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The foundations of Lock and Dam no. 26 - Alton, Illinois

John Joseph Livingston

THE FOUNDATIONS

OF

LOCK & DAM NO. 26 --- ALTON, ILLINOIS

BY

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A

THESIS

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ILLUSTRATIONS

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INTRODUCTION

The first improvement of any consequence on the Mississippi River was in the line of flood control, and dates back to 1717 when the engineer who laid out New Orleans provided for levees along the river front. By 1735, forty miles of levees had been constructed above this city.

In its original condition, 100 years ago, the navigable channel of this section of the Mississippi River had a natural depth in many places of only three and one-half to four feet at low water. The channel was divided by islands forming sloughs and secondary channels or chutes through which much of the flow was diverted to the detriment of navigation.

In accordance with the findings of a board of engineers which made an inspection of the Ohio and Mississippi Rivers from 1820 to 1822, snagging was begun in 1824 as the first step in improving the Mississippi for navigation below St. Louis.

The first permanent improvement built to direct the river was constructed in 1836 in St. Louis harbor by Lieutenant Robert E. Lee, the first district engineer. Further like improvements were made there in 1837 and 1844.

The closing of sloughs and chutes by means of dikes as a policy for improvement was adopted by Congress and begun in 1872. Thereafter, as appropriations were made, regulation works consisting of dikes of brush and stone were erected with a view to confining the low water volume to a single channel. Revetments were constructed to preserve the banks where necessary.

The project of 1881 for the first time prescribed a definite depth to be maintained. It planned a concentration of flow by means of dikes and revetments at low water with a view to obtaining an eight-foot channel from Cairo to St. Louis and a six-foot channel from St. Louis to the mouth of the Missouri River.

In 1896 dredging was introduced to temporarily gain a channel, where needed, pending the completion of the permanent improvement. Its main purpose was to aid the proposed project when the regularization works were insufficient. The success of such dredging led to the belief in 1903 that a suitable channel might be gained and maintained more cheaply by dredges than by permanent contraction works. This new policy proved unsatisfactory, however, and in 1910 the method proposed by the project of 1881 was revived with the major dependence upon dikes and revetments.

The River and Harbor Act of January 21, 1927, specified that a channel of nine-foot depth should be obtained from the northern boundary of St. Louis to Cairo. This upper boundary limit was later extended to the mouth of the Illinois River. This nine-foot channel was to be maintained ultimately in the open river by regulating works supplemented by dredging across bars. To this end, it was planned to progressively direct the channel and obtain banks where desired by use of the dikes. Then these banks were to be made permanent by use of revetments.

The present Upper Mississippi River Canalization Program provides for a channel depth of nine feet, with suitable widths, at low water, in the Upper Mississippi River between the Missouri River and Minneapolis, Minnesota, and was adopted by the River and Harbor Act of July 3, 1930, as amended by Public Resolution No. 10, Seventy-second Congress, First Session, approved February

24, 1932.

The establishment of this nine-foot channel requires the construction of twenty-six dams and accompanying locks--two of which were already constructed--at selected points throughout the distance between Minneapolis and the mouth of the Missouri River.

The first project to be constructed, Lock and Dam No. 19 at Keokuk, Iowa, was completed in 1913, and the second, No. 1 at St. Paul, in 1930. At the present time, 18 lock and dam projects have been completed. Of the remaining 6 projects, 2 locks have been completed, and there are now under construction 1 lock and 2 dams. Only two of the series, Dam No. 1 at St. Paul and Dam No. 19 at Keokuk, are used for the development of hydro-electric power.

This series of twenty-six locks and dams will effect an aggregate lift in the Mississippi River of 331.3 feet at extreme low water over a distance of 610 miles between Alton, Illinois, and the Northern Pacific Railroad bridge at Minneapolis.

Geologic History and Physical Characteristics of Substrata

This general area is covered by glacial and loessial drift, under which bedrock strata ranging in age from the Kimmswick formation of the Ordovician system to the Pottsville group of the Pennsylvanian system successively crop out from the southwest to northeast in accordance with their regional northeasternly dip. The St. Peter sandstone, of the Chazyan group of the Ordovician system, which is penetrated by wells but not exposed, is the oldest formation actually known in the area. See table No. 1, page 112 and Fig. 6, page 120.

The Devonian system in this area is represented by only a few feet of the Upper Devonian strata which are exposed in several places a few miles north and west of here; however, a stratum thirty feet in thickness was encountered two miles east of here in the Lindberg Park well, consisting of dolomite and limestone of the Senecan series. The outcroppings occur in beds of earthy, argillaceous, dolomitic limestone, sandy limestone, and quartzite sandstone. The sand in the strata suggests that they are littoral deposits and, therefore, mark a zone not far from the shore lines of the Devonian sea.

All of the Iowa (Lower Mississippian) series is present in this area, and several of these formations crop out along the bluffs of the Missouri and Mississippi Rivers or in the ravines of their tributaries. The chief outcropping formations of this system are the St. Louis and Ste. Genevieve. The St. Louis formation, in general, is exceedingly variable in character. It consists mainly of beds of sparingly fossiliferous limestone interbedded with dolomitic limestone and chert, but it contains also irregular bodies of chert, stringers, and inclusions of calcite, and thin partings of green shale or clay. The texture of the limestone beds varies from granular and coarsely crystalline to dense and lithographic, the latter being characteristic of the St. Louis formation. The beds of granular and dolomitic limestone are massive, whereas the dense limestone is thin-bedded. The rock is often brecciated.

The St. Louis formation crops out along the Mississippi River bluffs from Alton west for four miles, but to the north the area in which it crops out narrows rapidly because the thickness decreases, the formation dips steeply northeasternly, and the formation is overlapped by the Pennsylvanian strata. The maximum thickness of the St. Louis formation cropping out in this area is found in the Alton bluffs where a face of 183 feet was measured. Less than a mile west it is only about

120 feet thick. It thins rapidly to the northeast and north. East of here records of drill holes show the St. Louis formation is present in full thickness, 270 feet having been measured at the Lindberg Park well. The site of the Mississippi Lime Company quarry is located along these bluffs for almost a mile. Incidentally, the entire amount of coarse aggregate for the concrete in Lock and Dam No. 26 was supplied from this source.

The Ste. Genevieve limestone rests conformly on the St. Louis formation and resembles the St. Louis limestone in that it consists of limestone varying from massive to thin bedded and from coarsely crystalline to dense lithographic, but it differs in that the massive crystalline beds are much more dominant and in being remarkably pure and free from chert. Some beds near the top are arenaceous, and in them crossbedding is locally evident. The St. Louis and Ste. Genevieve formations can be readily distinguished, not only by their lithographic differences, but also by the fact that the fossil Platycrinus pencillus, so far as is known, occurs only in the Ste. Genevieve.

Outcroppings of the Ste. Genevieve formation are exposed in Rock Springs Park, Wood River bridge, and the quarries located near Seventeenth and Alby streets.

Very little, if any, evidences have been found indicating the presence of the Chester series of the Upper Mississippian in this area.

The Pennsylvanian strata, comprising beds of conglomerate, sandstone, shale, underclay, coal, and limestone, and dipping slightly eastward, overlap the truncated, more steeply dipping Mississippian beds, so that they lie on successively older formations. Pennsylvanian outcroppings are visible in the quarry at Seventeenth and Alby streets and the Alton Brick Company quarry. There is an unconformitory contact between the basal Pennsylvanian and the Ste.Genevieve formation in Rock Springs Park.

The pleistocene system includes unconsolidated glacial drift, loess, soil, and alluvial deposits which cover the bedrock except where removed by streams.

The glacial drift which mantles the area and fills some pre-glacial valleys varies in thickness from zero to nearly 200 feet. It consists mainly of blue till, or clay, in which are embedded sand, pebbles, cobbles, and occasionally large boulders.

Loess, which consists of dust, silt, and sand blown from the river flats, overlies the drift and caps the bluffs along the Mississippi River. It attains a maximum thickness of eighty feet and is calcareous below the zone leached by surface waters. The Mississippi and Missouri River bottoms constitutes the principal alluvial deposits of this area.

Preparatory to the design of the dam, wash borings were used to determine the foundation characteristics. This method of soil investigation is used extensively, and on the whole, gives reliable information except possibly in cases where fine silt or loam is encountered. These particular borings indicated that the underlying strata were composed of somewhat lenticular deposits of sand varying unevenly from very fine to coarse, with some silt, humus, and clay, and in some cases mixed with a small per cent of gravel. The Missouri bank at the site of the abutment consisted of gumbo clay underlaid by silt.

Adjacent to the Illinois shore limestone of the St. Louis formation was encountered by borings in two or three places ranging in elevation from 317 to 331 mean sealevel. No rock was encountered the remainder of the distance across the river down to elevation 302 mean sea level.

SITE

The Alton Lock and Dam comprises one unit, and marks the downstream end, of the twenty-six navigation locks and dams included in the general program of canalization of the Upper Mississippi River, being No. 26 and the largest of the series.

The site of the dam is at Alton, Illinois, only twenty-three miles upstream from St. Louis, Missouri, and about 203 miles upstream from the mouth of the Ohio River. It is the only one of the twenty-six projects which also forms a part of the Great Lakes--Gulf of Mexico Waterway. For this reason, twin locks were actually completed as a part of the project, whereas on the other projects provision is made for the completion of twin locks at a later date. No flood control benefits will be derived, and the development of power is not contemplated at present.

Unlike most construction projects of this nature, the location is immediately adjacent to the city of Alton proper, which necessitated the bypassing of the entire city sewage by means of a ten-foot by twelve-foot intercepter sewer around the lock basin.

The locks and dam, including the abutment on the Missouri shore, is founded in its entirety on piles--one of the boldest designs ever executed in the annals of engineering history, since it is the largest structure in the world to rest entirely on pile foundations.

TWIN LOCKS

The bid of John Griffiths and Son Company of Chicago, Illinois, of $3,269,565.00, was the lowest of five bids received on construction of the locks, and the contract was awarded to them on December 29, 1933. This appropriation was made from National Industrial Recovery funds. Work under the contract started officially on January 13, 1934, with 600 calendar days being allowed for completion; this was later increased because of change orders and high water.

The twin locks are located on the Illinois side of the river, immediately upstream from the Missouri and Illinois Bridge and Belt Railroad Company bridge and the Clark highway bridge of the Lewis and Clark Bridge Company. The latter has sufficient vertical clearance for navigation, but the railroad bridge is lower and is provided with a swing span. River traffic could pass the railroad bridge on either side of the pivot pier next to the Illinois shore, the clear openings being 200 feet wide.

During construction of the locks, provision was necessary for uninterrupted navigation through one of the movable spans. In order that there might be no interference with river traffic, it was decided to first construct the main lock and place it in operation prior to the initiation of construction on the auxiliary lock. To direct traffic through the bridge spans, it was necessary to construct both the land and intermediate walls of the Alton Locks approximately 600 feet longer than on other similar projects of the Upper Mississippi River. The intermediate wall was to be of sufficient size to inclose the pivot pier of the railroad bridge and the second pier of the highway bridge. Because of these facts both piling and concrete units were tremendously increased. It is of unique interest to note that the intermediate wall alone has as much concrete in its structure as some of the other lock projects complete.

The twin locks, as designed, comprise a main lock basin 600 feet long by 110 feet wide, with 45-foot gates, and an auxiliary lock basin 360 feet long 110 feet wide, with 27-foot gates upstream. This lock width is standard in all locks being constructed in this series, and is the same as that of the Panama Canal Locks. Small pleasure craft and light traffic will be accomodated in the smaller lock, with only the larger tows using the main structure. The filling and emptying process will be controlled by means of electrically-operated Tainter valves set in the tunnels at each end of the lock chambers.

The lock gates are of the mitering type, and will be operated by electrically-controlled machinery placed in recesses at the top of the lock walls.

Due to monetary losses caused by adverse river conditions, John Griffiths and Son Company gave up their contract in April, 1936, with the intermediate and land walls approximately ninety per cent complete and no work started on the auxiliary river wall.

The contract for the completion of this work was awarded to Engineering Construction Corporation, the contractors on the dam, for $1,200,000.00.

DAM

The bid of Engineering Construction Corporation,--composed of Spencer, White and Prentis, George A. Fuller Company, and Turner Construction Company, all of New York City--of $4,856,716.80, was the lowest of four bids received on construction of the dam, and the contract was awarded to them on May 14, 1935. This appropriation was made from Public Works and National Industrial Recovery funds. On May 27 work was launched on preparation of a supply yard and base of operations on the Missouri shore, and on June 15, 1935, work under the contract officially started, 785 calendar days being allowed for completion.

The dam is comprised of three steel roller gates and thirty Tainter gates, all of the submergible type, with concrete sills and separating piers, supported on timber and steel pile foundations, extending from the river wall of the auxiliary lock to the Missouri shore, and an earth embankment extending thence to an intersection with the embankment of the Missouri and Illinois Bridge and Belt Railroad Company, and thence along this embankment to a point about a mile from the river. The length of the movable section is 1,724 feet, giving this dam a greater gate area surface than any other in existence. The roller gates are eighty feet long between piers and twenty-five feet high above concrete sills, and are operated by fixed electric hoists housed on the tops of the piers. The Tainter gates will be forty feet long between concrete piers and thirty feet high above concrete sills, and are operated by individual electric hoist units mounted on the service bridge. All gates may be lowered three feet, and raised to clear an elevation of 6.7 feet above maximum high water. The forty-foot Tainter gates are the largest constructed to date, the movable parts of each gate weighing approximately 100 tons.

A steel deck-girder type service bridge will extend from the lock wall to the Missouri abutment, a distance of 1,724 feet, and will extend approximately 154 feet beyond the face of this abutment. It will be capable of supporting two fifty-ton cranes with capacities of thirty tons each. These cranes will be used for removing large drift and transporting and placing the emergency bulkheads and Poiree trestles used for unwatering the gates in case repairs are necessary.

The cofferdam of the Engineering Construction Corporation is of the straight-wall type and consists of two walls of steel sheet piling thirty feet apart held together by two rows of steel tie rods into outside timber walers. The structure is strengthened by berms on each side, the higher being on the inside to counteract river pressures (see Plate 22).

The dam was constructed in three sections necessitating the construction of three separate cofferdams (see Plate No. 1), as it was necessary to avoid undue restriction of the river. The first included the Missouri abutment and twelve piers, and inclosed an area of about six acres. The second included the four roller gate piers and three Tainter gate piers, and inclosed an area of about six acres. The third included the remaining thriteen Tainter gate piers and the auxiliary lock wall, and inclosed an area of about thirteen acres.

GENERAL ENGINEERING DATA

Upper pool elevation	419.0 M. S. L.
Maximum lift	25.2 ft.
Length of pool	38.5 miles
Extreme low water elevation (1933)	393.8 M. S. L.
Maximum high water elevation (1844)	432.4 M. S. L.
Minimum recorded discharge (1933)	14,800 C. F. S.
Maximum estimated discharge (1844)	460,000 C. F. S.
Mean stage elevation	405.7 M. S. L.
Discharge at mean stage (el. 405.7)	112,000 C. F. S.

Locks

Main lock	600' x 110'
Auxiliary lock	360' x 110'
Elevation top of lock walls	432.0 M. S. L.
Height of lock walls (between gates)	51.0 ft.
Length of land wall (including upper and lower guide walls)	2,160.0 ft.
Length of intermediate wall	1,464.0 ft.
Length of river wall	968.0 ft.
Height of lock gates:	
Main lock--upper and lower gates	45 ft.
Auxiliary lock--lower gate	45 ft.
Auxiliary lock--upper gate	27 ft.

Dam

Length of movable section	1,724'-7¼"
Tainter gates (30)	40' x 30' each
Roller gates (3)	80' x 25' each
Width of piers at base (piers battered 1" in 20'):	
Tainter gate	8 ft.
Roller gate	15 ft.
Elevation of top of piers	460.5 M. S. L.

Height of piers above sub-grade:

Tainter gate	82.5 ft.
Roller gate	82.5 ft.

Length of piers:

Tainter gate	85.0 ft.
Roller gate	100.0 ft.

Quantities

Designation	Unit	Locks	Dam
Piling, Round Timber	Lin. Ft.	417,400	419,000
Piling, Concrete	Lin. Ft.	138,500	1,040
Piling, Steel Sheet	Sq. Ft.	157,300	211,500
Concrete	Cu. Yds.	203,900	98,200
Steel, Concrete Reinforcing	Lbs.	3,027,500	2,640,000
Steel Structural	Lbs.	4,359,500	12,620,000
Miscellaneous Metals	Lbs.	537,800	1,226,000

PROCEDURE

Main Lock

John Griffiths and Son Company bid $ 0.33 per linear foot for all timber piling in place, using thirty-two foot piling as a base length. Piling of additional length under the specifications carried the following premium:

Increase in Length	Per Cent
0 to 5 feet, inclusive	5
6 to 10 feet, inclusive	15
Over 10 feet	25

The contractor was required to absorb the cost of an additional foot of length per individual pile to take care of the undersize butt where tapered to fit the hammer or the brooming and upsetting due to driving. All piles on the lock were left one foot high and this foot was cut off in advance of concrete placement. Provision was also made for payment of piles cut off other than the one foot mentioned at fifty per cent of the contract price. The causes for such cut-offs were refusal or encountering stone riprap at depth.

The initial lock contractor employed the wooden skid-rig type of driver with forty-foot long "A" frame skid and leads fifty-five feet in height. Four units were used, all equipped with the No. 1 Vulcan steam hammer which developed a striking blow of 15,000 foot-pounds from the 5,000-pound ram and the 35"-36" stroke. The overall length of the hammer was thirteen feet and the weight 9,600 pounds. For the No. 1 Vulcan hammer sixty to seventy blows per minute are recommended by the maker. Standard vertical boilers developing fifty horsepower at 125 pounds pressure furnished steam for the American $8\frac{1}{4}$" x 10" double drum type engine.

A fifth boiler and engine was held in reserve as a replacement unit. Plate No. 4, page 126, illustrates this type of driver.

All of the piledriver units were rented from R. C. Bolduc Company of Minneapolis, Minnesota, at $25.00 each per day, with Roy Bolduc of the above firm as general piledriver foreman at a reputed salary of $90.00 per week.

Actual driving on the land wall foundation piling was started on June 6, 1934, and on the intermediate wall on June 14, 1934, with one rig each. On June 17 the No. 3 rig was started on the intermediate wall, and on July 1 the No. 4 rig on the land wall. Fig. 5, page 64 shows the plan followed as well as the general foundation plans for the locks.

On May 11, in advance of the occupancy of the main cofferdam, a floating piledriver was used to drive 336 piles supporting a portion of the intercepter sewer which parallels the land wall for 800 feet. This method was very awkward at the start because of the inefficiency of the crews, but improved until sixty piles were being driven per eight-hour shift. After seven days of driving, this equipment was removed just before closing the cofferdam.

The only other use of floating drivers was made with the same unit in August, 1934, when two piles were driven in each cell of the upper cribbed section of the upper guide wall. After the timber cribbing was completed and sunk, a skid rig followed up and drove the remaining ten piles in each cell.

The plant used for the floating work had sixty-five-foot leads and was mounted on a barge four feet by twenty-six feet by eighty feet. Ample steam from the 150 horse-power locomotive type boiler was furnished at 125 pounds per square foot to the Number One Vulcan Hammer, engine, and jet pumps. This was rented from the Fruin-Colnon Contracting Company.

Railroad siding had been laid along the Illinois bank, parallelling the center line of locks. Concrete piling were unloaded directly from the incoming railroad cars with a 110-foot boom, stiff-leg derrick. This was set up part way down the old bank slope and landed the piles for storage near the land wall. See Plate No. 8.

Wood piling were received both by rail and truck and put into storage piles along the rail siding on the eastern part of the general storage yard. Some, of course, went directly to the pile driver and these, together with reloaded piling from storage, were delivered to the cofferdam working area by railroad cars served by one of the Orton cranes or the steam operated Brown hoist. When the storage piles within the inclosure were inaccessible to the pile driver, one of the two Caterpillar tractors pulled both wood and concrete piles to points ahead of the driver where they could be reached by the driver pile lines. As many as four wood piling could be towed by the fifty horse-power tractor, whereas, with the best of footing, only two concrete piling could be taken. With the four drivers working to capacity, it was frequently necessary to use both tractors.

The jet water supply was furnished by a Fairbanks-Morse centrifugal pump, rated at 1,000 gallons per minute at 300 pounds per square inch, and powered by a 200 horse-power motor. This was installed landward of the land wall and midway up and downstream. The six-inch main from this unit ran along the old bank to the limits of the job and was furnished with tees at 200-foot intervals. However, with four drivers jetting simultaneously through two and one-half inch hose and jet pipes, the pressure dropped to eighty from one hundred twenty-five pounds per square inch and appreciably cut down the rate of driving.

Later a second 1,000 gallons per minute pump of the same make but designed for 150 pounds per square inch was installed to serve one of the drivers on the land wall through a four-inch main.

PERSONNEL - MAIN LOCK

The construction over the past few years of permeable pile dikes on the Missouri and Mississippi Rivers, together with bridge foundations has trained an ample supply of experienced pile driver labor. It is estimated that fully 225 men, within a radius of twenty-five miles, are classified as pile drivers and, of this number, approximately 150 are fully experienced. Encouraged by P. W. A. regulations, the John Griffith & Son Company elected to use union labor. Hence, the Alton Carpenters Local No. 377 furnished pile drivers. The union initiation fee for this local was $60.00 and the dues $2.80 per month. The number of men used ranged from sixty to one hundred with a labor turnover of less than fifty per cent during the lock pile driving. During the pile driving on the main lock, there were twenty-nine lost time accidents from an exposure of 50,000 man hours, or a frequency rate of slightly less than 60 per 100,000 man hours. This can be compared to a frequency of 13 per 100,000 man hours for the entire year for all crafts.

The pile driver crews consisted of one foreman, one engineer, one fireman, and five leadsmen. The hourly wages of the crews under the P. W. A. specifications corresponded to the union scale in this vicinity and were as follows:

1 Foreman	-	$1.25	per	hour
1 Operating Engineer	-	1.25	"	"
1 Fireman	-	0.75	"	"
5 Leadsmen @ $1.00	-	5.00	"	"
		8.25		

Source and Character of Timber Piling - Main Lock

The John Griffiths & Son Company contracted directly with the Chicago Wood Piling Company to supply all wood piling for their contract. Under the specifications a Class "C" (using the American Society for Testing Materials code) peeled, hard wood pile was designated. This class specified a twelve-inch butt measured three feet from the end and an eight-inch tip with an allowable undersize of twenty-five per cent in number scaling down to eleven and one-half and seven and one-half-inch butt and tip, respectively. The maximum tolerance in crookedness was three-fourths of the pile diameter at the point of maximum crook.

For the main lock, 12,531 piling, totaling 444,801 linear feet, were driven with the following length classification:

20' - 32'	142,170
33' - 37'	242,761
55'	59,686
	444,617

Under the terms of the specifications jetting was permitted for all but the last five feet of penetration and cut-off permitted upon refusal--this being interpreted as eighty blows per foot using the 5,000 pound ram of the Number One Vulcan Hammer.

The difference in the penetration actually obtained at refusal and the theoretical penetration desired was termed paid cut-off. Arranged by lengths, this is as follows:

Lengths	Total Driven	Cut-off	Per Cent
20' - 32'	142,170	1,726	1.41
33' - 37'	242,761	1,940	.80
55'	59,687	2,996	5.00
	444,718	6,662	1.50

Of the piles received, ninety-eight per cent fell under the class of Oak with its family subdivisions in order of importance as follows: black, red, burr, white, pin, post and water. The remaining two per cent in order were: ash, hickory, elm, locust, walnut, persimmon, and pecan.

All piling was primarily inspected and graded at the shipping points by Government inspectors under the supervision of Joseph F. Gill of the Second Area office of the St. Louis Engineer District. Three to four inspectors were assigned to this work during the cutting season.

Again at the lock site, the incoming piling were inspected, tallied, and the lengths and sizes remarked on the butt, if necessary.

Of the total linear feet received on the job, 1.021 per cent were culled as follows:

Defect	Linear Feet	Per Cent
Undersize	2,191	49.2
Decay	967	21.7
Split	774	17.4
Crooked	280	6.3
Knots	188	4.2
Twisted Grain	57	1.2
	4,457	100.0

An additional one foot of non-payment length was provided for each pile to take care of the brooming and to clean up the taper of the butt where it had been shaped to fit the hammer. With12,531 pieces driven, this amounted theoretically to 12,531 linear feet.

Actually, this figure is very close, as it was entirely possible to drive the piling with a tolerance of two-tenths foot in grade when leaving the pile one foot high for cut-off.

