	Marble and terrazzo, etc.:				
-651	Quarry tile	1,860	Square foot	.94	1,748.85
	Total account No. 213				2,038,258.27

TABLE 32c.—*Final project cost—Details*—Continued

STRUCTURES AND IMPROVEMENTS—Continued

Account	Description	Quantity	Unit	Rate	Amount
214	Control building:				
-4	Concrete:				
-41	Superstructure concrete:				
-410	Concreting	3,490	Cubic yard	$21.46	$84,565.62
-411	Forms	164,506	Square foot	.66	107,923.51
-412	Reinforcing steel	601,320	Pound	.053	32,166.73
	Total superstructure concrete.	3,940	Cubic yard	57.02	224,655.86
-42	Lightweight concrete	138	Cubic yard	19.58	2,701.86
-6	Superstructure:				
-61	Interior masonry:				
-610	Tile partitions and furring	9,875	Cubic foot	.92	9,053.65
-616	Soundproofing or insulation				1,250.90
-62	Steel and iron:				
-620	Structural steel	1,202,714	Pound	.054	65,066.63
-621	Miscellaneous steel and iron:				
-6210	Grates, plates and miscellaneous structural steel.	262,136	Pound	.15	38,452.75
-6211	Handrails	36,605	Pound	.33	12,030.27
-624	Aluminum work				7,827.92
-625	Wire and steel partitions				5,297.79
-632	Kalamein and hollow metal work:				
-6322	Kalamein or hollow metal doors.	4,221	Square foot	13.20	55,701.06
-6324	Window frames and sash	5,022	Square foot	4.60	23,118.38
-64	Cement work:				
-640	Cement floor finish	83,474	Square foot	.35	29,395.63
-65	Marble and terrazzo, etc.:				
-652	Terrazzo floors	5,723	Square foot	.71	4,066.33
-653	Composition floors				2,566.65
-654	Burlap wall finish and photomurals.				837.47
-655	Structural glass	3,218	Square foot	2.62	8,418.20
-656	Precast terrazzo trim	922	Square foot	3.38	3,112.24
-66	Roofing and sheet-metal work:				
-660	Roofing (cork board insulation and built-up roofing).	6,200	Square foot	.45	2,820.38
-663	Sheet-metal work				2,889.87
-67	Plastering	3,651	Square yard	5.37	19,618.82
-68	Painting and glazing:				
-680	Painting (battery room)				413.08
-681	Glass and glazing	4,856	Square foot	.79	3,835.37
-8	Service work:				
-80	Plumbing:				
-800	Water piping				9,821.30
-801	Sewer piping				5,123.16
-802	Fixtures				4,281.81
-803	Water cooling and heating units.				2,118.32
-81	Powerhouse floor, roof and foundation drains.				27,020.64
-82	Heating:				
-820	Control cabinets				447.49
-821	Conduit and fittings	9,692	Pound	.27	2,641.91
-822	Wiring				1,145.13
-825	Fixtures (power plant and generator heaters).				6,790.72
-83	Air conditioning:				
-830	Ducts				9,182.71
-831	Conduit and cable				3,336.00
-835	Equipment and controls				20,611.94
-84	Ventilating system				11,382.06
-88	Passenger elevator				23,075.51
-89	Lighting:				
-890	Control panels				3,556.13
-891	Conduit work	45,932	Pound	.36	16,530.87
-892	Wiring				4,534.09
-895	Fixtures, switches, and receptacles.				14,569.01
	Total account No. 214				689,299.91

TABLE 32c.—*Final project cost—Details*—Continued

STRUCTURES AND IMPROVEMENTS—Continued

Account	Description	Quantity	Unit	Rate	Amount
215	Station filter plant:				
-2	Excavation and backfill:				
-23	Rock excavation				$1,897.94
-4	Concrete:				
-40	Filter plant concrete:				
-400	Concreting	214	Cubic yard	$19.00	4,065.80
-401	Forms	7,046	Square foot	.71	5,003.72
-402	Reinforcing steel	33,500	Pound	.049	1,655.62
	Total filter plant concrete	214	Cubic yard	50.12	10,725.14
-6	Superstructure:				
-61	Interior masonry:				
-610	Masonry walls	976	Cubic foot	.87	845.37
-62	Structural steel, miscellaneous iron and sheet metal.				2,316.70
-63	Doors, windows, glazing				801.33
-67	Plastering	147	Square yard	3.14	461.52
-8	Service work (ventilation, lighting, plumbing).				355.45
-9	Mechanical and electrical:				
-90	Conduit, wiring, control cubicles.				5,785.47
-91	Pumps	2	Pump	363.38	726.75
-92	Piping				7,053.93
-93	Gauges and controls				2,622.21
-94	Chemical equipment				3,862.88
-95	Air compressor	1	Compressor		297.60
-96	Tanks, 12,500 gallons each	2	Tank	1,341.00	2,682.00
	Total account No. 215				40,434.29
216	Miscellaneous buildings:				
-0	Observation building	7,315	Cubic foot	.61	4,466.99
-1	Warehouse	64,496	Cubic foot	.070	4,507.50
-2	Construction machine shop	187,575	Cubic foot	.068	12,821.35
-3	Brick storage building				858.64
	Total account No. 216				22,654.48
217	Air, water, and automotive terminals:				
-0	Airport				4,415.54
-1	Automobile parking area				7,502.96
	Total account No. 217				11,918.50
	Total structures and improvements.				2,873,341.77

RESERVOIRS, DAMS, AND WATERWAYS

Account	Description	Quantity	Unit	Rate	Amount
220	Reservoir:				
-0	Clearing	31,228	Acre	$93.86	$2,931,127.21
	Total account No. 220				2,931,127.21
221	Concrete dam and spillway:				
-0	Exploration of foundation				71,653.94
-1	Diversion and care of water:				
-11	Cofferdam No. 1				21,463.55
-12	Cofferdam No. 2				128,797.67
-13	Cofferdam No. 3				149,880.03
-14	Cofferdam No. 4				135,174.28
-15	Cofferdam No. 5				129,446.82
-16	Maintain lake level during construction.				1,583.12
-17	Control and disposal of miscellaneous flotage.				14,873.45
-19	Final closure				18,672.34
	Total diversion and care of water.				599,891.26

Table 32c.—***Final project cost—Details***—Continued

RESERVOIRS, DAMS, AND WATERWAYS—Continued

Account	Description	Quantity	Unit	Rate	Amount
221	Concrete dam and spillway—Continued.				
-2	Excavation and backfill:				
-23	Rock excavation	288,836	Cubic yard	$2.67	$772,547.13
-3	Foundation preparation and treatment:				
-30	Drilling grout holes (2½ inches) Wagon drill holes.	57,910	Linear foot	.55	31,704.71
-37	Pressure grouting	45,517	Cubic foot	2.02	92,111.25
-4	Concrete:				
-40	South abutment:				
-400	Concreting	4,117	Cubic yard	9.16	37,721.65
-401	Forms	6,424	Square foot	.88	5,651.69
	Total south abutment concrete.	4,117	Cubic yard	10.54	43,373.34
-41	North abutment (U. S. E. D.)				213,775.80
-42	Nonoverflow mass:				
-420	Concreting	179,278	Cubic yard	6.39	1,145,273.87
-421	Forms	575,356	Square foot	.55	315,703.34
-422	Reinforcing steel	157,580	Pound	.068	10,662.96
	Total nonoverflow mass concrete.	179,278	Cubic yard	8.21	1,471,640.17
-43	Overflow mass:				
-430	Concreting	274,265	Cubic yard	6.56	1,797,911.22
-431	Forms	821,442	Square foot	.69	565,929.24
-432	Reinforcing steel	375,480	Pound	.060	22,707.36
	Total overflow mass concrete.	274,265	Cubic yard	8.70	2,386,547.82
-44	Overflow piers:				
-440	Concreting	6,828	Cubic yard	10.25	70,020.36
-441	Forms	123,627	Square foot	1.31	162,370.83
-442	Reinforcing steel	561,540	Pound	.066	37,031.86
	Total overflow piers concrete.	6,828	Cubic yard	39.46	269,423.05
-45	Overflow training walls:				
-450	Concreting	3,928	Cubic yard	6.47	25,430.60
-451	Forms	33,772	Square foot	.60	20,336.40
-452	Reinforcing steel	434,660	Pound	.034	14,850.93
	Total overflow training walls concrete.	3,928	Cubic yard	15.43	60,617.93
-46	Energy dissipators:				
-460	Concreting	7,130	Cubic yard	6.72	47,895.22
-461	Forms	44,809	Square foot	.55	24,547.91
-462	Reinforcing steel	122,220	Pound	.035	4,239.86
	Total energy dissipators concrete.	7,130	Cubic yard	10.75	76,682.99
-47	Trashways:				
-470	Concreting	16,292	Cubic yard	6.52	106,145.67
-471	Forms	60,642	Square foot	.54	32,617.45
-472	Reinforcing steel	74,320	Pound	.072	5,318.85
	Total trashways concrete	16,292	Cubic yard	8.84	144,081.97
-48	Intake structure:				
-480	Concreting	57,127	Cubic yard	7.86	448,974.77
-481	Forms	418,066	Square foot	1.25	521,878.52
-482	Reinforcing steel	8,360,300	Pound	.039	324,776.54
	Total intake structure concrete.	57,127	Cubic yard	22.68	1,295,629.83

Table 32c.—*Final project cost—Details*—Continued

RESERVOIRS, DAMS, AND WATERWAYS—Continued

Account	Description	Quantity	Unit	Rate	Amount
221	Concrete dam and spillway—Continued.				
-5	Joints, stops, waterproofing and drains:				
-52	Metal water stops				$55, 650. 32
-53	Asphalt water stops				25, 648. 20
-58	Drains (formed or porous tile):				
-580	Concrete pressure relief piping.	9, 981	Linear foot	$1. 56	15, 617. 04
-581	Metal drain pipe				13, 472. 47
-6	Gates and appurtenances:				
-61	Crest gates:				
-611	Gates, frames, guides, and operating equipment.	4, 451, 571	Pound	. 12	512, 808. 84
-614	Floating bulkhead gate	98, 000	Pound	. 15	15, 045. 87
-62	Intake gates:				
-621	Gates, frames and guides	4, 174, 800	Pound	. 14	576, 582. 95
-623	Screens, supports and trashracks.	904, 718	Pound	. 095	85, 770. 28
-624	Trash handling and removing equipment:				
-6240	Traveling trash rake and rails.	70, 984	Pound	. 18	12, 712. 56
-6241	Sluice piping				2, 494. 99
-625	Gate handling crane, 85-ton traveling gantry.	1	Crane		64, 469. 39
-626	Gate handling crane rails	111, 283	Pound	. 12	13, 034. 01
-63	Draft tube gates:				
-631	Gates, frames and guides	408, 200	Pound	. 19	75, 537. 07
-635	Gate handling crane, 20 ton traveling gantry.	1	Crane		16, 301. 89
-636	Gate handling crane rails	68, 235	Pound	. 11	7, 462. 36
-64	Trashway gates, guides, and operating equipment.	137, 910	Pound	. 11	15, 326. 18
-7	Auxiliary structures or equipment:				
-74	Miscellaneous iron				2, 546. 44
-77	Pressure or other measuring devices.				11, 120. 17
-79	Utility building				8, 446. 37
-9	Electrical work:				
-92	Switchboards:				
-922	Power distribution and motor-control cabinets.				4, 329. 36
-93	Protective equipment:				
-931	Cut-outs and disconnecting switches.				764. 88
-932	Equipment grounds	5, 847	Pound	. 66	3, 874. 49
-94	Electrical structures:				
-941	Metal enclosures				2, 351. 27
-95	Conduit work:				
-950	Concealed steel	5, 563	Pound	. 27	1, 477. 08
-951	Exposed steel	42, 099	Pound	. 24	10, 245. 22
-953	Nonmetallic	48, 997	Pound	. 38	18, 473. 32
-96	Power wiring:				
-960	Braided cable				12, 787. 18
-967	Plug receptacles	30	Receptacle	13. 26	397. 91
-97	Control wiring:				
-970	Braided cable				819. 87
-98	Transformers:				
-980	Lighting	42. 5	Kilovolt-ampere	14. 62	621. 18
-981	Power	405	Kilovolt-ampere	7. 47	3, 024. 02
-99	Lighting:				
-990	Cabinets				1, 134. 61
-991	Conduit work	11, 911	Pound	. 36	4, 292. 86
-992	Wiring				1, 816. 75
-993	Fixtures	317	Outlet	2. 56	812. 88
	Total account No. 221				9, 126, 951. 47
226	Water conductors:				
	Tailrace:				
-80	Concrete slab and retaining wall:				
-800	Concreting	5, 188	Cubic yard	9. 27	48, 098. 50
-801	Forms	11, 887	Square foot	. 70	8, 287. 72
-802	Reinforcing steel	19, 195	Pound	. 042	798. 76
	Total tailrace concrete	5, 188	Cubic yard	11. 02	57, 184. 98
-8					
-83	Riprap	6, 852	Cubic yard	. 79	5, 398. 93
	Total account No. 226				62, 583. 91

TABLE 32c.—*Final project cost—Details*—Continued

RESERVOIRS, DAMS, AND WATERWAYS—Continued

Account	Description	Quantity	Unit	Rate	Amount
228	Channel improvements and navigation facilities:				
-0	Channel and sailing line improvements:				
-00	Special clearing and grubbing	872	Acre	$48.21	$42,043.24
	Total account No. 228				42,043.24
	Total reservoirs, dams, and waterways				12,162,705.83

WATER WHEELS, TURBINES, AND GENERATORS

Account	Description	Quantity	Unit	Rate	Amount
230	Foundations and miscellaneous steel and iron:				
-3	Draft tube pier nose castings	237,647	Pound	$0.073	$17,303.42
	Total account No. 230				17,303.42
231	Turbines, including scroll case, speed ring, and draft tube liners (propeller type, 45,000 horsepower).	2	Turbine	380,459.64	760,919.27
	Total account No. 231				760,919.27
232	Auxiliary equipment for turbines:				
-0	Governors	2	Governor	22,535.15	45,070.29
-2	Scroll case filling and drainage system:				
-20	Drainage system:				
-200	Pumps	7	Pump	2,538.77	17,771.38
-201	Piping				23,250.57
-21	Filling system				11,883.99
-9	Auxiliary equipment:				
-92	Intake vent piping				13,299.59
	Total account No. 232				111,275.82
235	Generators (2 of 36,000 kilovolt-amperes)	64,800	Kilowatt	14.15	917,241.24
	Total account No. 235				917,241.24
236	Auxiliary equipment for generators:				
-0	Unit exciter and direct-current distribution board.	2	Assembly	2,372.51	4,745.02
	Total account No. 236				4,745.02
239	Miscellaneous equipment for turbines and generators:				
-0	Lubrication and governor oil system:				
-00	Oil pump	1	Pump		270.92
-01	Oil piping system				3,708.26
-02	Oil storage tanks:				
	2 storage, of 5,000 gallons; 1 pressure, of 220 gallons				1,910.07
-03	Oil in storage	2,260	Gallon	.47	1,059.04
-3	Hydraulic gauge board	2	Board	2,701.02	5,402.04
-5	Bearing cooling system				13,890.77
	Total account No. 239				26,241.10
	Total water wheels, turbines, and generators.				1,837,725.87

TABLE 32c.—*Final project cost—Details*—Continued

ACCESSORY ELECTRIC EQUIPMENT

Account	Description	Quantity	Unit	Rate	Amount
241	Switchgear:				
-0	Assembled switchgear:				
-00	15-kilovolt metal-clad switchgear				$68,107.00
-01	Generator neutral oil circuit breakers.	2	Oil circuit breaker.	$2,050.66	4,101.32
-02	Potential housing	1	Housing		4,683.03
-3	Instrument transformers:				
	10 13,800-volt potential and 15 current, range 800–2,000 amperes.	25	Transformer	147.07	3,676.76
	Total account No. 241				80,568.11
242	Switchboards:				
-0	Control boards:				
-00	Main control board	1	Board		40,397.94
-01	Vertical auxiliary board	1	Board		32,524.16
-2	2,300-volt alternating-current boards	1	Board		38,419.59
-3	440-volt alternating-current boards:				
-30	Power distribution cubicles	8	Cubicle	2,309.25	18,474.03
-31	Switchboard "D"	1	Board		12,531.87
-32	Switchboard "E"	1	Board		11,275.85
-4	220-volt alternating-current board "L."	1	Board		4,356.37
-5	Battery board	1	Board		4,347.77
-8	Standard frequency source and load and frequency control.				15,570.97
-9	Annunciator system:				
-90	Annunciators				10,646.26
-91	Braided conductor for annunciation.				1,918.18
	Total account No. 242				190,462.99
243	Protective equipment:				
-2	Grounding system	27,290	Pound	.99	26,904.74
-4	Generator neutral reactor	1	Reactor		1,465.75
	Total account No. 243				28,370.49
244	Electrical structures:				
-1	Metal enclosures and barriers	15,558	Pound	.37	5,769.04
-4	Cable trays and supports				6,548.76
	Total account No. 244				12,317.80
245	Conduit work:				
-0	Concealed steel	112,654	Pound	.27	30,077.57
-1	Exposed steel	19,038	Pound	.45	8,514.26
-2	Nonferrous				1,001.02
-3	Nonmetallic	3,542	Pound	.34	1,202.09
-5	Boxes				3,453.78
	Total account No. 245				44,248.72
246	Power wiring:				
-0	Braided cable:				
-00	Auxiliary power				7,065.86
-01	High voltage				803.30
-1	Main power cable and potheads				3,419.41
-4	13-kilovolt bus and supports				3,999.85
	Total account No. 246				15,288.42
247	Control wiring:				
-0	Braided cable				9,876.31
	Total account No. 247				9,876.31
249	Station service equipment:				
-0	Transformers:				
-00	Auxiliary power	4,000	Kilovolt-ampere	3.37	13,475.44
-01	Lighting	300	Kilovolt-ampere	5.73	1,719.16
-1	Control battery and charging equipment.				8,826.04
-3	Transformer untanking crane, 75-ton overhead traveling.	1	Crane		17,259.43

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Table 32c.—*Final project cost—Details*—Continued

ACCESSORY ELECTRIC EQUIPMENT—Continued

Account	Description	Quantity	Unit	Rate	Amount
249	Station service equipment—Continued.				
-4	Transfer car and rails	79,013	Pound	$0.12	$9,680.57
-6	Transil oil storage and piping system:				
-60	Piping and tanks:				
-600	Piping				9,547.02
-601	Storage tanks; 2 of 12,000, 1 of 7,599 gallons.				2,910.01
-61	Oil treating equipment:				
-610	Oil purifier				9,419.63
-611	Pump	1	Pump		613.88
-9	Equipment:				
-91	Auxiliary gas-electric generator	1	Generator		8,670.66
	Total account No. 249				82,121.84
	Total accessory electric equipment.				463,254.68

MISCELLANEOUS POWER PLANT EQUIPMENT

Account	Description	Quantity	Unit	Rate	Amount
252	Station maintenance equipment:				
-0	Machine shop equipment				$9,320.40
	Total account No. 252				9,320.40
255	Cranes and hoisting equipment:				
-0	270-ton crane; traveling gantry, including crane rails.	1	Crane		119,726.74
-1	Service bay cranes, 4-ton overhead traveling.	2	Crane	$1,961.27	3,922.53
-2	100-ton freight transfer derrick	1	Derrick		45,129.21
	Total account No. 255				168,778.48
256	Compressed air and vacuum cleaning system:				
-0	Compressed air system:				
-00	Compressors and air receivers; 2 of 330, 1 of 95, 1 of 8 cubic feet per minute.	763	Cubic feet per minute.	13.47	10,277.47
-01	Piping				6,411.57
	Total account No. 256				16,689.04
258	Station service water system:				
-0	Fire protection:				
-00	Piping				1,357.83
-01	Hose, racks, and cabinets				1,241.25
-1	Raw water system:				
-10	Pump	1	Pump		700.39
-11	Piping				6,365.35
	Total account No. 258				9,664.82
259	Other miscellaneous equipment:				
-0	Communication system (intrasite):				
-00	Braided conductor				511.03
-01	Equipment				14,556.11
-02	Leaded and armored conductor				3,942.84
-03	Conduit (dam section only)	21,722	Pound	.19	4,189.40
-1	Fire extinguishing equipment:				
-10	Portable fire extinguishers (44 assorted).				1,261.36
-11	Carbon dioxide system				21,427.94
-2	Control building—furniture and equipment.				7,570.35
-4	Oscillograph	1	Oscillograph		2,239.66
-5	Gauges and indicating devices				8,222.68
-6	Carrier current relay and intersite communication.				48,088.58
	Total account No. 259				112,009.95
	Total miscellaneous power plant equipment.				316,462.69

TABLE 32c.—*Final project cost—Details*—Continued

ROADS, RAILROADS, AND BRIDGES

Account	Description	Quantity	Unit	Rate	Amount
260	Access roads for permanent use:				
-1	South approach road:				
-10	Grading	56,103	Cubic yard	$0.63	$35,467.72
-12	Surfacing	3,875	Square yard	1.57	6,082.64
-13	Structures (concrete)	104	Cubic yard	49.64	5,162.72
-14	Guardrails				691.16
-2	North approach road	27	Cubic yard	60.99	1,646.71
	Total account No. 260				49,050.95
262	Roadway on concrete dam:				
-4	Concrete:				
-40	Bridge slab:				
-400	Concreting	5,456	Cubic yard	13.51	73,690.43
-401	Forms	198,842	Square foot	.49	97,944.22
-402	Reinforcing steel	1,126,680	Pound	.040	45,548.78
	Total bridge slab concrete	5,456	Cubic yard	39.81	217,183.43
-41	Nonoverflow piers:				
-410	Concreting	2,172	Cubic yard	16.32	35,441.80
-411	Forms	54,418	Square foot	1.46	79,552.51
-412	Reinforcing steel	415,740	Pound	.055	22,724.64
	Total nonoverflow piers concrete.	2,172	Cubic yard	63.41	137,718.95
-5	Bridge drains				6,945.95
-6	Structural steel:				
-60	Low beam bridge	3,232,980	Pound	.050	162,035.71
-61	Ramp viaduct	2,278,520	Pound	.045	102,456.73
-62	Truss bridge	334,220	Pound	.073	24,340.40
-63	Bridge railings	888,000	Pound	.076	67,647.28
-9	Lighting:				
-90	Conduit	18,982	Pound	.20	3,880.98
-91	Wiring				2,163.77
-92	Fixtures	50	Fixture	66.66	3,333.22
	Total account No. 262				727,706.42
	Total roads, railroads, and bridges.				776,757.37