Classification of the entire piles received for the main lock was as follows:

Total linear feet accepted and in place	444,801
" " " paid as cut-off	6,662
" " " not paid for (1 ft. cut-off)	12,531
" " " rejected	4,457
Total Linear Feet	468,451

The timber piling used in the construction of the main lock was obtained from a total of 64 localities, most of which are in central and southern Illinois. 96.68% of the shipments came from Illinois, 2.61% from Indiana and 0.71% from Missouri. The Illinois localities listed in order of production are as follows:

Keyesport	Mill Creek	Fieldon
Okawville	Shasta	Springerton
Clay City	Big Boy	Brookport
Spur	St. Labory	Olney
Boulder	Bundas	Ullin
Metropolis	Mound City	Greenville
Bartelso	Venedy Station	Shattuc
Posey	Eloo	West Vienna
Millshoals	Ryder	Grand Chain
Baytown	Tamaroa	Hurricane
Cypress	Noble	Rock Bridge
Foreman	Dieteriach	Vandalia
Barnhill	Scanlen	Gaff
Brantsburg	Shawneetown	Kane
Perks	Dundas	Mounds
Chapel Island	Woodville	Miller City
Huey	Belknap	Iola
Unity	Lebanon	America
Buncombe	New Minden	New Memphis
Hagerstown	Covington	

The Indiana localities listed in order of production are as follows:

Crothersville	Seymour
Burns City	

The Missouri locality was Old Monroe.

Transportation was entirely by rail and truck. 8,584 piles or 68.5% were shipped by railroad and 3,947 piles or 31.5% were shipped by truck. Twenty-seven to fifty-one piles constituted a railroad car shipment and six to twelve piles were hauled per truck depending upon the size of the truck. None were shipped by barge or raft due to the distance to water of the points of origin.

The freight rate in most cases was $0.13 per hundred-weight, being slightly more on the longer hauls. The trucking companies met the railroad rate in all cases. The cost of shipping was paid by the timber contractors, e.g., piling was delivered for $0.17 per linear foot, f.o.b. Alton.

The average weight of the piles was approximately forty pounds per linear foot. At $0.13 per hundred, the freight amounted to $0.05 per foot. This allowed the piling subcontractor $0.17 minus $0.05, or about $0.12 per foot, out of which he had to pay for the labor of felling the trees, trimming, peeling off the bark, and hauling to the shipping point.

Under the terms of their contract, the Chicago Wood Piling Company did not trim the heads of piling to an eleven-inch diameter to fit the hammer. This was done by the John Griffith & Son Company, themselves.

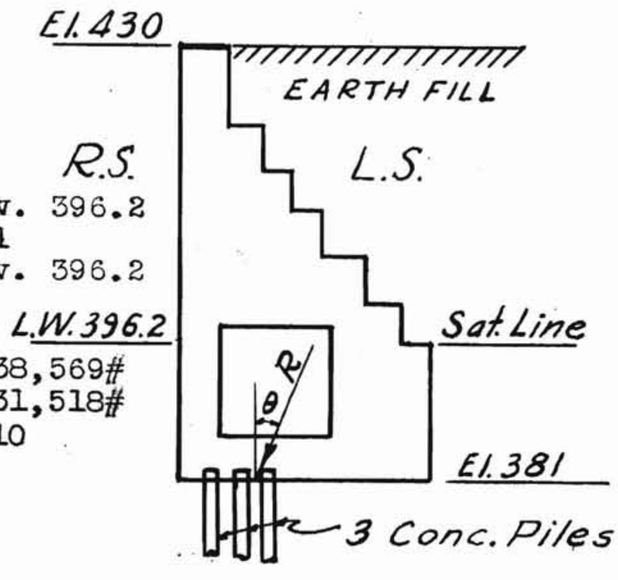
Concrete Piling

1. General. - From the earliest days of building construction, piles have played an important part in foundation work. Timber piles have come down through the ages as supports for buildings, but in later years piles made of steel and reinforced concrete have been used more extensively. A knowledge of materials has shown that concrete piles possess great strength and durability, and when made by modern manufacturing processes are practically indestructible. Concrete piles may be employed in locations, where timber piles would decay or be destroyed by parasites and they will not deteriorate when subjected to atmospheric conditions which are alternately wet or dry. Because of greater size and strength, concrete piles will also support greater loads than timber piles and their durability makes them superior to steel.

Concrete has high compressive strength but possesses a low tensile strength. Steel, on the other hand, has a high tensile strength. A combination of concrete and steel, therefore, has great strength, toughness and durability and the steel, being protected by concrete, is not subjected to corrosion. This combination possesses the advantage of both materials.

In order to attain the highest possible strength, concrete should be free from voids and the aggregate well bonded. Any manufacturing process which produces this result will develop a concrete which will possess the highest possible strength and durability.

2. Reasons for Placement of Concrete Piles. - In the construction of the twin locks of Lock and Dam No. 26, approximately 5,000 concrete piles were used at points where the greatest compression due to the overturning moment occurred, usually at the toe of the walls. In Figure No. 1 is shown a cross-section of the lower guide wall. The landside is backed by an earth fill. The maximum overturning moment occurs at the riverside toe when the river is at low water stage, elevation 396.2. This toe is supported by three concrete piles.



Conditions:-Water to Elev. 396.2
Earth to top of wall
Saturation Line Elev. 396.2

Resultant:-Hor. Comp. = 38,569#
Vert. Comp.=131,518#
Tan. θ = 0.3410

Fig. No. 1
LOWER GUIDE WALL

In Figure No. 2 is shown a cross-section of the intermediate wall between bays. The center recess is filled with sand. The maximum overturning moment occurs at the riverside toe when the water in the main lock is at upper pool elevation and the auxiliary lock is pumped out. This condition may be reversed, i.e., the water in the auxiliary lock at upper pool elevation and the main lock pumped out. Therefore both the riverside and landslide toes are supported by three concrete piles.

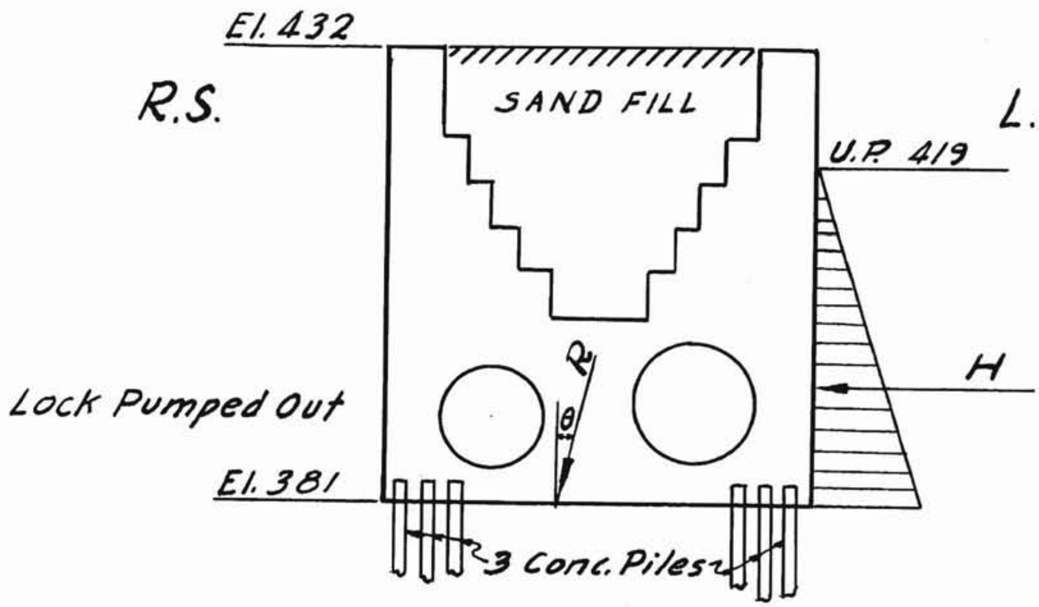


Fig. No. 2
INTERMEDIATE WALL BETWEEN BAYS

Conditions:-U.P. in Main Lock
Aux. lock pumped out - dry earth fill

Resultant:-Hor. Comp. = 45,125#
Vert. Comp. = 216,288#
Tan. θ = 0.20863

In Figure No. 3 is shown a cross-section of the riverwall between gates. The maximum overturning moment occurs at the landside toe, as in Case II, when the auxiliary lock is pumped out and the water on the exterior is at a lower pool elevation of 410. This toe is supported by three concrete piles. There is a reversal of this condition as in Case I, where the water in the auxiliary lock is at upper pool elevation and the water on the exterior is at low water elevation, but concrete piles are not used because the design of the wall is such that this condition is taken care of.

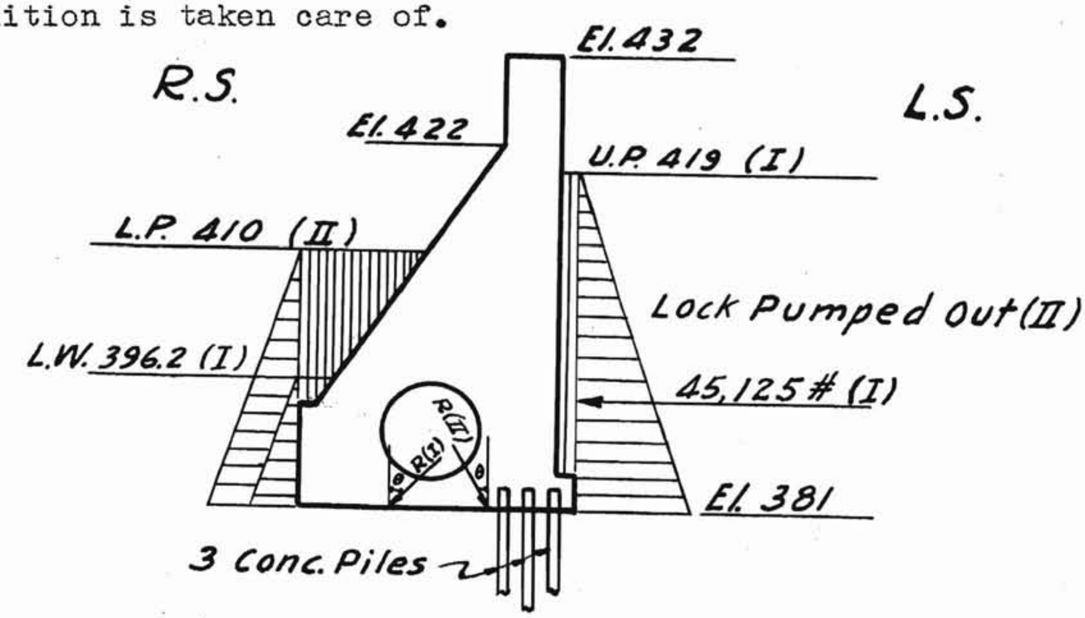


Figure No. 3
RIVER WALL BETWEEN GATES

Case I

Conditions:-U.P. in auxiliary lock
L.P. at low water
Resultant:-Hor.Comp.--37,905#
Vert.Comp.-52,871#
Tan. θ - 0.7169

Case II

Conditions:-Auxiliary lock pumped out
L.P. at top of lower Poiree Dam
Resultant:-Hor.Comp.-26,218#
Vert. Comp.- 103,507#
Tan. θ - 0.2539

A section of the walls approximately 85' in length above each gate is supported entirely by concrete piles. This is because of greater loads on the piles during construction, primarily, and it also serves to compensate for the excess longitudinal overturning moment produced in that particular section of the wall when the lock gates are open.

3. Pile loading tests. - Loading tests on constant and tapered section concrete piles were conducted under the supervision of the U. S. Army Engineers prior to the letting of the concrete pile contract. The location selected for these tests was under the land wall above the upper gate bay section. It was necessary to conduct these tests in about 10 ft. of water due to the length of time required for constructing the cofferdam.

Borings indicated that the foundation site consisted of sand deposited in strata of various degrees of coarseness. The test piles were loaded with rail and steel billets, weighted before loading.

The piles were given an initial load of 30 tons with a single reading at 20 tons; and above 30 tons, the load was added in increments of 20 tons at intervals of 4 hours. Readings were taken to nearest 1/1000 of a foot.

Comparison of the results of the loads on the concrete piles indicated that a 30 ft. penetration was sufficient, and that for all except loads well in excess of the maximum design load of about 52.5 tons, there was but little difference in settlement of the constant section and tapered type piles, under loads approaching 100 tons, however, the constant section piles showed less settlement.

The 100 tons load remained on the piles for a period of one week, after which the settlement was found to range from 0.007 ft. to 0.009 ft. After removing the load the settlement amounted to 0.004 ft. Due to the large number of variables, the following conclusions drawn from these tests are applicable only to the local foundation conditions:

Either a constant section pile with least lateral dimension of 16" or a tapered type with an 18" butt, with 30' penetration will provide adequate bearing for the maximum design loads for the twin locks, Lock and Dam No. 26. Tapered piles require fewer total blows for driving than constant section piles for a given penetration and final resistance to driving.

Concrete piles with a jet hole in the center may be driven to more accurate alignment and more rapidly than those jetted by an external jet pipe. Tapered concrete piles being lighter may be more easily handled than constant section piles for similar bearing capacity and may be driven about as rapidly as timber piles. Tapered concrete piles, in general, will sustain a greater load per square foot of surface area than a constant section pile of equal penetration.

Tapered concrete piles so reinforced as to have resistance to bending at the midpoint equivalent to that of a constant section pile, will have a greater resistance to bending from midpoint to butt. Concrete piles of either constant section or tapered type have a greater bearing capacity than indicated by the Engineering News Formula (based on comparison of .02 ft. settlement). For concrete piles the relationship between resistance to driving and settlement is more consistent than for timber piles.

The pile loading tests were conducted as a bid item at $500.00 for each complete test. Further details and graphs on this subject will be found in a "Pile Loading Report" dated August 1934 - U. S. Engineer Office, St. Louis District.

Elaborate lateral loading tests were made on single piles and groups of piles supporting test monoliths, to determine the allowable horizontal stress to which concrete and timber piles might be safely subjected. A complete account of the lateral loading tests may be found in a report on this subject by Mr. L. B. Feagin, Senior Engineer, or in Vol. 61, No. 9 (November 1935) of the Proceedings of the American Society of Civil Engineers.

Severe deflection tests were also made on both tapered and constant section piles by Westinghouse Electric and Manufacturing Company in the presence of U. S. Army Engineers.

On page 46 are deflection tests made August 1, 1934, on a Westinghouse 16" octagonal, constant section pile and a tapered pile 18" at the butt and 10-3/4" at the tip, 32 feet long. The latter was the type used on this job.

Although it is not applicable to this particular job, laboratory tests on 9½" point tapered hollow-spun concrete piles 20 feet long show an ultimate load value of 150 to 200 tons with the piles functioning as columns, i.e., as point bearing piles. Details of these tests are shown in a report from the University of Illinois. Also see Plate No. 37.

Load in Pounds	TAPERED Defl. in Inches	TAPERED Perm. Defl. in Inches	16" OCTAGONAL Defl. in Inches	16" OCTAGONAL Perm. Defl. in Inches	REMARKS
500	3/32	0	3/32	1/8	Note:- Fulcrum at midpoint
1000	1/4	0	1/4	9/32	Load applied 15'-10" from
1500	3/8	0	9/16	15/32	fulcrum, deflection ob-
2000	5/8	1/32	7/8	25/32	served 16'-6" from ful-
2500	7/8	-	1-1/8	1-1/32	crum. 5" hole at tip,
3000	1-3/16	1/8	1-1/2	1-3/8	$5\frac{1}{2}$" hole at midpoint.
3500	1-9/16	-	1-7/8	1-13/16	
4000	2-1/32	5/32	2-1/4	2-5/16	
4500	2-1/2	-	2-5/8	2-27/32	4-5/16 Perm.Defl. @ 5000#
5000	3	13/32	5000# pull applied for one minute 100 lbs. lost		
5000	3-3/32	7/16	" " " " " " 90 " "		
5000	3-3/16	7/16	" " " " " " 60 " "		
5000	3-3/16	15/32	" " " " " " 50 " "		
5000	3-7/32	15/32	" " " " " " 30 " "		
5000	3-1/4	15/32	" " " " " " 20 " "		
5000	3-5/16	1/2	" " " " " " 25 " "		
5000	3-5/16	1/2	" " " " " " 20 " "		
5000	3-11/32	1/2	" " " " " " 20 " "		
5000	3-11/32	1/2	" " " " " " 20 " "		
5000	3-11/32	17/32	1 min. lost 20#; 2 min. lost 35#; 3 min. lost 45#;		
4000	2-25/32	-	4 M. lost 60#		
5000	3-7/16	11/16*	*5000# pull applied 1 hr. 45 min. lost 250# but deflection		
5500	4	7/8	remained the same. Release in tension probably		
5500	4-3/16		due to cable stretch. Perm. defl. increased from 17/32"		
5650	4-3/4	-------------	Yield point. to 11/16"		
6000	5-7/8	-------------	Failure by constant pull.		

16" constant section with 8-3-1/4" ø bars - yield point 4,870#, Defl. 3-3/8".

Note:-Tapered pile with 8- 3/4" ø and 4 1/2" ø bars at 4,870# had 2-13/16" Defl.

Wm. J. Putmann, M. S.
Urbana, Illinois

COMPRESSION TESTS OF CONCRETE PILES

for Westinghouse Electric & Manufacturing Co.
South Bend, Indiana

Pile No.	Dia. Top In.	Base In.	Core Top In.	Hole Base In.	Area Top Sq.In.	Length Ft. In.	Max. Load Lb.	Unit Stress Lb./Sq.In.
1	9.53	14.49	2.55	5.60	66.22	20'-1/8"	184,200	2780
2	9.52	14.52	3.00	6.12	64.11	20'-1/8"	229,500	3580
3	9.56	14.52	2.55	4.30	66.67	20'-1/8"	275,000	4120
4	9.54	14.51	3.08	5.45	64.03	20'-1/8"	229,500	3580
5	9.53	14.50	3.03	4.20	64.12	20'-0"	299,500	4670
*1-A	10.54	14.49	3.22	5.60	79.11	16'-0"	399,000	5040

*Pile No. 1-A was pile No. 1 after testing, from which four feet was cut from the top, or small end, by means of a concrete saw, and the remainder tested as a sixteen foot pile.

The above piles were shipped to the laboratory from the St. Louis plant of the Westinghouse Electric & Mfg. Co. by Mr. H. F. Hedderich. One pile was broken in shipment so that only five remained to be tested. They were said to be approximately seven weeks old when tested and made of a 1/3 mix of cement and lead chats.

The large end of the pile to be tested was bedded in plaster of Paris on a cast iron bearing and the top, or small end, capped with plaster of Paris and a machined cast iron bearing plate to form plane ends. Load was applied, through a spherical bearing block at the small end, at a uniform rate of 0.10 inches per minute in a Riehle 600,000 pound vertical testing machine.

Failure occurred in each case by compression of the concrete at the small end, In no case was there appreciable buckling of the pile.

Respectfully submitted,

Dec. 14, 1929

Wm. Putmann

4. Selection of Westinghouse. - The final loading test was made on a tapered concrete pile with 10 3/4" tip and 18" butt. In view of the fact that for loads less than 90 tons this pile showed, in general, less settlement than any of the other concrete piles with 30' penetration, and since a tapered pile complying with the provisions of the specifications requiring resistance to bending at the mid-point equivalent to that of the constant section pile would necessarily provide a greater resistance to bending between the midpoint and the butt, it was decided to allow the contractor the choice of supplying either a constant section pile with least lateral dimension of 16", or a tapered pile with not less than 10-3/4" diameter point and an 18" butt. The contractor elected to furnish the latter.

Westinghouse Electric and Manufacturing Company, of 3850 Bingham Ave., St. Louis, Missouri, by virtue of being the lowest bidder, was selected to furnish the contractor with concrete piling for the twin locks of Lock and Dam No. 26.

5. Manufacture. - Westinghouse piles are manufactured by a unique process perfected after years of intensive research. In this process the concrete, after

careful preparation, is placed in sealed cast iron molds in which the reinforcing elements have been assembled. The molds are then rotated at a high speed, thus producing a compact mass free from voids, which is practically unexcelled for density, durability, toughness and strength. The centrifugal force developed during the rotation of the molds compresses the concrete mixture closely around the reinforcing structure and at the same time forms a cylindrical opening which extends the entire length of the pile; hence the name"Hollowspun".

Formulae - The various batch formulae used in the manufacture of the hollowspun concrete piles are shown in Table No. II, page 51.

Two batches, approximately 5/8 cubic yard each were used per pile. The first batch comprising the tip portion usually consisted of 50% gravel and 50% sand and the second batch comprising the butt portion usually consisted of 60% gravel and 40% sand. In a tapered section pile of this sort, greater workability was required in the smaller portion, hence the greater percentage of sand in the first batch.

These percentages varied from time to time due to changes or variations in the gradation of the gravel and the percentages of moisture in the sand. In Table No. II are shown columns of % add and % cut. This was used to compensate for any fluctuations that may occur due to variation of the moisture content of the aggregates, usage of the concrete for test cylinders, wastage, etc.

It was desirable to have a small amount of concrete left in the mixer after the second batch was poured rather than be short. If this surplus was utilized a percent add was used on the second next batch and if the surplus became excessive a percent cut was subtracted on the second next batch.

When conditions were more or less perfect a 10% add was placed on the second batch of the first pile and a 10% cut was placed on the second batch of the last pile of the shift.

TABLE II

35.8 Cu. Ft. Loose										
							REGU-LAR	5% CUT	10% CUT	10% ADD
17.9 for half mix. -										
			40-60 mix.							
7.15	cu.	ft.	gravel	at 95.5	lbs.		685	650	615	755
10.75	"	"	sand	" 108	"		1160	1100	1040	1275
5.91	"	"	cement	" 94	"		555	530	500	610
			45-55 mix.							
8.06	cu.	ft.	gravel	at 95.5	lbs.		770	730	690	845
9.84	"	"	sand	" 108	"		1060	1010	955	1165
5.91	"	"	cement	" 94	"		555	530	500	610
			50-50 mix.							
8.95	cu.	ft.	gravel	at 95.5	lbs.		855	810	770	940
8.95	"	"	sand	" 108	"		965	920	870	1060
5.91	"	"	cement	" 94	"		555	530	500	610
			55-45 mix.							
9.84	cu.	ft.	gravel	at 95.5	lbs.		940	895	845	1035
8.06	"	"	sand	" 108	"		870	825	780	955
5.91	"	"	cement	" 94	"		555	530	500	610
			60-40 mix.							
10.75	cu.	ft.	gravel	at 95.5	lbs.		1025	975	920	1125
7.15	"	"	sand	"108	"		770	730	690	845
5.91	"	"	cement	" 94	"		555	530	500	610
			65-35 mix.							
11.63	cu.	ft.	gravel	at 95.5	lbs.		1110	1055	1000	1220
6.27	"	"	sand	" 108	"		675	640	605	740
5.91	"	"	cement	" 94	"		555	530	500	610
			70-30 mix.							
12.52	cu.	ft.	gravel	at 95.5	lbs.		1195	1135	1075	1315
5.38	"	"	sand	" 108	"		580	550	520	635
5.91	"	"	cement	" 94	"		555	530	500	610

General. - As has been said the concrete piles selected for the twin locks or Lock and Dam No. 26 was of the tapered type, 10-3/4" in diameter at the tip, 18" in diameter at the butt and 32' long. Three feet at the butt was of constant section and the remaining 29' had a taper of 1/4" per foot. This pile had an approximate weight of 5800 pounds and the surface area was 120.3 square feet.

Reinforcing. - The reinforcing consisted of eight 3/4" x 31'-7" round deformed steel bars and four 1/2" x 17'-0" round deformed steel bars. The 3/4" bars extended the entire length of the pile and the 1/2" bars began $17\frac{1}{2}$" from the butt and extended tipward. This feature of extra 1/2" bars in the butt section provided for greater stability against buckling or lateral thrust.

6. Process. - In assembling the reinforcing, the spacer rings were electrically welded to the first long bar and the other bars were fastened to the rings by means of accurately spaced clips which were a part of the spacer ring assembly. The exact centering of the reinforcement in the molds was accomplished by means of non-ferrous metal chairs riveted rigidly to the steel spacer rings. The assembly of bars held rigidly together by the spacer rings was placed in a lathe which

turned at approximately 100 R.P.M. and was then wrapped with a continuous spiral of 0.162" diameter steel wire (ASTM) 490'-9" long with a pitch varying to meet the strength demanded. Beginning $2\frac{1}{2}$" from the butt there were 36" of 19 turns at 2" pitch, 14'-0" of 56 turns at 3" pitch, 11'-7" of 23 turns at 6" pitch, and 36" of 19 turns at 2" pitch, ending 2" from the tip. The butt was protected by 12" of 1" x 2" wire mesh .080" in diameter and the tip was protected by 6" of the same. This entire reinforcing assembly was known as a "Cage", See Plates No. 27 and No. 28.

Molds. The molds were made of semi-steel and weighed approximately 11,500 pounds. They were divided into two longitudinal sections held together on each side by 5/8" bolts 12" on center. Each end was closed by bolting on a 2" steel plate. The cage assembly was placed into a half section of the mold and the other half was inverted and bolted to the first section. See Plate No. 28.

A $2\frac{1}{2}$" W. I. coupling 5" long was fastened to the mold 18" from the butt end to provide for a jet hose connection. Four feet of a .1055" diameter steel wire was passed about six reinforcing bars forming a 3" equilateral triangle in the center of the cage 12" from the butt for the purpose of backing the paper packing for the butt plug.

The mold containing the cage was then placed on a horizontal rack which lowered the tip end into a pit whereupon the mold reached a vertical position ready to be filled with concrete.

Filling the Molds. The concrete for these piles was poured with a 3/4 cubic yard Ransome mixer. Two batches totalling approximately 1-1/4 cubic yards were required per pile. See Plate No. 29. All material that went into the piles was weighed, except the water which was measured. The water temperature was 170 degrees Fahrenheit. The weight operator kept a record of the weights of material in each pile. These weights fluctuated due to the percent cut or add on account of varying moisture in materials. The second batch contained 1½% $CaCl_2$ (8.3 pounds) to insure that portions of the pile at the larger diameter curing as rapidly as that at the smaller diameter. When the mold was completely filled a butt plate was bolted on and the tip end was raised until the mold reached a horizontal position.

A total of 16 molds were employed in the manufacture of the concrete piles. With respect to time, this was the minimum number required to complete the cycle of spinning, draining, hot water curing, stripping, cleaning, reinforcing, installation, and filling with concrete.

Spinning. The filled mold was then moved by an overhead traveling crane to a lathe where it was horizontally spun for nine minutes at speeds varying from 90 R. P. M. to 400 R. P. M., the latter speed being maintained for four minutes. See Plate No. 30.

Accuracy in spinning was insured by the use of a centrifugal tachometer and a timing instrument attached to the lathe. An automatic recording tachometer installed in the office made it possible to check the time and rate of spinning on any particular pile after it was spun.

Draining. When the spinning process was completed the pile was moved by means of a crane to a drain rack where the tip was held slightly elevated. Plugs were removed from each end and the effluent consisting of excess water, laitance and any foreign material was drained from the hole in the spun pile. In spite of the fact that the mold was completely filled with concrete, the truncated conical hole in the pile usually ran approximately 4 inches in diameter at the tip and 7 inches at the butt. This fact illustrates the density produced by the centrifugal force of the spinning process.

Occasionally the effluent from the cylinders was dehydrated and the amount of water extracted by centrifugal force determined. The residue left after dehydrating the sludge or effluent was analyzed for the cement content and found to average 4%.

While the pile was in this position the condition of the inner walls was inspected with a strong flashlight. In some instances in the case of rough handling or slightly dropping the pile on the rack, a slight caving of the upper wall was noted. Occasionally this caved material would plug the jet hole. Such piles were then made solid by filling them with concrete and driving was accomplished with the aid of an outside jet. Occasional dehydrating tests were run on the effluent from the pile. Approximately 5.75 gallons of water per sack of cement was used in the original mix. After spinning,only 4.9 gallons per sack were retained in the spun concrete. In other words, in the spinning process, sufficient water for plasticity can be used without danger of weakening the conrete as only the water retained in the spun concrete affects the strength. Naturally, the volume change is correspondingly due to the low water cement ratio.

High Temperature Curing. After an inspection of the center hole the end plugs were inserted and the mold and spun pile were removed to a hot water curing tank where they remained from $2\frac{1}{2}$ to 3 hours. See Plates 30 and 31. The temperature of the water in this pit was maintained at 125 degrees Fahrenheit to 150 degrees Fahrenheit. Accurate records of the water temperature in the pit were maintained by means of an automatic recording thermometer. The capacity of the pit was 14 piles.

The purpose of this high temperature curing was to insure a high early strength for handling after the pile was stripped of the molds. Handling was conducted by means of 2 hooks 15 feet apart attached to a bar which in turn was attached to an overhead traveling crane. These hooks were placed under a pile, one 6 feet from the butt and the other 11 feet from the tip. See Figure 4.

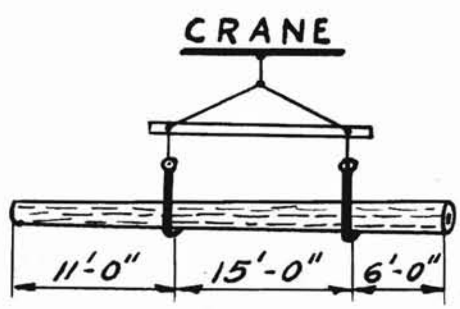


Fig. No. 4
PILE PICK-UP DEVICE

Stripping. The piles were moved from the curing tank to racks where the molds were unbolted by means of electrical wrenches and removed. By this time the piles were 3 or more hours old and had compressive strengths ranging from 800 to 1500 pounds per square inch.

At this point the piles were given a thorough inspection for checking, circumferential cracks, handling cracks, size of hole, slumping, caving, etc. For future reference, immediately after stripping, the pile was painted on butt end and side with date, shift, and serial number.

Normal Curing. The piles were moved from the stripping rack to the curing bay within one hour after stripping. See Plate No. 32. They were racked about 10 high and hydrated for 14 days by means of an intermittant spray. The piles were sprinkled with hot water the first 24 hours to prevent checking caused by too rapid cooling, after which water at tap temperature was used. The ends of the piles were kept wet by means of a hose.

Records of temperature and humidity of this curing bay were continuously recorded by an automatic thermometer and hygrometer.

Plugging and Coring. At sometime during the period of hydration the pile was plugged at the butt and cored at the tip. The butt plug consisted of a rich mix of conrete with a slight amount of $Ca\ Cl_2$ added, filling the butt for a distance of 12 inches. That portion of the surface of the inside wall of the pile that came in contact with the plug was roughened while the green pile was resting on the drain rack to give the plug a good adhesive effect. When pile driving began some longitudinal cracking or splitting was noted at the pile butt. It was thought that the plugs were driving down but it was later discovered that inefficient cushion blocks were the cause. A plug flush with the butt was found to be the best.

For the purpose of obtaining an efficient hydraulic orifice at the tip of the pile, a 12 inch tapered nozzle made of 28 gage sheet iron was grouted in place. The inside diameter of the tip of the pile was approximately 4 inches. This was reduced to 2 inches in diameter in a distance of 12 inches.

Loading. After the 14 day period of hydration was completed the piles were loaded into railroad cars in the rear of the plant. See Plates No. 33 and No. 34. Twenty-one piles were loaded into each car, 3 tiers of

7 each. An intricate system of bracing and blocking was employed to keep the piles from sliding through the car end in the event of the brakes being quickly applied. As many as 7 cars per day were shipped. A final inspection was given the piles during loading.

Laboratory. Westinghouse Electric and Manufacturing Company maintained a concrete testing laboratory at the plant. The equipment of this laboratory consisted of the following:

A Riehle Brothers mechanical testing machine was used for testing the compressive strength of the cylinders. Its capacity was 200,000 pounds with a 10% overload factor, totalling 220,000 pounds. See Plate No. 36. The capacity of this machine was exceeded many times by the 90 and 180 day cylinders.

A lathe was used for spinning the 6" x 12" cylinders the same time and rate that the piles were spun. The speed was indicated by a centrifugal tachometer attached to the lathe. See Plate No. 35.

A Eureka balance and Chapman flasks were used for determining the specific gravities of the various materials.

Steam and gas cabinets were used for curing or drying specimens or materials. Other equipment consisted of a rotap machine, flow table, vicat apparatus, balances, weights, flasks, graduates, beakers, test tubes, chemicals, etc.

Cylinders. Each day 6 or 8 cylinders 6 inches in diameter by 12 inches long were made. The samples were taken from the mixer at the pile. See figure 12. These cylinders being hollowspun had holes in the center ranging from 2 inches to 2-5/8 inches. Two 7 day and two 14 day cylinders were broken for each day piling was manufactured. There were alternate breaks for each day on cylinders of 3 hours and 21, 28, 90, 180 and 365 days.

A cylinder having a $2\frac{1}{2}$ inch hole would have a cross-sectional area of 23.4 square inches. In the case of this particular cylinder exceeding the compressive capacity of the testing machine, which has happened, there would be a compressive strength of 220,000/23.4 or 9400 pounds per square inch, a value almost unheard of in practical concrete.

Test Results. The following table shows the results of the compressive stresses of the cylinders made in connection with the concrete piles.

AGE	COMPRESSIVE STRESS	NO. OF TESTS
3 Hours	1167 Lb./sq.in.	94
1 Day	4020 "	26
7 Days	4750 "	136
14 Days	5180 "	136
21 Days	5775 "	86
28 Days	6180 "	71
90 Days	7747 "	81
180 Days	8630 "	36
365 Days	9200 + "	15

Two cylinders were picked at random, sawed in two, polished and minutely examined for concentric banding and their respective hardness. See Figure No. 7.

The fact that coarse material was apparently concentrated at a point inside of the periphery is contrary to ordinary reasoning, that the heavier the particle, the more likely it is to seek an outside position on spinning. Further, it was of great interest to note that this point of concentration was at the center of mass of the cylinders.

$$\text{Thus;} \quad K = \sqrt{\frac{r_1^2 + r_2^2}{2}} = 2.3''$$

Materials. Incor cement was used in the manufacture of the first 100 piles. Due to the hot water curing the results from this cement were not as good as those from a standard Portland cement. The Incor cement was followed by Atlas and then by Missouri Portland. A test on each car of cement was submitted by the Pittsburgh testing laboratory.

The fine aggregate was Meramec sand and the coarse aggregate was Meramec gravel. A sample was then taken from 25 different places in each car of aggregate. This sample was screened and dried by a Ro-tap machine. The percentagesretained on each screen were added to form what is termed as the fineness modulus of the materials. The latter was used for constructing the proper mixture for the spun piles.

History. For the twin locks of Lock and Dam No. 26, concrete production began on May 10, 1934. Two 8 hour 7 day shifts were worked in the beginning, after a short time a third 8 hour shift was added but this did not prove economical and was soon dropped. Each shift produced about 32 piles under fair conditions. Ten men constituted a shift.

September 9, 1934, the plant went on a 4 hour shift, 5 days per week schedule, producing 16 piles per shift. Production was terminated on November 26, 1934.

The percentage of rejects on the job was very small, about 0.3 of 1%.

Driving

Pattern Followed. The general procedure followed on driving was to begin on one side of the wall and move laterally to each successive pile location until the other side of the wall was reached after which the rig moved forward one row and began driving in the opposite direction. See Figure No. 5. As a rule separate drivers were employed to drive the concrete piles. Although there were only 2 or 3 rows of concrete piles the above procedure was followed.

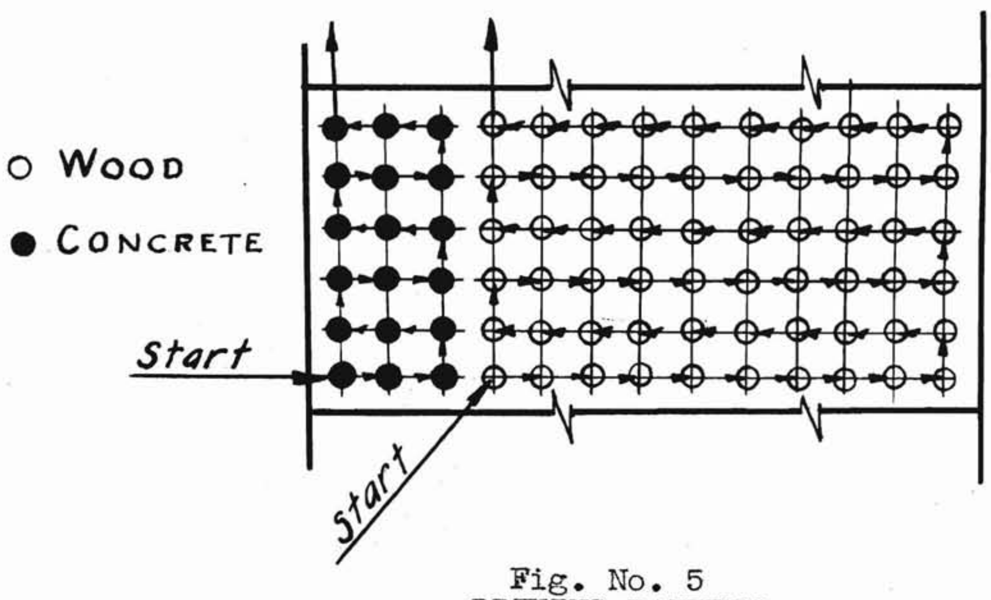


Fig. No. 5
DRIVING PATTERN

Pile delivery to driver. The piles were taken from the stock pile in the yard to the cofferdam area with an Orton locomotive crane that ran out on a trestle from the landside. From this point the piles were pulled by means of a caterpillar tractor to the various drivers. A tractor would pull 2 to 7 wood piles but only one concrete pile due to its weight. These tractors also supplied the drivers with coal, hauling about one ton per load in a slip.

Occasionally on the intermediate wall the Orton locomotive would unload piles from a flat car on the cofferdam directly to the driver. (A standard gage railroad track ran the entire length of the cofferdam).

Timber

Production. The general average over the entire period of the job was approximately 460 ft. per 6 hour shift. The daylight shifts as a rule exceeded this figure while the night shift dropped under it. The above figure includes delays of which there were no accurate data kept but the average taken from daily observance was about 55 minutes per 6 hour shift. The delays were due to waiting on piling, waiting on steam, minor repairs, moving, etc. The best driving in any one shift was on May 18, 1934, when one driver drove 62 33-ft. piles or 1950 paid linear feet in nine hours. This is an average of 217 feet per hour or 7 33-ft. piles.

A total of 746 26-ft. piles were driven in the 31 struts in the lock floor. A standard rig was first used and drove 622 piles but due to the relatively small number of piles and the long move from strut to strut this proved very costly. The maximum production was approximately 24 per 12 hour day. Later a Northwest crane with swinging leads was used to fine advantage. The maximum production of this rig was approximately 40 piles per 12 hour day or an increase of 66% over the standard skid rig.

Jetting. The wood piles were driven with the aid of an outside jet pipe with a pressure around 200 pounds per square inch. This pressure varied widely due to pumping conditions. Five feet before the desired penetration was reached jetting was discontinued and the pile was driven with the hammer to secure final penetration.

Bearing values. Bearing values were figured by the Engineering News Formula. These values varied between 15 and 100 tons with an average of 55 tons. Design loadings ran from 1.5 tons to 30 tons per pile.

Butt Cut-off. The pile butts were sawed off to grade by a Wolf Link air driven pile saw, manufactured by the Reed-Prentice Corporation and costing $765.00. Two were used. For operation each required 3 men; 2 carpenters and a carpenter's helper. The cost of labor amounted to \$3.10 per hour or \$37.20 per day per saw, since there were two 6 hour shifts. One saw would cut off about 80 butts per 12 hour day, at a cost of \$0.465 each.

Driving Costs. The following costs are based on 411,592 linear feet of piling as of February 1, 1935.

Unit	Cost	Material	\$0.17
"	"	Miscellaneous	.01
"	"	Labor	.19
"	"	Equipment	.04
"	"	Indirect	.08
			\$0.49

These costs may be broken down as follows:

Material included piling only delivered at the lock site of Lock and Dam No. 26.

Miscellaneous included pipe, cable, rope, timbers, skid grease, etc.

Labor included driving, butt cut-off, unloading, handling, maintenance, sharpening, etc.

Equipment included plant rental, repairs, depreciation, storage, taxes, coal, lubricants, etc.

Indirect was divided into two classes as follows:

Legal	Bond ($1\frac{1}{2}$% of total job) Employees Liability Ins. (Average 10%) Marine Insurance Interest on Capital Invested (4%)
Indirect	Mobilization Demobilization Distributive Overhead

The total indirect cost usually ran about 16% of the total direct costs.

Concrete

Production. The general average over the entire job was about 325 feet or approximately 10 piles per 6 hour shift which included delays. The maximum driven in one nine hour shift was 1216 linear feet or

135 feet per hour.

Jetting. Jetting was accomplished by attaching the jet hose directly to the coupling in the pile. It was found that by the use of the jet hole in the center, the piles were driven to more accurate alignment and more rapidly than those jetted by an external jet pipe. Due to the weight of the concrete pile, a considerable portion of its length went down without the aid of driving. The last 5 feet or more was driven without a jet to secure desired penetration of a minimum of 70 blows per foot.

Bearing Values. Bearing values were figured by the Engineering News Formula. These values ran between 30 and 150 tons with an average of approximately 70 tons. Design loadings ran from 24 tons to 52.6 tons per pile.

Butt Cut-off. Generally the concrete piles were driven to grade. Driving to grade or butt cut-off was optional after 20 feet of penetration and when the resistance to driving exceeded 150 blows per foot with the full jet.The contractor utilized this option to cut-off 50 piles with a total linear footage of 245 or approximately 5 to the pile. The concrete was chipped away until the reinforcing bars were exposed, after which they were burned in two with a torch. The operation required about 2 hours and cost approximately $2.00 per cut-off. This cost was carried under the item of labor in driving costs.

Cushion Block. As a protection to the top of the piles, cushioning material was placed between the anvil block of the hammer and the top of the pile. Various materials such as wood, rope, belting and old hose are adaptable to this. However, 3" pine blocks were used due to having a large amount of salvage 3" pine decking on hand. These blocks were changed about twice per 18 hour day or one block served on an average of 20 piles.

The pile head breakage ranged from small hair cracks to complete shattering of 8" to 10" of the butt. In most cases the head breakage was decreased by frequent changing of cushions. However, there were so many instances where the head cracked after the cushion was changed that it is impossible to draw a direct ratio.

Follower Blocks. The driving of 50 piles to elevation 372 and 449 piles to elevation 376.5 was accomplished by the use of follower blocks made from the butt portions of oak piles. These followers ranged from 9 to 17 feet in length and 14 to 19 inches in diameter.

The upper end was capped with an iron head to fit the hammer block and the lower end was encased in a 1/2" iron band 2 feet long, which extended 3 or 4 inches producing a hood effect over the pile head. Four such followers were made up at an approximate cost of $30.00 each. This method of driving was found to be very satisfactory where 3 or 4 rows of piling were considerably below the grade of the others, which made it impracticable to drop the skid rig to this lower elevation. This was also advantageous where in such cases, the sand was not always excavated to the lower grades. The only disadvantage was the decreased production. By careful reference to bench marks and alignment it was possible to drive with accuracy comparable to that of the normal driving.

Driving Costs. The following costs are based on 126,758 linear feet of piling as of February 1, 1935.

Unit	Cost	Material	$1.10
"	"	Miscellaneous	.01
"	"	Labor	.25
"	"	Equipment	.06
"	"	Indirect	.27
		Total	$1.69 per foot

These costs may be broken down as follows:

Material included piling only delivered at the lock site of Lock and Dam No. 26.

Miscellaneous included pipe, connection nipples, cable,rope, timbers, skid grease, etc.

Labor included driving, butt cut-off, unloading, handling, maintenance, etc.

Equipment included plant rental, depreciation, repairs, storage, taxes, coal, lubricants, etc.

Indirect included (a) Legal; - Bond, Employees Liability Insurance, Marine Insurance, Interest on capital invested and (b) Indirect: - Overhead, mobilization and distributive.

The total indirect cost usually ran about 16% of the total cost.

Critique on Construction of Main Lock

General Procedure. - It is surprising in a pile job of this magnitude that some new and more effecient method of pile handling, plan of driving or work incentive payment was not evolved. On the contrary, although 413,217 linear feet of wood piling and 126,758 linear feet of concrete piling were used, totaling $377,202 in payment, this phase of the work was sadly neglected.

Partly in consequence, the contractor is reported to have suffered a loss of $36,960 aside from a very considerable time delay.

Actual driving on the land wall was started on June 6, 1934, at approximately station 3 + 50B and on the intermediate wall on June 14, 1934, at approximately station 3 + 60B. Due to a large extent to the plan and progress of driving, concrete was not poured until August 20, 1934. Had two drivers been centralized on each wall progressively advancing it would have been possible to start concreting within four weeks or on July 14, 1934. Although records of delay are very meager, it is estimated that 20% delay from all causes would be fairly conservative. By close planning the unavoidable delays could have been reduced to 10% or less.

Expensive delays in driving operation and material supply were caused by the tremendous excess of sand and water remaining in the cofferdam when it was unwatered. The sand was due to poor estimation or measurement of the depth to dredge. To secure ample room for driving and concreting piles of sand were rehandled by dragline and clamshell as much as four or five times. Of course, this was a tremendous economic waste. The first few weeks of driving, especially on the land wall, were done in from 2 to 4 feet of water. This reduced the accuracy of the work and was extremely expensive. Later, more well points were installed and water was not such an obstacle.

With the added month of working season and based upon November concrete production of 34,500 cubic yards, the main lock should have been 90% complete on March 11th, 1935.

The extremely low unit bid price of $0.33 per foot for wood piles in place demanded the keenest of planning and execution.

A "lost time" or "delay and causes" report would have been invaluable. Upon a request from the contractor, the Government inspectors could very handily have kept such a record.

At this time, with 34,000 cubic yards of concrete still to be placed and 80% of the structural steel still to be set for the main lock, and again on May 15, 1935, the contractor experienced flood stage, with its consequent loss of working time and tremendous damage. With the additional 4 weeks of work utilized in the fall working season the job should have progressed to the point that damage from flooding would have been negligible with, of course, a vast saving to the contractor.

The fifth driver used as a spare should have proven economical as a cushion for long delays. This could have been "spotted" in a locality removed from the immediate driving vicinity and in such condition that a crew could have occupied it at once in the event of their own rig going out of commission. With the estimated 20% delay time this would have filled up the weekly operation.

In most cases the driver foremen and crews were high grade as such. All were experienced men on floating and skid rig drivers. However, they lacked a work incentive. A bonus on driving footage or a competitive spirit could have been injected into the work with most surprising results. But to have any such scheme effective the pile driver rental contract would have needed revision, with the drivers rented as they were on a day basis from the owner who was in turn placed in charge at an attractive figure as general foreman. In fact, the contractor's engineer could very readily have planned and supervised the driving, a saving of dollars, and at the same time tending to more direct execution of the work.

Lateral Displacement and Upheaval. In mass pile driving little consideration at present is given to lateral displacement and upheaval, although these conditions are prevalent to a certain degree in practically all jobs of any magnitude.

Substrata, through which piling are driven, will compact only a certain amount depending upon the various physical characteristics. At a depth of 20 feet there would be a pressure of approximately one ton per square foot, superimposed by the surcharge of earth. It would be reasonable to assume that further compacting would be practically nil. At this depth a pile may be only half way to grade, therefore, continuation of the driving of mass piling spaced on 3-foot centers, is bound to produce a certain amount of displacement in some direction. It is impossible for an appreciable displacement to be in a downward direction, therefore, it must be lateral or upward. Ordinarily the displacement would be lateral, providing there are no restriction obstacles such as ledge rock, bridge piers, steel sheet pile, curtain walls, etc. If lateral movement is restricted then upheaval must result.

Charles Terzaghi, Professor of Foundations at the Massachusetts Institute of Technology, states that the amount of work required for rapidly reducing the volume of one cubic foot of laterally confined clay by two cubic inches may be 100 times as great as the amount of work required for producing the same volume change slowly. He terms this as the hydrodynamic stress phenomena. This inevitably develops as a result of rapid application of loads or pressures on water-soaked materials with a low degree of permeability, however, sand, which comprises the substrata of this structure, has a high degree of permeability.

Upheaval was appreciably minimized on this job due to the fact that the steel sheet piling was driven after the wood and concrete piles were in place, which allowed for lateral displacement. In one instance on the land wall between stations 0 + 93A and 1+ 80A the sheet piling was driven before the others, which resulted in a lateral displacement of the steel amounting to as much as 2 feet. This condition, however, was remedied by excavating on the landside of the steel and pulling the top back or approximately back on line.