DIRECT COST OF TRANSMISSION PLANT

STRUCTURES AND IMPROVEMENTS

Account	Description	Quantity	Unit	Rate	Amount
422	General yard improvements:				
-0	Grading and landscaping:				
-00	General grading	2,177	Square yard	$0.32	$701.27
-01	Surfacing (sodding and crushed stone).	2,048	Square yard	1.62	3,321.67
-2	Retaining walls, fences, gates, railings:				
-21	Fences, gates, and railings	696	Linear foot	2.95	2,052.44
	Total account No. 422				6,075.38
	Total structures and improvements.				6,075.38

STATION EQUIPMENT

Account	Description	Quantity	Unit	Rate	Amount
430	Outdoor substation structure:				
-1	Foundations for structures and equipment.	660	Cubic yard	$18.67	$12,324.90
-3	Superstructures (structural steel)	389,107	Pound	.091	35,404.50
-7	Cable and pipe tunnels:				
-70	Excavation and backfilling	3,515	Cubic yard	2.30	8,075.80
-74	Concrete	1,136	Cubic yard	38.11	43,296.63
-76	Cable trays and supports and miscellaneous iron.				6,311.55
-77	Inserts	1,536	Linear foot	.42	639.77
-8	Switchyard drainage and sprinkler system.				150.91

TABLE 32c.—*Final project cost—Details*—Continued

STATION EQUIPMENT—Continued

Account	Description	Quantity	Unit	Rate	Amount
430	Outdoor substation structure—Con.				
-9	Lighting:				
-90	Transformer	37.5	Kilovolt-ampere	$10.63	$398.67
-91	Cabinets				401.08
-92	Conduit	3,851	Pound	.39	1,512.94
-93	Wiring				616.14
-94	Fixtures	104	Outlet	10.46	1,087.39
	Total account No. 430				110,220.28
431	Switchgear:				
-1	Oil circuit breakers	6	O. C. B.	33,032.17	198,193.02
-2	Disconnecting switches: 12 manual, 5 motor operated 161-kilovolt; 1 manual 46-kilovolt.				35,576.10
	Total account No. 431				233,769.12
433	Protective equipment:				
-0	Arrestors, gaps				4,672.46
-1	Fuses, gaps				631.67
-2	Grounding system	13,674	Pound	.37	5,003.20
	Total account No. 433				10,307.33
435	Conduit work:				
-0	Concealed steel	16,733	Pound	.22	3,635.80
-1	Exposed steel	3,355	Pound	.29	965.50
-4	Concrete envelopes	50	Cubic yard	23.22	1,160.77
-5	Boxes				365.26
-6	Manholes and covers				509.12
	Total account No. 435				6,636.45
436	Power wiring:				
-1	Leaded cable				2,127.86
-3	Bare cable				2,676.66
-4	Bare busses, connections, and fittings.				5,344.42
-5	Insulators and bushings				13,498.61
	Total account No. 436				23,647.55
437	Control wiring:				
-1	Leaded cable				12,830.37
	Total account No. 437				12,830.37
438	Main conversion equipment:				
-0	Power transformers:				
-00	Main power (3 of 24,000 kilovolt-amperes, 13,800/154,000 volts).	72,000	Kilovolt-ampere.	2.21	159,312.99
-01	Auxiliary power	750	Kilovolt-ampere.	6.25	4,689.61
	Total account No. 438				164,002.60
439	Station service equipment:				
-6	Transil oil storage and piping system:				
-60	Piping				3,981.99
-62	Oil in storage	8,000	Gallon	.31	2,462.93
	Total account No. 439				6,444.92
	Total station equipment				567,858.62

DIRECT COST OF GENERAL PLANT

VILLAGE

	Land improvements:				
	Roads				$24,940.36
	Culverts				4,058.65
					28,999.01

TABLE 32c.—*Final project cost—Details*—Continued

VILLAGE—Continued

Account	Description	Quantity	Unit	Rate	Amount
	Dwellings:				
	Permanent brick dwellings (15 brick dwellings, 7 garages costing $2,112.83).	296, 854	Cubic foot	$0. 42	$125, 354. 02
	Semipermanent dwellings (14 frame dwellings, 3 shed-type group garages costing $1,094.34).	119, 839	Cubic foot	. 26	30, 685. 41
	Low-cost dwellings (remodeled)	28, 020	Cubic foot	. 28	7, 746. 83
	Dwelling (old construction personnel office).	10, 350	Cubic foot	. 24	2, 527. 66
					166, 313. 92
	Other buildings and structures:				
	Service building (includes special equipment in amount of $2,411.46).	33, 360	Cubic foot	. 55	18, 232. 10
	School building	26, 228	Cubic foot	. 16	4, 199. 69
	Clubhouse	7, 560	Cubic foot	. 24	1, 796. 98
					24, 228. 77
	Sewer system				27, 536. 53
	Water system:				
	Storage tank, piping and meters				21, 245. 92
	Fire protection				1, 830. 69
					23, 076. 61
	Electric distribution system:				
	Overhead				10, 668. 90
	Underground				18, 409. 23
					29, 078. 13
	Miscellaneous equipment				6, 276. 42
	Total village				305, 509. 39

OTHER

Account	Description	Quantity	Unit	Rate	Amount
	Communication System:				
	Wheeler Dam—Town Creek trunk line				$7, 519. 89
	Total general plant, other				7, 519. 89
	Total general plant				313, 029. 28

INDIRECT CONSTRUCTION COSTS

Superintendence, accounting, and timekeeping: Salaries and expenses of superintendent of construction and his immediate assistants in the field and office, also salaries and expenses of field accounting and timekeeping offices	$449, 216. 43
Transportation: Personal transportation and other expense incurred by field superintendents and others in maintaining contact with working units	100, 286. 23
Office supplies and expense: Miscellaneous expense of field office, such as stationery and office supplies, blue prints, photostats, telephone and telegraph, maintenance of office space, etc	185, 094. 42
Construction plant expense: Planning, design, and expenses of the construction plant and equipment, and studies of related operating costs	59, 737. 79
Camp operation: Net cost of operation of camp, including depreciation and training activities during construction period. (See p. 106)	542, 862. 99
Provision for medical service: Provision for costs of medical examinations and treatment for service-connected injuries or disabilities. (Allocations based on pay roll distribution)	220, 799. 87

TABLE 32c.—*Final project cost—Details*—Continued

Malaria prevention: Control of mosquitoes	$16,848.09
Police and guide service and accommodation of guests: Police and guide service maintained for benefit of project and accommodation of guests when on official visits	76,420.31
Other credits: Liquidated and miscellaneous damages, less bonus payments and other costs incurred in cancellation of contracts due to change in design or policy	[1] *18,485.30*
Public facilities: Maintenance of facilities for visitors, such as observation platform, drinking fountains, parking area for visitors' automobiles, etc.	3,427.05
Total camp and other indirect costs	[2] 1,616,207.88
DISTRIBUTIVE GENERAL EXPENSE	
DESIGN AND CONSTRUCTION ENGINEERING COSTS	
Engineering administration: Provision for salary and expenses of chief engineer's general administrative office. (Allocation based on pay roll distribution.)	78,019.85
Engineering—field and office: Salaries and expenses of executive and supervisory engineers, cost engineers and assistants, concrete technicians, inspectors and assistants, shop tests, and inspection of miscellaneous materials not assigned to specific items of property, also engineering for control lines, bench marks, etc.	892,936.17
Dam site and regional geology: Geologic examinations and studies made of the dam foundation and reservoir rim	11,652.62
Design: Design of dam, powerhouse, switchyard, and other related structures, including $273,520.96 for U. S. Bureau of Reclamation charges, architecture of dam and powerhouse, and landscape treatment of dam site	459,366.95
Consulting service: Salary and travel expenses of consultants	2,622.40
Architectural engineering and design: Architectural engineering and design charges by the Land Planning and Housing Division in connection with the employees' village	8,304.16
Total design and construction engineering costs	[2] 1,452,902.15
EXECUTIVE AND ADMINISTRATIVE COSTS	
General administrative expense: Provision for costs of general offices including board of directors, general manager's office, finance department, legal department (exclusive of land condemnation costs), materials department, personnel department, and office service department. (Allocation based on pay roll distribution.)	1,295,325.81
Division administrative costs: Provision for costs of general office expense of the various divisions, such as construction and maintenance, reservoir clearing, highway and railroad, cemetery relocations, etc. (Allocation based on pay roll distribution.)	71,455.72
Total executive and administrative costs	[2] 1,366,781.53
OTHER GENERAL COSTS	
Maps and surveys: Establishment of basic horizontal and vertical positions around the reservoir, surveys and mapping of the topography at the dam site, aerial photography and mosaic construction, surveys to provide the basic data for studies of silt deposits in the reservoir, and other general surveys and mapping	92,037.58

[1] Denotes credit.
[2] Expanded by activities. (See p. 301.)

TABLE 32c.—*Final project cost—Details*—Continued

OTHER GENERAL COSTS—Continued

Hydraulic data: Pro rata share of the cost of collection and compilation of basic hydrographic and hydrologic data, such as river forecasting, rainfall and evaporation studies, measurements of the amount of silt carried by various streams and deposited in reservoirs, stream flow measurements, ground water investigations, laboratory tests of hydraulic structures, etc.	$100,342.16
Project planning: Pro rata share of the cost of investigations and studies of projects proposed or under construction, such as studies made to determine the feasibility of the project, preliminary plans and estimates, studies of stream flow and flood data, determination of most economical power installations, navigation studies, etc.	25,493.71
Regional planning studies: Miscellaneous preliminary investigations and plans for the use of land in Wheeler Reservoir area	12,590.76
Final project report: Cost of assembling, editing, and printing a report on the planning, design, construction, and initial operation of the project	9,320.90
Property records: Analysis of actual costs of each inventoriable item, the recording of same in permanent property records, and the preparation of the final report on the cost of the project	27,815.78
Total other general costs	267,600.89

TABLE 33.—*Details of certain indirect construction costs and distributive general expenses by activities*

	Relocations and protecting structures[1]	Reservoir	Dam and powerhouse construction	Village	Total
INDIRECT CONSTRUCTION COSTS					
Camp and other indirect costs:					
Superintendence	$15,911.57	$102,805.56	$83,327.78	$13,743.15	$449,216.43
Accounting and timekeeping	43,279.77		190,148.60		
Transportation	11,031.70	37,681.06	51,573.47		100,286.23
Office supplies and expense	11,842.02	19,939.20	153,313.20		185,094.42
Construction plant expense			39,737.79		39,737.79
Camp operation			[2] 542,862.99		542,862.99
Provision for medical service	14,463.21	19,626.18	183,262.55	3,447.93	220,799.87
Malaria prevention			16,848.09		16,848.09
Police and guide service, accommodation of guests and safety activities			76,420.31		76,420.31
Public liability	842.33				[3] *18,485.30*
Other credits less bonus payments			[3] *19,327.63*		
Public facilities			3,427.05		3,427.05
Total	97,370.60	180,052.00	1,321,594.20	17,191.08	1,616,207.88
DISTRIBUTIVE GENERAL EXPENSE					
Design and construction engineering costs:					
Engineering administration	8,971.29		65,794.88	3,253.68	78,019.85
Engineering—Field and office	252,213.83	104,827.48	535,894.86		892,936.17
Dam site and regional geology			11,652.62		11,652.62
Design by Authority			185,845.99		459,366.95
Design by Bureau of Reclamation			273,520.96		
Consulting services			2,622.40		2,622.40
Architectural engineering and design				8,304.16	8,304.16
Total	261,185.12	104,827.48	1,075,331.71	11,557.84	1,452,902.15
Executive and administrative costs:					
General administrative costs	63,028.79	307,053.91	905,672.48	19,570.63	1,295,325.81
Division administrative costs	18,297.80	52,915.11		242.81	71,455.72
Total	81,326.59	359,969.02	905,672.48	19,813.44	1,366,781.53

[1] Relocating highways, railroads, and other structures and improvements; protecting existing structures and improvements.
[2] See p. 106 for details of camp operation.
[3] Denotes credit.

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*TVA staff member.

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Aikin, H. B., general.
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Draper, E. S., social and economic studies, employee housing, archaeology.
Eckel, E. C., geology.
Fitts, W. C., Jr., general.
Fowler, H. H., general.
Frierson, R. E., land purchase control.
Fruend, H. L., Decatur industries adjustments.
Fry, A. S., ground water, silt investigations, and model studies.
Garity, C. H., materials procurement.
Glenn, William, photographs.
Gorthy, W. C., planning and construction.
Hammacher, J. W., illustrations.
Harton, T. G., initial operations.
Henle, Maurice, general.
Holden, W. R., reservoir clearance.
Hopkins, R. A., electrical engineering design.
Hume, Ritchey, general.
Hunt, Granville, photographs.
Jessup, G. P., general.
Johnson, W. R., aggregate and concrete.
Jones, Barton M., design.
Junior, F. E., highway and railroad relocation.
Kelly, H. J., relocation of cemeteries, electric power, telephone and telegraph lines.
Kimball, J. H., preliminary studies and designs; Decatur water supply and sewage system adjustments.
Kohler, E. L., costs.
Krutch, Charles, photograhs.
Moreland, J. E., bridge and highway relocation.
Nagel, R. H., general.
Osborn, K. D., employee housing.
Pollock, W. J., costs.
Poole, L. H., illustrations.
Rich, G. R., mechanical engineering design.

Riegel, R. M., civil engineering design.
Sayford, N. H., reservoir activities.
Schweier, Arthur, general.
Slover, George, personnel.
Snyder, J. I., land acquisition.
Steffey, Ina M., general editorial.
Stromquist, W. F., malaria control.
Sullivan, J. B., railroad relocation.
Sunda, A. G., appendixes.
Turner, C. M., reservoir activities.
Tuttle, R. E., illustrations.
Vasey, H. B., historical and project design.
Webster, F. W., access, and highway and railroad relocation.
Werner, P. E., design.
Wessenauer, G. O., plant operations.
Whitmore, G. D., surveys and mapping.
Wilson, M. A., reservoir activities.
Wood, Dana M., power studies.
Woodward, S. M., general.

APPENDIX A

STATISTICAL SUMMARY

Authorized by Tennessee Valley Authority Act, May 18, 1933; Tennessee Valley Authority Board of Directors, Sept. 21, 1933.

Work started:	
On lock	January 1933
On dam	Nov. 21, 1933
Placed in operation:	
Navigation lock	Dec. 1, 1936
Unit No. 1	Nov. 9, 1936
Unit No. 2	Apr. 14, 1937
Unit No. 3 (Scheduled for completion)	1941
Unit No. 4 (Scheduled for completion)	1941
Project dedicated	Sept. 10, 1937

Location

At Tennessee River mile 274.9; in Lauderdale and Lawrence Counties, Ala.; 15.5 miles above Wilson Dam; 29.5 miles below Decatur, Ala.; 95 miles north of Birmingham; 18.5 miles from Southern Ry. at Sheffield, Ala.; 8.5 miles from Southern Ry. at Town Creek, Ala.

Stream Flow

Drainage area at dam	29,590 square miles
Gaging station records: Florence, Ala., Nov. 7, 1871, to date.	
Drainage area	30,810 square miles
Maximum flow, approximate (at Florence, Mar. 19, 1897)	470,000 cubic feet per second
Average flow estimated at dam site (1894–1936)	50,000 cubic feet per second
Minimum natural flow, estimated (at Florence, 1925)	4,070 cubic feet per second

Reservoir and Water Elevations

Counties affected:

State of Alabama — Lauderdale, Lawrence, Limestone, Madison, Marshall, and Morgan

Operating levels at dam:	
Maximum design flood (area 76,800 acres)	Elevation 558.3
Top of gates (area 68,300 acres)	Elevation 556.3
Malaria control:	
Upper (area 62,900 acres)	Elevation 555.0
Lower (area 50,600 acres)	Elevation 552.0
Minimum:	
For low water releases (area 43,000 acres)	Elevation 550.0
In advance of floods (area 38,700 acres)	Elevation 548.0
Length (backwater to Guntersville Dam)	74.1 original river miles
Average width at elevation 556.3	1.42 miles
Clearing line (area cleared 31,228 acres)	Elevation 556.3
Length of shore line at elevation 556.3 (to Guntersville Dam)	1,063 miles
Main shore	899 miles
Islands	164 miles
Original river area	17,600 acres
Storage (flat pool assumption):	
Uncontrolled storage (elevation 556.3–558.3)	145,000 acre-feet
Total volume at elevation 556.3	1,150,000 acre-feet
Controlled flood storage (elevation 556.3–548.0)	429,000 acre-feet
Volume for low water releases (elevation 555–550)	263,000 acre-feet

TAIL WATER

Maximum expected level (650,000 cubic feet per second)	Elevation 510.8
Maximum recorded level (approximate) (1897)	Elevation 510.0
Average (at 50 percent of average annual flow)	Elevation 505.2
Minimum level (minimum Wilson flat pool)	Elevation 503.0

HEAD (GROSS)

Maximum expected (aproximate) (elevation 556.3–503.0)	53 feet
Average (approximate) (elevation 553.5–505.2)	48 feet
Minimum (approximate) (elevation 548.0–508.0)	40 feet

NAVIGATION FACILITIES

Length of channel for 9-foot navigation (to Guntersville Dam)	69 sailing miles
Minimum flat pool level to maintain depth for 9-foot navigation	Elevation 550
Length of dredged navigable channel:	
Below Wheeler lock	0.5 mile
Upper end of Wheeler pool	3.0 miles

NAVIGATION LOCK

Location	At right (north) end of dam
Lock chamber, clear	60 by 360 feet
Lift:	
Maximum (elevation 503–556)	53 feet
Normal (elevation 506–555)	49 feet
Guard sills:	
Upper	Elevation 534.3
Lower	Elevation 491.3
Depth over upper guard sill	Minimum 13.7 feet
Depth over lower guard sill	Minimum 11.7 feet
Lockage time (checking to regaining speed) estimated	40 minutes
Concrete in total structure	121,100 cubic yards
Lock walls:	
Width:	
Base	53 feet
Top	8 feet
Height (from elevation 488.3 to elevation 560.3	72 feet
Guide walls:	
Upper:	
Length	264 feet
Width:	
Base	35 feet
Top	8 feet
Height (from elevation 494.3 to elevation 560.3)	66 feet
Lower:	
Length	299 feet
Width:	
Base	14 feet
Top	5 feet
Height (from elevation 488.3 to elevation 512.3)	24 feet
Guard walls, lower:	
Length	144 feet
Width:	
Base	14 feet
Top	5 feet
Height (from elevation 488.3 to elevation 512.3)	24 feet
Gate bay walls:	
Width:	
Base	53 feet
Top	23.5 feet
Height (from elevation 488.3 to elevation 560.3)	72 feet
Provision for future lock	Space provided for 110- by 600-foot lock

NAVIGATION LOCK—continued

Design assumptions:

Upper pool .... elevation 561.3
Lower pool .... elevation 505.3
Earth fill behind upper guide wall assumed saturated to .... elevation 561.3
Earth fill behind all other land walls assumed saturated to elevation 505.3 with dry earth from elevation 505.3 to the top of the wall.
Full hydrostatic pressure, effective over 50 percent of the area of the base.
Bedrock .... elevation ±496
Keys negligible.
For the design of the upper and lower bay land walls, the lock chamber was assumed pumped out with the lock gate in mitered position, and the culvert filled with water.
For the design of the sills, the gates were considered as horizontally framed with no pressure from the gate on the sills. For the upper miter sill and emergency dam sill, the emergency dam was considered in place and the lock pumped out.
Units used in calculations:

Weight of water	62. 5 pounds per cubic feet
Weight of concrete	145 pounds per cubic feet
Weight of dry earth	100 pounds per cubic feet
Weight of saturated earth	125 pounds per cubic feet
Pressure of dry earth on a vertical plane	33. 33 pounds per cubic feet
Pressure of saturated earth on vertical plane	83. 33 pounds per cubic feet

LOCK VALVES

Type .... Segmental
Operation .... Electrical
Time required to open or close .... 60 seconds
Weight each valve: Swinging portion .... 24,000 pounds
Total, including embedded material .... 34,200 pounds
Ports:
Inlet .... 6—5 by 8 feet
Chamber .... 20—4 by 3 feet
Outlet .... 12—4 by 5 feet
Headwater .... elevation 555
Tailwater .... elevation 502
In position 1 (when the valve is raised ¼ of the total lift), it is assumed that the water level in the chamber has risen to elevation 502.89 in 15 seconds.
In position 2, it is assumed that the water level in the chamber has risen to elevation 504.5 in 30 seconds.
In position 3, it is assumed that the water level in the chamber has risen to elevation 507.1 in 45 seconds.
In position 4 (when the valve is raised the total lift), it is assumed that the water level in the chamber has risen to elevation 511.53 in one minute.
Coefficient of friction for lubricated joints 0. 27
Coefficient of friction for sealing strip 0. 435

LOCK EMERGENCY BULKHEADS

Upper pool .... elevation 555. 3
Lower pool (pumped out) .... elevation 491. 8
Unit weight of water (per cubic foot) .... 62. 5 pounds
Weight of gate .... 7,400 pounds
Weight of gate submerged .... 6,475 pounds
Size of culvert .... 8 by 10 feet
Size of gate .... 9. 5 by 11 feet

LOCK GATES

Type:
Upper gate .... Horizontal framed
Lower gate .... Arch
Operation .... Electrical

LOCK GATES—continued

Weight:
- Upper gate, swinging portion — 186,200 pounds
- Lower gate, swinging portion — 698,200 pounds

Size:
- Upper gate, each leaf — 35 feet ⅜ inch wide by 24 feet high
- Lower gate, each leaf — 37 feet wide by 65 feet 8 inches high

Time required to open or close — 60 seconds

Dam

Material and type — concrete gravity; nonoverflow sections and gate-controlled spillway section

Length:
- Dam, powerhouse, and lock (station 4+67 to 68+09) — 6,342 feet
- South abutment (station 2+97 to 4+67) — 170 feet
- Road and bridge (station 4+70 to 69+72) — 6,502 feet
- Spillway section (including 60 piers 5 feet thick) — 2,700 feet
- Trashways (2) over-all center to center of piers — 90 feet

Maximum height, foundation to top of piers — 72 feet

Maximum width at base:
- Spillway section only, elevation 496.3 — 61.2 feet
- Including integral apron — 124 feet

Roadway (elevation 568.28 rising to elevation 619.78 over lock) — 20 feet wide

Foundation — Fort Payne limestone

NONOVERFLOW SECTION

Top of parapet wall — elevation 560.3

Top of dam — elevation 557.3

Width of top of dam — 4.71 feet

Design assumptions:
- Alignment of vertical portions of upstream face of dam straight and continuous through spillway, trash gate, and nonoverflow sections.
- Weight of bridge per linear foot — 4,500 pounds
- Effect of sand and gravel deposits — None
- Constants:
  - Unit weight of water (per cubic foot) — 62.5 pounds
  - Unit weight of concrete (per cubic foot) — 150 pounds
  - Coefficient of friction of concrete on rock, or concrete on concrete — 0.65
  - Ultimate strength of concrete in shear — 400 pounds per square inch
- Stresses and loads:
  - Vertical stresses vary as straight lines from upstream to downstream face of dam at all elevations.
  - Stresses at faces of dam are acting in directions parallel to the slopes of the faces and are equal to the calculated vertical stresses divided by the squares of the cosines of the angles between the faces and the vertical directions.
  - Uplift pressures vary as a straight line from full reservoir pressure at the upstream face of the dam to zero pressure at the downstream face, or to tail water pressure at locations where the plane being analyzed is below the elevation of the tail water surface.
  - Uplift pressures according to the above curve act over two-thirds the horizontal area of the base and over two-thirds the horizontal areas of the concrete sections analyzed at elevations above the base, uplift pressures being assumed to act in the pores of the concrete as well as along the plane of contact between the concrete and the foundation rock.
  - Maximum earthquake assumed to have an acceleration equal to one-tenth of gravity, a period of vibration equal to one second, and a direction of vibration at right angles with the axis of the dam.
  - Maximum effect of earthquake load during the empty condition of the reservoir occurs when the base of the dam is in the upstream position; that is, when the inertia force of the dam caused by the earthquake is acting in an upstream direction.

NONOVERFLOW SECTION—continued

Design assumptions—Continued.
  Stresses and loads—Continued.
    Maximum effect of earthquake load during the full condition of the reservoir occurs when the base of the dam is in the downstream position; that is, when the inertia force of the dam and water caused by the earthquake is acting in a downstream direction.
    All loads assumed to be carried by gravity action.
    Resistance to failure by shear at any elevation assumed to be increased by the coefficient of friction times the summation of vertical forces acting at the elevation.
    Lowest foundation level assumed to be elevation 485 in spillway, abutment, and trash gate; and elevation 498 in power plant section.

SPILLWAY SECTION

Length (including 60 piers 5 feet thick) 2,700 feet
Permanent spillway crest elevation 541.3
Design assumptions:
  Radii of principal parts of ogee curve section 16 and 48.53 feet
  Point of tangency of ogee curve with downstream face elevation 524.3
  Thickness of spillway crest at elevation 524 28.43 feet
  Spillway control, 14-foot radial gates. (1 foot was later added to make the gates 15 feet high.)
  Top of radial gates in closed position elevation 556.3
  Weight of radial gates per linear foot 1,075 pounds
  Distance center of pin to center of gravity of radial gate in closed position 16.5 feet
  Distance from center of pin to axis of crest 16.25 feet
  Required thickness of concrete on upstream side of gate hoist chamber 5.5 feet
  Bottom of gate hoist chamber elevation 523.3
  Alignment of vertical portions of upstream face of dam, straight and continuous through spillway, trash gate and nonoverflow sections.
  Weight of bridge per linear foot 4,500 pounds
  Constants: See Nonoverflow Section.
  Stresses and loads: See Nonoverflow Section.
Crest gates 60—tainter, 40 feet wide by 15 feet high, weight 19.02 tons including arms
  Hoists 60—(operate singly or in sets of 5)
  Structural steel—class A, Federal Specifications QQ–S–711 "Structural Steel for Bridges."
  Cold-finished steel:
    Ultimate tensile strength 60,000 pounds per square inch
    Elastic limit 40,000 pounds per square inch
    Elongation in 2 inches 20 percent
    Reduction in area 45 percent
  Rubber seals:
    Ultimate tensile strength 3,000 pounds per square inch
    Ultimate elongation 600 percent
  Maximum capacity, water elevation 558.3 (all gates open) 687,000 cubic feet per second

TRASHWAYS

Permanent crest elevation 550.3
Radii of principal parts of ogee curve in trash gate section 9.0 and 20.13 feet
Point of tangency of ogee curve with downstream face elevation 542.69
Thickness of crest at elevation 542.41 15.55 feet
Top of trash gates in closed position elevation 556.3
Distance from center of gravity of trash gates to axis of crest (upstream) 0.23 ft.
Requirements and positions of gate hoist chamber and operating gallery, same as for spillway section.
Alignment of vertical portions of upstream face of dam straight and continuous through spillway, trash gate, and nonoverflow sections.

TRASHWAYS—continued

Weight of bridge per linear foot ---------- 4,500 pounds
Constants: See Nonoverflow Section.
Stresses and loads: See Nonoverflow Section.
Trashway gates ---- 2 fixed roller, 39 feet 4 inches by 6 feet 11½ inches, 7 tons
Net opening ---------- 37.5 by 6 feet
Hoists ---------- 5-horsepower electric
In addition to the two principal trashways there is a small trashway 6 feet wide by 6 feet 8 inches high between the lock and the north abutment.

Power Plant

INTAKES

Number ---------- 8 (3 bays for each of 8 units)
Dimensions at waterway ---------- 18 feet wide by 49.5 feet high
Gross area ---------- 891 square feet per intake
Design assumptions:
Weight of bridge per intake unit (76 feet) ---------- 336,000 pounds
Weight of three emergency gates ---------- 330,000 pounds
Maximum water surface upstream ---------- elevation 558.3
Tail water surface against service gate ---------- elevation 505.3
Unit stresses:

	Pounds per square inch
Concrete:	
Flexure:	
Extreme fibre stress in compression	700
Extreme fibre stress in compression adjacent to supports of continuous or fixed beams or of rigid frames	800
Axial compression—In short columns with lateral ties	450
Bearing—Load over entire area	500
Shear:	
Beams—No web reinforcement nor special anchorage of longitudinal steel	40
Beams — No web reinforcement — longitudinal bars specially anchored	60
Beams—Web reinforcement and longitudinal bar specially anchored	180
Bond—Deformed bars	100
Steel reinforcing bars—deformed, intermediate grade billet steel: Tension	16,000

Pier loadings—Various combinations of emergency and service gates closed in outer bays and emergency gates closed in interior bays.
For the section downstream from the gate slots, all the foundation load carried by a 7-foot strip because of the cushion under the interior piers.
Forces due to unbalanced gate loads are resisted by bending in the structure downstream from the gate slots, this resistance distributed over a horizontal plane as a uniform (or triangular) load.
The portal girder section, forming the roof of the intake, is very deep in proportion to its span, and as the intermediate piers and side walls are relatively thin, the walls and piers are considered fixed at the top.
One-half of the water load transmitted to the gate slot is carried by adjacent upstream and downstream 6-foot pier sections respectively.
Gates ---------- 7 active; weight, 58.75 tons; size, 20 feet 11⅛ inches by 38 feet 3¾ inches, 18 feet by 37.8 feet clear opening. Fixed roller lift type 17 inactive; weight, 45 tons; size, 20 feet 11⅛ inches by 38 feet 3¾ inches, 18 by 37.8 feet clear opening.
Loading: Unbalanced water pressure against the gate with flowage through the intake.
Head ---------- 60 feet
Structural steel—Class A, Federal Specifications QQ-S-711 "Structural Steel for Bridges."
Cold-finished steel:
Ultimate tensile strength ---------- 60,000 pounds per square inch
Elastic limit ---------- 40,000 pounds per square inch
Elongation in 2 inches ---------- 20 percent
Reduction in area ---------- 45 percent

INTAKES—continued

Cold-finished steel—Continued.

SAE 2330 steel "Nickel Alloy":

Ultimate tensile strength	100,000 pounds per square inch
Elastic limit	80,000 pounds per square inch
Elongation in 2 inches	20 percent
Reduction in area	60 percent

Cast steel—medium grade, Federal Specifications QQ-S-681.

Alloy cast steel:

Ultimate tensile strength	95,000 pounds per square inch
Yield point	70,000 pounds per square inch
Elongation in 2 inches	15 percent
Reduction in area	30 percent

Monel metal:

Ultimate tensile strength	60,000 pounds per square inch
Yield point	45,000 pounds per square inch
Elongation in 2 inches	35 percent

Rolled bronze:

Ultimate tensile strength	60,000 pounds per square inch
Yield point	30,000 pounds per square inch
Elongation in 2 inches	30 percent

High tensile bronze:

Ultimate tensile strength	90,000 pounds per square inch
Yield point	50,000 pounds per square inch
Elongation in 2 inches	15 percent
Reduction in area	15 percent

Emergency gates — 2—weight, 45 tons; size, 19 feet 11½ inches by 42 feet 11 inches; 18 by 42.4 feet clear opening

Trash racks (present installation) — 6 of 8 sections each

Area	972 square feet gross for each intake
Head	10 feet
Structural steel: Maximum allowable stress	18,000 pounds per square inch

Trash rake load:

Load	1,000 pounds
Speed	25 feet per minute
Factor of safety	5

Crane — 1—85-ton gantry with 5-ton monorail hoist

POWER STATION

Generating capacity:

Initial installation (2 units) — 72,000 kilovolt-amperes; Two additional scheduled for completion in 1941. 64,800 kilowatts; 90,000 horsepower

Ultimate installation provided for (8 units) — 288,000 kilovolt-amperes; 360,000 horsepower

Type of construction — Outdoor generators—concrete substructure

Principal outside dimensions:

Powerhouse and service bay:

Length	690 feet
Width (including intake)	181 feet
Height	125 feet

Separate control building:

Length	156 feet
Width	48 feet
Height (5 stories plus basement and penthouse)	99 feet

Design assumptions:

Discharge draft tubes — 10,200 cubic feet per second

Pier nosings: Maximum allowable stress=16,000−52 (L/r) or approximately 14,000 pounds per square inch.

Concrete unit stresses—See Spillway.

POWER STATION—continued

Design assumptions—Continued.	Pounds per square inch
Structural steel stresses:	
Tension, net section	18,000
Compression, in columns and struts, gross section	$18,000-70\frac{L}{r}$
With a maximum of	15,000
Bending, in extreme fibres of rolled shapes and built-up members:	
Tension flange, net section	18,000
Compression flange, gross section except as noted below	$21,000-275\frac{L}{b}$
Compression flange, gross section in beams and girders with top cover of channel section and in double web girders	$21,000-250\frac{L}{b}$
Maximum compressive stress under the above	18,000
Bending in extreme fibres of pins	27,000
Shear in webs of rolled shapes and built-up members, gross section:	
Where the thickness of the web exceeds 1/60 of the clear distance between the flanges	12,000
Where the thickness of the web does not exceed 1/60 of the clear distance between the flanges	$\frac{18,000}{1+\frac{h^2}{7,200t^2}}$
Shear, in pins and shop rivets	13,500
Shear, in turned bolts and field rivets	10,000
Bearing, on pins and shop rivets:	
Single shear	22,000
Double shear	27,000
Bearing on turned bolts and field rivets:	
Single shear	16,000
Double shear	20,000
Bearing on outstanding legs of stiffener angles and other parts in contact	27,000
Bearing on rollers per linear inch	$600d$

Symbols used in the foregoing table (all in inches):

L=Unsupported length of member.
r=Radius of gyration.
t=Thickness of web of girder.
b=Width of compression flange.
h=Clear distance between flanges or rivet lines of stiffeners, the smaller dimensions to govern.
d=Diameter of roller.

POWER STATION FLOOR LIVE LOADINGS

Control building[1]: Room	Elevation	Pounds per square foot
Penthouse roof	597.28	75
Roof (except penthouse floor)	588.78	75
Penthouse floor	588.78	300
All rooms and observation terrace	569.03	100
All rooms east of F line	552.28	100
Cable spreading room and lobby	540.28	200
Lobby (between F and D lines)[2]	540.28	100
Laboratory, etc., west of D line[2]	540.28	150
All rooms	528.28	150
Entire floor	513.28	[3] 500
Interior stairs		100
Exterior stairs		100

[1] See exhibit 36, appendix E.
[2] See exhibit 40, appendix E.
[3] Or concentrations caused by 20-ton truck.

POWER STATION FLOOR LIVE LOADINGS—continued

Service bay: Room	Elevation	Pounds per square foot
Deck	568.28	[4] 1,000
All rooms	552.28	100
Deck	540.28	1,000
Switchgear room	528.28	850
Balcony	525.28	100
Floor	513.28	[5] 500
Oil pump room, etc	498.78	200
Sump floor	495.78	200
Sump floor	479.28	200
Unit 1:		
Cable gallery roof	552.28	200
Deck	540.28	1,000
Floor slabs	525.28	500
Pipe Gallery	498.78	200
Unit 2:		
Observation terrace	552.28	200
Generator deck	540.28	1,000
Operating gallery	525.28	500
Northwest and southwest rooms	516.28	300
Pipe gallery	498.78	200

HYDRAULIC TURBINES

Number — 2 initially; 8 ultimately
Type — Fixed-blade propeller; right hand; vertical shaft with rubber-lined water-lubricated guide bearing
Rated capacity — 45,000 horsepower at 48-foot net head
Manufacturer's turbine rating at generator capacity — 44,800 horsepower at 44.5-foot head
Gross head — 44 to 53 feet
Normal head — 50 feet
Speed at 48-foot head — 85.7 revolutions per minute
Specific speed — 144 revolutions per minute
Run-away speed (by test) — 155 revolutions per minute at 54-foot head
Discharge at full gate and generator capacity (44-foot head) — 10,400 cubic feet per second
Efficiency (best guaranteed) — 88 percent at 43-foot head; 89.5 percent at 48-foot head
Shaft — Forged carbon steel, 33½ inches diameter with 8-inch center hole; 23 feet 9½ inches long
Diameter of runner — 264 inches
Diameter of stay ring (outside) — 34 feet 7 inches
Height of stay ring — 13 feet 4 inches
Spacing of units center to center — 76.0 feet
Distance from center line of distributor to floor of draft tube — 58 feet
Center line of distributor — Elevation 508.3
Bearing — Goodyear Cutless rubber type, water lubricated, 37½ inches diameter
Intake openings — 3—18 by 33.4 feet
Height of wicket gate — 9 feet 3⅞ inches
Leakage (Flow meter calibration test) — 37.5 cubic feet per second at 50-foot head
Casing — Concrete scroll case (4,200 cubic yards) with cast steel stay ring and distributor ring
Clearances:
- Runner clearance in throat ring — 3/16 inch
- Guide bearing clearance — 0.016 inch

[4] Or one 20-ton truck or plant crane on rails.
[5] Or concentrations caused by 20-ton truck.

HYDRAULIC TURBINES—continued

Weight of principal parts (approximate):

Item	Number of sections	Total weight, pounds
Head cover barrel	1	40,600
Inner head cover	2	118,000
Outer head cover	4	106,700
Guide vanes	24	156,000
Operating ring	4	24,000
Couplings and levers	24	21,700
Cylinder bays, heads, piston	2	14,100
Runner blades	6	114,000
Runner hub	1	54,000
Stay ring	6	224,000
Distributor ring	4	60,000
Throat ring	4	88,400
Main shaft	1	92,000
Draft tube liner	1	62,500
Lower pit liner	6	46,000

Shipping weight of principal parts — 1,480,000 pounds
Complete assembly, consisting of turbine shaft, runner and head cover to be lifted by powerhouse crane — 432,000 pounds
Turbine flywheel effect in pounds-feet-squared — 4,500,000
Turbine rotating parts — 267,000 pounds
Turbine rotating parts plus unbalanced hydraulic thrust of runner — 1,300,000 pounds
Tensile strength of materials (pounds per square inch):
- Castings not subject to stresses — 20,000 to 30,000
- Castings under stress — 38,000 to 42,000
- Steel not under stress — 63,000 to 68,000

DRAFT TUBES

Type — Elbow, dividing into 3 discharge passages at lower end; throat diameter 22 feet; cast steel intermediate pier noses in draft tube discharge passages.
Horizontal length (centerline of turbine to downstream face) — 85 feet
Net area at outlet opening — 1,320 square feet
Gates — 3 sliding type, 18- by 24.5-feet clear opening, 20 tons each
Crane — 1 20-ton gantry

GOVERNORS

Rated capacity (300 pounds pressure) — 600,000 foot-pounds
Operating pressure — 250 to 300 pounds in system; minimum time to close gate, 4 seconds.
Motor-driven flyballs from separate generator on main generator.
Speed droop adjustment — 0 to 7.8 percent

GENERATORS

Number installed — 2 initially
Rated capacity, each — 36,000 kilovolt-amperes at 0.90 power factor, 60 cycles, 3-phase, 13,800-volt, 85.7 revolutions per minute.
Type — Vertical shaft—outdoor with combination thrust and lower guide bearing below rotor and upper guide bearing between rotor and exciter.
Thrust bearing — General Electric typical springs type, 2,000,000 pounds capacity—unit load 432 pounds per square inch. Thrust and lower guide bearings, combined in same oil reservoir.
Rotor — Spider consists of laminated steel rim and radial arms, center bolted at top and bottom to circular steel plates which are bolted to a cast steel hub; 84-pole, outside diameter 29 feet 3 inches; WR^2, 61,260,000 pounds-feet-squared.
Stator — Star connected with 12 terminals; in 4 sections; outside diameter 35 feet.
Cooling — Air recirculated through 8 water-cooled radiators within steel housing. Cooling water, 1,080 gallons per minute, at 30° C.

GENERATORS—continued

Exciters---------- Main exciter 200 kilowatt; pilot 17 kilowatt; both 250 volt, directly driven.
Brakes------------- 8 units with mineral asbestos compound faced shoes; air pressure 90-100 pounds per square inch. 7½ minutes to stop from half speed.
Jacks------------------- Hydraulic, using brakes with oil at 1,200-pound pressure from portable hand pump. Capacity 840,000 pounds; lift at least ½ inch.
Efficiencies from field tests----------- Actual over-all at 0.90 power factor, including exciters and rheostats: 94.45 at 25 percent rated kilovolt-amperes; 96.70 at 50 percent; 97.32 at 75 percent; and 97.48 at 100 percent.
Short-circuit ratio (calculated from field test)---------------------------- 1.10
Maximum output-------------- 41,400 kilovolt-amperes, continuous, at maximum safe indicated temperature (120° C.) as determined by detectors when ambient water is at 30° C.
Condenser operation--------- Maximum capacity not less than 20,000 kilovolt-amperes; losses not more than 800 kilowatts.
Maximum line charging capacity 31,680 kilovolt-amperes.
Damper windings-- None
Voltage regulation----------------- General Electric high speed, noncontinuously vibrating type. Exciter response ratio not less than 1.0 with ceiling voltage of 380 volts.
Oil capacity----------------- For thrust and lower guide bearings 1,375 gallons approximately, and for upper guide bearing 50 gallons approximately.
Weight of principal parts:

Shaft	128,000 pounds
Lower bearing bracket (including sidearm)	120,000 pounds
Upper bearing bracket	75,800 pounds
Thrust bearing	20,000 pounds
Rotor, less shaft	380,000 pounds
Exciter	26,000 pounds
Pilot exciter	10,000 pounds
Stator, frame, and windings	305,800 pounds
Total weight, each unit	975,000 pounds

Outside diameter of housing-- 43 feet

TRANSFORMERS

One bank of 3 single-phase, 24,000 kilovolt-amperes, 13.2–89 kilovolts, oil-insulated, self cooled with 2 2½ percent taps above and 2 2½ percent taps below 89 kilovolts, connected 13.2-kilovolt delta, 154-kilovolt grounded wye.
Weights of each transformer---------- core and coils, 56,000 pounds; case and fittings, 43,000 pounds; oil, 38,000 pounds; total, 137,000 pounds.
Dimensions-------------------------------- floor area, 175 by 156 inches; height of case, 209 inches; height including terminals, 298 inches.

SWITCHYARD

STRUCTURES

154-kilovolt 2-bus, 6-bay, latticed galvanized steel, 207 feet long, 96 feet wide, and 105 feet high.

FOUNDATION MATERIAL

Weight:
Dry weight-- 110 pounds per cubic foot
Net saturated weight------------------------------ 65 pounds per cubic foot
Allowable soil bearing:
With ice, wind, and wire pulls--------------- 4,000 pounds per square foot
When safety factor 1.75 has been applied to column forces------------------------------ 7,000 pounds per square foot
Horizontal active pressure—Rankine's formula, $P=\frac{1}{2}\ C\ wh^2$:
For dry material $C_e w=$------------------------- 18 pounds per square foot
For material below ground water $C\ w=$ -------- 80 pounds per square foot

SWITCHYARD FOOTINGS—GENERAL

Modification of applied forces—The true values of the moments, shears, and direct forces due to wind, ice, and wire pulls, which are to be resisted by the footings, were each converted to failure values by application of: (1) the factor 1.75 for the compression design; and (2) the factor 1.40 for the uplift design. On an ultimate loading basis, therefore, the following design values were used:

Allowable soil bearing........................ 7,000 pounds per square inch

Bending stress in lb. per sq. in.................. f_c=1,200 and f_s=30,000

Other stresses—Values obtained by multiplying customary allowable stresses by the factor 1.75.

SWITCHYARD FOOTINGS AT BRACED BAY

Case for uplift design:

High tailwater.. elevation 512.3

Origin of resisting and overturning moments was taken at upper corner of mat in contact with the tunnel wall projection where hinge action takes place.

Effective earth volume used was not greater than the volume of earth engaged or effective over the area of the footing, including that within an angle of 20° from a vertical line at the outer corner of the footing.

Allowable upward reaction on each tunnel wall projection was figured as 35 percent of dead weight of a 32-foot-long tunnel unit (between contraction joints), plus the net weight of rock fill over the projecting bottom slab of tunnel at each side of footing mat.

Case for compression design:

Low tailwater .... elevation 505.3

Origin of resisting and overturning moments was taken at lower corner of mat at point of contact with tunnel.

Effective earth volume used was not greater than that directly over mat.