A few instances where a portion of a pile fell outside of the face of the wall was not only due to incorrect pile location, (because any of the foremen could "spot" a driver much closer than this 18-inch discrepancy) but was also due to a certain amount of lateral displacement.

The writer made some 20 unofficial observations in the vicinity of the highway pier where driving conditions were somewhat restricted and found upheaval amounting to as much as 0.5 feet with an average of about 0.2 feet. These piles had low penetration on account of submerged riprap from the highway structure. This is not a criterion for the job. On the other hand, it probably represents the worst condition.

Generally there was no apparent upheaval due to the fact that the pile butts were sawed off to grade. By this time a stabilized position had been reached since most of the upheaval occurs at or immediately after the time of driving in piles adjacent to or in close proximity to those being driven.

Driving Formulae. There are approximately 85 driving formulae in use by various construction engineers throughout the world. The one that was used on this job and which seems to be most generally in use is the Engineering News Formula which was empirically derived by A. M. Wellington about 1889 from driving a limited number of wooden piles in sand by use of a drop hammer. It does not take into consideration any of the changes made in modern methods and in fact is such that it can not be modified. The formula is as follows:

For drop hammers:

$$P = \frac{2\ W\ H}{S + 1} \tag{1}$$

For single acting steam hammers:

$$P = \frac{2\ W\ H}{S + 0.1} \tag{2}$$

For double acting steam hammers:

$$P = \frac{2\ H\ (W + a\ P)}{S + 0.1} = \frac{2\ E}{S + 0.1} \tag{3}$$

in which P = safe load on pile, in pounds
W = weight of drop hammer, or striking parts of steam hammer, in pounds
H = fall of hammer or striking parts in feet
S = average penetration in inches per blow for last 10 to 20 blows
E = calculated energy of the ram in foot pounds per blow
a = effective area of piston in square inches
p = mean effective steam pressure in pounds/square inch

The above formula does not take into consideration the differential between the varying weights of piles, wherein one pile may weigh 2000 pounds and be driven with a 5000 pound ram and compared to another type of pile weighing 8000 pounds, but driven with the same ram.

If the safe supporting power of a pile is to be determined from the blows required to drive it, then certainly some consideration must be given to the relation of the weight of the pile, or mandrill, to the weight of the hammer. A practical knowledge of mechanics teaches that a 10 pound bar can not be driven as far into the ground with a one pound hammer as can a one pound tube, the same size as the bar, with a 10 pound hammer. The inertia of the mass has a pronounced effect.

Bearing values were figured on this job from the Engineering News formula where an identical hammer was used for wood piles weighing as little as 1000 pounds and concrete piles weighing as much as 6000 pounds; a matter of 600% difference in weight.

Wellington introduced constants to give this formula a reasonable value. Theoretically, it has a factor of safety of 6, that it, a wooden pile which, by the Engineering News formula, gives a working value of 15 tons, has an ultimate value of 90 tons, which is questionable for a small wooden pile.

To cover the use of steam hammers then coming into use, Wellington arbitrarily changed his denominator $S + 1$, to $S + 0.1$, giving no good reasons except that the more rapid blow of the steam hammer ought to give that much better results. Empirical evidence is too fragmentary to warrant such a step. As a matter of fact, there is much variation in the rapidity with which steam hammer blows are struck. The Vulcan single-acting hammer is nearly as slow as the drop hammer and the double-acting hammer is much faster.

The Engineering News Formula, now incorporated in many building codes, has done a great deal of harm, because its official character has misled owners and engineers as to its reliability. Lazarus White, President of Spencer, White & Prentis of New York, states that he has encountered cases where pile foundation conscientiously driven to the Engineering News formula, have failed and he has repeatedly checked results obtained by using the formula, testing the driven piles with hydraulic apparatus, and has found the formula decidedly inaccurate. The use of the Engineering News formula has checked progress. Because of its official adoption, engineers have often computed pile values by it and have not made tests which they would otherwise have made. In this connection, it should be noted that the results of a loading test on a single pile should not be conclusive to a group. Each pile of a rather closely spaced group, will on individual test, have a much higher value than the total of the group divided by the number of piles.

When piles are supported entirely by the friction between their sides and the earth, the load is transmitted to a deep ground level in a conoid of pressure through the earth above it. Such piles should be driven so far apart, or to such a depth, that the increased area of bearing developed by the conoid, or bulb, of pressure, which has the required altitude to contain the frictional resistance, reaches a level whose material will afford the required support before it intersects the corresponding conoid of an adjacent pile.

The only safe tests are those on a sufficiently large group to eliminate overlapping effects. Such tests will demonstrate the wastefulness of spacing of piles closely when not driven to rock, or an equivalent bearing.

The penetrations to be used in the Engineering News Formula should not be taken unless it has been at a reasonably uniform or uniformly decreasing rate. If the penetration is at an uneven rate it is probable that the pile is passing boulders or logs. If the penetration is practically zero, it is probable that the pile is against an impenetrable stratum or is already crushed. When the pentration has reached a small amount and the hammer rebounds considerably, it is safe to conclude that the limit of safe driving of that pile has been reached. Of course, the apparent penetration due to the brooming or shattering of the head, or the crushing of the body of the pile, or the bruising of the point should not be used in the formula for computing the bearing power. Care should be taken that the test blows are struck on sound wood or concrete as otherwise the observed pentration may be much too small and consequently the computed supporting power be much too great.

In view of the failure of the Engineering News formula to consider the varying weights of piles, it is suggested that the Navy Formula be given careful consideration. The Navy formula is as follows:

For single-acting steam hammers:

$$P = \frac{2\ WH}{S\left(1 + \frac{0.3w}{W}\right)}$$

Combining this with Engineering News formula No. 3, we have for double-acting steam hammers:

$$P = \frac{2\ H\ (W + ap)}{S\left(1 + \frac{0.3w}{W}\right)} = \frac{2E}{S\left(1 + \frac{0.3w}{W}\right)}$$

In which P, W, H, E, a, p and S are the same as in the Engineering News formula.

w = weight of pile in pounds including all appliances thereto such as hood, etc.

Charles Terzaghi in his various writings and publications has dealt the Engineering News formula some staggering blows. From his lifetime study and experimentation on soil mechanics and foundations he has drawn the following equations relative to the resistance against penetration of a pile under impact.

Based on the assumption that the bearing capacity of a pile can be computed from the effect of the blow -- let

R = weight of hammer.

G = weight of the pile.

L = length of the pile.

F = area of cross-section of the pile.

E = modulus of elasticity of the pile material.

h = distance the hammer drops.

s = penetration produced by one blow.

m = coefficient of elasticity of the impact; $m = 0$ for perfectly non-elastic impact; and $m = 1$ for perfectly elastic impact.

Q_d = resistance against penetration of the pile, under impact.

Q = ultimate bearing capacity of the pile under static load.

C = empirical constant depending on nature of the pile and the resistance against penetration = $\frac{Q_d L}{2 FE}$

The theory of semi-elastic impact leads to the following equation:

$$Q_d = \frac{F}{L} E \left[- s \pm \sqrt{s^2 + \frac{2\,Rh}{E} \; \frac{R + m^2 G}{R + G} \; \frac{L}{F}} \right] \qquad (1)$$

The value, m, is usually assumed equal to 0.5 (semi-elastic impact). For **m** = 0 the equation becomes Redtenbacher's formula, which is quite extensively used in Europe. On the other hand, if perfect elastic impact is assumed, m = 1, equation (1) becomes:

$$Rh = Q_d \, s + \frac{1}{2} \frac{Q_d^2}{F} \frac{L}{E} = Q_d \left(s + \frac{1}{2} \frac{Q_d^2}{F} \frac{L}{E} \right) \qquad (2)$$

$$\text{or, } Q_d = \frac{Rh}{s + \frac{1}{2} \frac{Q_d}{F} \frac{L}{E}} \qquad (3)$$

In view of the fact that the term, $\frac{1}{2} \frac{Q_d}{F} \frac{L}{E}$, depending on both the nature of the pile and the resistance against penetration, is disregarded, and if this variable term is replaced by an empirical constant, C, independent of all these factors, $Q_d = \frac{Rh}{s + C}$ (4)

which is none other than the Engineering News formula.

Of course, the most authentic means of obtaining local pile bearing values will always be vertical pile loading tests, such as used on this project. These could be carried somewhat further and from the results, curves of blows against actual load capacity could be developed. Such curves could be made for the actual pile groupings to be encountered. From these curves the needed penetration in blows could be used for actual driving.

A certain amount of thought and research might also be devoted to the construction or derivation of a driving formula or formulae that would be applicable to any and all conditions encountered in pile driving, taking into consideration such factors as weight, style and composition of pile; and cohesion, buoyancy, friction, plasticity and gradation of driving media.

A formula of such flexibility could be altered by means of constants to meet the widely varying physical conditions in each and every district or area under the jurisdiction of the United States Engineer Department.

PROCEDURE

Dam

General. In view of the fact that the general procedure on the dam was somewhat the same as that on the Main Lock it will be needless to elaborate extensively on this subject.

Specifications. The specifications stated that "timber piles shall be of oak, hickory, white ash, beech, rock elm, yellow birch, sugar maple, locust, black walnut, pecan, persimmon, or dense (close-grain) southern yellow pine conforming to the Tentative Specifications of the American Society for Testing Materials, for 'Timber Piles', serial designation D 25 - 34 T, 'Class C Piles', except that the minimum tip diameter for any length pile shall be 8 inches and all outer bark shall be removed. Other suitable, approved species may be used provided the butt and tip dimensions are sufficient to indicate resistance to shock and bending stress equivalent to the resistance of the above specified yellow pine".

All jetted piles were seated by driving not less than 5 feet after jetting had been stopped with the exception of some that encountered difficult driving in a few localities.

Payment for each pile driven as required, including test piles incorporated in the permanent work, was made to the nearest tenth of a foot at the respective contract price per linear foot, for round timber piling, up to 32 linear feet in place below the cut-off elevations as directed.

The following percentages cover the various increases in length for lengths greater than the base of 32 feet.

Increase in Length	Percentage
0.0 to 5 feet, inclusive	5
5.1 " 15 " "	15
15.1 " 25 " "	30
25.1 feet or greater	50

Payment was made for the cut-off portion of any pile at the rate of 50 per cent of the contract prices plus percentages for increased length provided above. The contractor bid $0.55 a linear foot in place on the 32-foot base length.

Procurement. The Collins Lumber Company and the Ross Lumber Company were subcontractors for the delivery of wood piling requirements, with the Ross Lumber

Company delivering less than 1,000 piles. In the neighborhood of 35 per cent of the piling was furnished from localities within a hundred-mile radius with the remainder being shipped from Tenessee, Kentucky, Missouri, Indiana, Arkansas and Louisiana. The piles from Louisiana were approximately 200 cypress used in the first section. It will be noted here that on account of the Main Lock partly depleting the local supply of suitable timber, it became necessary to import piling from a greater distance.

Shipments were made in the following ratio:

Barge - 40 per cent

Rail - 40 per cent

Truck and raft - 20 per cent

Inspection. Government inspection was made at the site, as well as in the field, by inspectors at the shipping points. Less than $1\frac{1}{2}$ per cent were rejected at the site, due principally to undersize, decay and excessive splitting. The rejected piling and the piling held over from one cofferdam to the next which showed excessive splitting was used for temporary construction in the cofferdam and Gantry trestles.

Driving. Piling for the first cofferdam was started August 30 and was completed December 16, 1935. Two skid-rig drivers were used for the piers, sills, and those piling under the downstream Gantry trestle. See Plate No. 23. Swinging leads were unsuccessfully used in driving approximately 100 piles in the apron foundation. Production in the first section of the cofferdam averaged 81.5 linear feet an hour or an average of 2.8 piles an hour by each rig for the 5,100 piles driven. Delays attributable to all causes amounted to 29 per cent of the crew operating time.

In the second section of cofferdam a barge-mounted skid-rig driver drove the temporary piling for the upstream trestle bents and enough piling at the start of the apron to receive and support the skid-rig driver. As shown in Plate No. 24, this driver continued the apron piles for the full length of the second section to enable the construction of the downstream Gantry trestle. This also partially shows the storage of piling along the cofferdam berms. After the completion of the apron piling, two skid-rig drivers, also shown in this picture, drove the balance of the piling for piers and sills advancing toward Illinois. The average driving production for the second section of cofferdam increased to 95.1 linear feet each crew-hour or 2.92 piles for the approximately 5,000 piles driven.

The drivers had been overhauled which

resulted in less delay attributable to repairs and the distribution of piling to serve the rigs was bettered.

The third section of cofferdam, it must be remembered, encompassed the contract for the completion of the auxiliary lock. Greater use was made of floating equipment. Three barges remained inside carrying one Whirley revolving crane and two standard skid-rigs. The upstream trestle piling was driven as well as the trestle piling running along the axis of the auxiliary lock by floating equipment. Sufficient start was also made by floating equipment on the apron so that a skid-rig driver could be unloaded. As shown in Plate No. 25, the apron piling was pushed to completion so that the downstream Gantry trestle could proceed and two drivers advanced toward Illinois completing all of the pier and sill piling. A third driver was started above the upper gate bay section on the river wall driving both concrete and timber piling for the river wall as it proceeded downstream. Pile driving started December 19, 1936, and was finished on April 25, 1937, with the exception of piling for the upper wall and miscellaneous struts. Production in the third section for the piling under the dam contract was again increased to 99.8 linear feet each crew hour or 3.1 piles each rig.

In lieu of requiring a one-foot cut-off wherever the pile butts showed no cracking or brooming, the piling was driven to grade in order to secure an additional foot of penetration. Eight railroad spikes were placed around the circumference of the pile approximately 10 inches from the top on all piling for the sills and aprons which had a tapered butt to fit the driving block. This was not required for the pier piling.

Tests. Provision in the contract was for 33 pile loading tests at a contract price of $500. each. However, only 13 tests were conducted. As shown in Figure No. 8 the actual loading results are compared with those computed from blows per last foot by the Engineering News Formula. Due to the 15 previously conducted pile tests within the main cofferdam of the twin locks contract, it was reasonably safe to assume the same conditions due to comparable material at depth underlying the dam foundation. Pile tests conducted verified this assumption. Based upon this, 37-foot piles were used for the piers, 32-foot for the sills, and 27-foot for the aprons. In addition, 24,287 linear feet of 43-foot piles were driven in lock areas where weakness was indicated.

The following pages will show the average results as obtained from pile tests conducted on the dam foundation.

Table No. IV

Bearing Values - Dam

Pier Nos.	Average Length of Supporting Piles	Average Computed Bearing Capacity of Supporting Piles
33	37.7 Feet	39.0 Tons
32	39.06 "	36.9 "
31	37.77 "	38.0 "
30	37.58 "	36.0 "
29	38.2 "	42.0 "
28	38.1 "	40.6 "
27	38.0 "	41.2 "
26	38.0 "	38.5 "
25	37.9 "	35.8 "
24	37.6 "	41.0 "
23	37.7 "	41.0 "
22	38.2 "	37.9 "
21	38.2 "	38.2 "
20	37.6 "	38.0 "
19	37.8 "	35.1 "
18	38.3 "	34.8 "
17	38.9 "	35.4 "
16	38.5 "	34.8 "
15	38.4 "	35.5 "
14	37.5 "	35.1 "
13	37.5 "	32.2 "
12	37.7 "	35.1 "
11	37.7 "	38.1 "
10	38.0 "	34.3 "
9	37.9 "	33.7 "
8	37.8 "	33.4 "
7		

VERTICAL PILE LOADING TEST NO. 1

Location - Pier No. 30, 3/ 11.8C - 9.0A

Kind of wood - Oak, Size of Pile: Butt 12½, tip 9½, Length, 38'.
Final penetration without jet, 15'; cut-off elevation, 384; penetration, 36'.
No. of blows last 12" - 15; safe load in tons (E. N. Formula)= 16 tons.

TABULATION OF TEST RESULTS

Load Tons	Time Loaded	Time Read	Settlement Feet	Accumulated Settlement
0	-	9:30 AM	-	-
10	10:45 AM	11:15	0.006	0.006
		11:40	.001	.007
20	11:45	11:50	.0055	.0125
		12:40 PM	.0065	.019
30	12:45	12:50	.0025	.0215
		1:40	.0025	.024
35	1:45 PM	1:50	.0035	.0275
		2:40	.0005	.028
40	2:45	2:50	.004	.032
		3:40	.003	.035
50	3:45	3:50	.0065	.0415
		7:40	.0125	.054
60	7:45	7:50	.003	.057
		*7:45 AM	.0085	.0655 9/14/35
		1:40 PM	.0035	.069 "
0	Unloaded 1:45	2:00		0.069
				0.031 Recover
			Permanent Settlement	0.038

* Jack lost 15 tons pressure between midnight and 8:00 A. M. Reading taken on 45 tons.

Uplift on Anchor Pile No. 1 = 0.0125; No. 2 = 0.0125; Av.= 0.01250

Recovery Do. = 0.0045; Do. = 0.008; Av. = 0.00625

Permanent Uplift = .00625

VERTICAL PILE LOADING TEST NO. 4

Location - Abutment. Sta. 1/15.0C - Station 16.0B

Kind of wood - Oak. Cut-off elev. 381.0. Penetration 42 ft.
Size of pile: Butt 13' - Tip 8" - Length 43'. Final penetration without jet - No. jet.
No. of blows last 12" - 25. Safe load in tons (E. N. Formula) = 25.

TABULATION OF TEST RESULTS

Load Tons	Time Loaded	Time Read	Settlement Feet	Accumulated Settlement
0 Sept.28		10:25 AM	-	-
10	10:30 AM	10:35 AM	.0055	.0055
		11:25 AM	.000	.0055
20	11:30 AM	11:35 AM	.0065	.012
		12:25 PM	.0005	.0125
25	12:30 PM	12:35 PM	.0035	.016
		1:55 PM	.000	.016
30	2:00 PM	2:05 PM	.002	.018
		2:55 PM	.004	.022
40	3:00 PM	3:05 PM	.003	.025
		3:55 PM	.002	.027
50	4:00 PM	4:05 PM	.003	.030
		-		
" Sept.29		8:00 AM	.007	.037
		5:00 PM	.005	.042
" Sept.30		8:00 AM	.0015	.0435
		4:00 PM	.000	.0435
" Oct. 1		8:00 AM	.0005	.043
		4:00 PM	.001	.042
" " 2		8:00 AM	.001	.041
		4:00 PM	.0005	.0415
" " 3		8:00 AM	.001	.0405
		1:25 PM	.001	.0395

VERTICAL PILE LOADING TEST NO. 4(Continued)

Load Tons	Time Loaded	Time Read	Settlement Feet	Accumulated Settlement
30	1:30 PM	1:50 PM	.005	.0345
10	2:00 PM	2:20 PM	.0125	.022
0	2:30 PM	3:00 PM	.0105	.0115

Permanent Settlement

Uplift on Anchor Pile No. 1 = + 0.0485
No. 2 = + 0.0215. Av. = + 0.035

Recovery " " No. 1 = 0.0160
No. 2 = 0.0105. Av. = 0.01325
Permanent Uplift = 0.02175

VERTICAL PILE LOADING TEST NO. 7

Physical Data of Test Pile No. 7 Located in Pier No. 18 at the Intersection of Station 937.604C with Station 59.0B

Length	43'-0"
Butt Diameter	12-3/4"
Tip Diameter	9"
Kind of Wood	Oak
No. of Blows last Foot of Penetration	38
Penetration	42.5'
Final Penetration without Jet	No Jet Used
Computed Safe Load in Tons Using the Engineering News Formula	35 Tons
Cut-off Elevation	380.0

Foundation Properties as Revealed by Wash Borings Prior to Driving Operations

Test Hole No. 33, located 110.2 feet towards the Missouri shore and 59.0 feet upstream from the test pile, indicates a stratum of saturated medium sand from Elevation 382.7 to Elevation 349.4 and thence saturated fine sand to Elevation 302.7.

This investigation indicates the character of the bearing sub-strata to be almost entirely saturated medium sand with perhaps a few feet of saturated fine sand at the tip of the pile.

RESULTS OF PILE LOADING TEST
(Load applied by use of hydraulic ram and jack beam)

Load in Tons	Time Loaded 7/9/36	Time Read	Settlement in Feet	Accumulated Settlement
0		6:50 AM		
10	7:00 AM	7:10 AM	.0025	.0025
		7:50 AM	.000	.0025
20	8:00 AM	8:10 AM	.002	.0045
		8:50 AM	.0005	.005
30	9:00 AM	9:10 AM	.003	.008
		9:50 AM	.000	.008
40	10:00 AM	10:10 AM	.0025	.0105
		10:50 AM	.0005	.010
50	11:00 AM	11:10 AM	.004	.014
		11:50 AM	.000	.014

RESULTS OF PILE LOADING TEST (Continued)

Load in Tons	Time Loaded 7/9/36	Time Read	Settlement in Feet	Accumulated Settlement
60	12:00 AM	12:10 P. M.	.006	.020
		12:50 PM	.002	.022
70	1:00 PM	1:10 PM	.0025	.0245
		1:50 PM	.002	.0265
80	2:00 PM	2:10 PM	.0045	.0310
		2:50 PM	.0035	.0345

At 3:00 PM the superimposed load was reduced from 80 tons to 40 tons and at 3:30 PM a recovery of .005 feet was noted. Then the entire load was removed and at 4:00 PM a recovery of .0195 feet was noted. The total recovery was .0245 feet as compared with a permanent settlement of .010 feet.

VERTICAL PILE LOADING TEST NO. 12

Physical and Driving Data of Test Pile No. 12 Located in Pier No. 7 at the Intersection of Station 1567.40 with Station 40.0B - Pile D

Length of Pile Driven	38 Ft.
Butt Diameter	14 In.
Tip Diameter	$8\frac{1}{2}$ In.
Kind of Wood	Oak

- -

No. of Blows Last Foot of Penetration	36
Length Below Cut-off Elevation (Subgrade Elevation 372.0 at time of Test)	36 Ft.
Actual Penetration	28 Ft.
Computed Safe Load in Tons Using the Engineering News Formula	34 Tons

NOTE: Physical data given within dashed limits applies to pile at time of test, but is not to be considered as final since this pile was later driven to grade. Top of pile at time of test was at Elevation 382.0, as compared with a final specified cut-off elevation of 380.0.

- -

Test Pile No. 12, one of a group of 15 piles in the downstream foundation area of Pier No. 7, was driven January 12, 1937, in the wet by Floating Rig No. 2, using a Vulcan No. 1 Hammer. The water level within the cofferdam was at Elevation 384.0, as compared with a river level at Elevation 402.5 recorded at 7:00 A. M. No jet was used in driving operations.

At the start of the loading test Skid Rig No. 2 was driving in Sill No. 8; at the finish of the recovery test the same rig was driving in Sill No. 7.

Foundation Properties as Revealed by Wash Borings Prior to Driving Operations

Test Hole No. 30, located 82.3 feet towards the Missouri shore and 40.0 feet upstream from the test pile, indicates the following stratification: Elevation 382.6 (Natural River Bed as of date wash boring was taken) to Elevation 368.6, saturated medium fine sand; Elevation 368.6 to Elevation 338.6, saturated fine and very fine sand with a trace of pea gravel; Elevation 338.6 to Elevation 328.0, saturated coarse sand with a trace of pea gravel; Elevation 328.0 to Elevation 308.6, saturated medium sand; Elevation 308.6 to Elevation 302.6, saturated fine sand.

With reference to the boring log, it is quite obvious that the bearing sub-strata consists almost entirely of saturated sand, varying from coarse to very fine, with perhaps a trace of pea gravel.

RESULTS OF PILE LOADING TEST
(Load applied by use of hydraulic ram and jack beam)

Load in Tons	Time Loaded 1/28/37	Time Read	Settlement in Feet	Accumulated Settlement
0		8:55 AM	----	----
10	9:00 AM	9:05 AM	.000	.000
		9:55 AM	.001	.001
20	10:00 AM	10:05 AM	.0035	.0045
		10:55 AM	.000	.0045
30	11:00 AM	11:05 AM	.0025	.007
		11:55 AM	.0005	.0075
40	12:00Noon	12:05 PM	.0045	.012
		12:55 PM	.001	.013
50	1:00 PM	1:05 PM	.0035	.0165
		1:55 PM	.001	.0175
60	2:00 PM	2:05 PM	.0065	.024
		2:55 PM	.001	.025
70	3:00 PM	3:05 PM	.012	.037
		3:55 PM	.0055	.0425
80	4:00 PM	4:05 PM	.014	.0565

This 80-ton load was sustained until the following day, January 29, 1937, at 8:45 A.M., when an additional settlement of .0345 feet was noted, making the accumulated settlement .091 feet.