Footing is free of tunnel.

Required reinforcement—For cases not governed by calculated stresses, reinforcement used was not less than specified below:

Piers—12¾-inch-round main rods symmetrically placed and tied with 2 sets of ½-inch-round binders 12 inches on centers.

Mats—⅝-inch-round, 12 inches on centers, each way, top and bottom except where the mat overhang from face of piers is less than mat thickness, ½-inch round, 12 inches on centers, was used for spacing the main rods in other direction.

OIL CIRCUIT BREAKERS

Six 1,200-ampere, 2,500,000-kilovolt--amperes, interrupting capacity, Westinghouse Electric and Manufacturing Co. oil circuit breakers. De-ion grid, 161 kilovolt; 1 each on transformer bank, bus tie, and each of 4 outgoing feeders: Norris, Wilson, Columbia, and Guntersville. Oil capacity 6,640 gallons each.

Foundations:

Loads—

Dead load (tanks empty)................................ 55,300 pounds

Weight of oil, 3 tanks (each tank 8,900 pounds)...... 26,700 pounds

Impact.. 200 percent of dead load

Types of soil reactions asumed..Uniform distribution; parabolic distribution

Design case IIa: Dead load, plus weight of oil for center tank only, no impact. Soil reaction distribution—uniform for determination of bottom reinforcement, and parabolic for top reinforcement.

Design case IIb: Dead load, plus weight of oil for center tank only, no impact. Soil reaction distribution, uniform for bottom reinforcement.

Design case II b: Dead load, plus weight of oil for 2 end tanks only, no impact. Soil reaction distribution, parabolic for top reinforcement.

DISCONNECT SWITCHES

Twelve 161-kilovolt Delta Star manually operated disconnect switches; 5 161-kilovolt Delta Star motor-operated disconnect switches.

CABLE TUNNEL

Control-------- Concrete reinforced cable tunnel 741 feet long by 7 feet high and 7 feet wide extending from cable spreading room in control building to far bay of switching structure. All control cable supported on cable trays.

Contraction joints. See chapter 3.

Seals—Annealed copper strips, 20 gage, were used in all concrete joints. A bent seal, Type M, was provided at all contraction joints. For all doweled construction joints not protected by membrane waterproofing a straight seal, Type N, was provided.

Type of design—Rigid frame design based on dimensions from center to center of members.

Loading:

H20 truck loading (see Bridge), plus impact as figured by formula,

$$I=\frac{50}{125+L}\times \text{load.}$$

Effective width for resisting concentrated loads was determined by formula:

$E=0.78+w$, in which

E =Effective width in feet for 1 wheel.

S =Span in feet.

d =Depth of fill in feet.

w =Width of tire in feet.

$E_1=\frac{1}{2}(E+C)$ when effective widths overlap.

C =Distance center to center of wheels.

Earth surcharge, 2 feet 6 inches.

Flotation factor of safety—A minimum factor of safety, 1.5 was provided against flotation. The volume of earth engaged or effective over the projecting bottom slab was assumed as that within an angle of 20° from a vertical line at the outer corner of slab.

Reinforcement for concrete shrinkage and temperature change—The total longitudinal reinforcement required for each frame member was figured as not less than 0.35 percent of the concrete sectional area; ⅔ of the amount for the exposed face and ⅓ for the opposite face.

OIL SYSTEM

Oil lines from oil filter plant and oil storage tanks in service building to all oil-filled equipment in yard.

CARRIER CURRENT RELAY SYSTEM

Carrier current relaying equipment is provided for protecting the 154-kilovolt transmission line. Utilizes phase A and ground with coupling capacitors connected to phase A at both ends of the transmission lines. The direction of power flow at one end of the line in relation to the direction of simultaneous flow at the opposite end determines whether or not an electrical fault has developed on the protected line.

CARRIER CURRENT TELEPHONE SYSTEM

Carrier current telephone equipment, utilizing the B and C phases of the 154-kilovolt lines provides two-way telephone communication between Wheeler, Wilson, Chickamauga, and Norris. Any extension telephone instrument at either station can be selectively called by the usual method of dialing.

HIGHWAY BRIDGE

ASSUMPTIONS

Train of motor trucks or equivalent loads assumed to occupy traffic lanes, each having a width of 9 feet.

Truck train loads shall be used for loaded lengths of less than 60 feet, but may be used for greater loaded lengths. Each train shall consist of 1 truck of 20 tons gross weight followed by, or preceded by, or both followed and

HIGHWAY BRIDGE—ASSUMPTIONS—continued

preceded by, a line of trucks of indefinite length, each of the following or preceding trucks having a gross weight of 15 tons. The trucks in adjacent lanes shall be considered as headed in the same direction. The 20-ton trucks shall have a wheel base length of 14 feet with a load of 32,000 pounds on the rear wheels, each rear tire being 20 inches wide. Front wheels carry total load of 8,000 pounds. Gage of wheels 6 feet. No wheel nearer than 1 foot 6 inches to curb.

The 15-ton trucks carry a total load of 24,000 pounds on rear wheels with 15-inch tires and 6,000 pounds on front wheels. Trucks are spaced so loads of rear wheels of preceding truck and front wheels of following truck are 30 feet from front and rear wheel loads, respectively, of intermediate trucks.

Equivalent loadings shall be used only for loaded lengths of 60 feet or greater. Each lane loading shall consist of a uniform load of 640 pounds per linear foot of traffic lane combined with a single concentrated load so placed on the span as to produce maximum stress. The concentrated load shall be considered as uniformly distributed across the lane on a line normal to the center line of the lane. For the computation of moments and shears, different concentrated loads shall be used. 18,000 pounds concentrated load shall be used in computing the stresses in members in which the greater part of the stress is produced by bending moments. 26,000 pounds concentrated load shall be used when the greater part of the stress in a member is produced by shearing force.

The loadings shall be applied by that one of the following methods which produces the greater maximum stress in the member considered, due allowance being made for the reduced load intensities hereinafter specified for roadways having loaded widths in excess of 18 feet.

Each traffic lane loading shall be considered as a unit, and the number and position of the loaded lanes shall be such as will produce maximum stress.

The roadway shall be considered as loaded over its entire width with a load per foot of width equal to $\frac{1}{9}$ of the load of 1 traffic lane. This shall apply to both uniform and concentrated loads.

H20 loading, "Standard Specifications for Highway Bridges and Incidental Structures," adopted by the American Association of State Highway Officials, 1931 edition.

If the loaded width of the roadway exceeds the two-lane width of 18 feet, the specified loads shall be reduced 1 percent for each foot of loaded roadway width in excess of 18 feet. Thus for 20-foot roadway the loads shall be reduced 2 percent. If the loads are lane loads, the loaded width of the roadway shall be the aggregate width of the lanes considered; if the loads are distributed over the entire width of the roadway, the loaded width of the roadway shall be the full width of roadway between curbs.

Sidewalks shall be designated for a live load of not less than 100 pounds per square foot of sidewalk area.

IMPACT (EXCEPT SIDEWALK LOADS)

The amount of this allowance or increment is expressed as a fraction of the live load stress and was determined by the formula:

$I = \frac{50}{L+125}$, in which

I = Impact fraction.

L = Length in feet of the portion of the span which is loaded to produce the maximum stress in the member considered.

LONGITUDINAL FORCE

Provision was made for the effect of a longitudinal force of 10 percent of the live load on the structure, acting 4 feet above the floor.

LATERAL FORCES

Wind 30 pounds per square foot on 1½ times the area of the structure, as seen in elevation, including the floor system and railings and on ½ the area of all trusses or girders in excess of 2 in the span.

HIGHWAY BRIDGE—LATERAL FORCES—continued

The lateral force due to the moving live load and the wind force against this load was considered as acting 6 feet above the roadway and was 200 pounds per linear foot.

The total assumed wind load was not less than 300 pounds per linear foot on girder spans.

Auxiliary Equipment

CRANES

270-ton gantry (generator erecting crane):
8-motor, double trolley, double leg 175-lb. rails.
2 135-ton hooks with lifting beam; 20-ton auxiliary hook.

	Motor horse-power	Maximum lift	Feet per minute	Travel	Span
		Feet		*Feet*	*Feet*
Main hooks	2-60	72	5.1		
Auxiliary hooks	2-60	85	32.8		
Trolleys	2-15		29.5	40	
Bridge	2-60		72-78	210	69

Loads:

Dead load—includes the weight of the bridge and legs, gantry drive mechanism, platforms, trucks, and operator's cage. The effect of the eccentricity of the locations of the gantry drive mechanism, platform, cage, and other equipment was considered.

Live load—includes the weight of the hook loads, hooks, trolleys, ropes, and attachments. Wheel loads were treated as concentrated loads in determining bending moments and other stresses.

Impact—live load was increased 10 percent for impact.

Trolley tractive force—10 percent of the weight of the trolley with full load and was considered as applied at the top of the trolley rail. Three-fourths of this force was considered as resisted equally by all of the gantry wheels at either end of the gantry.

Gantry tractive force—considered as the sum of the following forces:

10 percent of the weight of the trolley with full load applied at the top of the trolley rail in such a position as would give maximum stresses in either the shear legs or bridge girders.

10 percent of the dead weight of the bridge girders equally distributed between the top and bottom flanges.

10 percent of the dead weight of all the remaining structure and machinery applied at its center of gravity.

Wind loads on the crane were taken as 10 and 30 pounds per square foot. The wind area was taken as the vertical projection of the structure. The wind area of the hook loads was taken as 300 square feet with the center at elevation 570.

Bridge and leg load combinations:

Dead and live loads, 10-pound wind load on crane and hook load, impact loads, and trolley tractive force.

Dead and live loads, 10-pound wind load on crane and hook load, and trolley and gantry tractive forces.

Dead and trolley loads with no load on hooks, and a 30-pound wind load.

Collisions with track stops when the gantry or trolley is traveling with full load at full speed, with the power off, in combination with dead and live loads.

Unit stresses (tension, net section) .... 14,000 pounds per square inch

20-ton gantry (stop log crane) .... 100-pound rails

	Motor horse-power	Maximum lift	Feet per minute	Travel	Span
		Feet		*Feet*	*Feet*
Main hoist	20	84	9.5		
Bridge	20		99.5	700	16

CRANES—continued

20-ton gantry—Continued.

Loads:

Upper platform and legs—load combinations:

Dead and live loads, 10-pound wind load on crane and gate leaf, and impact loads.

Dead and live loads, 10-pound wind load on crane and gate leaf, and gantry tractive forces.

Dead loads with no load on hooks, and a 30-pound wind load.

Dead loads and the rated breakdown torques of the motors.

Dead and live loads, and collision of the bumpers and track stops at full speed with power off.

Impact—25 percent of live loads.

Gantry tractive force on each end of the crane is considered as 10 percent of the resultant maximum wheel loads, considering the weight of the crane and hook loads.

A factor of safety of six, based on the rated capacity of the crane (but exclusive of impact, acceleration, and retardation) was used in the design of all mechanical parts.

Unit stresses .... 14,000 pounds per square inch

75-ton gantry (transformer untanking):

Main hoist 75 tons; auxiliary hoist 1 ton; 100-pound rails.

	Motor horse power	Maximum lift	Feet per minute	Travel	Span
		Feet		*Feet*	
Main hoist	40	40	5–7	....	
Auxiliary hoist	1¾	38	20–35	....	
Trolley	5	....	8–10	14	
Bridge	10	....	10–15	26	23 feet 2¾ inches.

Loads:

Trolley tractive force—as 10 percent of the weight of the trolley fully loaded and considered as applied to the top of the trolley rails. Three-fourths of this was considered as resisted equally by all of the bridge wheels at either end of the crane.

Impact .... 10 percent of live load

Bridge tractive force was considered as the sum of the following forces:

10 percent of the weight of the trolleys with full load applied at the top of the trolley rails in such position as would give maximum stresses in either the truck or bridge girders.

10 percent of the dead load of the bridge girders equally distributed between the top and bottom flanges.

Unit stresses (tension) .... 14,000 pounds per square inch

85-ton gantry (powerhouse intake crane):

Two 42½-ton main hooks; 5-ton auxiliary hook; 100-pound rails.

	Motor horse-power	Maximum lift	Feet per minute	Travel	Span
		Feet		*Feet*	*Feet*
Main hoist	60	83	7	....	....
Auxiliary hoist	10	75	25	....	....
Trolley	5	....	25	35	....
Monorail	¾	....	25	58	....
Bridge	60	....	100	656	53

Loads:

Service gates (70 tons) required lifting effort .... 85 tons

Impact .... 15 percent of live load

Tractive forces .... 10 percent of weight on the rails

Lateral load for trolley .... 10 percent of its weight with full load

Longitudinal load .... 10 percent of maximum wheel load

Wind load .... 30 pounds per square foot

CRANES—continued

Loads—Continued.

Combination loadings—bridge and legs:

Dead and live loads, 10-pound wind load on crane and gate leaf, impact loads, and trolley tractive force.

Dead and live loads, 10-pound wind load on crane and gate leaf, trolley and gantry tractive forces.

Dead and trolley loads with no load on hook, and a 30-pound wind load.

Allowable unit stress ---------- 14,000 pounds per square inch

Factor of safety:

Mechanical equipment—based on the rated capacity of the crane (but exclusive of impact, acceleration, and deceleration) ---------- 6

Auxiliary hoist ---------- 5

100-ton stiffleg ---------- 70-foot radius, 85-foot boom, 50-foot mast

	Capacity	Lift	Motor horse-power	Feet per minute
	Tons	*Feet*		
Main hoist	100	70	150	12
Auxiliary hoist	25	75	25	50

100-ton transfer car:

Loads—

Dead load is the weight of the car including all parts permanently attached to complete the car for its duty.

Live load is the weight of the load transported.

In determining bending moments and other stresses, wheel loads were treated as concentrated loads.

Wind loads were taken as 10 pounds per square foot. The wind area was taken as the vertical projection of the live load.

Tractive force is considered as 10 percent of the weight of the live load acting through its center of gravity, which is approximately 10 feet above the cross rails on the car.

Unit stresses (tension) ---------- 16,000 pounds per square inch

ELEVATOR

Type ---------- Gearless, automatic electric

Capacity ---------- 5,000 pounds, live load

Motors ---------- 40-horsepower motor-generator set, 30 horsepower, 2–1 gearless

Lift ---------- 70.25 feet

Speed ---------- 250 feet per minute

Platform ---------- 7 feet 8 inches by 7 feet 3¾ inches, outside dimensions

Landings ---------- Bottom landing at elevation 498.8—top landing at elevation 569 with 4 intermediate landings.

Construction Plant

MATERIALS

Cement ---------- Type B modified; tricalcium silicate not more than 55 percent; tricalcium aluminate not more than 8 percent; fineness 1,600 to 2,200 (centimeters) 2/grams Wagner turbidimeter.

Initial set ---------- Gillmore, not less than 60 minutes

Final set ---------- Gillmore, not more than 10 hours

Quantity ---------- 833,068 barrels

Price per barrel f. o. b. barges Wheeler Dam ---------- $1.8798

Aggregates:

Source of material ---------- Tennessee River, 30 to 50 miles downstream

Sand ---------- Chiefly quartz, weight 90 pounds per cubic foot

Gravel -- Porous chert, 70 percent; solid chert, 15 percent; quartz, 10 percent

Sizes No. 4 to ¾ inch; ¾ to 1½ inches; 1½ to 3 inches.

Weights per cubic foot ---------- 87 to 90 pounds

Produced, washed, screened, and delivered to mixing plants by Cumberland River Sand Co.

Quantity ---------- All sizes, 1,058,533 tons

CONCRETE MIXING

Mixing............ 4 floating concrete plants, each equipped with a 2-cubic yard Davis tilting mixer; mixing time 1½ minutes; average cycle 2.4 minutes.
Plant capacity............... Capacity of each plant, 50 cubic yards per hour
Plant output................... Average all plants, 37.12 cubic yards per hour
Mixes used......................... Slump ½ to 1½ inches for mass concrete
Placing.......... By buckets from deck of mixer barges to forms by means of whirley cranes mounted in cofferdam; 2-cubic-yard buckets for majority of work; 3 internal and 1 surface vibrator per crew. Height of lifts, 5 feet; form area, 3.1 square feet per cubic yard of dam mass concrete.
Tests.................................... Total number of job cylinders, 5,300
  Average 7-day strength...................... 2,890 pounds per square inch
  28-day strength......................... 4,750 pounds per square inch
  Durability tests showed loss of 5 percent by weight of concrete after 160 cycles of alternate freezing and thawing.
  Weight, mass.................................. 147 pounds per cubic foot
  Specific gravity................................ Sand, 2.63; gravel, 2.45
Total concrete poured (does not include concrete for lock)... 631,150 cubic yards

PRINCIPAL CONSTRUCTION EQUIPMENT [1]

Aggregate handling............. 4 American cranes mounted on floating mixing plants 75-foot booms, 2-cubic-yard clamshell buckets.
Cement unloading............................. 4 Fuller-Kinyon 4-inch pumps
Compressors............... 3 stationary Sullivan 2,448 cubic feet per minute; 2 portable Worthington 240 cubic feet per minute.
Concrete placing........... 6 gantry-mounted revolving Clyde cranes, 95-foot boom; 23 concrete buckets, 1- and 2-cubic-yard sizes; one 60-cubic-yard per hour concrete pump; 38 electric vibrators.
Drilling................ 16 wagon drills, drifter mounted, for line drilling; 20 jackhammers; 15 paving breakers; three 5½-inch gasoline shot core drills; one 36-inch electric shot core drill.
Excavation shovels.......... 1 Marion, 2½ cubic yards; 1 Bucyrus, 2½ cubic yards; 1 Osgood, 1 cubic yard; 1 gas-electric, 1¼ cubic yards.
Excavation trucks......... 9 White with 10-cubic-yard Boulder Dam type bodies
Hoists........... Two 2-drum 60-horsepower electric hoists; 2 locomotive cranes with 60-foot booms, 12 and 25 tons. 2 guy derricks, 20 and 50 tons (see also aggregate handling and concrete placing).
Marine equipment........... 3 launches, 6 towboats, 14 barges, 1 derrickboat—22-ton crane. Rented—peak periods: 2 dipper dredges, 6 derrickboats, 3 towboats, 14 barges, 1 launch, 1 quarterboat, 1 tender.
Unwatering pumps........... 13, size 10 to 14 inches; 40, size 2½ to 6 inches; and 10 smaller sizes.

CONSTRUCTION QUANTITIES

Construction south of the lock:
  Excavation, approximate............................ 548,000 cubic yards
  Formwork, approximate............................ 3,021,400 square feet
  Reinforcing.. 8,450 tons
  Concrete, approximate.................................... 631,150 cubic yards
Lock concrete by U. S. Army Engineers..................... 121,100 cubic yards

[1] For further details see pp. 134–8.

APPENDIX B

REPORTS OF CONSULTANTS

The following reports contain the joint recommendations and approvals following studies by the consultants acting as a board. Individual reports of consultants working separately are not included because in most instances they are voluminous and cover detailed engineering or geologic work ordinarily done by full-time members of the Authority's staff.

Spillway Capacity, Powerhouse, and Earth Dam, December 12, 1933

It was agreed that the spillway capacity should be 650,000 cubic feet per second, with a surcharge of three feet; that is, a water level of 558. It was understood that the tainter gates would be 40 feet long by 14 feet high. Apparently 60 gates would supply such a discharge so that the length of spillway between abutments would then be about 2,760 feet.

It was agreed to recommend that at this time all the under water work for the final completed powerhouse be installed with the remainder of the dam.

It was also agreed that no earth dam should be constructed at one end of the dam for these three reasons:

1. Because it would put an earth dam on a rock foundation, under conditions where a concrete section seems much more desirable.
2. Such an earth dam would always have water against the lower part of the downstream base, resulting in complete saturation of the lower part of the dam.
3. There would be no important saving in cost.

C. H. Locher.
J. L. Savage.
C. H. Paul.
B. M. Jones.
S. M. Woodward.

Determination of Ultimate Generating Capacity, January 27, 1934

At a conference between the undersigned on January 27, 1934, the following conclusions were reached with reference to the generating capacity which should be provided for at Wheeler Dam:

1. Intakes should be provided for an ultimate installation of eight 35,000 kilowatt units, or a total of 280,000 kilowatts.
2. At the present time the excavation should be completed and the downstream end of the draft tubes constructed for four of the above units.
3. Serious consideration should be given to the completion of the powerhouse for two of the above units at the present time—installing one unit complete and purchasing only the speed ring and draft tube liner for the second unit.

It should be pointed out that the original scheme was for the installation of 10 units which would involve the completion of 10 intakes, and it was also proposed to complete the draft tube excavations below these intakes. On a basis of a total of 8 units, as now proposed, there will be a saving in intakes and draft tubes excavation of $1,040,000. To complete 1 unit would require the additional expenditure of $1,686,000 or a net additional expenditure beyond that which was contemplated of $646,000. With regulation by Norris Reservoir this one unit would produce about 300,000,000 kilowatt-hours of primary power at a cost of about 0.2 mill on the above basis. Even though the saving of $1,040,000 was not deducted from the additional cost of one unit this power could be produced for 0.5 mill per kilowatt hour. This would be by far the cheapest capacity on the system and at the earlier stages of development might save expensive steam generation.

4. It is believed that the Bureau of Reclamation should make studies to determine what economy might be effected by construction of the so-called outdoor type of power station as opposed to the usual type of superstructure.

JAMES S. BOWMAN.
BARTON M. JONES.
ROSS M. RIEGEL.

GENERAL INSPECTION, MAY 8, 1935

The Consulting Board made an inspection of the work at Wheeler Dam on May 6, 1935.

We found good progress being made on all features of the work, and there seems to be no doubt that, barring unforeseen contingencies, the job will be completed within the scheduled time.

Cleaning and preparation of the rock foundation are being well handled and seem to be satisfactory in every respect.

Concrete work appears to be highly satisfactory both as to quality and as to handling and placing.

Examination was made of the trimming and preparation of the south abutment rock, and the procedure being followed at that point is approved.

L. F. HARZA.
C. H. LOCHER.
J. L. SAVAGE.
S. M. WOODWARD.
CHARLES H. PAUL.

GENERAL INSPECTION, APRIL 6, 1936

The members of the Consulting Board spent a part of Wednesday, April 1, on inspection of the work at Wheeler Dam, and observation of the flood flow passing over the spillway and through the intake structure of the power plant extension.