RECOVERY DATA

Remaining Load in Tons	Time of Load Reduction	Time Read	Recovery in Feet	Diminished Settlement
40	9:00 AM	9:05 AM	.0105	.0805
		9:55 AM	.003	.0775
20	10:00 AM	10:05 AM	.0085	.069
		10:55 AM	.0015	.0675
0	11:00 AM	11:05 AM	.0135	.054

It will be noted that the permanent settlement is .054 feet as compared with a recovery of .037 feet. It is interesting to note that five minutes after the load was increased in the loading test or decreased in the recovery test an appreciable settlement or recovery was noted, whereas a reading fifty minutes later in the loading or recovery test indicated no appreciable settlement or recovery, as the case might be.

GROUTING UNDER BASE OF DAM

When excavation for the concrete aprons at the toe of the dam was done, it was disclosed that percolation of water had caused removal or subsidence of the sand under some of the downstream ends of the pier bases and sills, leaving voids between the foundation concrete and the underlying sand.

Prior to the concrete placement for the aprons, two-inch grout pipes were inserted in these voids leaving the upper ends of the pipes about three feet above the grade of the concrete. Twenty-eight of these pipes were used in the first section.

This operation was carried out during the month of February, 1936. Grout was injected by means of a home-made grout machine in conjunction with a 100 cubic foot per minute air compressor. In general, neat cement was used for the first batch, followed by a 1 : 2 mix of cement and sand. A uniformly increasing pressure was applied up to an adopted refusal of ninety pounds per square inch. This pressure, however, could not be reached in cases where the grout began to flow from the weep holes in the downstream edge of the apron, which occurred in some instances with pressures as low as ten to twenty pounds. In view of this, it was believed that the voids were not completely filled and, as a result, eleven holes were drilled in the centers of sills. An average of 21.5 cubic feet of grout was pumped into each hole and pipe.

Grouting of the second section was done in September, 1936. The procedure was much the same as that carried out in the first section. The maximum air pressure was reduced from ninety pounds per square inch to fifty pounds per square inch in sills No. 15 and No. 16, there being danger of forcing grout into the uplift pressure boxes located in the bases of these sills. Thirty-one grout pipes were installed in this section, having an average capacity of 12.7 cubic feet.

A similar procedure was carried out in the third section.

When grouting was completed the pipes were burned off flush with the surface of the concrete.

Glossary of Terms Used in Foundations

Berm--	A bank of sand or earth usually used for retaining a structure and/or extending beyond the outcrop of the hydraulic gradient or plane of saturation.
Boil--	A run of wet material, usually quicksand, on the bottom of an excavated hole or inclosure or under the sheeting of an excavation, due to greater water pressure on the outside than on the inside (an ordinary spring is a boil carrying water only).
Brace--	A strut or pusher, usually horizontal, though sometimes inclined, of timber or steel, acting in compression to hold earth material or a structure against lateral movement.
Bulb or Conoid of pressure--	The mass of compressed material under a pile, group of piles, or a footing.
Burn--	To cut off with an oxyhydrogen or oxyacetylene flame.
Burning Torch--	Equipment for burning with an oxyhydrogen or oxyacetylene flame.
Caisson--	Watertight box or cylinder, usually of wood, concrete, or steel sheeting, used for the purpose of making an excavation. Caissons may be either open (that is, open to the free air) or pneumatic (that is, under compressed air).
Cant--	To turn over; to tilt; to slant.
Cant Hook--	A lever or hook for turning over timbers or piles.
Clay--	Heavy, compact, cohesive soil which is stiff and smooth to the touch when wet and hard when dry, composed largely of aluminum and silica compounds.
Cofferdam--	A temporary enclosing dam constructed of

earth, wood, or steel sheeting, or a combination of these, built in the water and pumped dry for the purpose of expediting the construction of subaqueous footings or foundations. This term is usually applied to large enclosures, there being practically no limit to the extent of area.

Cut-- An excavation, usually a trench; the differential between original ground and an established grade below ground.

Follower-- A member interposed between the hammer and a pile to transmit blows to the latter when below the foot of the leads.

Footing-- The lowest element of a foundation; that part which bears on the substratum.

Foundation-- That on which a structure is built; base.

Grillage-- Horizontal reinforcing members, usually of steel, and embedded in concrete, for spreading or distributing the load of a structure over its footing or foundation.

Grout-- A mixture of cement, sand, and water usually used for filling ordinarily inaccessible voids or cavities; also used to insure a good bond where concrete is poured on "set" concrete. In the former case, it is usually applied by means of air pressure.

Hardpan-- A compact, cemented mass of clay with sand, gravel, or boulders, or any two or all of them, usually of glacial origin. The term is rather loosely used but always implies a very dense material difficult to excavate without picks or the equivalent.

Hammer-- A weight or machine used to deliver blows to a pile to secure its penetration.

Drop Hammer--One which is raised by means of a line and then allowed to drop.

Steam Hammer--One which is automatically raised and dropped a comparatively short distance by the action of a steam cylinder and piston supported in a frame which follows the pile.

Air Hammer--Same as steam hammer except that it is powered by compressed air.

Anvil Block--A heavy steel disc that rests on the head of the pile and receives the blow from the ram.

Ram--That part of a steam or air hammer that delivers the blow.

Head-- Potential force or pressure of water line to difference of elevation.

Hood-- An encasing of steel attached to bottom of hammer usually used for protection of heads of concrete piling.

Hydraulic Jack-- A hydraulic cylinder and piston used to raise weights, to force piles, cylinders, and the like into the ground; or to pretest piles, footings, braces, shores, etc. The jack may be operated by a separate hydraulic pump, or the jack and pump may be built as one unit.

Jet-- A pipe end or nozzle from which water is emitted under pressure. Also the water emitted from the nozzle. As a verb, it means the process of using such a jet to loosen the ground to facilitate the driving of a pile or to loosen and suspend the material adjacent to a pile so that it may be removed by pulling.

Lateral Movement-- Horizontal movement or displacement of a structure, substratum, sheeting, or bracing.

Lateral Pressure-- The horizontal component of the force due to a wedge of earth (and surcharge, if any) moving or tending to move downward along its natural cleavage plane.

Lost Ground-- Material which runs into an excavation under or through the sheeting or cribbing, or as a boil in the bottom; or material outside the sheeting which moves downward as a result of the run or boil or because of voids left behind the sheeting when it was placed or because of movement of the sheeting; or material lost by percolation of water.

Mat-- A footing (or grillage) of timber, steel, or concrete, to support a post, shore, or column.

Muck-- Excavated material or material to be excavated; also a soil largely composed of vegetable matter in the early stages of decomposition.

Neat Cement-- Portland cement alone, that is, without sand.

Net Line (or Neat Line)-- The limit within which no material may be left in an excavation. It refers to the sides, not to the bottom or subgrade of an excavation.

Orange Peel-- A type of self-opening and closing bucket in the shape of a half orange peel cut into segments used on a line to excavate soft material from a sump, caisson, or elsewhere.

Pile-- A column of wood, reinforced concrete, or steel, usually less than twenty-four inches in diameter, driven and/or jetted, or jacked into the ground and deriving its support from the underlying strata and by the friction of the ground on its surface. The usual functions of a pile are (a) to carry a superimposed load; (b) to compact the surrounding ground; (c) to form a wall to exclude water and soft material, or to resist the lateral pressure of adjacent ground.

Pile Head--The upper end of a pile; the top.

Pile Foot--The lower extremity of a pile.

Pile Butt--The larger end of a pile. The diameter is usually measured three feet from the butt.

Pile Tip--The smaller end of the pile.

Batter Pile--One driven at an inclination to resist forces which are not vertical.

Bearing Pile--One used to carry a superimposed load.

Disc Pile--One having a disc attached to its foot to provide a larger bearing area.

Screw Pile--One having a broad-bladed screw attached to its foot to provide a larger bearing area.

Sheet Pile--Piles driven in close or interlocking contact in order to provide a tight wall to prevent leakage of water and soft materials, or driven to resist the lateral pressure of adjacent ground.

Pile Cap--A metal block used to protect the head of a pile and to hold it in the leads during driving.

Cushion Block--Material such as laminated wood, rope, etc., placed between the ram and the anvil, or directly in contact with the pile, as in concrete, to protect head of the pile from shattering or brooming.

Ring--A metal hoop used to bind the head of a wood pile during driving.

Shoe--A metal protection for the point or foot of a pile.

Pile Driver-- A machine for driving piles.

Leads-- The upright parallel members of a pile driver which support the sheaves used to hoist the hammer and piles, and which guide the hammer in its movement.
Niggerhead-- A winch head or drum on a pile driver hoist engine around which a line can be placed by hand and the hoisting done by pulling by hand on the line to give it a friction grip on the drum. The jet pipe is usually handled in this manner.

Sheave--A fixed, pulley-like wheel around which a line is passed.

Snatch Block--A movable sheave or pulley block with hook attached, arranged so that a bight of rope can be entered.

Skid--The bottom timbers of a pile driver which slide over caps or "skid timbers" drifted to tops of piles.

Winch--A small hoist usually operated by steam, gasoline, electricity, or compressed air.

Quicksand-- Fine sand (frequently with an admixture of clay- and water which easily runs and boils.

Run-- A flowing of material, usually provoked by hydrostatic head, into an excavation, either under or through the sheeting, or into voids left behind sheeting.

Running Ground--Either water-bearing sand or very dry sand which will not stand up without sheeting.

Sand-- Granular silicious material, the grains being larger than dust and smaller than gravel.

Seal--	To close off permanently the bottom of a form, caisson, or other excavation, usually by pouring in grout or concrete so that the water or earth can not flow in.
Settlement--	Downward movement of a structure, part of a structure, fill, etc.
Sheating (or Sheeting)--	Horizontal or vertical members of wood or steel placed in contact with earth, usually on a vertical plane for the purpose of retaining earth or excluding earth or water.
Shore--	An inclined brace of timber or steel.
Soft Ground--	Earth as contrasted to "hard ground" or rock. Usually with depth, soft ground is water-bearing and semi-fluid.
Spreader--	A brace between two wales.
Subgrade--	The final level to which an excavation (or fill) is to be made.
Sump--	A small depression or excavation in which water is collected for pumping.
Test--	To find the bearing capacity of a pile, pile group, pier, or footing usually by means of loading or hydraulic jacks. Such tests are not usually made up to the full bearing power of the pile or footing, but to some predetermined limit, frequently about 200% of the maximum design load.
Tongue and Groove--	Sheeting (usually wood) in which one edge of the sheet is cut with a projecting tongue which fits into a corresponding groove or recess in the edge of the next sheet.
Underpinning--	The addition of new permanent support to existing foundations, either to provide additional capacity or additional depth.

STRATIGRAPHY OF THE ALTON VICINITY

Table No. I

Era	Period	Epoch	Stage	Rock	Geological Events
		Recent		Alluvium	Weathering, erosion, alluviation.
Cenozoic	Pleistocene			Drift, till, loess	Continental Glaciation.
	Tertiary			Gravel and clay	Area probably above sea level mainly erosion.
Mesozoic				None	
			Unconformity		Elevation; slight tilting in direction north of east.
	Pennsylvanian			Shale, sandstone, limestone, coal	Many cycles of elevation and submergence; alternation of marine and terrestial sedimentation.
			Unconformity		Eastward tilting; slight folding and long erosive interval.
		Chester		None	
			Unconformity		Regional tilting and beveling of eastward dipping strata.
		Iowa Series: Meramec	Ste. Genev., St. Louis	Limestone	Marine sedimentation.
			Unconformity		Partial withdrawal of sea toward southeast.
			Salem		
Paleozoic	Mississippian	Osage	Warsaw, Keokuk, Burlington, Fern Glen	Limestone	Marine sedimentation; oscillatory conditions during Warsaw and Salem stages.
			Unconformity		Erosion and regional tilting toward east.
		Kinderhook	Hannibal	Shale	Marine sedimentation.
			Unconformity		Down-warping to south.
			Sweetland Crk.	Shale	Marine sedimentation.
			Unconformity		Erosion.
	Devonian	Senecan		Limestone	Marine sedimentation.
			Unconformity		Tilting to northwest; erosion of higher land masses on Ozark flanks which continued as land through early Devonian times.
	Silurian	Niagran		Dolomite	Marine sedimentation.
			Unconformity		Emergence; erosion.
		Alexandrian	Sexton Creek, Edgewood	Limestone	Marine sedimentation; regional depression and tilting.
			Unconformity		Emergence and tilting.
	Ordovician	Cincinnatian	Maquoketa	Shale	Marine sedimentation; local down-warping.
		Mohawkian	Kimmswick, Plattin	Limestone	Marine sedimentation.
		Chazyan	Joachim		
			St. Peter	Sandstone	Marine sedimentation.

EQUIPMENT CHART — Table No. III

for Main Lock Pile Driving

No.	Designation	Trade Name	Size or Capacity	Power	Purchase Price New	Book Value	Salvage Value	Yearly Depreciation	Daily Rental
4	Pile Driver*	Skid-Rig		Steam (35 hp.)					$25.00
1	Crane	Brownhoist	15	Steam	$16,000.	$11,300.	$800.	10%	9.42
2	Crane	Orton	5½	Gasoline	6,000.	4,375.	120.	14%	4.23
1	Crane	Orton	11	Gasoline	9,000.	7,980.	150.	14%	7.71
1	Crane** (Dragline)	Northwest	1¼ C.Y.	Gasoline (80 hp.)					20.00
1	Tractor	Caterpillar		Diesel (50 hp.)	4,500.	4,500.	100.	20%	6.75
1	Bulldozer	Bulldozer			1,453.	1,453.	53.	10%	2.33
1	Derrick-Stiffleg		110' Boom	Electric (100 hp.)	5,000.	2,000.	50.	13%	1.82
1	Jet Pump	Fairbanks-Morse	5-Inch	Electric (100 hp.)	1,299.	1,299.	49.	17%	2.22

*Rented from R. C. Bolduc Company, Minneapolis.

**Rented from O. B. Avery, St. Louis.

Acknowledgements

The work herein described was designed by and constructed under the direction of the United States Engineer Department. For the Corps of Engineers Major E. P. Ketchum was in immediate charge of the Lock and Dam Section of the St. Louis District under Lieutenant-Colonel P. S. Reinecke, District Engineer, with Mr. L. B. Feagin as Senior Engineer in charge of Engineering, J. A. Adams, Associate Engineer, in charge of construction and Harry S. Pence, Associate Engineer, Resident Engineer on Lock and Dam No. 26.

For the Locks contractor, John Griffiths & Son Company, Mr. Bruce A. Gordon was General Superintendent, James Rittenhouse, Superintendent, and S. B. Christopher, Engineer. For the Dam contractor, Engineering Construction Corporation, Frederick B. Spencer was Works Manager, with Robert Dunlap, George Ferris, and D. C. Andrews as Superintendents.

The writer is under obligations to Harry S. Pence for helpful information, advice and various services which required much of his valuable time; to Lazarus White and Edmund A. Prentis of Spencer, White & Prentis, for useful and pertinent information gleaned

through technical discussions of the subject foundations; and to other members of the Lock and Dam organization who had occasion to assist in the assembly of data contained herein.

John J. Livingston, Assistant Engineer
United States Engineer Department

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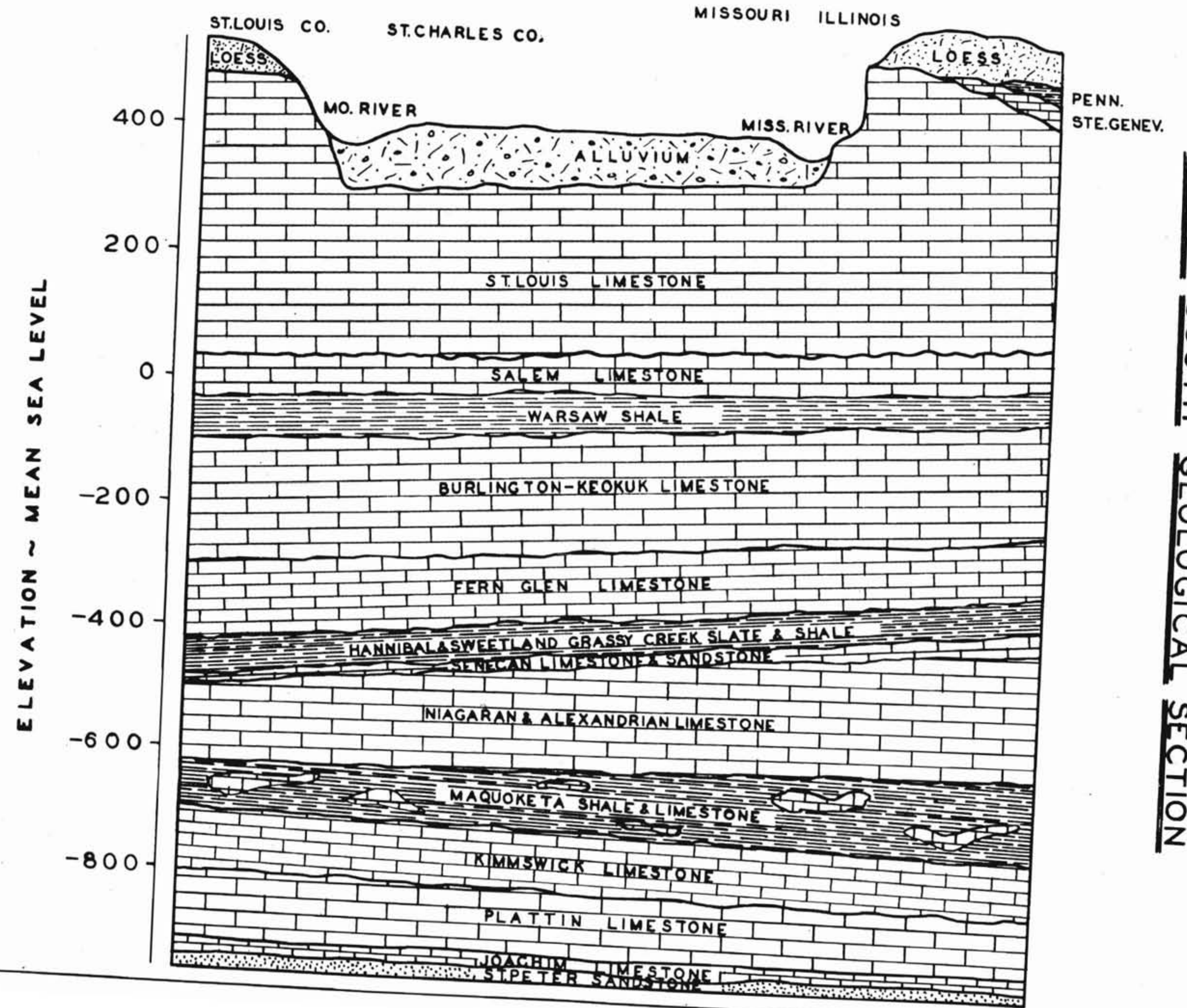
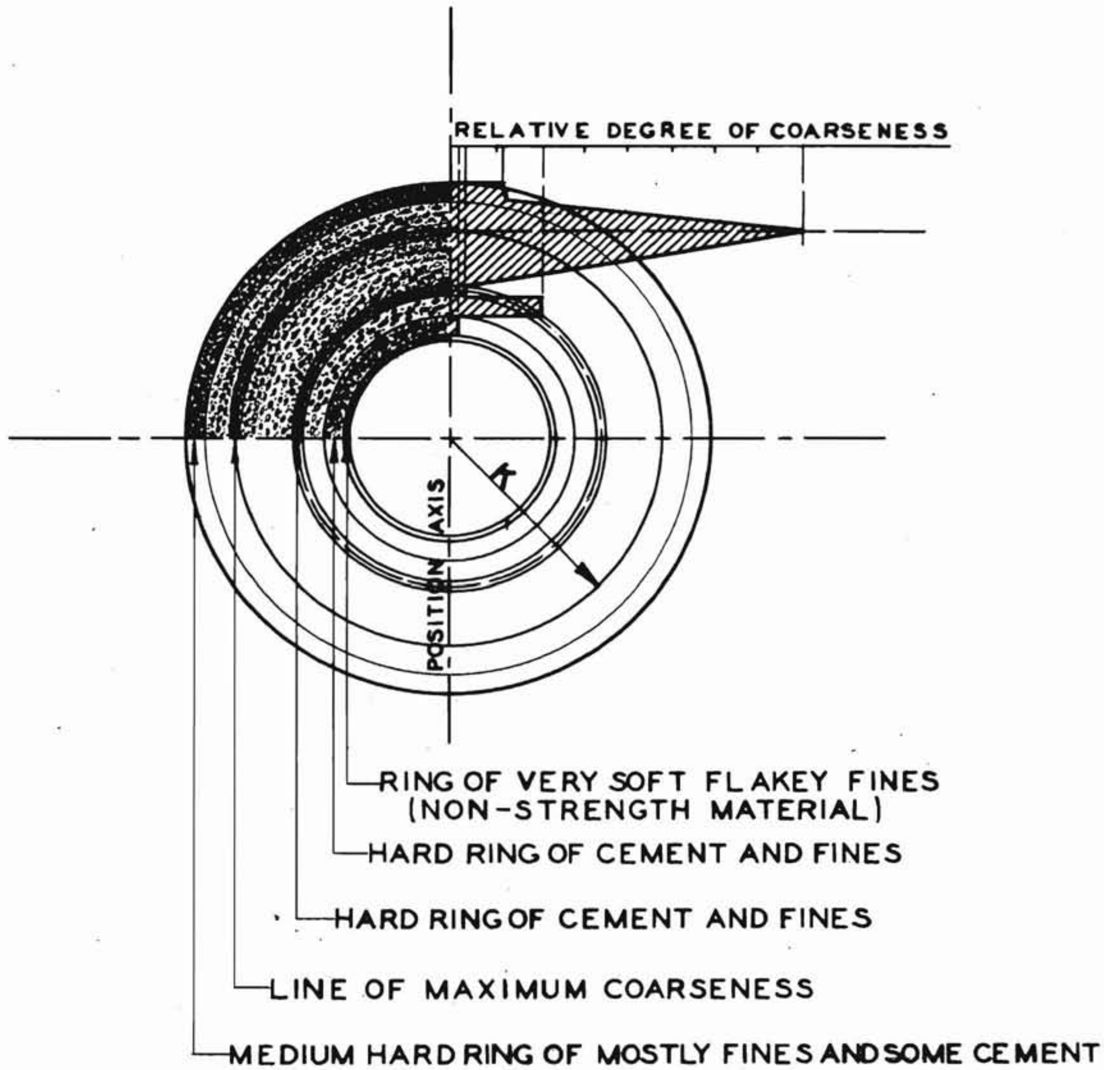

NORTH-SOUTH GEOLOGICAL SECTION
ELEVATION ~ MEAN SEA LEVEL
400
200
0
-200
-400
-600
-800
ST.LOUIS CO.
ST.CHARLES CO.
MISSOURI
ILLINOIS
LOESS
LOESS
MO. RIVER
MISS. RIVER
PENN.
STE.GENEV.
ALLUVIUM
ST.LOUIS LIMESTONE
SALEM LIMESTONE
WARSAW SHALE
BURLINGTON-KEOKUK LIMESTONE
FERN GLEN LIMESTONE
HANNIBAL & SWEETLAND GRASSY CREEK SLATE & SHALE
SENECAN LIMESTONE & SANDSTONE
NIAGARAN & ALEXANDRIAN LIMESTONE
MAQUOKETA SHALE & LIMESTONE
KIMMSWICK LIMESTONE
PLATTIN LIMESTONE
JOACHIM LIMESTONE
ST.PETER SANDSTONE