The construction work on the dam and appurtenant structures appears to be of excellent character.

There is no evidence of seepage around the structure on either side, indicating that so far as revealed by service conditions to date the grouting and cutoff work has been effective.

L. C. GLENN.
L. F. HARZA.
WARREN J. MEAD.
J. L. SAVAGE.
CHARLES H. PAUL.

GENERAL INSPECTION, JULY 16, 1936

The members of the Board spent a part of the forenoon of Thursday, July 16, 1936, on inspection of the work at Wheeler Dam.

The work on the dam itself is practically complete, and progress on the powerhouse is satisfactory.

In our opinion the character of the work is excellent.

CHARLES P. BERKEY.
O. N. FLOYD.
L. C. GLENN.
G. W. HAMILTON.
L. F. HARZA.
C. H. LOCHER.
WARREN J. MEAD.
J. L. SAVAGE.
CHARLES H. PAUL.

General Inspection, May 30, 1937

The members of the Board spent part of the forenoon Sunday, May 30, on a general inspection of the work at Wheeler Dam.

Since our last visit the odds and ends of construction on the dam have been entirely completed, the installation of both units of the powerhouse has been finished and they are now in operation. The character of all this work appears to be excellent.

Our attention was called to the small leak which has developed around the south abutment of the dam, entering the rock apparently a few hundred feet upstream from the dam, and appearing in the gulch which joins the river several hundred feet downstream from the site. Up to date the flow appears to vary with the reservoir head, and the time element has not become a factor in affecting the flow. It is expected that this leak will be stopped or greatly reduced as soon as the reservoir is drawn down to the point where its inlet can be reached. In the meantime nothing serious has been indicated in connection with this condition.

O. N. Floyd.
L. C. Glenn.
L. F. Harza.
C. H. Locher.
J. L. Savage.
Charles H. Paul.

Inspection of Leak in Left Abutment, July 26, 1939

In the morning of July 26, the undersigned Consultants visited the Wheeler Dam, made a general inspection of the dam and powerhouse, and gave particular attention to the leak around the left abutment.

Whenever the reservoir elevation reaches 548.5, water finds its way through rock seams around the left abutment cut-off and comes out in a small draw which heads up in the abutment region about 1,200 feet beyond the end of the dam. The leakage increases uniformly as the reservoir is raised above elevation 548.5 and amounts to about 5 second-feet with reservoir surface at elevation 556. The water emerges from rock bedding seams and comes out clear. There has been no perceptible increase in flow under full reservoir conditions during the past 18 months. However, the discharge measurements are affected by local rainfall as well as by reservoir level, and therefore accurate comparisons can be made only during dry weather periods.

We feel certain that this leakage has no effect whatever on the safety of the dam and that its practical significance relates only to the small water loss involved. We believe that it should be observed and recorded systematically for a period of years until it can be determined definitely whether it is increasing or decreasing, and particularly so as to be informed immediately of any marked increase, should that occur. In that case, extension of the cut-off or other treatment might be advisable.

In order to make the records more complete, we suggest that a rainfall record be plotted along with the discharge records, so as to determine, if possible, the effect of rainfall on the measured discharges.

With the situation thus under systematic observation, we feel that there need be no other concern, unless or until some significant change should make further consideration advisable.

Charles P. Berkey.
O. N. Floyd.
L. C. Glenn.
Charles H. Paul.
W. F. Prouty.
J. L. Savage.

APPENDIX C

SPECIAL TESTS

AGGREGATE AND CONCRETE TESTS

Both the U. S. Bureau of Reclamation[1] and the Tennessee Valley Authority[2] conducted extensive tests to determine the quality and characteristics of the aggregate, cement and concrete considered for use at Wheeler Dam. Tests on aggregate were undertaken to determine which of the material available was best suited for concrete. In addition to the tests on the aggregate, tests also were made to determine the durability of concrete made from these aggregates. Additional concrete tests were made to determine the permeability, volume change, thermal properties, and temperature and heat generation. Temperature tests were also carried on to determine the actual temperature rise in the dam and the effect of 5- and 10-foot lifts. Strength and elasticity were determined on both field and laboratory mixes, and, as a check, tests were made on cores drilled from the mass of the dam.

General references describing the apparatus used throughout these tests and the procedure adopted for casting, curing, and testing are shown in table 34. The technique and apparatus pertaining to tests which are not described in the above references will be mentioned briefly in connection with the discussion of the test results.

TABLE 34.—*References—Test methods and apparatus*

Subject	Reference
Normal consistency	Paragraphs 23 and 24, A. S. T. M. Standards, Designation C77-32.
Time of set	Paragraphs 33 and 34, A. S. T. M. Standards, Designation C77-32.
Soundness	Paragraphs 26 to 29, inclusive, A. S. T. M. Standards, Designation C77-32.
Specific gravity	Paragraphs 2 and 3, A. S. T. M. Standards, Designation D55-25.
Fineness	Paragraphs 13 to 18, inclusive, A. S. T. M. Standards, Designation C77-32. Paragraph 31, Specification No. 566, Portland Cement for Boulder Dam. "A Rapid Method for the Determination of the Specific Surface of Portland Cement," Proceedings, A. S. T. M., vol. 33, pt. II, 1933.
Chemical analysis	U. S. B. R. Paragraph 29, Specification No. 566, Portland Cement for Boulder Dam.
Compound composition	U. S. B. R. Paragraph 30, Specification No. 566, Portland Cement for Boulder Dam.
Adiabatic calorimeter rooms.	U. S. B. R. Technical Memorandum No. 302, Sept. 9, 1932. Journal of the A. C. I., vol. 5, No. 1, Sept.-Oct. 1933, p. 21.
Machine mixing	U. S. B. R. Technical Memorandum No. 313, Oct. 15, 1932. Section on Uniformity.
Vibratory compaction	U. S. B. R. Technical Memorandum No. 296, June 1, 1932.
Large specimens	Proceedings, A. S. T. M., vol. 33, pt. II, 1933.
Comparison of cements	Journal of the A. C. I., vol. 5, No. 1, Sept.-Oct. 1933, p. 9. Technical Memorandum No. 371, sec. II.
Elasticity	Journal of the A. C. I., vol. 5, No. 1, Sept.-Oct. 1933, p. 41. Technical Memorandum No. 305, Sept. 1, 1932.
Thermal properties	Journal of the A. C. I., vol. 5, No. 1, Sept.-Oct. 1933, p. 35.
Permeability	For Publication in the Journal of the A. C. I., March-April 1935.
Aggregate soundness	A. S. T. M. Standards. Designations C88-32T and C89-32T.

AGGREGATES

Two samples of aggregates were tested by the Bureau of Reclamation, the first being furnished from the gravel plant used for the construction of a lock adjacent to the dam and the second from the deposit in the river proposed for use in the concrete in the dam.

The aggregate itself appeared none too desirable structurally as a concrete ingredient. The gravel and coarse sand was composed principally of chert

[1] Vidal, E. N., and Meissner, H. S., *Concrete Investigations for Wheeler Dam,* U. S. Bureau of Reclamation Technical Memorandum No. 440, February 25, 1935.

[2] Johnson, W. R., *Concreting at Wheeler Dam,* Tennessee Valley Authority, Technical Monograph No. 31. November 1937.

and cherty materials with quartz secondary. A great deal of the chert appeared to have been calcitic at some time, but the lime had been leached out by circulating waters and removed elsewhere. This aggregate, as a result, was very porous, with absorption tests showing values as high as 6 percent in some cases for 24-hour immersion. The particles were extremely smooth and for the most part lenticular in shape. The presence of abundant sea life in the past was observed from the remains embedded in the gravel.

As the aggregate size decreased, the amount of quartz increased until in the 100-mesh size and smaller the material was practically all quartz. The specific gravity of the sand was about average, although the unit weight was somewhat less than ordinarily obtained for a sound siliceous sand.

In addition to the regular physical tests of aggregates, which included fineness modulus, silt, specific gravity, unit weight (dry and rodded), voids (dry and rodded), absorption, and calorimetric tests of sand, tests were undertaken to determine the durability of coarse aggregate and structural strength of sand.

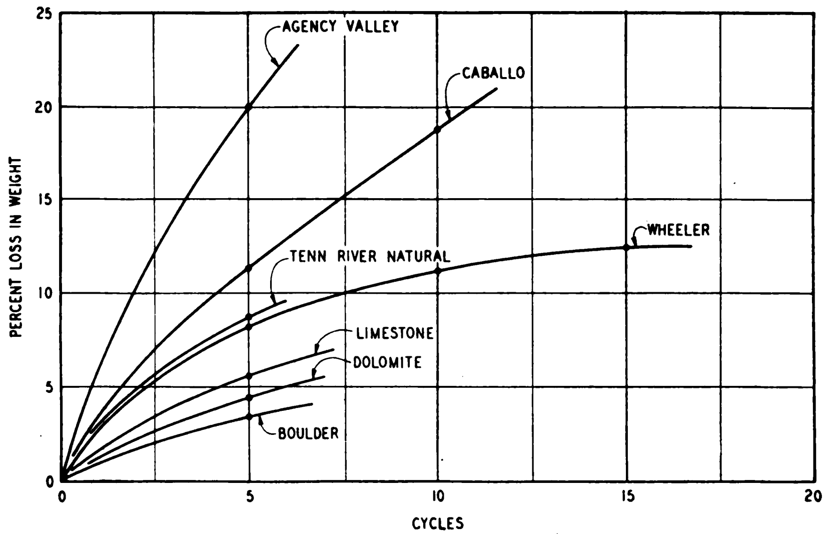

FIGURE 135.—*Results of sodium sulphate tests—Aggregate durability.*

Soundness tests of coarse aggregate.

At one time it was thought that the excavated rock from the foundation of the dam might be used as concrete aggregate, and tests were made by the Authority to determine the suitability of this rock for this purpose. For purpose of comparison, tests were also made on gravel obtained from the Tennessee River.

Two predominating types of rock were found in the powerhouse excavation and cofferdam No. 1:

1. A compact siliceous limestone, dark gray in color, occurring in thin laminated sheets.

2. A coarse crystalline limestone, light gray in color, of more or less granular composition and containing a great deal of quartz.

The gravel was mostly chert with a small percentage of quartz. A great deal of chert was of limestone origin, but most of the lime appeared to have been leached out by circulating water and removed elsewhere.

The tests were carried out in accordance with standards[3] of the American Association of State Highway Officials. Generally, the tests followed the procedure of immersing 10 small samples of the rock with total weight of approximately 100 grams in a saturated solution of sodium sulphate at 70 de-

[3] Method of Tests for Soundness of Coarse Aggregate, Tentative Method T–9.

grees Fahrenheit. The samples were left in this solution for 20 hours, after which they were placed in an oven maintained at 100 degrees Centigrade for 4 hours. This procedure of immersing, heating, and reimmersing was continuous and was repeated until conclusive results were obtained. The conditions of the rock as to soundness at the end of the tests were noted, and samples with marked disintegration were considered to have failed the tests.

After five cycles of the treatment described above, the 2 limestone samples from the dam excavation had commenced to fail rather badly and, after 10 cycles, were completely disintegrated. The compact siliceous limestone failed by breaking down into thin pieces, which could be easily broken with the hand. The coarse crystalline limestone broke down into small granular particles and after complete failure looked like a pile of coarse sand.

Of the 10 pieces of gravel, one which was a soft white cherty limestone, broke into 5 small pieces after 5 cycles. Two other pieces, which were somewhat thin and elongated and composed of a harder chert, broke into 2 parts. A fourth piece, which was quartz, also broke into 2 parts. The other 6 pieces were not affected. At the end of 10 cycles no further disintegration had taken place.

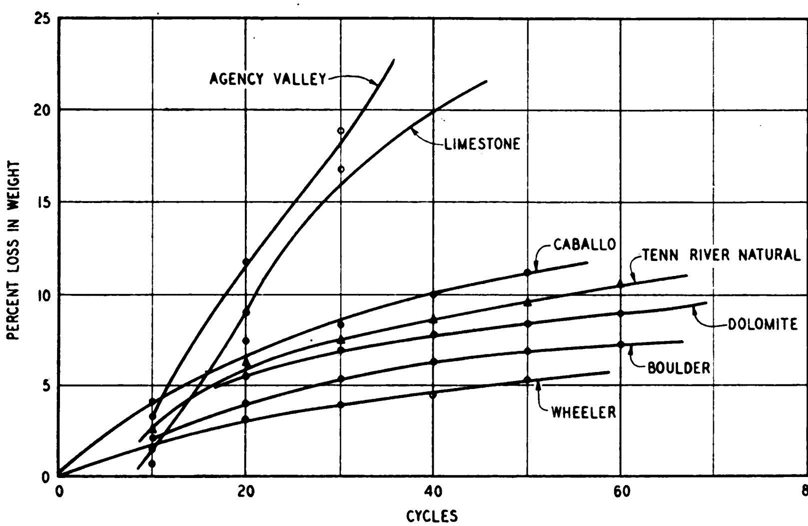

FIGURE 136.—*Results of freezing and thawing tests—Aggregate durability.*

The results of these tests indicated that the excavated material was not nearly as suitable for concrete purposes as the river gravel.

Durability.

Durability tests were made on both the aggregates and the concrete made from these aggregates. These tests were undertaken by the U. S. Bureau of Reclamation.

Aggregate.—The soundness of the aggregate was determined in terms of its resistance to attack by sodium sulphate solution and to deterioration from alternate cycles of freezing and thawing.

The percent loss in weight of these aggregates when repeatedly soaked in concentrated sodium sulphate solution, following the procedure of the A. S. T. M. Standards, Designation C88–32T and C89–32T, is shown in figure 135. Figure 136 shows the losses sustained when the aggregates were repeatedly frozen when submerged in water and thawed at 70 degrees Fahrenheit. These figures show that the Wheeler Dam aggregates resisted the freezing and thawing tests better than any of the other six aggregates, whereas they did not appear to such good advantage when tested in the sodium sulphate solution.

Concrete.—Durability tests were conducted on concrete mixes made with each of the seven different aggregates used in the aggregate durability tests. The results of the resistance of these concrete mixes when subjected to alternate freezing, thawing, wetting, and drying may be compared in figures 137 and 138. Six 3- by 6-inch cylindrical specimens were made from each of the concrete mixes. Three of these contained stainless steel points embedded concentrically in their ends for measurement of length changes and the others were made with plain ends which were ground plane. All cylinders were weighed periodically to determine weight losses as another measure of durability. Before beginning the test the specimens were cured for 90 days at 70 degrees Fahrenheit in a moist room.

After curing, the cylinders were weighed, measured, and then dried in an oven at 120 degrees Fahrenheit for 72 hours. They were then cooled to 70 degrees Fahrenheit, weighed and measured again, and then immersed in 70-degree Fahrenheit water for another 72 hours. For freezing, the specimens were nested in 2 layers on end, 4 to each layer in galvanized iron containers open at the top. These containers were filled with water and placed in brine maintained at —15 degrees Fahrenheit. The containers remained in the refrigerating brine until the temperature of the concrete, determined by means of resistance thermometers in dummy specimens, was lowered to approximately 5 degrees Fahrenheit, requiring approximately 1½ hours. The containers were removed from the brine and were next submerged in a tank of circulating water kept tempered to 70 degrees Fahrenheit. After another 1½ hours, in

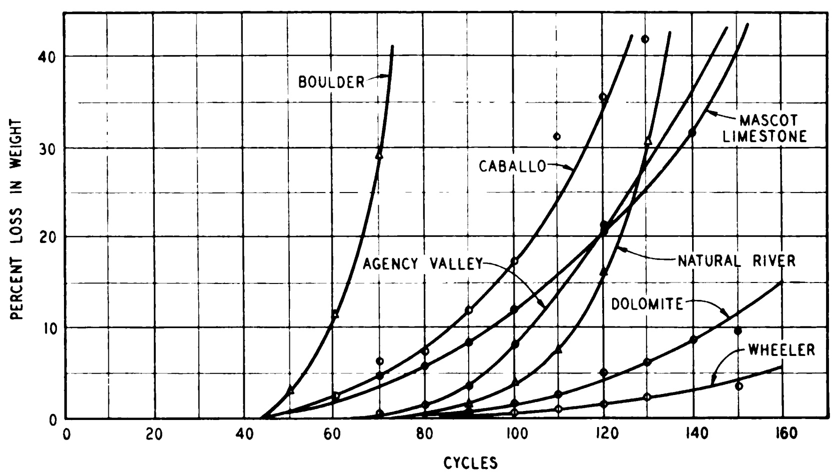

FIGURE 137.—*Results of freezing and thawing tests on 3- by 6-inch cylinders showing percent loss in weight.*

which time thawing was complete, another freezing cycle was begun. If night intervened, the specimens remained in the 70-degree Fahrenheit water. After 10 cycles of alternate freezing and thawing, length and weight measurements were taken. These operations were then repeated in the above sequence until complete disintegration was obtained.

From every consideration, the concrete containing Wheeler Dam aggregate may be judged superior in resistance to the durability test to the other six materials. Although the results are in disagreement with the character of the aggregate and its mediocre performance in the sodium sulphate test and in agreement with its performance under freezing and thawing upon the aggregate alone, aggregate soundness tests do not seem to have any relation to concrete durability from the data obtained. This statement is evidenced particularly by the results upon Agency Valley and Boulder Dam materials. Boulder Dam materials resisted the sodium sulphate and freezing and thawing tests better than any other, and yet the concrete made with the Boulder materials deteriorated most rapidly. On the other hand, Agency Valley Dam aggregate, found definitely unsound, made concrete that appeared superior for some time and near the end of the tests compared quite favorably.

A study of the mineral classification of these various aggregates reveals a clue to the durability of the concrete made from these aggregates. Boulder

Dam material is observed to be very heterogeneous, while that of Agency Valley Dam is homogeneous. Caballo aggregate and a natural Tennessee River gravel found near Chattanooga are also composed of a variety of minerals, while the material purposed for use as Wheeler aggregate found below the site, dolomite, and limestone are homogeneous. The Boulder, Caballo, and natural Tennessee River concretes may be selected as the least durable, containing the miscellaneous aggregates, and the concretes composed of the more uniform aggregates, Agency Valley, Wheeler, and dolomite, may be chosen as the most resistant. Such grouping suggests the possibility that differential expansion and contraction within concrete composed of assorted materials with various coefficients of expansion, contributes in no small degree to failure. The limestone concrete cannot be aligned with either group, although, in view of the foregoing, more favorable results should have been expected. However, this aggregate gave very bad performance under freezing and thawing tests.

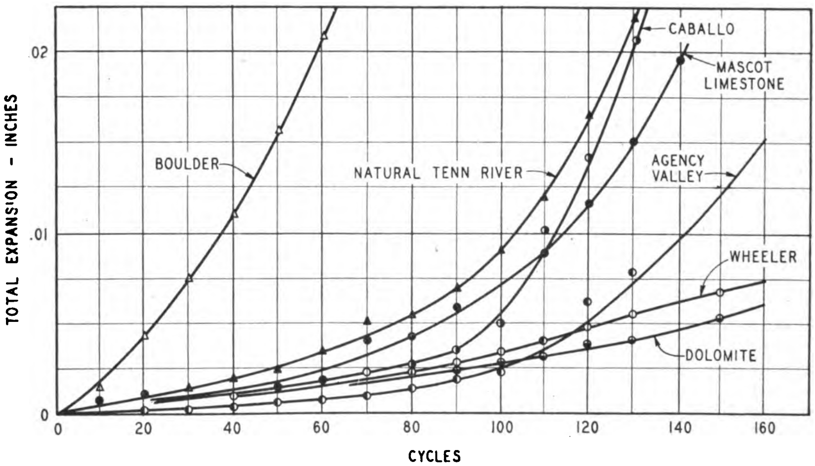

Figure 138.—*Results of freezing and thawing tests on 3- by 6-inch cylinders showin expansion in inches.*

Structural strength of sand.

These tests undertaken by the Bureau of Reclamation were made under a modification of A. S. T. M. Standards, Designation C87–32T. A sufficient quantity of the sand to be tested was mixed into a slurry composed of two-thirds cement and one-third water by weight, so as to produce a consistency of 1 to 1½ inches in the resulting mixture as measured by a 2- by 4- by 6-inch slump cone. The strength of the sand was found to be average and comparable with the material used at Boulder Dam.

CEMENT

The chemical composition and physical properties of the two cements used in the tests were determined. The cements used were the normal and a modified portland cement. The normal portland cement was an ordinary, standard product, except that the clinker was retreated before grinding so as to give the finished material more desirable heat generation properties. The modified portland cement was the type B cement and was typical of that purchased by the Authority.

CONCRETE

Concrete tests were conducted on laboratory and field mixes to determine the strength and elastic properties of mass mixes used in the main body of the dam and of mixes of smaller maximum size aggregate used in thin sections and reinforced walls. As a check on the strength and elastic properties, tests were made on cores drilled from the mass of the dam. In addition, the work included tests for determination of permeability, volume change, thermal properties, and temperature rise and heat generation. Temperature rise tests were also carried on in the dam structure to determine the effect of 5- and 10-foot lifts.

Strength.[4]

The results of the strength tests were entirely satisfactory and were considerably higher at all ages than had been expected. At the time of casting, the smooth surfaces of the larger aggregates appeared to shed the cement paste, and indications were that the bond between the two would be poor. This was not the case, and the crushing tests showed an average condition of failure through both mortar and aggregate.

The gross water-cement ratios were in every case somewhat high, but when computations were made for the abnormal absorption of the aggregate particles, the net ratio was found to be a reasonable value. In every series of tests, the relation between water-cement ratio and strength agrees with the Abram's law.

Mass (adiabatic) curing provided strength increases at ages of 7 and 28 days similar to those obtained previously in numerous investigations with other cements and aggregates. The strength-age relationship for wet-screened 6- by 12-inch specimens was quite normal and indicates satisfactory strength gains at later ages.

Elasticity.[4]

The values of the elastic properties, in general, were not as high as those obtained for sound quartz or limestone aggregates. The modulus at 28 days was about 4,000,000 pounds per square inch for the full mass mix and almost 5,000,000 pounds per square inch for the 1½-inch maximum size aggregate mix. Poisson's ratio was roughly 0.15 and 0.18, respectively, for these two mixes.

Tests on 18-inch concrete cylinders and drill cores.

Twelve 18-inch concrete drill cores and six 18- by 36-inch molded concrete cylinders were tested[5] for the Authority by the Bureau of Reclamation. The drill cores ranged from 34 to 42 inches in length and varied from 18 to 18¾ inches in diameter.

The molded cylinders were cast in molds of standard 18- by 36-inch size. Prior to shipment, the molded concrete cylinders were cured by sprinkling with water for 14 days and left in the air thereafter. All cores except two were exposed to the weather until crated, and these two were crated immediately after drilling and thus were cured under slightly different conditions. All specimens were shipped in wooden boxes with no packing. The drill cores contained many noticeable cracks when they arrived at the laboratory.

Preparation for tests.—Only seven of the drill cores were cut to the standard 36-inch length. The remaining five had to be cut to the maximum length of the shortest core or about 31½ inches. To obtain plane end surfaces, all specimens were ground. However, extreme hardness of the aggregates made it very difficult to keep within the tolerance of 0.002 inch allowed for standard tests, but no more than 0.004 inch was exceeded. The cores were stored in a dry place until tested.

Several cores were drilled for inserts for elasticity apparatus, but when the first tests were made, it was decided not to test any cores with large cracks because of the erratic results which would be obtained. The short cores could not be tested for elasticity because they were too short for the apparatus available.

Several of the tops sawed from the large concrete cores were drilled for 3-inch-diameter cylinders, 3 to 4 inches long, for durability tests.

Owing to the prominence of cracks in some of the core specimens, a very accurate record of each was kept in order to discover the possible effect of such cracks on the ultimate strength of the specimens. In testing the cylinders, however, very few tests appeared to be affected by the cracks.

Very little preparation was needed to make the molded concrete cylinder specimens ready for testing. Both ends of the cylinders were ground and each cylinder was equipped with inserts to be used by the elasticity apparatus. The cylinders were stored in a dry place from the time they were received until they were tested.

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[4] Vidal E. N., and Meissner, H. S. *Concrete Investigations for Wheeler Dam, Tennessee Valley Authority,* U. S. Bureau of Reclamation, Technical Memorandum No. 440, February 25, 1935.

[5] Byrne, W. S., and Vidal, E. N. *Tests on 18-inch Concrete Drill Cores, Wheeler Dam—TVA*, U. S. Bureau of Reclamation Technical Memorandum, No. 554, January 26, 1937.

Summary of results.—The unit strength of the concrete drill cores ranged from 4,780 to 7,600 pounds per square inch, with an average of 6,130 pounds per square inch for the 12 specimens. The modulus of elasticity values varied from 3,300,000 to 4.300,000 pounds per square inch and Poisson's ratio varied from 0.08 to 0.13 for 3 specimens. In a few cases the cores appeared to fail along the cracks, but an examination of the strength results shows that it made little difference so far as the compressive strength was concerned. The large variation in strength results was probably due to the difference in ages of the specimens, which ranged from 312 to 612 days, and to the different lengths of the cores. Table 35 shows the tabulation of the results.

TABLE 35.—*Results of U. S. Bureau of Reclamation tests on Wheeler Dam concrete drill cores and molded cylinders*

TESTS ON CONCRETE DRILL CORES

Laboratory No.	Mix by weight	F. M. of sand	Cement content (barrels per cubic yard)	W/C ratio weight	Slump (inches)	Unit weight (test age) (pounds per cubic foot)	Unit strength (pounds per square inch)	Modulus of elasticity (pounds per square inch)	Poisson's ratio	Age at breaking date days	6- by 12-inch control cinders unit strength (pounds per square inch)			Remarks
											7 days	28 days	1 year	
1	1:2.6 :5.2	2.70	1.13	0.59	½	141.4	[1] 4,780			370	2,470	4,110		Specimen size, 18 7/16 by 31 5/8 in. Core in good shape.
2	1:2.6 :5.2	2.70	1.13	0.59	½	138.0	[1] 4,880			370	2,470	4,110		Specimen size, 18 5/8 by 31 5/8 in. Best appearing core of lot.
4	1:2.8 :5.0	3.25	1.15	0.49	1¼	144.6	6,770			519	2,560	4,340		Specimen size, 18 9/16 by 36 3/8 in. Badly cracked, excessive voids.
7	1:2.8 :5.0	3.25	1.15	0.49	1¼	142.8	[1] 7,040			534	2,560	4,340		Specimen size, 18 9/16 by 31 3/4 in. Badly cracked, few voids.
3	1:2.8 :5.0	3.13	1.14	0.54	1	140.6	[1] 6,630			532	3,360	5,230		Specimen size, 18 5/8 by 31 3/4 in. Good specimen, no cracks.
12	1:2.8 :5.0	3.13	1.14	0.54	1		7,600	4,300,000	0.11	510	3,360	5,230		Specimen size, 18 1/2 by 36 1/4 in. Good specimen, very few cracks.
8	1:2.45:5.0	2.47	1.18	0.51	1	146.4	7,010	4,000,000	0.08	593	2 970	4,190		Specimen size, 18 1/2 by 36 1/2 in. Good specimen, very few cracks.
11	1:2.45:5.0	2.47	1.18	0.51	1	142.0	5,660			593	2,970	4,190		Specimen size, 18 3/4 by 36 3/8 in. Bad circumferential crack.
5	1:2.45:5.0	2.49	1.18	0.51	1¼	142.7	5,820			612	3,250	5,140		Specimen size, 18 3/4 by 36 7/16 in. Several horizontal and vertical cracks., failed in compression along cracks.
6	1:2.45:5.0	2.49	1.18	0.51	1¼	143.2	5,820			612	3,250	5,140		Specimen size, 18 5/8 by 36 3/8 in. Few cracks, excessive voids.
9	1:2.80:5.0	2.75	1.14	0.56	½		5,380	3,300,000	0.13	312	3,310	4,420		Specimen size, 18 1/2 by 36 1/4 in. Excessive voids, no cracks.
10	1:2.80:5.0	2.75	1.14	0.56	½	141.8	[1] 6,060			332	3,310	4,420		Specimen size, 18 1/2 by 31 5/8 in. same voids, no cracks.

TESTS ON MOLDED CONCRETE CYLINDERS

1	1:2.80:5.0	3.05	1.14	0.51	½	143.7	7,300	5,400,000	[2] 0.27	366	3,640	5,340	6,410	Specimen in good condition.
2	1:2.80:5.0	3.05	1.14	0.51	½		7,300			372	3,640	5,340	6,410	Do.
3	1:2.80:5.0	3.05	1.14	0.51	½	146.0	7,360	5,100,000	.13	372	3,640	5,340	6,410	Do.
Average							7,330	5,100,000	.13					
4	1:2.80:5.0	3.12	1.13	0.54	¾	145.6	7,000	5,000,000	.17	367	4,130	5,490	6,830	Do.
5	1:2.80:5.0	3.12	1.13	0.54	¾	145.1	7,020	4,700,000	.12	367	4,130	5,490	6,830	Do.
6	1:2.80:5.0	3.12	1.13	0.54	¾	145.8	7,020	4,700,000	.13	367	4,130	5,490	6,830	Do.
Average							7,010	4,800,000	.14					

[1] Strengths corrected for length of specimen according to A. S. T. M. Designation C42–31.
[2] Not included in the average.

NOTE.—6- by 12-inch concrete control cylinders were fabricated and cured in the field.

Excellent results were obtained from tests on the molded concrete cylinders. The strength varied from 7,300 to 7,360 pounds per square inch for those taken from the mix for block 8–N–30 and, for those taken from block 1–N–49 the results varied from 7,000 to 7,020 pounds per square inch. The modulus of elasticity varied from 4,700,000 to 5,400,000 pounds per square inch, and Poisson's ratio from 0.12 to 0.17. The test age was approximately 1 year for all specimens. These results are also shown in table 35.

Discussion of results.[6]—As both the 18-inch cores and the 18-inch molded cylinders were packed and shipped in the same manner, and as none of the molded cylinders were cracked, the cracks found in the cores were no doubt caused by handling when the cores were being removed from the dam. In order to break off a large core at the bottom of the hole after drilling, a small charge of dynamite was used to loosen it.

The report indicates that the unit weight of the molded cylinders was in general greater than that of the cores. Tests made at Wheeler on thirty 6-inch cores taken from three locations show a slightly greater unit weight than obtained from either the 18-inch cores or molded cylinders of the same size.

This condition may be accounted for in that most of the cores were taken from blocks placed before the speed of the concrete vibrators was increased from 3,600 to 4,200 revolutions per minute. The higher speed gave greater placing efficiency and more uniform concrete, particularly in the dry mixes (slumps of less than 1 inch).

In view of the nonuniform unit weight shown by the 18-inch cores, indications were that it would be good practice not to place concrete with less than 1-inch slump unless uniform consolidation was assured. The tests on the 18-inch molded cylinders indicated that uniform results could be obtained with concrete of less than 1-inch slump if the vibration was thorough at all times.

Permeability.

The Bureau of Reclamation conducted permeability tests made on the full mass concrete mix with 3-inch maximum size aggregate. The specimens were 12- by 12-inch cylinders, mass (adiabatically) cured, sealed for 28 days, and then tested in suitable containers at a pressure of 400 pounds per square inch for 15 days. The permeability apparatus was such that the water pressure was applied to the top surface only and the flow passed through the specimen in the same direction as the concrete was cast.

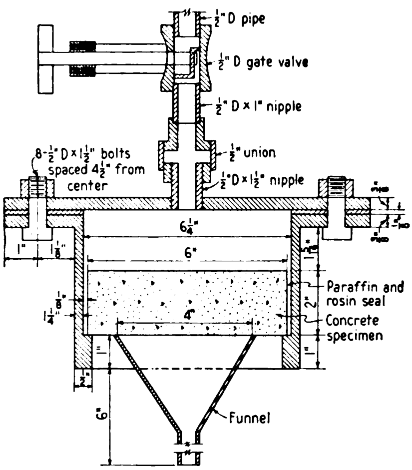

FIGURE 139.—*Apparatus for determining permeability of concrete.*

The value of the coefficient of permeability for the concrete was unusually high when compared to those obtained from Boulder, Norris, and Grand Coulee Dams concrete. However, there would be no need to anticipate difficulties due to percolating waters, as the amount of water flowing into the concrete was so small that it would evaporate as soon as an exposed surface was reached.

The Authority conducted an additional group of permeability tests using 6- by 2-inch concrete discs molded in the laboratory and concrete cores of the same size cut from the dam. The method of conducting the tests and the apparatus used were a duplication of the Portland Cement Association tests. A sketch of the apparatus is shown in figure 139. The specimens were sealed against leakage along the sides by a mixture of hot resin and paraffin. A water pressure of 80 pounds per square inch was maintained against the upper face of the specimen and the amount of water percolating through the specimen in a given period of time was measured.

[6] Johnson, W. R. *Concreting at Wheeler Dam,* Tennessee Valley Authority, Technical Monograph No. 31, November 1937.

The results of the permeability tests on both molded specimens and the cores are in line with results obtained at the Portland Cement Association's laboratory with concrete of the same water-cement ratio.[7]

Volume change.[8]

Six 4- by 4- by 40-inch bars of concrete with 1½-inch maximum size aggregate and one 8- by 8- by 48-inch bar of concrete with 3-inch (mass mix) maximum size aggregate were cast in molds lined with 7-ounce dead soft sheet copper. Immediately after casting the concrete, the casings were hermetically sealed by soldering. The molds were removed at 1-day age and length changes were then observed. These bars were adiabatically cured for 28 days and then subjected to several temperature changes over a range of approximately 100 degrees Fahrenheit. The average coefficient of expansion and contraction, observed after the adiabatic period, was 0.0000064 inch per degree Fahrenheit for the 4- by 4- by 40-inch bars and 0.0000067 inch per degree Fahrenheit for the 8- by 8- by 48-inch bar.

Volume change tests made on 4- by 4- by 40-inch bars indicated that the contraction of the concrete after one year's exposure in air was as low or lower than concrete made from other aggregate on similar jobs.

Thermal properties.[8]

The values for thermal conductivity, specific heat, and diffusion constant were obtained upon the full mass concrete mix with 1½-inch to 3-inch aggregate replaced by the same weight of ¾-inch and ⅜-inch sizes. The general procedure in manufacturing specimens was to crush larger aggregate to the limiting 1½-inch size, but in this case the coarse aggregate was considered to be so uniform mineralogically that a substitution of sizes would not alter its thermal properties.

Excellent agreement was noted between individual specimens when tested for specific heat, and results were about average when compared with other concretes tested. The thermal conductivity was somewhat above average and the value for diffusivity was quite high.

Temperature and heat generation.[8]

In the sample tested the initial temperature of the fresh concrete was 70.9 degrees Fahrenheit, and in a 28-day adiabatic period the temperature increased a total of 71.2 degrees Fahrenheit, resulting in a maximum temperature of 142.1 degrees Fahrenheit. This was an unusually high figure for mass concrete and was occasioned by the higher cement content of 1.18 barrels per cubic yard compared to the more customary barrel per cubic yard used in Boulder and Norris Dams mixes.

Computing the heat generation from the temperature rise, a total of 83.2 calories per gram was found to have been produced by the modified cement at the end of 28 days. From tests made on six individual modified cements, using Boulder Dam aggregates, an average heat of hydration of 80.1 calories per gram was obtained. The difference between this value and that obtained on the modified cement in the Wheeler Dam temperature rise test lies in the slight difference in cement samples. Their calculated potential heats, from their compound compositions, are in the same relation as obtained upon concrete adiabatically cured.

Temperature tests.

During the winter of 1935-1936, resistance thermometers were placed in blocks N–41 and N–42 to determine the temperature rise of the concrete. Block N–41 was placed in 5-foot lifts, while block N–42 was placed in 10-foot lifts, the purpose being to determine, if possible, any differences in temperature rise due to the height of the lifts.

A concrete placing schedule was worked out for the two blocks, but, because of cold weather and high water encountered at this time, it was not possible to follow the schedule as outlined. The schedule was different for the two

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[7] Data of Portland Cement Association tests taken from *Basic Principles of Concrete Making*, by F. R. McMillan.

[8] Vidal, E. N., and Meissner, H. S., *Concrete Investigations for Wheeler Dam, Tennessee Valley Authority*, U. S. Bureau of Reclamation Technical Memorandum No. 440, February 25, 1935.

blocks. Block N–41 was brought up in advance of block N–42, thus allowing the heat to be dissipated into the air through all four sides of block N–41 during the first three or four weeks; whereas block N–42 was placed between blocks N–41 and N–43, allowing heat dissipation into the air through the upstream and downstream faces and into the concrete in blocks N–41 and N–43 through the two inside faces. When the concrete temperature in the 5-foot lifts of block N–41 was reaching its peak, the additional heat from blocks N–42 and N–40 undoubtedly raised the temperature still higher. This probably accounts for the generally higher temperature rise obtained in the 5-foot lifts of block N–41.

While the tests did not show any satisfactory results in the comparison between 5- and 10-foot lifts owing to the conditions mentioned above, they did show that the thickness of section had a considerably greater effect on internal temperatures than did the height of the lifts.

The tests indicated that in a dam of comparatively low height, where the section is small compared to a section of a dam such as Norris or Boulder, the concrete temperatures become stable in a comparatively short time and thereafter are affected mostly by the temperature of the air.

To obtain satisfactory results in a study of temperature effects, it is suggested that the tests be started in the spring or early fall of the year, when air temperatures are more uniform. It is also suggested that, if a study is to be made to determine the effect of the height of lift, the concrete of three adjoining blocks of each type be placed together in order to take into account the effect of the adjoining concrete. In other words, if a 10-foot lift is to be studied, the block containing the thermometer should have blocks poured in 10-foot lifts on either side of it; and if a 5-foot lift is to be studied, the adjoining blocks also should have 5-foot lifts.

Figure 140 shows the effect of thickness of sections, in blocks N–41 and N–42, on the time required for the temperature of the concrete to fall from its maximum to 70 degrees Fahrenheit.

The curve merely illustrates the comparative effect of the heat-retaining qualities of a block of concrete, the thickness of which increases from top to bottom. It should be used only in a general way, as there are too many factors influencing dissipation of heat in blocks of such comparatively thin sections; that is, thin compared to such dams as Norris, Boulder, and other high gravity dams. Among the more important factors affecting the rate of dissipation of heat are the placing temperature of the concrete, air temperature following the placing of concrete, type of cement, thickness of sections, time allowed between lifts, and the effect of adjoining concrete.

A few resistance thermometers were placed in one of the 5-foot lifts of block N–22, which has the same dimensions as blocks N–41 and N–42. Block N–22 was placed in the late summer and early fall during comparatively warm weather, whereas blocks N–41 and N–42 were placed in the late fall and through

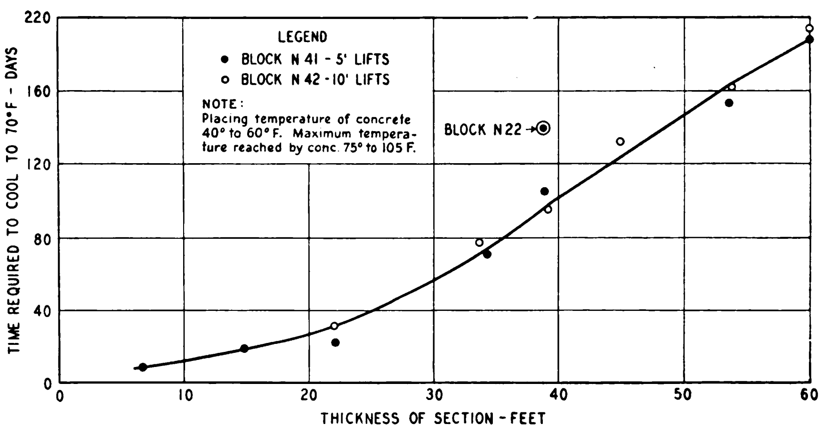

FIGURE 140.—*Effect of section thickness on dissipation of heat.*

the winter during comparatively cold weather. The placing temperature of the concrete going into block N–22 at elevation 508 was 82 degrees Fahrenheit, whereas the placing temperature of the concrete going into blocks N–41 and N–42 was 45 and 41 degrees Fahrenheit, respectively. The air temperature for 30 days following the placing in block N–22 ranged from 43 to 104 degrees Fahrenheit, whereas the air temperature for blocks N–41 and N–42 for 30 days following the placing ranged from 17 to 76 degrees Fahrenheit. The highest temperature reached by block N–22, at elevation 508, was 117 degrees Fahrenheit, while the highest temperature reached by blocks N–41 and N–42 at the same elevation was 105 and 93 degrees Fahrenheit, respectively.

HYDRAULIC MODEL STUDIES FOR THE DESIGN OF DRAFT TUBES

A study[9] was made by the United States Bureau of Reclamation of the performance of the draft tubes and of the tailwater conditions in the vicinity of the power plant.

The enormous size of the unit in this installation made it desirable to use a draft tube design which would be efficient and at the same time be relatively inexpensive to install. The elbow type, having these qualities, as evidenced by recent practice, was favored. In order to further reduce the installation cost for this design, it was proposed to shorten the vertical section by reducing the length of the conical section immediately beneath the runner. A seemingly insignificant change in the shape of a draft tube may have far-reaching effects upon its performance. It was therefore decided to make a series of comparative tests on models of various draft tube designs in order to ascertain as nearly as possible what results might be expected from the proposed alteration.

It was impossible in the short period of time between the decision to make these tests and the scheduled placing of concrete in the powerhouse substructure to conduct a series of highly refined tests. The results obtained, however, were consistent and reasonably accurate.

Purpose of tests.

In any medium- or low-head plant, the effectiveness of the draft tube in utilizing the energy of the water flowing through it is of much importance because every foot of head regained may be an appreciable percentage added to the operating head of the plant. In general, Wheeler Dam is of the usual design of low-head plant except that it is larger in every respect. The runner discharge diameter considered in these studies was approximately 22.25 feet, and as the ratio of discharge diameter to depth of draft tube is usually from 1:2.2 to 1:2.5, the problem of excavation for draft tube in this case was a serious one. In addition, it was proposed to use propeller or Kaplan-type runners, which discharge the water at high velocities, leaving much energy to be regained by the draft tubes. Available data on draft tube tests indicated that some energy was recovered by increasing the length of the horizontal section of an elbow draft tube, especially at high angles of whirl at the runner discharge. The length of the draft tube, therefore, involved the question of economic balance, the data for which could be obtained only through model experimentation.

In order to reduce cavitation to a minimum, the runners are set very low, and this fact, together with the large size of runner, makes great depth of excavation necessary for the draft tubes. It was therefore suggested that the outflow end of the elbow draft tube be sloped upward. Before making this

As the efficiency of energy regain from the high velocity at the runner discharge was the criterion for the draft tube tests, it was necessary to test the various types of tubes under different conditions for efficiency and performance. Among the draft tube designs submitted for tests, some provided for one pier change, it was thought desirable to investigate the limits of this slope.

and others for two piers in the horizontal section. It was entirely possible that one pier might work more satisfactorily in a given design, while two piers might give decidedly better results in another design. It was therefore thought advisable to make a comparison of the two conditions in this series of tests.

[9] Hornsby, G. J., *Hydraulic Model Studies for the Design of Draft Tubes for Wheeler Dam*, U. S. Bureau of Reclamation Technical Memorandum 456, May 21, 1935.

The river channel immediately below the powerhouse is very shallow so that, when all runners were discharging at full capacity, the tailwater attained a much higher velocity than that of the draft tube discharge. This created a condition such that the water left the draft tube at one velocity, passed through the tailrace at a much lower velocity, and again increased to nearly twice the draft tube discharge velocity as it flowed out into the river bed. The question arose as to whether this was a desirable condition or whether corrective measures from a standpoint of economy should be considered.

The purpose of the tests, therefore, resolved itself into five different points of consideration:

1. To determine the most economical height for the horizontal section of the draft tube.
2. To ascertain the effect of sloping the horizontal section of the draft tube upward beyond certain limits.
3. To compare the efficiencies of various types and designs within the limits described in (1) and (2).
4. To compare the performance of the various types of draft tubes with two piers versus one pier.
5. To determine the effect of energy regain by allowing the draft tube outflow velocity to be less than the velocity of the river bed below the power plant.