Fig. No. 6

CROSS-SECTION OF TYPICAL TEST

CYLINDERS-PICKED AT RANDOM



ONE-HALF SCALE

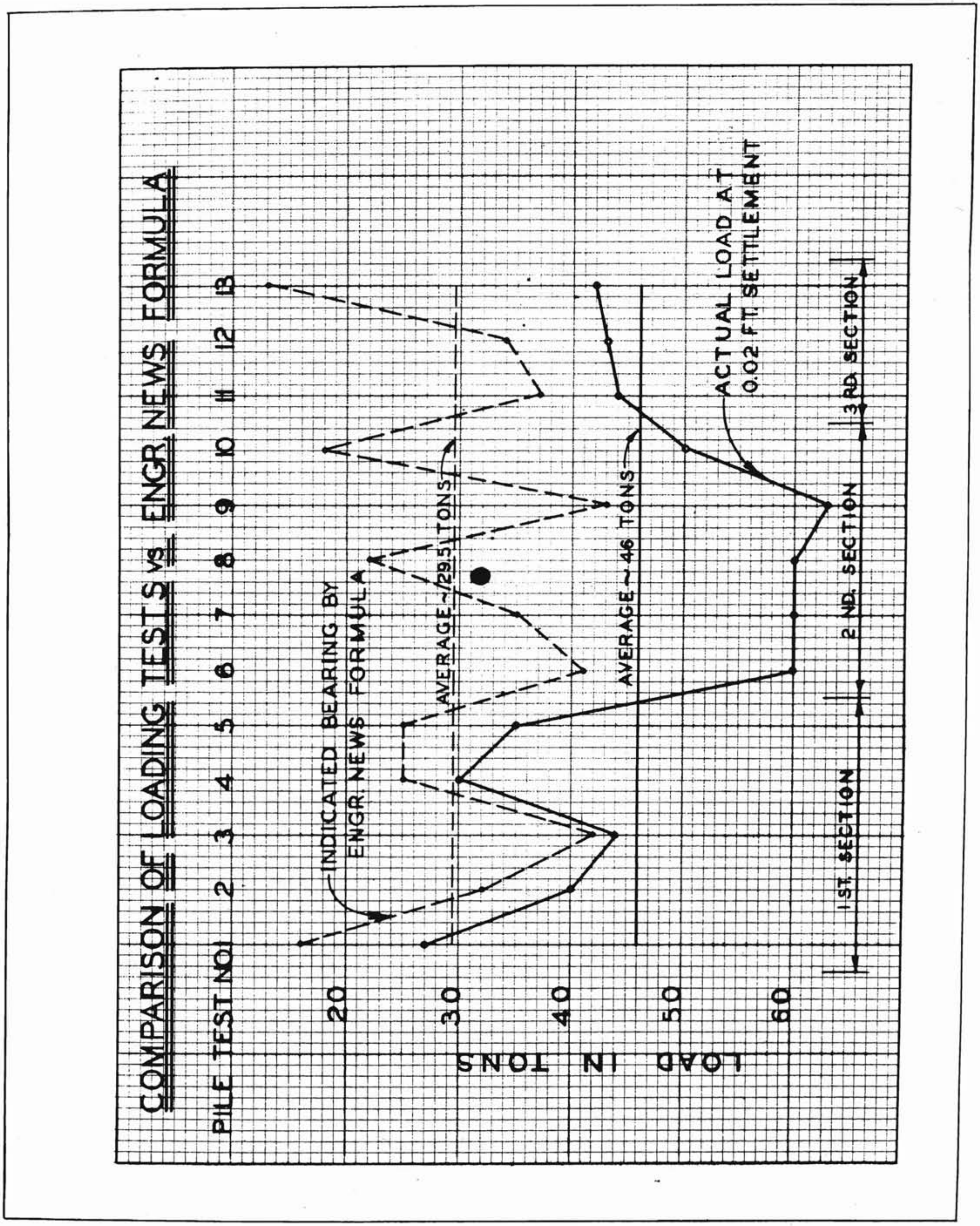

COMPARISON OF LOADING TESTS VS ENGR. NEWS FORMULA
PILE TEST NO. 1 2 3 4 5 6 7 8 9 10 11 12 13
LOAD IN TONS
20
30
40
50
60
INDICATED BEARING BY ENGR. NEWS FORMULA
AVERAGE ~ 29.5 TONS
AVERAGE ~ 46 TONS
ACTUAL LOAD AT 0.02 FT. SETTLEMENT
1ST. SECTION
2 ND. SECTION
3RD SECTION

Fig. No. 8

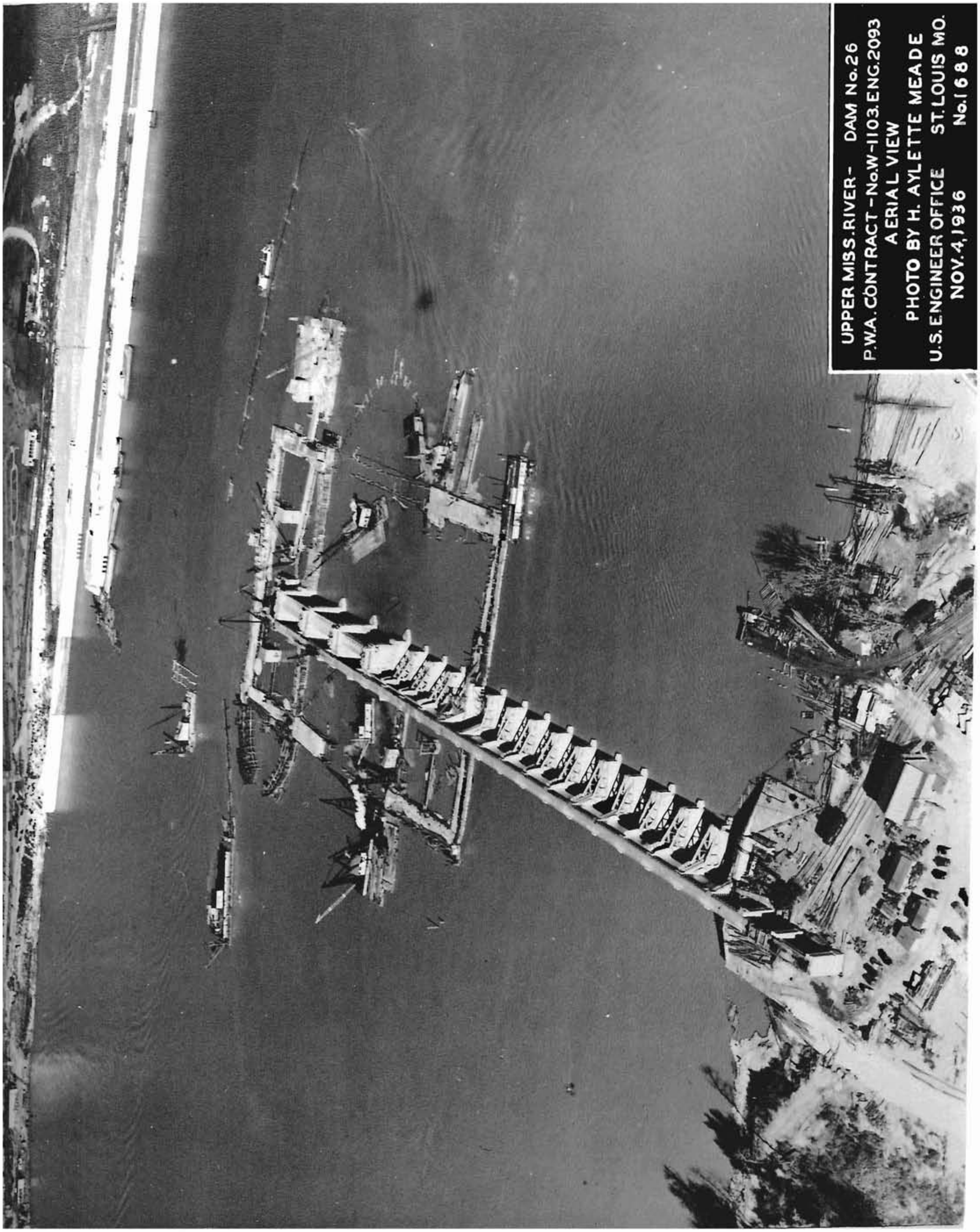

UPPER MISS.RIVER- DAM No.26
P.W.A. CONTRACT-No.W-1103.ENG.2093
AERIAL VIEW
PHOTO BY H. AYLETTE MEADE
U.S.ENGINEER OFFICE ST.LOUIS MO.
NOV.4,1936 No.1688

PLATE 1


UPPER MISS. RIVER-LOCK NO. 26
PWA-CONTRACT-NO. W-1103 ENG. 1706
GENERAL VIEW DOWNSTREAM
U. S. ENGINEER OFFICE ST. LOUIS, MO.
JUNE 29 NO. 172

PLATE 2

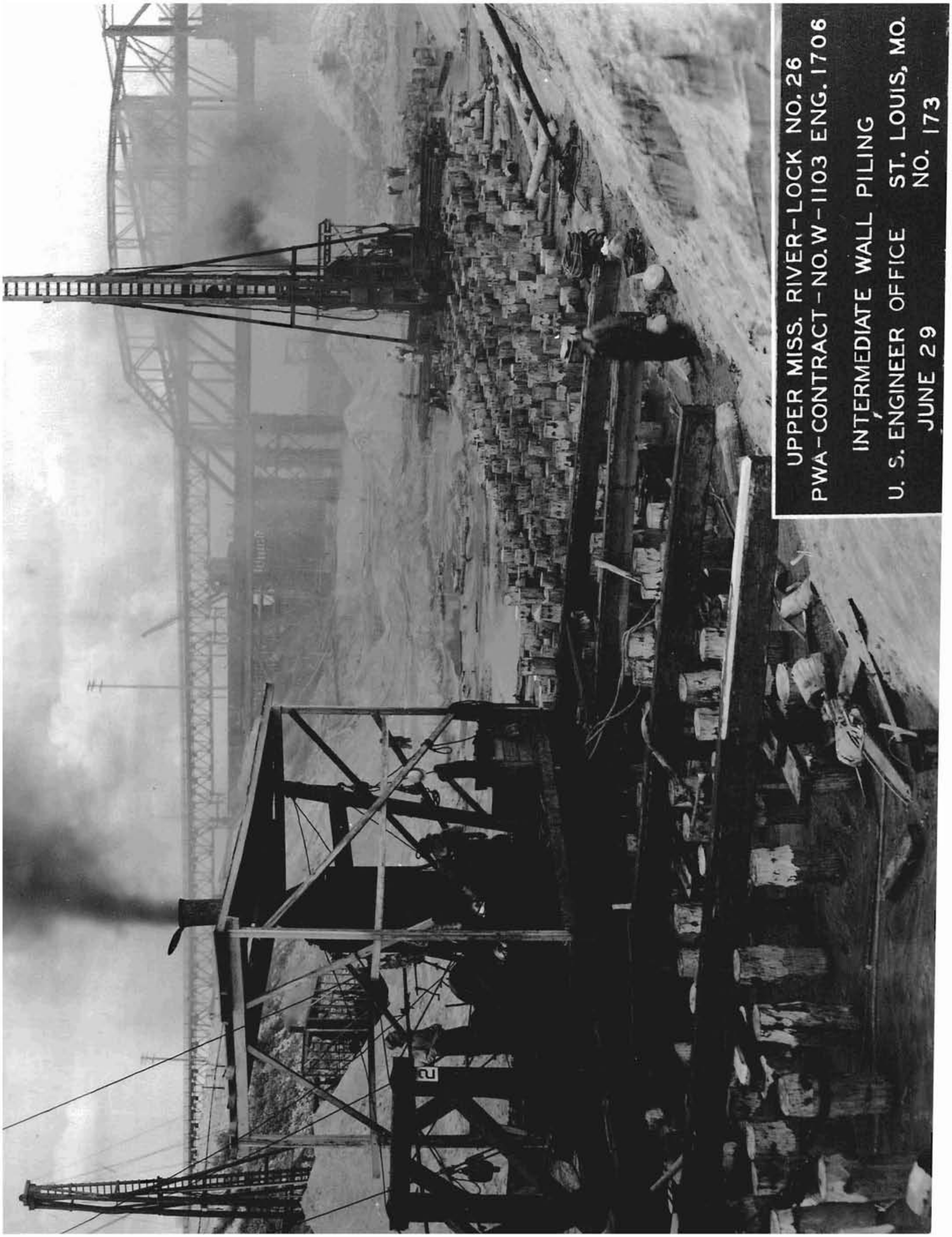
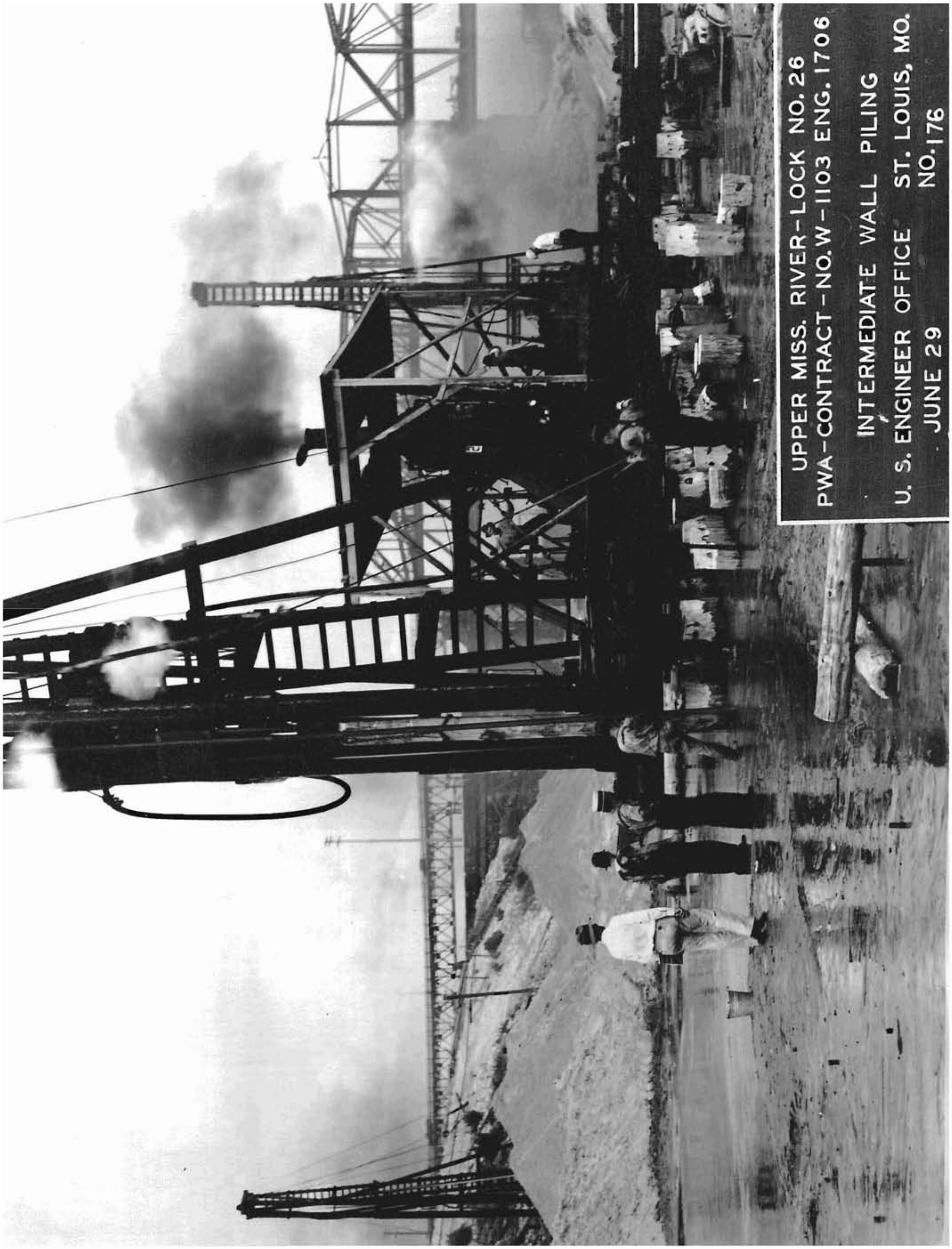

UPPER MISS. RIVER-LOCK NO. 26
PWA-CONTRACT-NO. W-1103 ENG. 1706
INTERMEDIATE WALL PILING
U. S. ENGINEER OFFICE ST. LOUIS, MO.
JUNE 29 NO. 173

PLATE 3


UPPER MISS. RIVER-LOCK NO. 26
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INTERMEDIATE WALL PILING
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PLATE 4

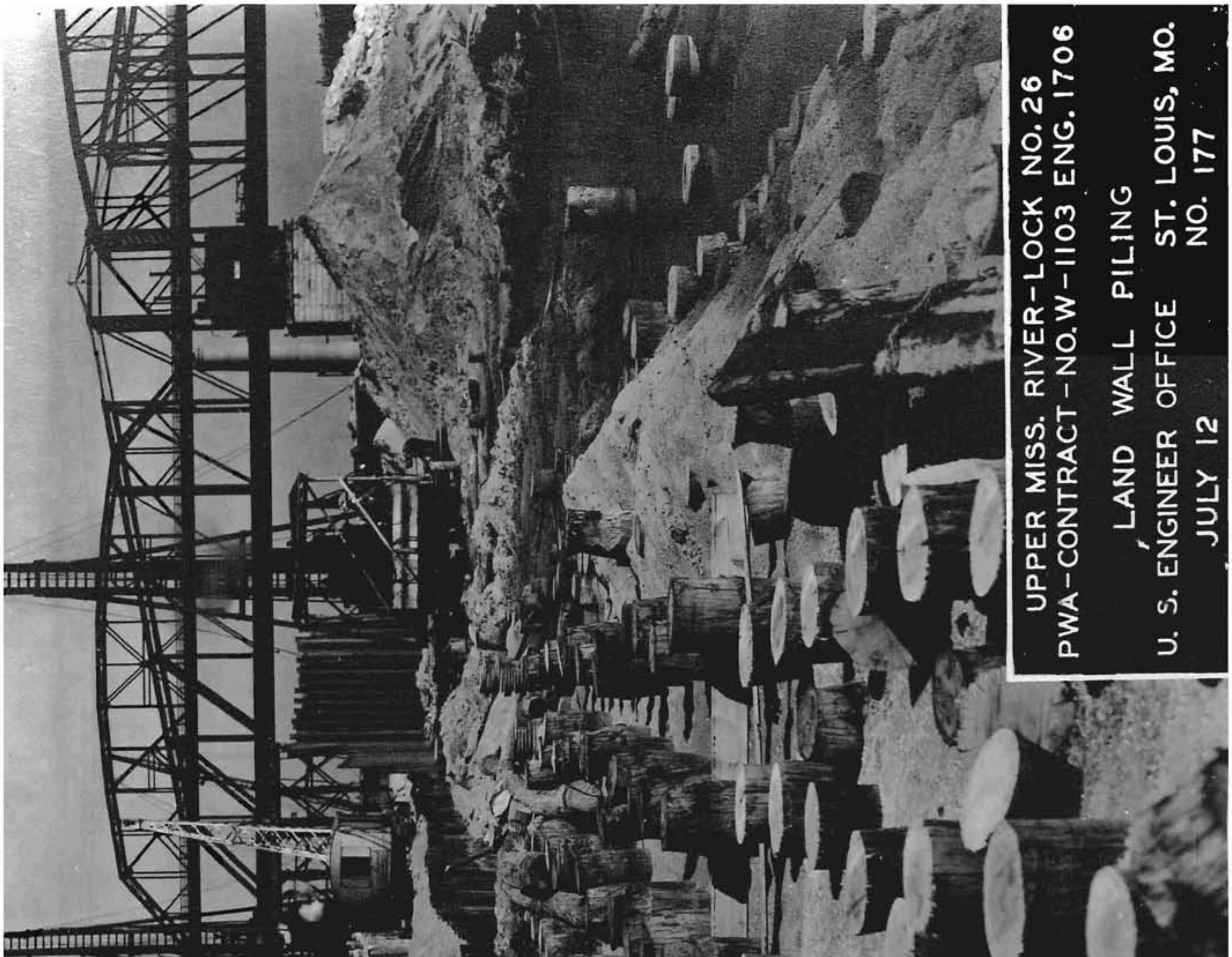
UPPER MISS. RIVER-LOCK NO. 26
PWA-CONTRACT-NO. W-1103 ENG. 1706
LAND WALL PILING
U. S. ENGINEER OFFICE
ST. LOUIS, MO.
JULY 12
NO. 177

PLATE 5


UPPER MISS. RIVER-LOCK NO. 26
PWA-CONTRACT-NO. W-1103 ENG. 1706
INTERMEDIATE WALL PILING
U. S. ENGINEER OFFICE ST. LOUIS, MO.
JULY 12 NO. 179

PLATE 6


UPPER MISS. RIVER-LOCK NO. 26
PWA-CONTRACT-NO.W-1103 ENG. 1706
INTERMEDIATE WALL PILING
U.S. ENGINEER OFFICE ST. LOUIS, MO.
JULY 12 NO. 180

PLATE 7


UPPER MISS. RIVER-LOCK NO. 26
PWA-CONTRACT-NO. W-1103 ENG. 1706
VIEW FROM HIGHWAY BRIDGE
U. S. ENGINEER OFFICE ST. LOUIS, MO.
AUG. 14 NO. 207

PLATE 8

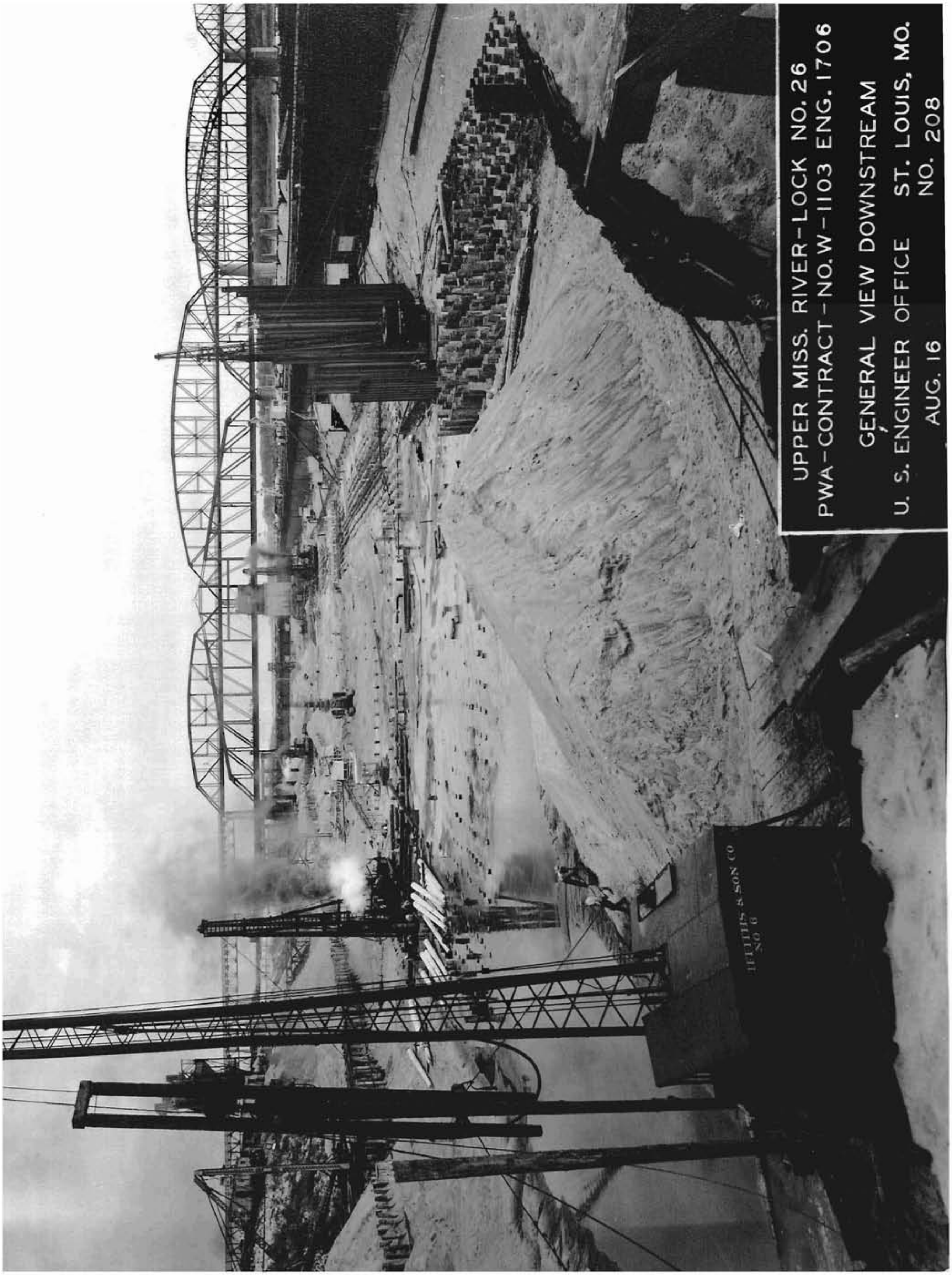
UPPER MISS. RIVER-LOCK NO. 26
PWA-CONTRACT-NO.W-1103 ENG. 1706
GENERAL VIEW DOWNSTREAM
U. S. ENGINEER OFFICE ST. LOUIS, MO.
AUG. 16 NO. 208

PLATE 9

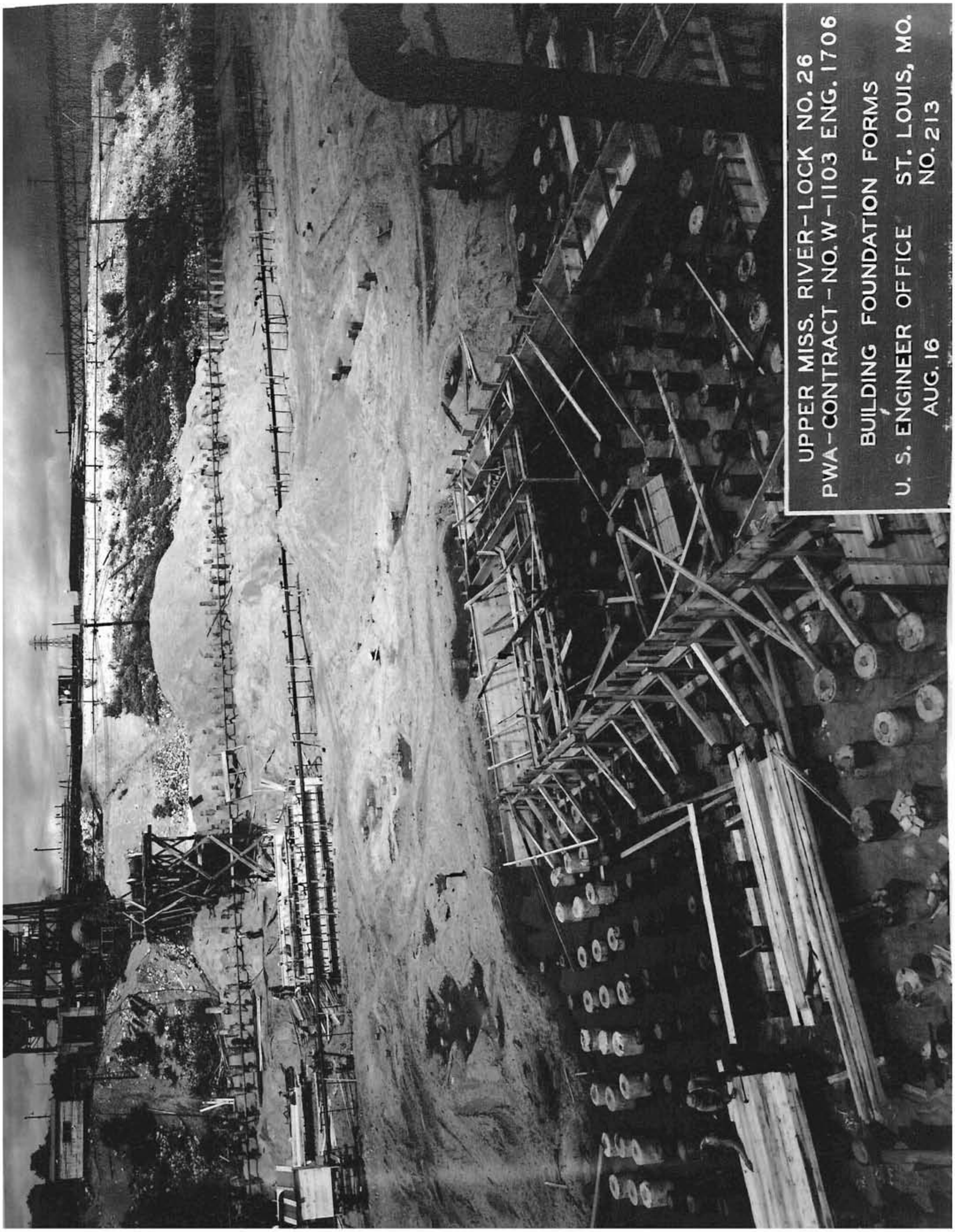
UPPER MISS. RIVER-LOCK NO. 26
PWA-CONTRACT-NO.W-1103 ENG. 1706
BUILDING FOUNDATION FORMS
U. S. ENGINEER OFFICE ST. LOUIS, MO.
AUG. 16 NO. 213

PLATE 10

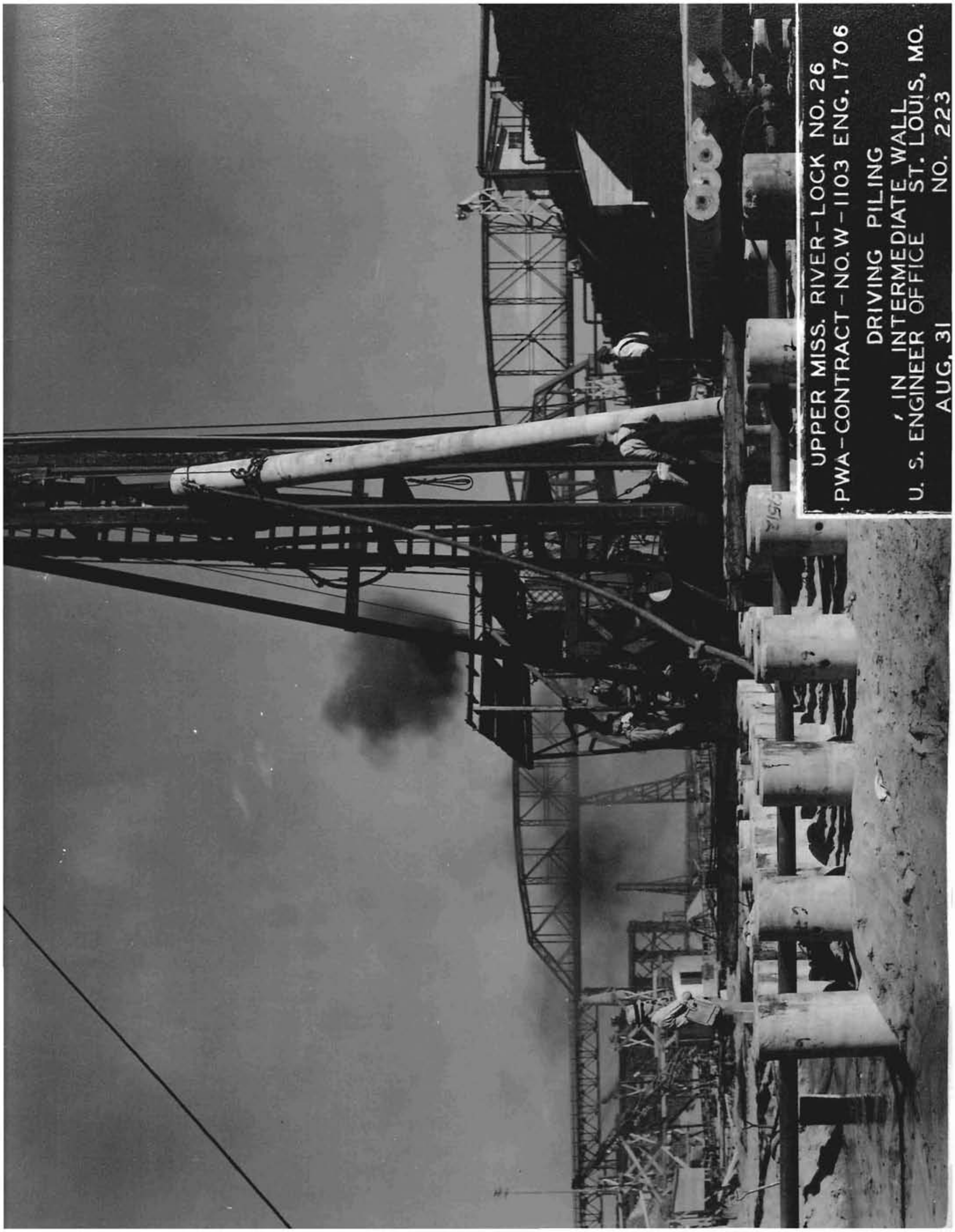
UPPER MISS. RIVER-LOCK NO. 26
PWA-CONTRACT-NO. W-1103 ENG. 1706
DRIVING PILING
IN INTERMEDIATE WALL
U. S. ENGINEER OFFICE ST. LOUIS, MO.
AUG. 31 NO. 223

PLATE 11

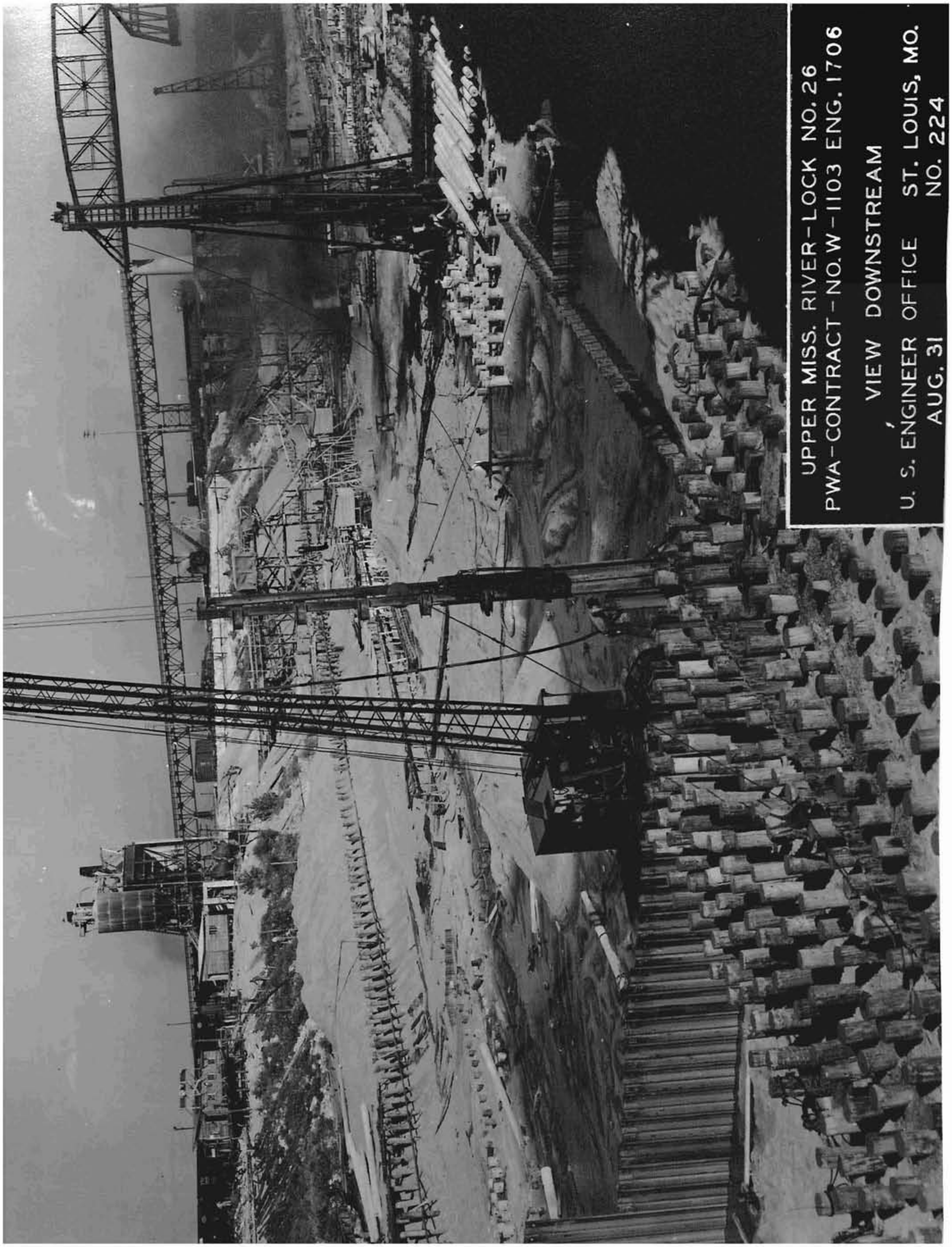
UPPER MISS. RIVER-LOCK NO. 26
PWA-CONTRACT-NO.W-1103 ENG. 1706
VIEW DOWNSTREAM
U. S. ENGINEER OFFICE ST. LOUIS, MO.
AUG. 31 NO. 224

PLATE 12

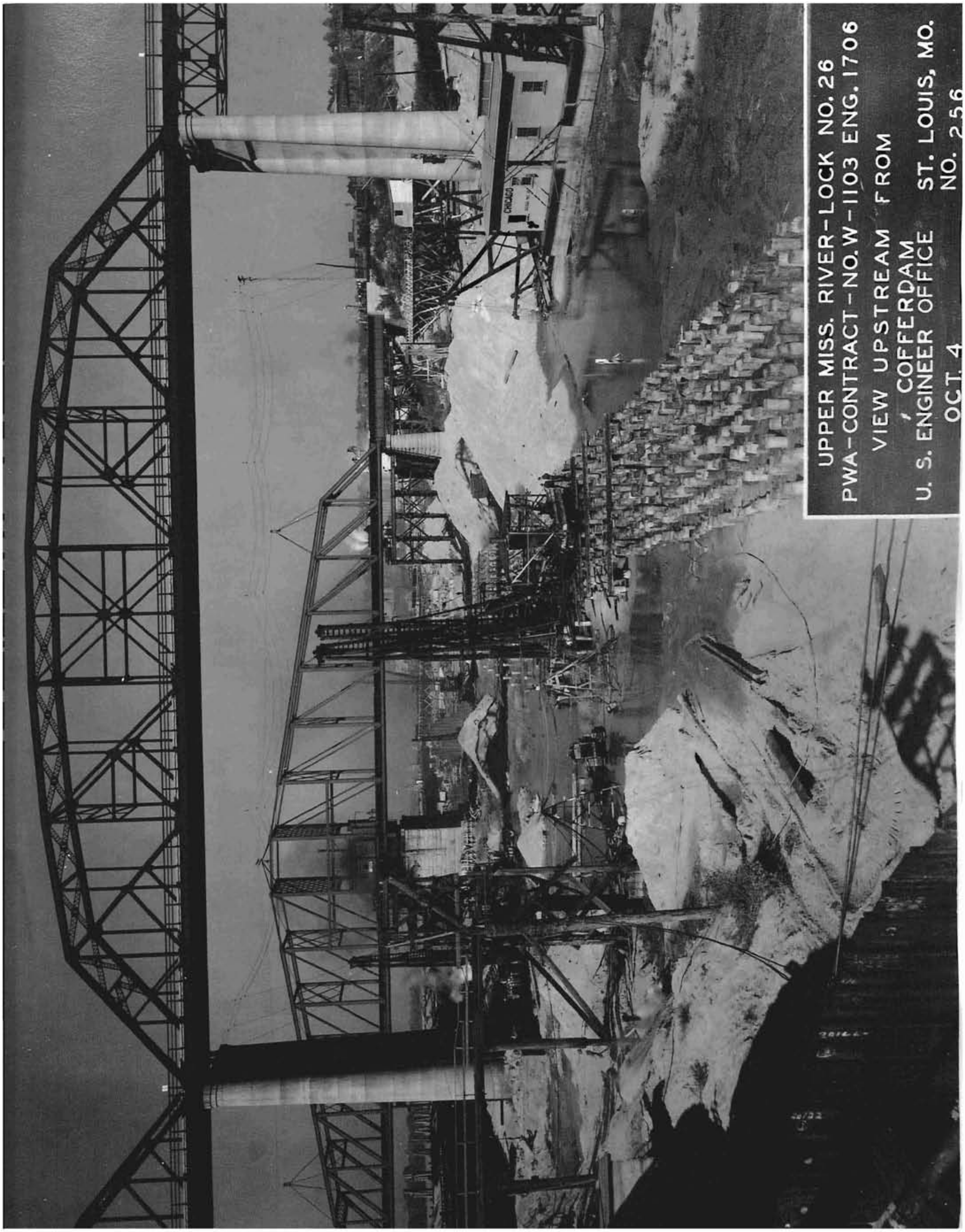
UPPER MISS. RIVER-LOCK NO. 26
PWA-CONTRACT-NO.W-1103 ENG. 1706
VIEW UPSTREAM FROM
COFFERDAM
U. S. ENGINEER OFFICE ST. LOUIS, MO.
OCT. 4 NO. 256

PLATE 13


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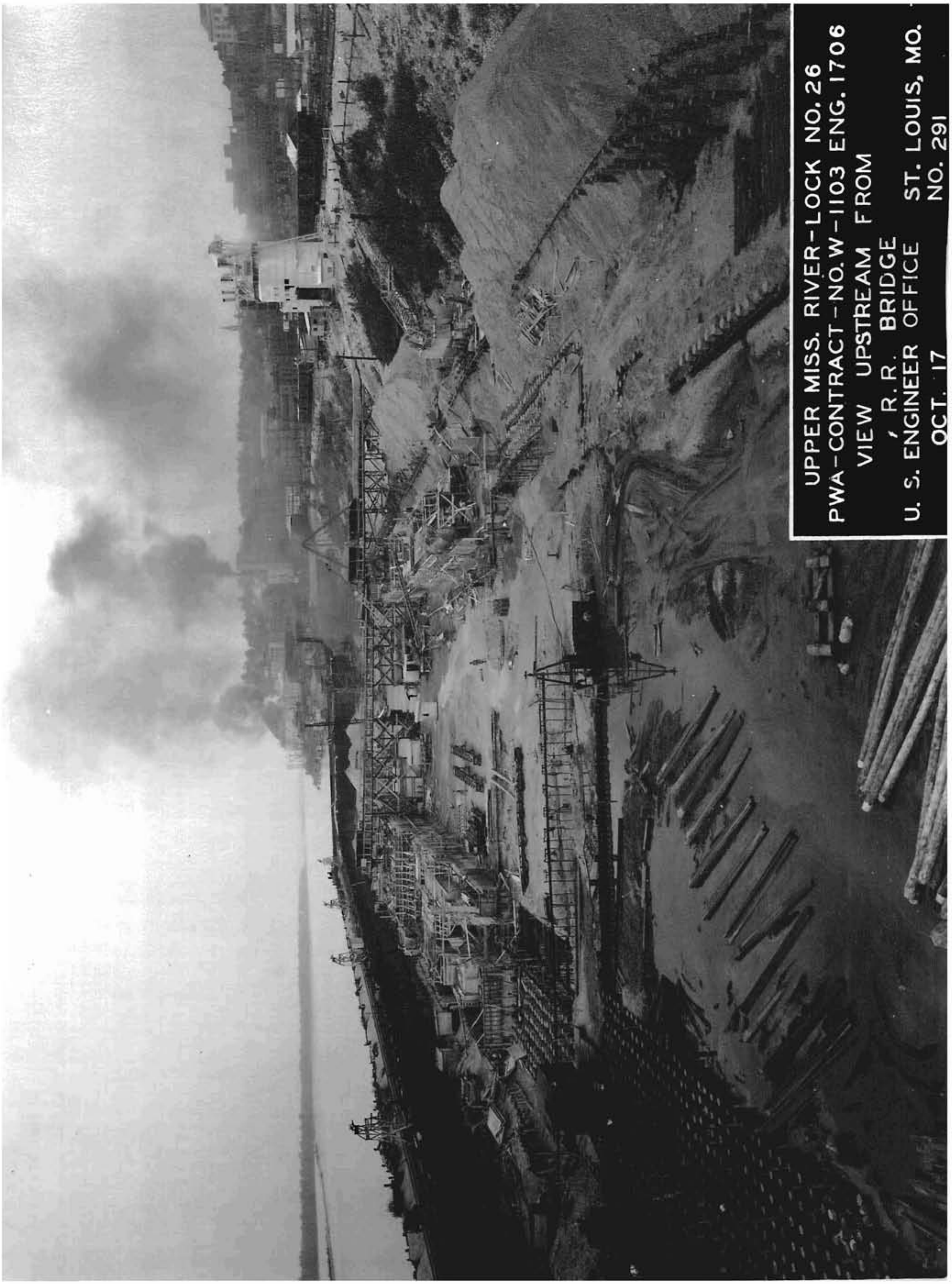
VIEW DOWN STREAM
FROM R.R. BRIDGE
U. S. ENGINEER OFFICE ST. LOUIS, MO.
OCT. 16 NO. 288

PLATE 14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VIEW UPSTREAM FROM
R. R. BRIDGE
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VIEW UPSTREAM FROM
COFFERDAM
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PLATE 16


UPPER MISS. RIVER-LOCK NO. 26
PWA-CONTRACT-NO.W-1103 ENG. 1706
VIEW UPSTREAM FROM
R. R. BRIDGE
U. S. ENGINEER OFFICE ST. LOUIS, MO.
OCT. 26, 1934 NO. 296

PLATE 17

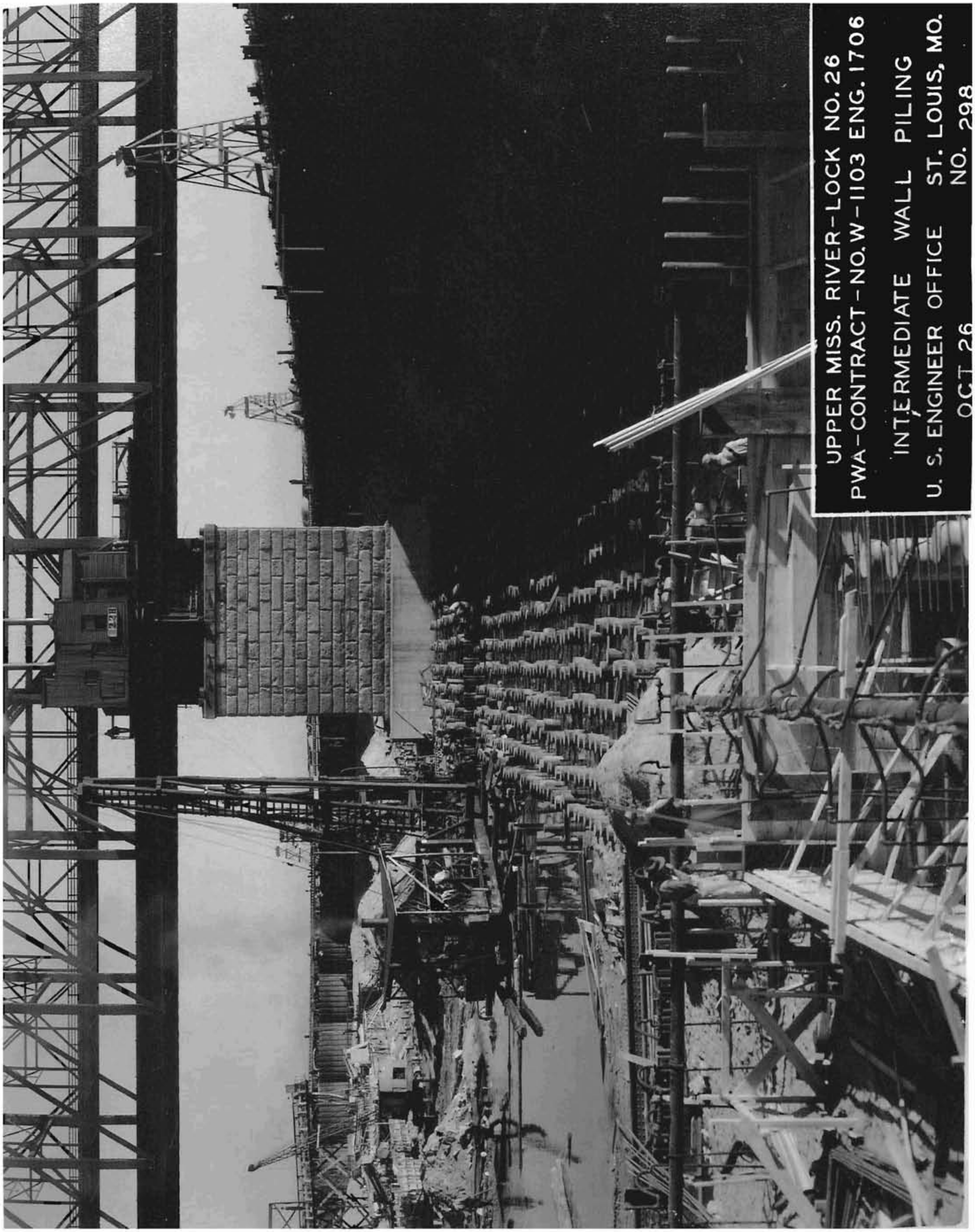
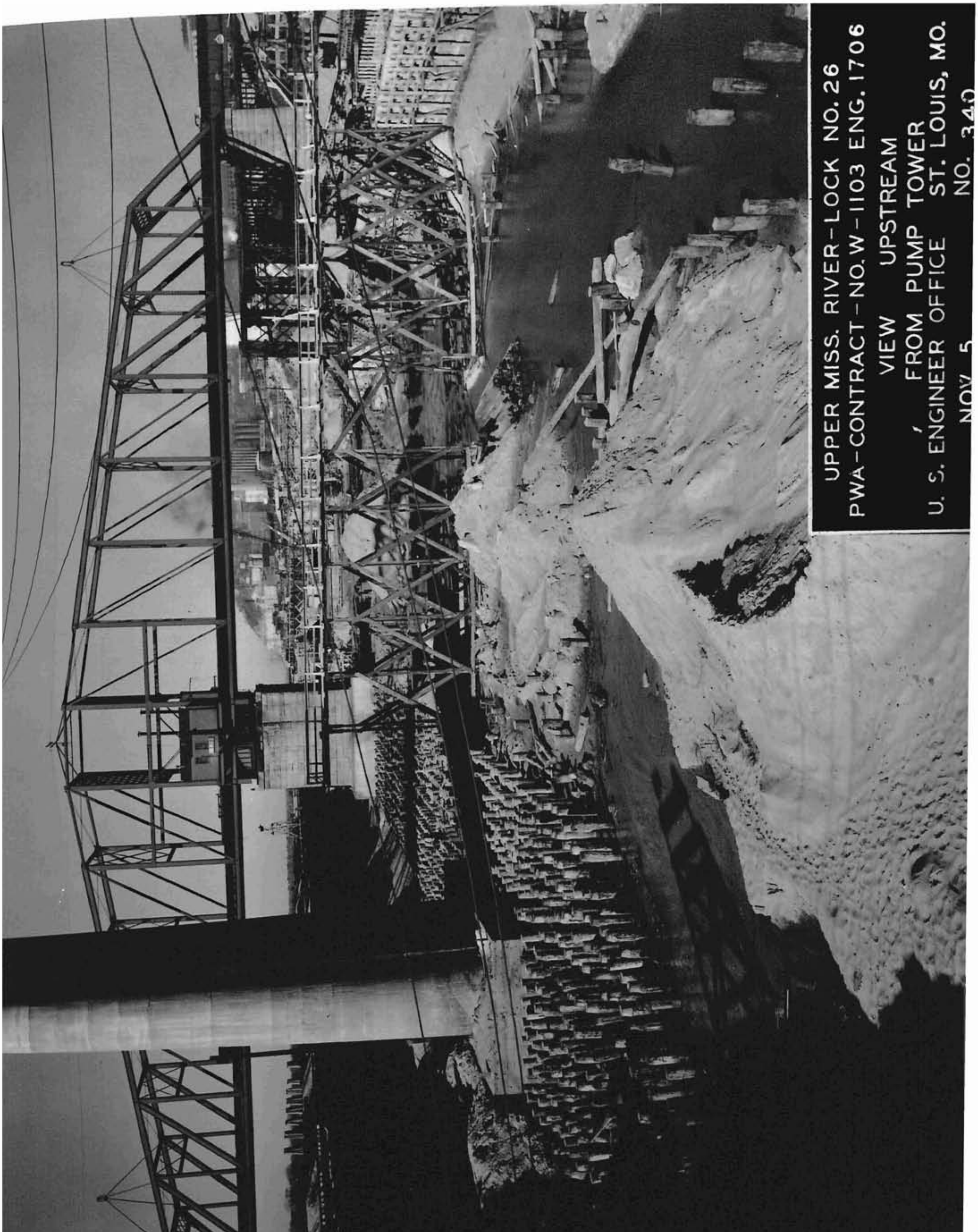