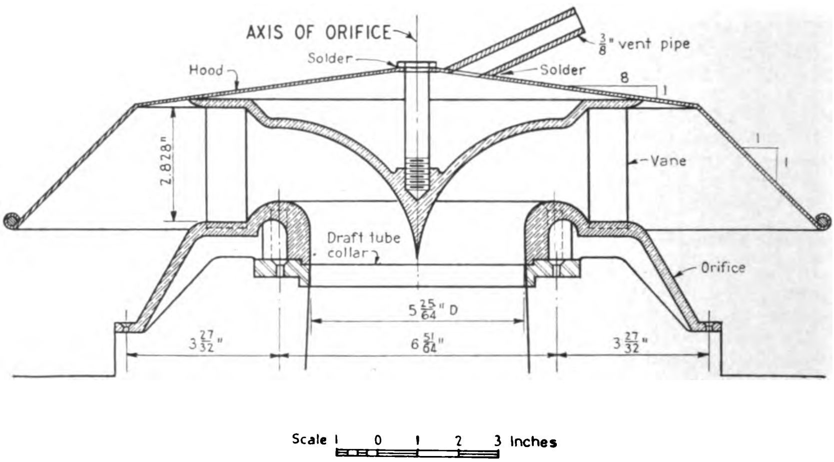

FIGURE 141.—*Orifice and vane details.*

The model.

To conduct the above tests, a model, with necessary upstream and downstream sections, was constructed to a scale of 1:50. Gages, weirs, and water circulating equipment were provided where required.

Since it was impractical to construct a model exactly like the prototype, the action of the water discharging from the propeller blade was simulated by use of deflector vanes. These deflector vanes were soldered to a flat ring which was made to fit in a groove of a casting made of brass and machined to conform to the lower gate ring and in about the relative position of the speed ring. The crown plate was machined to conform to the flow lines of the water entering the runner. Four sets of vane and cover-plate assemblies were used; one with vanes in radial position, one with vanes 15 degrees off the radial, one with vanes 30 degrees off the radial, and one with vanes 45 degrees off the radial. Each angle was turned on the arc of a given radius from the axis of the orifice. The deflector vanes were placed so as to give the water a whirl in a clockwise direction. Twenty vanes, equally spaced, were placed in each deflector vane set and the sets were interchangeable. Over the deflector vanes was placed an inverted bowl to exclude air at low head. Details of the orifice and deflector vane are shown in figure 141.

Models tested.

Six basic draft tube designs were tested in these studies. Draft tube design A is shown in figure 142. All of the designs tested except design E were similar. The principal differences in the designs of the various types of draft tubes are outlined below:

A–1. Same as design A except the outflow edge of the roof was lowered to reduce the outflow area from 84.1 square inches to 74.4 square inches. This was done for the 100-foot prototype length as well as the 77-foot length.

A–2. Same as design A except that the roof was curved upward from the point of tangency at the downstream end of the elbow, thus increasing the discharge area at 77-foot prototype length from 71.17 square inches to 100.28 square inches, or 32 percent. This was 54 percent greater than design A–1 at 77-foot prototype length.

A–3. Same as design A except that a splitter was introduced into the horizontal section.

B. Differed from design A principally in that the horizontal section had a steeper upward slope and the areas were slightly less than corresponding areas in design A.

C. Similar to design A, the greater difference was that the height of the downstream end of the elbow was much less than that in design A.

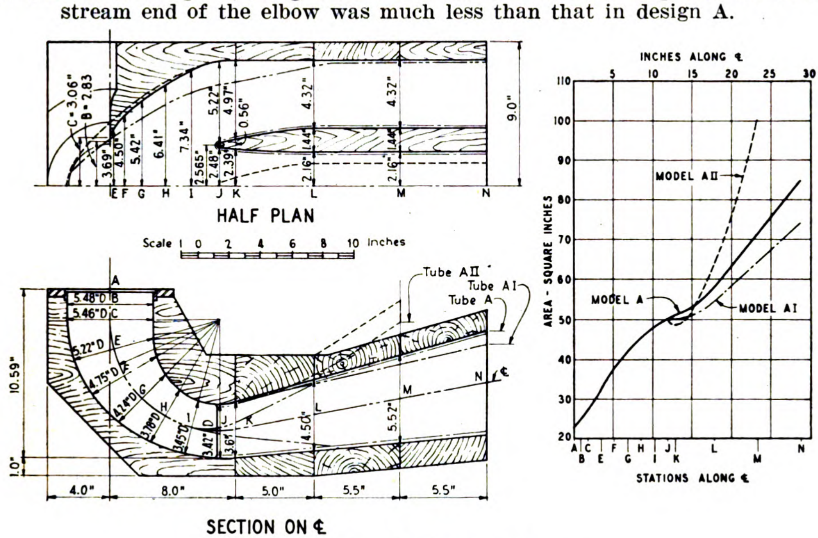

FIGURE 142.—*Draft tube design A.*

D. Similar to design A, the principal difference was that the height at the downstream end of the elbow was greater than in design A.

D–1. Similar to design D except that a splitter was introduced in the elbow in a manner similar to European practice.

E. Radically different from any of the other designs tested. It consisted of a vertical barrel and a horizontal section with an easy transition from one to the other, in which a sort of hydraucone action was supposed to take place.

F. The elbow section was identical with design B. The horizontal section was similar except that it had a much steeper slope than any of the other designs.

All draft tube models were made of wood to facilitate construction and to make alterations easier. The elbow sections were split in a vertical plane through the transverse center line of the unit. The horizontal section was generally divided in a plane lying midway between the floor and the roof. Considerable difficulty was encountered with these wooden models due to swelling. Corrections were made and check runs ascertained the true performance of the models.

Steel rings or collars were made to fit the discharge side of the orifice casting and a rubber gasket was used to make a tight seal. These rings were fitted to the throat of the models when constructed and when installed were bolted tightly to the orifice outlet, thus holding the model in place for testing.

A row of piezometer openings, all connected together to give an average value, was placed along the extended center line of the draft tube model on the slope representing the tailrace excavation. Another row was placed in like position on the center line of an adjacent unit. As both gave the same results, the one placed along the center line of the operating unit was used in these tests.

Performance of model.

In order to determine the amount of work being done by the draft tube models, it was necessary to measure the head between the forebay and the tailwater level, and the quantity of water flowing.

Runners of a conventional design when operating at or near the point of best efficiency discharge the water in an axial direction. When overloaded, the discharge is spiral in a direction opposite that of the runner and vice versa when underloaded. As no runner was used in these tests, the vanes were set in the crown plate for the purpose of simulating the discharge conditions of

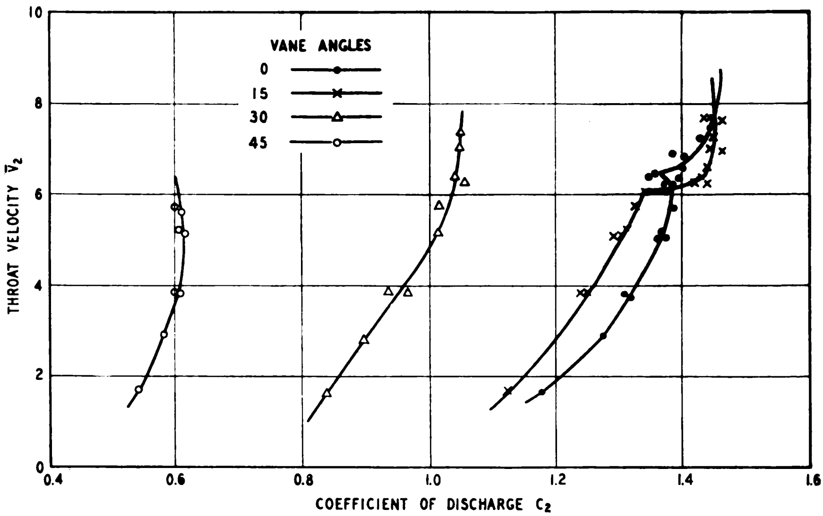

FIGURE 143.—*Relation of typical throat velocity to coefficient of discharge—Tube A.*

runners operating at various gate openings and constant speeds. These were referred to as "vane angles" and were fixed at 0, 15, 30, and 45 degrees. As the models were symmetrical, the vanes were set to cause the water to whirl in a clockwise direction only.

In order to arrive at a basis of comparison for the effectiveness of the various draft tubes, it seemed best to compare their absolute efficiencies. Methods of arriving at efficiencies of draft tubes indicated in their derivation that certain data, which would not be practicable to obtain in the original set-up and with the time available, would be required to compute the absolute efficiencies of draft tubes. Since the data obtainable was only sufficient to give a comparison of efficiencies of the draft tubes and as the coefficient of discharge (C_2) would give the same thing, it was decided to make the comparison on this basis.

The discharge coefficient (C_2) was determined from the formula

$$C_2 = \frac{Q}{A\sqrt{2gh}}$$

where Q is the quantity of water flowing in cubic feet per second, A is the area of the draft tube throat in square feet, and h is the head on the model in feet. The throat velocity in feet per second was calculated from the formula

$$\overline{V}_2 = \frac{Q}{A}$$

The coefficient of discharge (C_2) was calculated for the different vane angles and for different throat velocities ranging from about 2 feet per second to 7.5

feet per second on the model. The normal model velocity was 4 feet per second to correspond to the normal velocity of 27 feet per second in the prototype.

A throat velocity versus discharge coefficient curve was plotted for each vane angle for each draft tube tested. Inasmuch as it is generally conceded that the discharge coefficient and the effectiveness of the draft tube is independent of the velocity above a certain point, a velocity of 6 feet per second was selected as a point at which to compare the effectiveness of the model draft tubes in these tests. Figure 143 shows a typical curve.

Taking a throat velocity of 6 feet per second for each vane angle, C_2 was plotted against angle of whirl in degrees. These curves showed coefficient of discharge from the draft tubes for angles of discharge ranging from 0 degrees whirl up to 45 degrees.

Calculations were made and curves plotted showing a comparison of discharge coefficients for each draft tube model tested at 54-, 77-, and 100-foot lengths, using one and two piers, insofar as it was considered advisable to test all three lengths. A comparison of the performance of all draft tube models tested with two piers at 77-foot length, except design F, is shown in figure 144; and a comparison of all draft tube models tested with one pier at the 77-foot length is shown in figure 145.

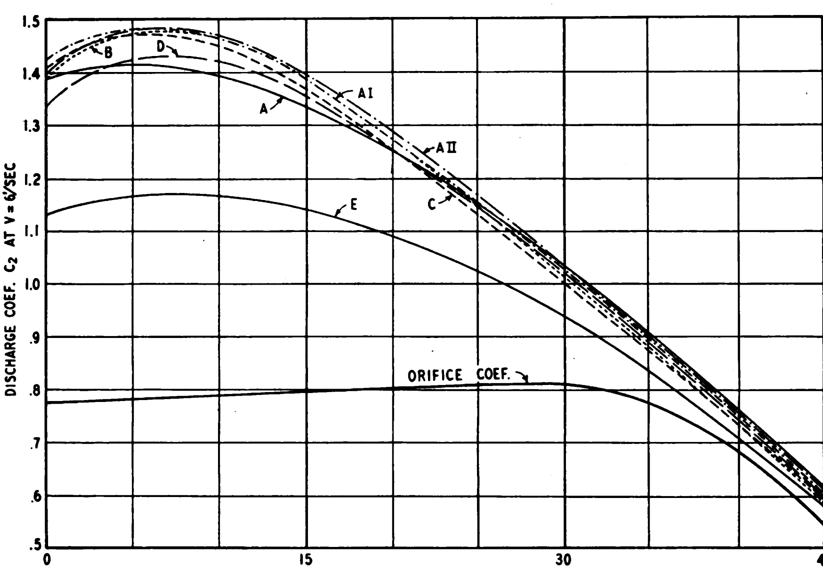

FIGURE 144.—*Comparison of discharge coefficients, Tubes A, A–I, A–II, B, C, D, and E. Prototype length = 77 feet. Two piers equally spaced.*

At the discharge end of draft tube design B using 77-foot length the outflow passages were raised 4 feet. This made it possible to reduce the necessary tailrace excavation by this amount. The floor representing the tailrace excavation in the model was raised correspondingly for the test, and this had to be considered in evaluating the merits of this draft tube. The floor was not raised in the case of design F.

Internal pressure and flow conditions.

In draft tubes the size of those required, the stresses imposed by inside pressure conditions are considerable. Pressure tests were made on draft tube designs A, F, and F–1. A row of piezometer openings was placed along the center line of the roof and one along the center line of the floor, both rows extending from the outlet back through the horizontal section and up to the throat. A row of piezometer openings was also placed in the roof of each of the side passages of the horizontal section. Pressure readings were taken on each model operating at normal discharge of 10,500 cubic feet per second

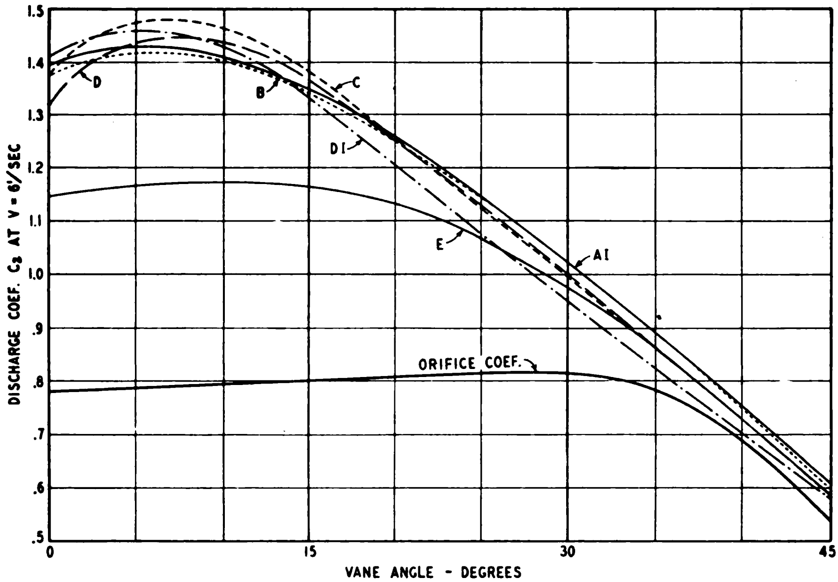

FIGURE 145.—*Comparison of discharge coefficients, Tubes A–I, B, C, D, D–I, and E. Prototype length = 77 feet. One pier in center.*

and at 0-, 15-, 30-, and 45-degree angles of whirl. The results for tube A are shown in figure 146. It is interesting to note the changes in pressure in the side passages as the angle of whirl increased.

Paint tests were made on design A–2. Where the velocities were sufficiently high, the flow lines etched themselves into the fresh paint. Where eddies occurred, there was a piling up of paint. Also, at the point where the water failed to flow, the surface of the passage wall was clearly marked. It must be kept in mind that flow lines indicated only the surface conditions and that the internal conditions may be entirely different.

Tailrace flow conditions.

In order to study flow conditions in the tailrace and on the river bed as the water left the tailrace excavation, profiles of the water surface and tailrace excavation were made. The various conditions tested are shown in the following tabulation:

Test No.	Unit No.	Discharge (per unit) cubic feet per second	River surface elevation	Remarks
1	1	10, 500	505	No restriction on left bank.
2	1	10, 500	505	Retaining wall installed, extending straight downstream from the left side of draft tube outlet.
3	3	10, 500	505	Unit No. 3 operating alone.
4	All 8	10, 500	505	All 8 units operating at normal discharge.
5	All 8	10, 500	508	All 8 units operating at normal discharge.
6	All 8	10, 500	512	All 8 units operating at normal discharge.

When only one unit was operating, the water spread over the river bed as it flowed from the tailrace so that it was difficult to determine the velocities. From observation, however, it was apparent that the velocity was very near the critical velocity at a point in line with the draft tube outlet.

With all units operating at normal discharge of 10,500 cubic feet per second each, and with the river water surface at elevation 505, a distinct hydraulic jump occurred just below the edge of the tailrace excavation. As the tail-

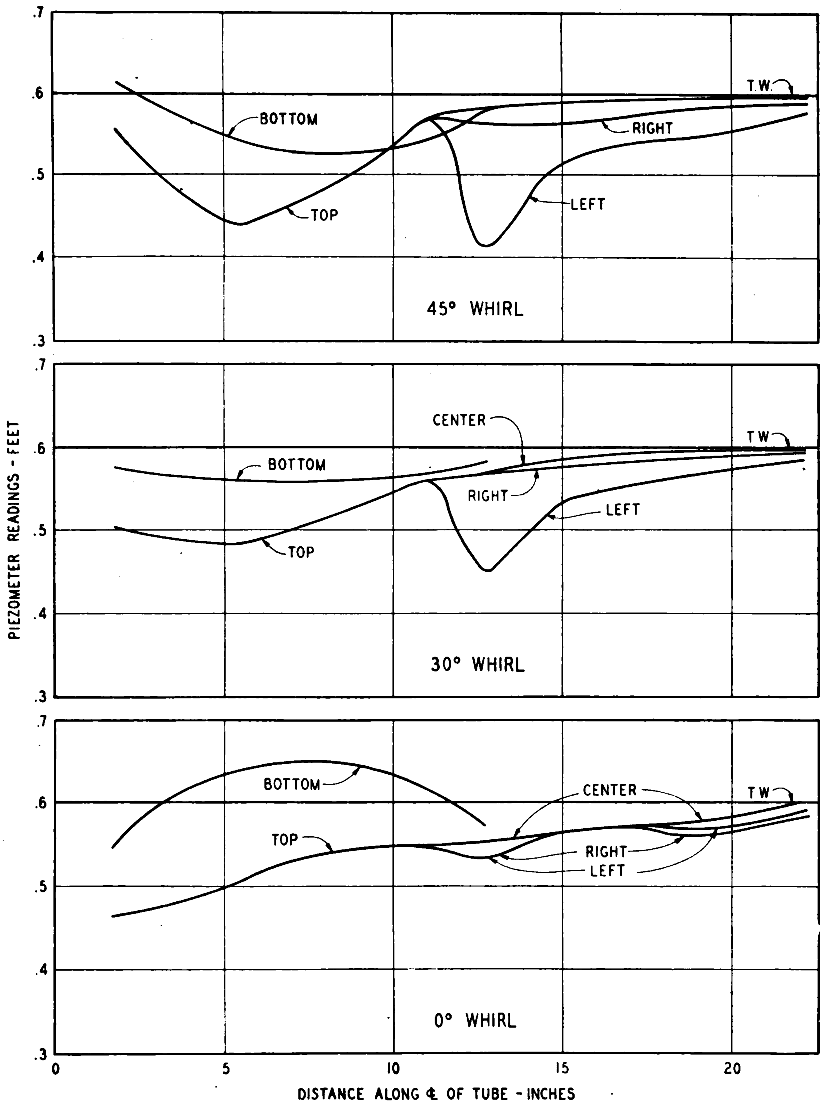

FIGURE 146.—*Internal pressure—Design A.*

water level rose, the hydraulic jump moved farther upstream and finally disappeared at the point of critical flow, where

$$V=\sqrt{gd}$$

g being acceleration of gravity in feet per second and d the depth at critical point in feet. This condition can do no harm insofar as the efficiency of the plant is concerned. On the other hand, it might do much good by cutting out the river bed below the tailrace excavation and thus lower the level of the water at the draft tube outlet.

Results of these tests indicated that it would be desirable to raise the elevation of Wilson Dam pool in order to increase the tail water depth at Wheeler Dam. This would decrease the operating head at Wheeler but would increase the head at Wilson.

Conclusion.

These tests indicated that to increase the length of the horizontal section of the draft tube from 77 to 100 feet increased its efficiency. It was doubtful, however, that this increase would be sufficient to outweigh the additional cost of constructing the additional length of the horizontal section of the draft tube. From these tests, it seemed that the limit of upward slope for the horizontal section of the draft tube of this particular design was reached. Design B and design F were similar except for the slope of their horizontal sections. Design B had a slope of about 13 degrees, while design F had a slope of about 18 degrees. The results indicated that design B was slightly superior to design F. This may indicate that the upper slope limit has been reached for this particular design of draft tube, but it does not indicate that it has been greatly exceeded. It is believed that the permissible slope depends a great deal upon the design of the elbow section and that much may be gained by an investigation of this feature.

It may be seen from the discharge coefficient-throat velocity curves (see fig. 144) that a very unstable condition developed when the throat velocity reached a value of approximately 6 feet per second. A great many check runs were made at this point to determine the reason for this break in the curve, and it was concluded that the cusp was caused by vortices which formed in the forebay and extended into the throat of the model. To correct this would have necessitated a major alteration in the flume. A better way might have been to use V_2 at a higher value for the C_2-vane angle curves. This would have necessitated raising the height of the flume to give the required value for V_2.

Designs A–1 and B both showed better performance with two piers, while designs C, D, and E showed better performance with one pier.

After the water leaves the draft tube outlet and the shock loss caused by the sudden change in velocity is sustained and accounted for, any other changes in velocity where the water is unconfined will not affect the head on the turbine. The head required to supply the energy for the hydraulic jump obtained under some conditions of test may be added to the gross operating head of the plant by excavating the river bed to a depth sufficient to keep the velocity less than that of critical flow as it leaves the tailrace excavation.

Taking the sum of the heads on Wheeler Dam and Wilson Dam into consideration, it would seem that by raising the pool elevation of Wilson Dam within certain limits, while decreasing the head on Wheeler Dam a lesser amount, the sum of the two heads will increase. Little advantage is to be gained by raising the pond elevation of Wilson Dam more than 4 feet.

APPENDIX D

PERSONNEL

Organization.

The chart shown in figure 147 represents the organization of the Authority and particularly the departments for water control in the river channel at the close of the project. Several organizational changes occurred during the construction period and this chart shows the final arrangement effected. Resolutions of the Board of Directors giving the Authority's organization structure in detail are shown on pages 334 to 365 of the annual report of the Authority for the fiscal year ended June 30, 1937.

Personnel.

Wilfred M. Hall, construction engineer, and Lee H. Huntley, construction superintendent until February 1935, succeeded by George P. Jessup, construction superintendent from February 1935 until the end of the job, were in direct charge of the engineering and construction connected with the dam, lock, and powerhouse, and related features. The other principal persons contributing to the Wheeler project are listed on pages 347 to 352. This list has been limited to those whose responsibility placed them in policy-making positions, insofar as those positions affected the Wheeler project. Included in the list are a number of employees not directly connected with the actual construction work and who did not devote their efforts entirely to the Wheeler project but who served in like capacities on several construction projects. It is regretted that space does not permit the listing of all persons who were identified with the project.

Wage scale.

The hourly wage scale was adjusted from time to time so that it might be kept consistent with wages existing in the Tennessee Valley region. The different rate schedules in effect during the construction period are shown in table 36, pages 353 and 354.

Employee relationship policy.

The policy governing the relationship between the Authority and its employees was adopted by the Board of Directors as a result of long study and following a series of discussions in which both the employees and management participated. This is given on pages 804–7 of the report on the Norris Project, Tennessee Valley Authority Technical Report No. 1.

BOARD OF DIRECTORS[1]

Arthur E. Morgan, chairman, chief engineer until June 17, 1937.
Harcourt A. Morgan, vice chairman.
David E. Lilienthal, director.

GENERAL MANAGER

John B. Blandford, Jr., coordinator (until May 1936), general manager (after May 1936).

ENGINEERING AND CONSTRUCTION DEPARTMENTS

Carl A. Bock, assistant chief engineer.

PLANNING DEPARTMENT

Sherman M. Woodward.

PROJECT PLANNING

James S. Bowman.
Clifton T. Barker.
David E. Donley.
George W. Foster.
Roland A. Kampmeier.
Joseph H. Kimball.
Edward J. Rutter.
Arthur Schweier.
William L. Voorduin.
Gabriel O. Wessenauer.
Dana M. Wood.

[1] James P. Pope succeeded Arthur E. Morgan as director on January 27, 1939. Harcourt A. Morgan was made chairman of the board March 23, 1938, and David E. Lilienthal vice chairman January 27, 1939

Geology

Edwin C. Eckel.
Nicholas A. Rose.
Ernest L. Spain, Jr.

Hydraulic Data

Albert S. Fry.
Gene N. Burrell.
Van Court M. Hare.
Benham E. Morriss.
Jackson H. Wilkinson.

Inspection and Testing

Perry J. Freeman.
Frank W. Groh.
Edgar R. Kendall.
Franklin H. Stamps.
Charles D. Susano.

Engineering Services

Ned H. Sayford.
Harry Wiersema.

Surveys

George D. Whitmore.
Felix W. Truss, assistant.
Roscoe W. Anderson.
Dewitt C. Bishop.
William H. Keen.
Clarence C. Miner.
Paul Morris.
Melvin C. Thomas.
Charles H. Wright.

Drafting and Mapping

Harry P. McKean.
Thomas Benson.
Paul C. Klyce.
Jesse E. Means.
Frank W. Ray.
Harry Tubis.

Land Purchase Control

Robert E. Frierson.
Paul F. Meredith, assistant.

Railroad Engineering

Henry L. Fruend.
Robert H. Gleaves.
Clifford C. Muhs.
J. Butler Sullivan.

Cemetery Removal

Fred W. Wendt.

Utility Relocation

H. Jervey Kelly.

DESIGN DEPARTMENT

Project designed by United States Bureau of Reclamation with some participation by the following TVA forces.

Verne Gongwer, liaison officer.

Civil and Structural

Ross M. Riegel.
Bernard R. Fuller.
John A. Howe.

Electrical

Raymond A. Hopkins.
Edwin P. Almond.
Harry B. Barnhill.
Richard E. Behnke.
Joseph P. Kennell.

Mechanical

William R. Chambers.
Charles F. Ellis.
James M. Lloyd.
Frank H. Moore.
Donald H. Mattern.
J. Frank Roberts.

Highway Engineering

Frank W. Webster.
James E. Moreland, assistant.
Erwin Harsch.
E. Dudley Jeffries.
Francis E. Junior.

CONSTRUCTION DEPARTMENT

Theodore B. Parker, chief construction engineer (after November 1935).
Ross White, Superv. construction superintendent (after April 1935).

Dam and Powerhouse Engineering

Wilfred M. Hall, construction engineer.
Assistants:
- George K. Leonard (December 1933 to March 1936).
- Franklin E. Bell (after March 1936).

Office Engineering

Joseph P. Laws.

Special Reports

Walter E. Bundy.

Field Engineering

Joseph C. Gilbert.

Cost Engineering

William J. Pollock.
Raymond L. Adams.
Carl J. Hartman.
Harry G. Worsham.

Lay-Out and Inspection

Joseph K. Black.
James C. Blalock.
Henry L. Bodfish.
Claude F. Howison.
Harold W. Hunt.
Joel B. Justin.
Eugene P. Kavanaugh.
Richard E. Meade.
Sampson A. Powers.
Gerard Rozendale.
John Stewman.
Charles E. Wattles.

Electrical Engineering

Franklin E. Bell.
William P. Fegley.
Giles U. Mason.
Theodore R. Mausen.

Drafting Room and Design

Wiliam J. Bailey.
J. Roscoe Sharp.

Materials Engineering

William R. Johnson.
John S. Kennoy.
Earl V. Neiswanger.

Equipment Materials

Milton E. Ober.
Milo Singer.
Coordinator, sand and gravel contracts, Walter E. Bundy.

Dam and Powerhouse Construction

Construction superintendent:
- Lee H. Huntley (November 1933 to February 1935).
- George P. Jessup (February 1935 to September 1937).

Assistants:
- Benjamin S. Philbrick (December 1933 to December 1935).
- William McLean (February 1935 to December 1936).
- Arthur W. Sherman (April 1935 to March 1936, December 1936 to June 1937).
- Burt M. Sloan (February 1934 to October 1936).

MECHANICAL

Detler H. Roby (May 1934 to April 1936).
Henry M. Stafford (May 1934 to December 1935).
Ray D. Rockefeller (May 1936 to September 1937).

CONCRETING

Robert V. Sass.
Herbert H. Jones.

RIGGING

Joseph F. McNeill (May 1934 to March 1936).
John R. Hite (May 1936 to July 1937).

CARPENTRY

Frank Healy.
Thurston R. Koonce.
Marshall S. Plier.

COFFERDAMS

Homer W. Penney.

REINFORCING STEEL

Robert B. Brannin.
Almos L. Foster.

PAINTING

Brice M. Thompson.

ELECTRICAL

George C. Dunagan (February 1934 to December 1935).
Lewis M. Thompson (January 1936 to July 1937).

CONCRETE FINISHERS

William V. Byrne.

LABOR

Horace E. Nunley.
Frank Austin.
Walter K. Johnson.

MARINE

Conway Graden.
John Thomas.

WAREHOUSE

William E. Wiechers.
Owen E. Elliott.
William H. Patterson.

CLERICAL

Wallace S. Wooten.
Oakley Z. Brown.
Earle W. Mullan.

CAMP

Superintendent:
Alexander W. Jordon (August 1934 to December 1935).
Henry B. Sherrill (March 1934 to July 1937).
Lorenza C. Chapman, chief of police.

CONSTRUCTION PLANT

Adolph J. Ackerman.
Robert T. Colburn, assistant.
Operations, Philip H. Kline.
Design, Robert T. Colburn.
Costs, Howard P. Maxton.

MISCELLANEOUS CONSTRUCTION

Raymond H. Foss.
Leslie R. Ancill.
John C. Burns.
Joseph B. Davis
Thomas D. Lebby.
Lee M. Ragsdale.
Irving L. Treat.

RESERVOIR CLEARANCE

Louis N. Allen.
Howard E. Davis.
William R. Holden.
Pat A. Miller (January 1934 to September 1935).
Jake W. Williams (September 1935 to September 1936).

GENERAL OFFICE ENGINEER

Olaf Laurgaard (February 1936 to January 1937).
Harry Wiersema (after January 1937)

BUDGETS

Burgess B. Brier.
Albert B. Wilkinson.

ORGANIZATION AND REPORTS

George E. Tomlinson.
Earle B. Butler.
Thomas G. Harton.
Charles M. Turner.
Harold B. Vasey.

CONSULTANTS

ENGINEERING, GEOLOGY, AND CONSTRUCTION

Charles P. Berkey.
Albert J. Eldridge.
Ozro N. Floyd.
Leonidas C. Glenn.
George W. Hamilton.
Leroy F. Harza.
Barton M. Jones.
Charles H. Locher.
Charles H. Paul.
John L. Savage.
Sherman M. Woodward.

DECATUR ADJUSTMENTS

Robert L. Clare.
John W. Whittemore.

RESERVOIR FAMILY REMOVAL

William G. Carnahan.
Alto Lee Snell, assistant.

LAND ACQUISITION

John I. Snyder.
Allen J. Roulhac, assistant.
Joe L. Burdette.
Robert J. Coker.
Leo L. Cole.
William P. Hamphill.
John W. Newman.
W. Edward Sanford.
Harrison C. Underwood.

LAND PLANNING AND HOUSING

Earle S. Draper.
Tracy B. Augur, assistant.
Architecture, Roland A. Wank.

ARCHAEOLOGY

William S. Webb.
T. Levron Howard.

FORESTRY

Edward C. M. Richards.
Bernard Frank, assistant.

POWER OPERATIONS

J. A. Krug.
Aaron H. Sullivan.
Charles L. Karr.
W. Warren Woodruff.
Percival E. Clement.

PERSONNEL

Floyd W. Reeves (until January 1936).
Gordon R. Clapp (after January 1936).
Arthur S. Jandrey, assistant (after April 1937).

CENTRAL OFFICE

Classification

Milton V. Smith (until December 1936).
Carl L. Richey (after December 1936).

Employment

Carl L. Richey (until January 1936).
George Slover (after January 1936).

Labor relations

Clair C. Killen.

Personnel relations

Edwin B. Shultz.

Training

J. Dudley Dawson (until February 1, 1935).
Maurice Seay (after February 1935).

FIELD OFFICE

Norman D. Huff (until February 1936).
Luther E. Reynolds (until May 1936).
John H. Hales (until February 1937).
William C. Lindsey (after February 1937).
Roy Boyd.

PROCUREMENT OF MATERIALS

Charles H. Garity.
Assistants:
- Jack S. Beauchamp.
- Andrew H. Fickes.
- William J. Hagan, Jr.
- Richard F. McLaughlin.

Administrative, Clifford L. Edington.
Contracts, John G. Werneke.
Inspection, Frank J. O'Brien.
Property:
- Addison E. Hook.
- Roy S. Virtue.

Purchasing:
- John W. Almquist.
- Ralph Semmes.

Specifications, James H. Cheston.
Traffic, Louis B. Rockwell.
Warehouse and stores, Rufus Holland.

FINANCE

Frank J. Carr.
Paul W. Ager, assistant.
Jerry F. Stone, adviser.
Auditing, Anson J. Robertson.
Plant Records, William J. Pollock.
Construction Accounting, Fred L. Cavis.
General Accounting, Glenn P. Smith.
Treasurer:
- Mrs. Florentine D. Goodrich (until January 1937).
- J. Ed. Campbell (after January 1937).

LEGAL

James Lawrence Fly.
Evans Dunn.
William C. Fitts, Jr.
Herbert S. Marks.
Joseph C. Swidler.

HEALTH AND SAFETY

Eugene L. Bishop.
James A. Crabtree, assistant.

MALARIA CONTROL

Robert B. Watson.

ENVIRONMENTAL SANITATION

Walter G. Stromquist.

SAFETY

Paul F. Stricker (June 1934 to February 1935).
Donald F. McMurchy (after April 1935).
Construction Safety, H. E. Haley.
Public Safety:
- Kenneth A. Rouse (May 1935 to May 1936).
- R. Morris Hoisington (after May 1936).

MEDICAL STAFF

Dam:
- John J. Eberhart (January 1934 to March 1936).
- Walter T. Davis (May 1935 to October 1936).
- Charles B. Olim (October 1936 to April 1937).

Reservoir:
- Floyd G. Estridge (February 1934 to August 1934).
- Herschel Penn (July 1934 to October 1934).
- Eugene B. Glenn (August 1934 to September 1935).
- Gillian S. Hicks (September 1935 to September 1936).

OFFICE SERVICES

John F. Pierce.

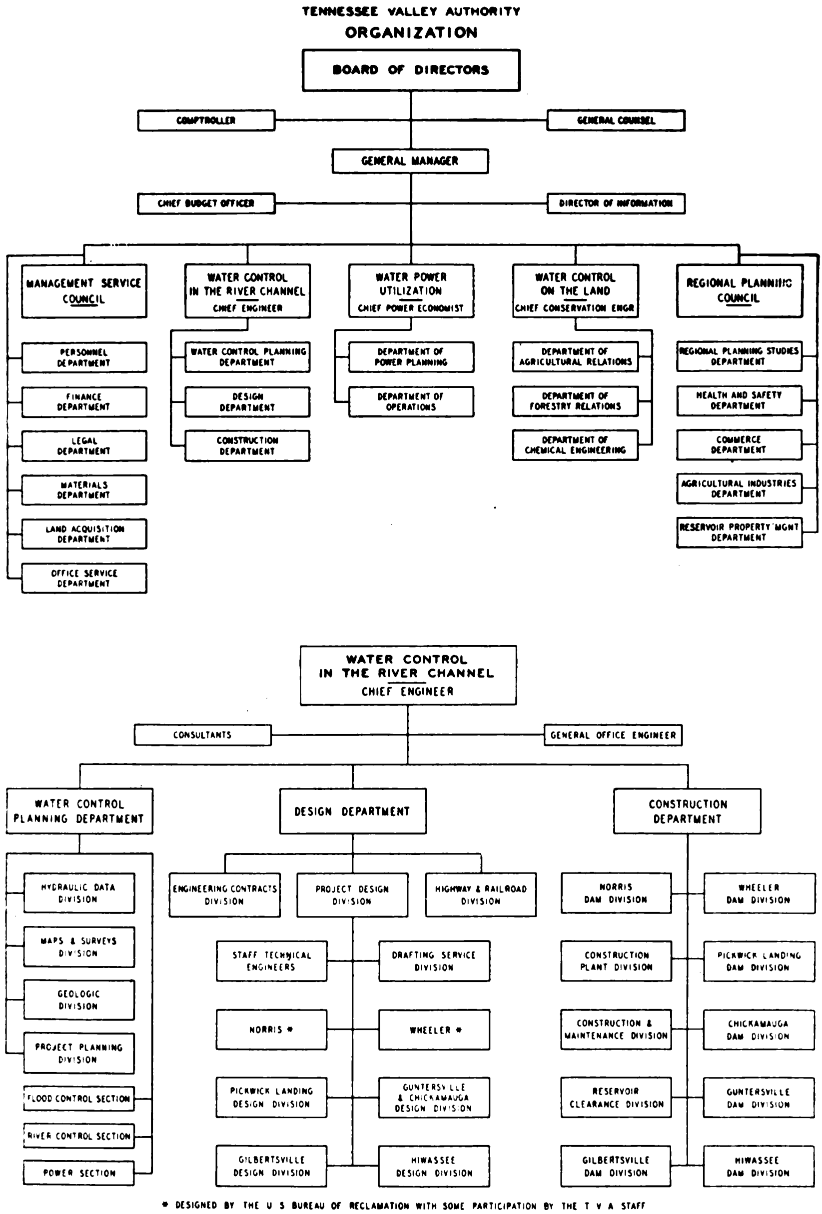

FIGURE 147.—*Organization chart.*

TABLE 36.—*Labor classifications and hourly rates of pay*

Unless otherwise indicated, rates shown were for all types of construction. Rates for "Dam" applied to only the dam structures, while "Miscellaneous" rates applied to such other work as buildings, highways, and railroads.]

Classification	Prior to Dec. 1, 1933	Effective Dec. 1, 1933	In effect Oct. 29, 1935 [1]	Effective Feb. 1, 1936	Effective Jan. 1, 1937
Apprentices and helpers, dam		$0.60-$0.75	$0.60-$0.75	$0.60-$0.75	$0.60-$0.75
Miscellaneous		.55	.60- .75	.60- .75	.60- .75
Asbestos worker (heat insulation)				1.00	1.10
Blacksmith		1.00	1.00	1.00	1.10
Boilermaker		1.00	1.00	1.00	1.10
Building trades, unclassified		1.00	1.00	1.00	1.10
Cableway operator				1.50	1.50
Carpenter (form and finish)	$0.60-$0.75	1.00	1.00	1.00	1.10
Compressor operator, dam		.60- .75	.60- .75	.60- .75	.60- .75
Miscellaneous		.80	.60- .75	.60- .75	.60- .75
Heavy duty compressor operator					1.00
Concrete finisher		1.00	1.00	1.00	1.10
Concrete mixer operator, dam		.75	.75	.60- .75	.60- .75
Miscellaneous		.80	.75	.60- .75	.60- .75
Concrete puddler		.45	.55	.55	.55
Concrete rodder and spreader		.75			
Core drill operator		1.00	1.00	1.00	1.00
Crane or derrick operator, dam		1.50	1.50	1.50	1.25- 1.50
Miscellaneous		1.25	1.25	1.25	1.25- 1.50
Dinkey operator		.75	.75	.75	.75
Drill-dresser machine operator		1.00	1.00	1.00	1.10
Electrician		1.00	1.00	1.00	1.10
Elevating grader operator				1.00	1.00
Fireman		.75	.75	.75	.75
Flagman				.60	.60
Foremen—labor	1.00	[2] .80 1.00	[3] .90 1.00	1.00	1.10
Skilled Trades		1.25	[2] 1.10 1.25	[3] 1.25 1.375	[4] 1.25 1.35 1.50
Foremen, sublabor	.75		.75	.75	.75
Skilled trades				[3] 1.10 1.20	[4] 1.10 1.20 1.35
Glaziers				1.00	1.10
Hoist operator:					
2 drums		1.00	1.00	1.00	1.10
1 drum		1.00	1.00	1.00	1.00
Iron worker, ornamental				1.125	1.25
Jackhammer operator, dam		.60	.60	.60	.60
Miscellaneous		.55	.60	.60	.60
Labor, unclassified	.30	.45	.45	.45	.45
Lathers		1.00	1.10	1.00	1.10
Le Tourneau operator					1.00
Lineman and armature winder				1.00	1.10
Locomotive operator		1.00	1.00	1.00	1.10
Machinist and general mechanic		1.00	1.00	1.00	1.10
Marine engineer:					
50 tons or over		1.00	1.00	1.00	1.10
Less than 50 tons		1.00	1.00	1.00	1.00
Marine pilot:					
50 tons or over			1.00	1.00	1.10
Less than 50 tons			1.00	1.00	1.00
Masons, brick and stone		1.00	1.00	1.125	1.25
Mason tender		.55	.60	.60	.60
Millwright		1.00	1.00	1.00	1.10
Mortar mixer		.55	.60	.60	.75
Motorboat and tugboat operator		.60- .75		.75	.60- .75
Nozzlemen—Sluicing		.60	.60		
Oiler		.60	.60	.60	.60- .75
Painters and decorators		1.00	1.00	1.00	1.10
Painters, sign				1.00	1.10
Pile driver		1.00	1.00	1.00	1.10
Pipefitter	.90	1.00	1.10	1.00	1.25
High pressure permanent				1.125	1.25
Plasterers		1.00	1.10	1.00	1.25
Plumber and steamfitter		1.00	1.10	1.125	1.25
Powder man		.75	.75	.75	.75
Pump operator, dam		.60	.60	.60-.75	.60 - .75
Miscellaneous		.80	.60	.60-.75	.60 - .75
Rigger	.70	1.00	1.00	1.00	1.25
Road machine operator		.80	.75	.75	.75
Road roller operator		.80	.75	.75	.75
Roofer		1.00	1.00	1.00	1.10

For footnotes, see end of table.

Table 36.—*Labor classifications and hourly rates of pay*—Continued

[Unless otherwise indicated, rates shown were for all types of construction. Rates for "Dam" applied to only the dam structures, while "Miscellaneous" rates applied to such other work as buildings, highways, and railroads.]

Classification	Prior to Dec. 1, 1933	Effective Dec. 1, 1933	In effect Oct. 29, 1935 [1]	Effective Feb. 1, 1936	Effective Jan. 1, 1937
Saw filer		$1.00	$1.00	$1.00	$1.10
Sewer layer		.55	.60	.60	.60
Shaft and tunnel miner		.75	.75	.75	.75
Sheet metal worker		1.00	1.00	1.00	1.10
Shingle splitter		.75	.75		
Shovel or dragline operator:					
Over ¾ yard, dam		1.50	1.50	1.50	1.50
¾ yard and under, dam		1.50	1.50	1.50	1.25
Miscellaneous		1.30	1.25	1.25	1.25 –1.50
Signalman (cableway)				1.00	1.25
Steam engine operator		1.00	1.00	1.00	
Steel worker, reinforcing (bending, placing, tying)		.75	1.00	1.00	1.00
Steel worker, structural (erecting, riveting, heating)		1.00	1.10	1.125	1.25
Teamster			.45 –.60	.45 –.60	.45 – .60
Tool dresser		1.00	1.00	1.00	1.10
Tractor and grader operator, dam		.75	.75	.75	.75
Miscellaneous		.80	.75	.75	.75
Trenching machine operator		1.25	1.50	1.00	1.10
Truck operator:					
All sizes, miscellaneous	$0.55	.55			
4-ton, dam		.75			
1½-ton, dam		.60			
Over 5-ton			.75	.75	.75
5-ton and under			.60	.60	.60
Wagon drill operator		.60 –.75	.75	.75	.75
Watchman	.45	.45	.45	.45	.45
Welder		1.00	1.00	1.00	1.25
For certified welding				1.125	

RESERVOIR CLEARANCE

Classification	Effective Dec. 1, 1933	In effect Oct. 29, 1935	Effective Feb. 1, 1936	Effective Jan. 1, 1937
Foremen, labor		$0.85	$0.85	$1.00
Foremen, sublabor	$0.625	.75	.75	.75
Labor:				
Skilled	.85			
Semiskilled	.50 –.625	.625	.625	
Unskilled	.375	.45	.45	.45
Log scaler	.625			
Machine operator		.85	.85	.85
Motorboat and tugboat operator		.625	.625	
Saw filer	.625	.70	.70	.75
Tallyman		.625	.625	
Timber rigger		.625	.625	.625
Tree climber		.625	.625	
Truck driver		.60	.60	

[1] Between December 1933 and October 1935, rates for individual classifications were changed as such changes were considered appropriate. All changes after October 1935 became effective as of the date shown.
[2] Rates shown for 8-hour and 5½- to 6-hour shifts respectively.
[3] Rates shown for supervising trades with $1 and $1.125 rates respectively.
[4] Rates shown for supervising trades with $1, $1.10, and $1.25 rates respectively.

APPENDIX E

CONSTRUCTION DRAWINGS

"Drawings for the Wheeler Project," issued separately by the Tennessee Valley Authority as Technical Monograph No. 42, August 1939. Price $2.50.

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