UPPER MISS. RIVER-LOCK NO. 26
PWA-CONTRACT-NO. W-1103 ENG. 1706
INTERMEDIATE WALL PILING
U. S. ENGINEER OFFICE ST. LOUIS, MO.
OCT 26 NO. 298

PLATE 18


UPPER MISS. RIVER-LOCK NO. 26
PWA-CONTRACT-NO.W-1103 ENG. 1706
VIEW UPSTREAM
FROM PUMP TOWER
U. S. ENGINEER OFFICE ST. LOUIS, MO.
NOV. 5 NO. 340

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GUIDE WALL CRIB
U. S. ENGINEER OFFICE ST. LOUIS, MO.
NOV. 12
NO. 351


UPPER MISS. RIVER-LOCK NO. 26
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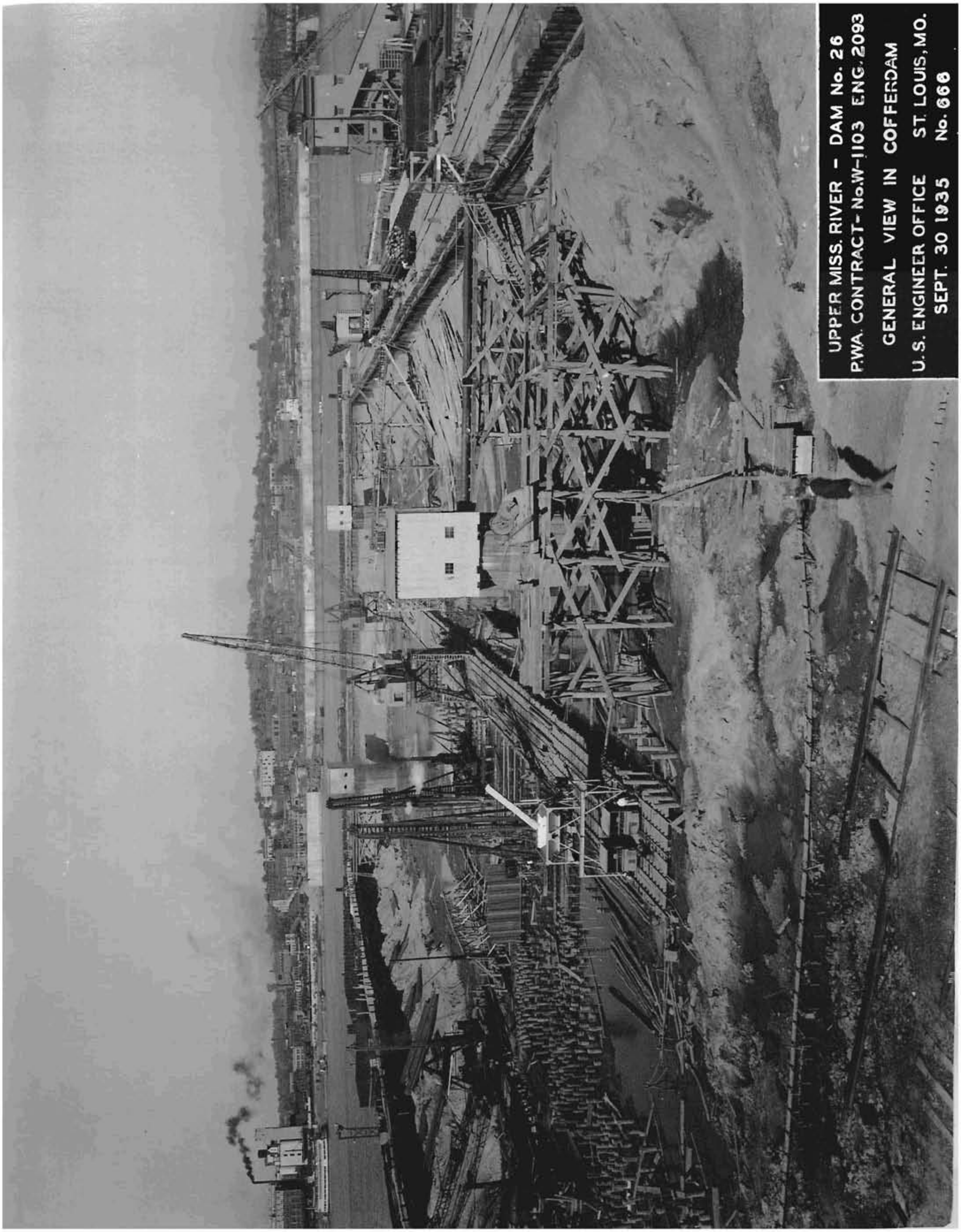
VIEW DOWNSTREAM
FROM CEMENT RESERVOIR
U. S. ENGINEER OFFICE ST. LOUIS, MO.
FEB 12 1935 NO. 450

PLATE 21


UPPER MISS. RIVER - DAM No. 26
P.WA. CONTRACT- No.W-1103 ENG. 2093
GENERAL VIEW IN COFFERDAM
U.S. ENGINEER OFFICE ST. LOUIS, MO.
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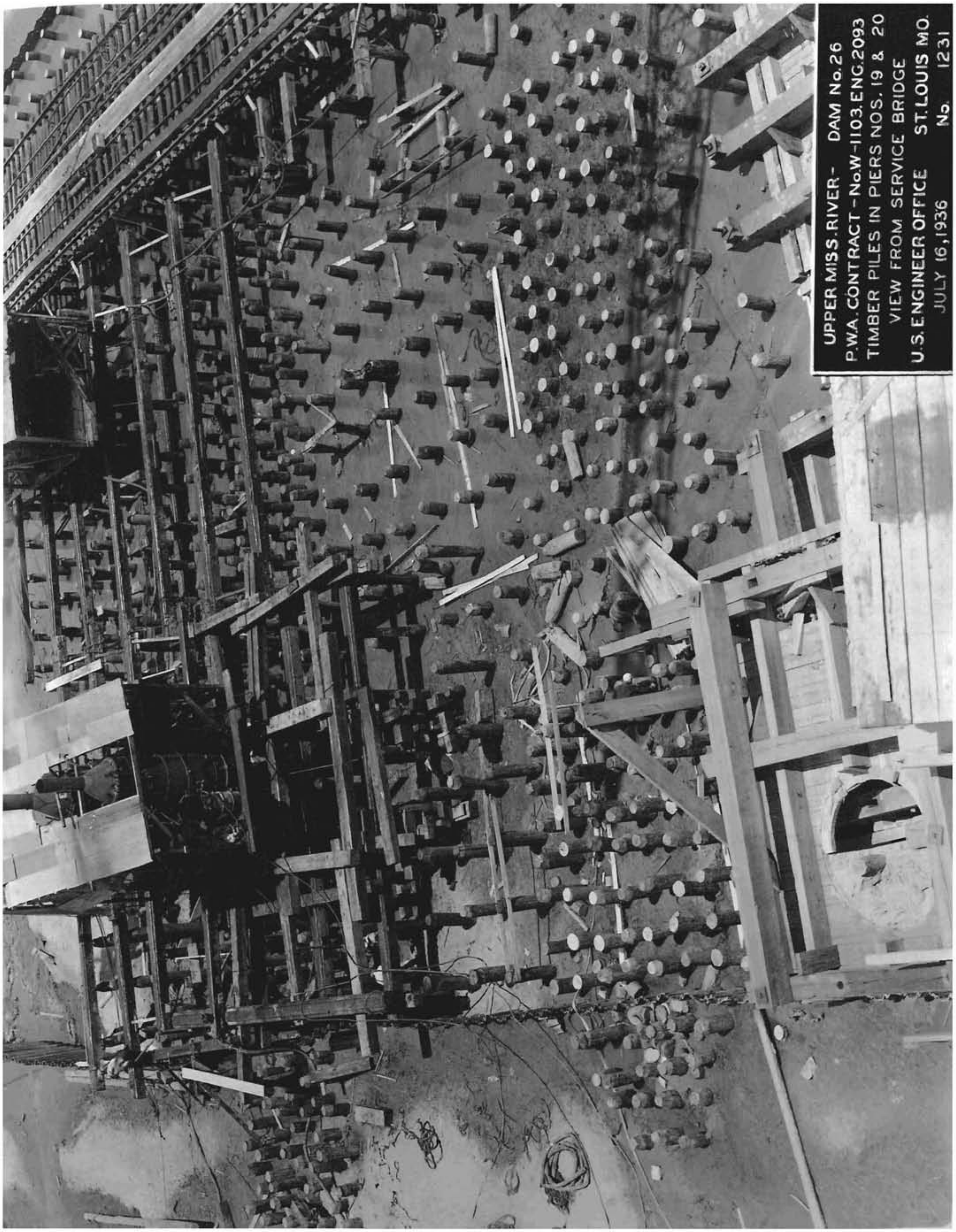
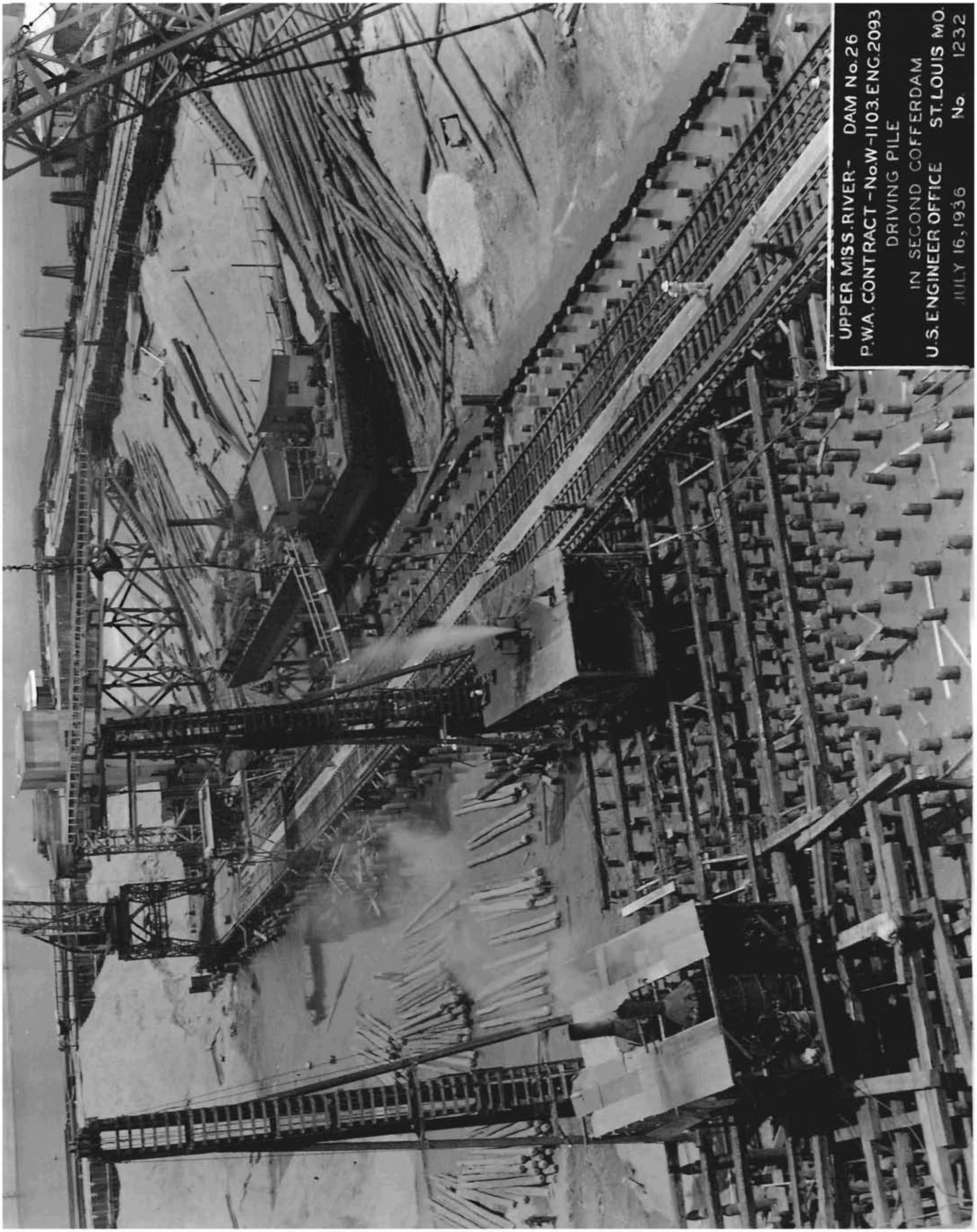

UPPER MISS.RIVER- DAM No.26
P.W.A. CONTRACT-No.W-1103.ENG.2093
TIMBER PILES IN PIERS NOS. 19 & 20
VIEW FROM SERVICE BRIDGE
U.S.ENGINEER OFFICE ST.LOUIS MO.
JULY 16,1936 No. 1231


UPPER MISS. RIVER - DAM No.26
P.W.A. CONTRACT - No.W-1103.ENG.2093
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IN SECOND COFFERDAM
U.S. ENGINEER OFFICE ST. LOUIS MO.
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PLATE 24

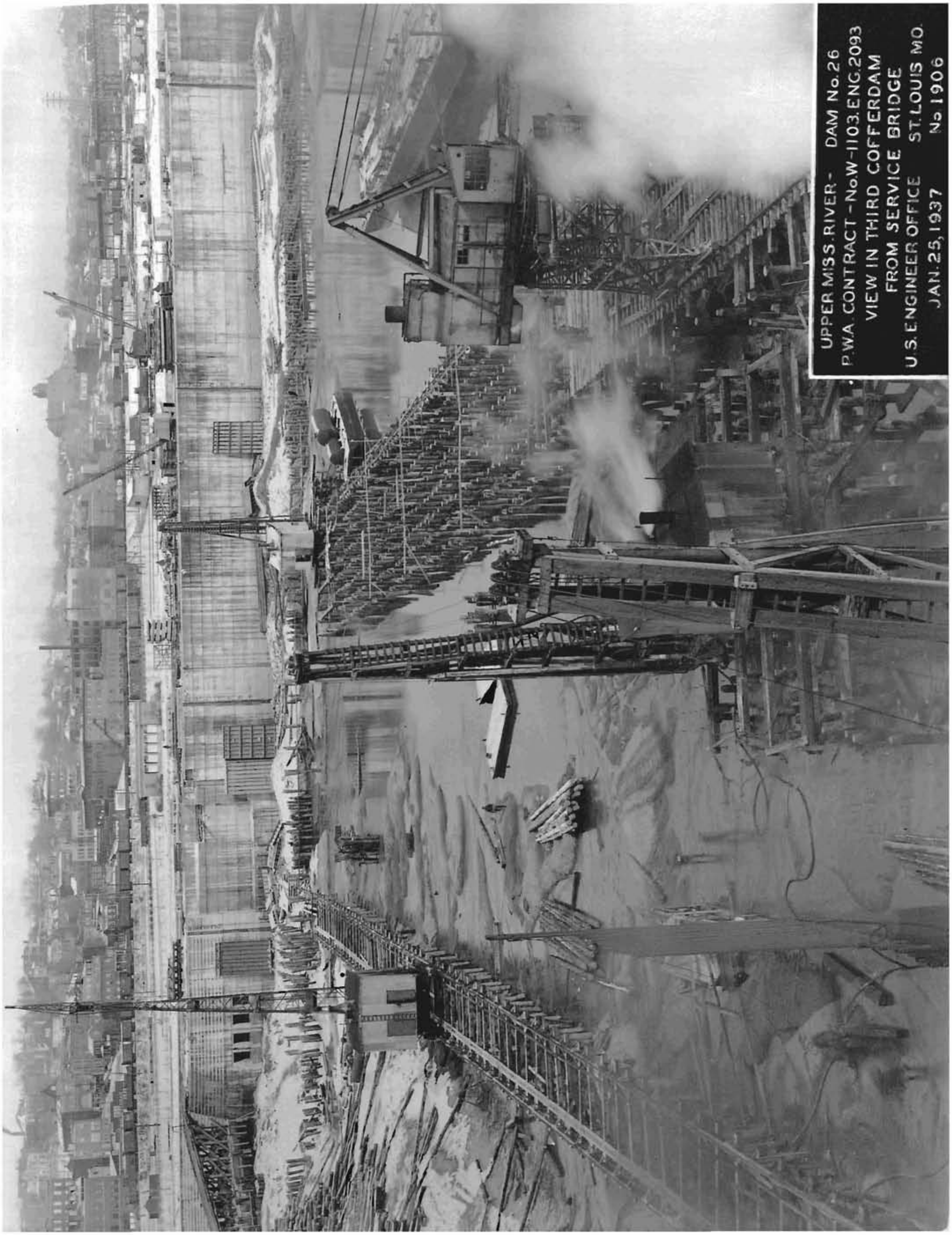
UPPER MISS. RIVER - DAM No. 26
P.W.A. CONTRACT - No. W-1103. ENG. 2093
VIEW IN THIRD COFFERDAM
FROM SERVICE BRIDGE
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PLATE 25

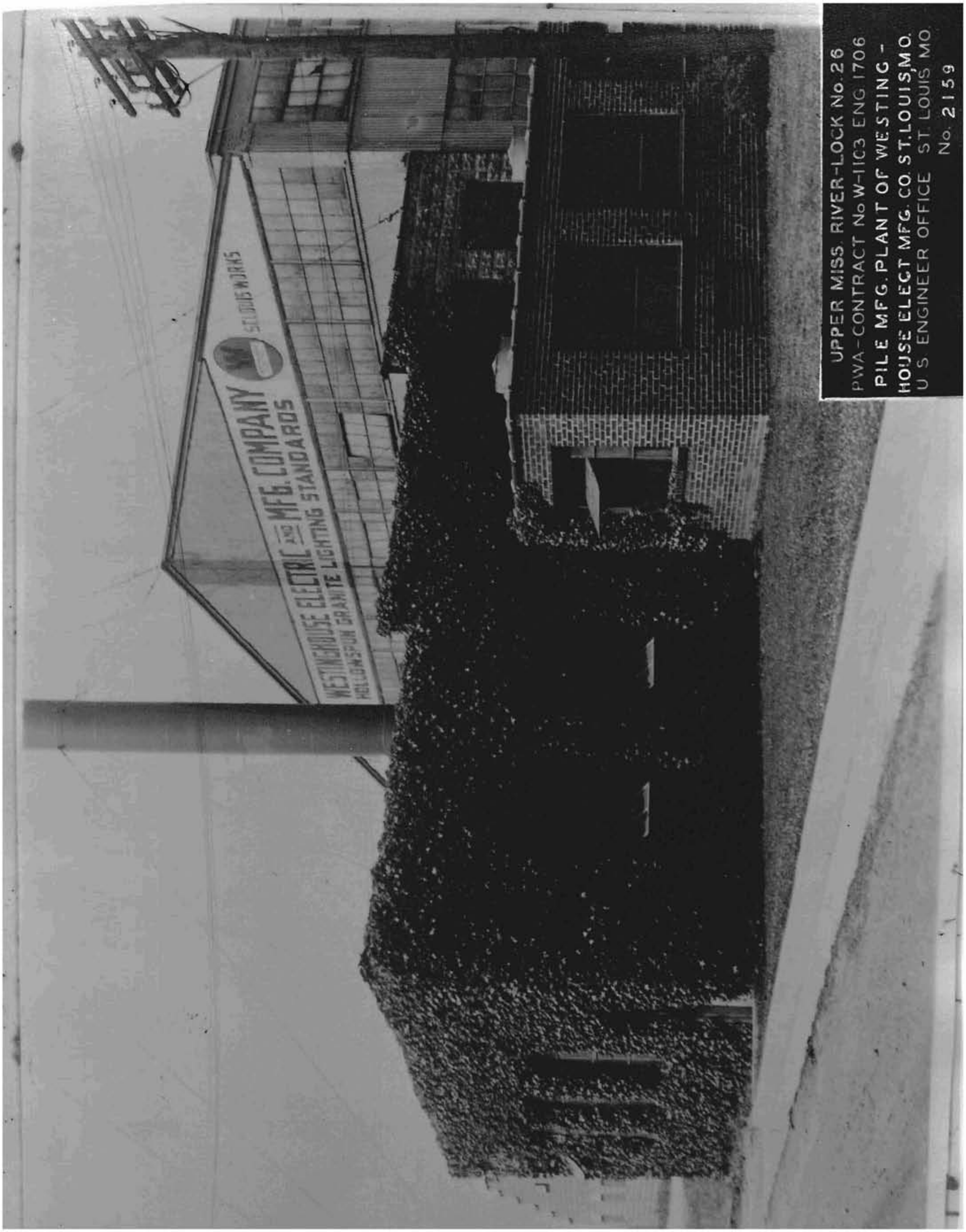

UPPER MISS. RIVER-LOCK No. 26
PWA-CONTRACT No W-11C3 ENG 1706
PILE MFG. PLANT OF WESTING-
HOUSE ELECT MFG. CO. ST. LOUIS, MO.
U S ENGINEER OFFICE ST. LOUIS MO.
No. 2159
WESTINGHOUSE ELECTRIC AND MFG. COMPANY
HOLLOWSPUN GRANITE LIGHTING STANDARDS
ST. LOUIS WORKS

PLATE 26

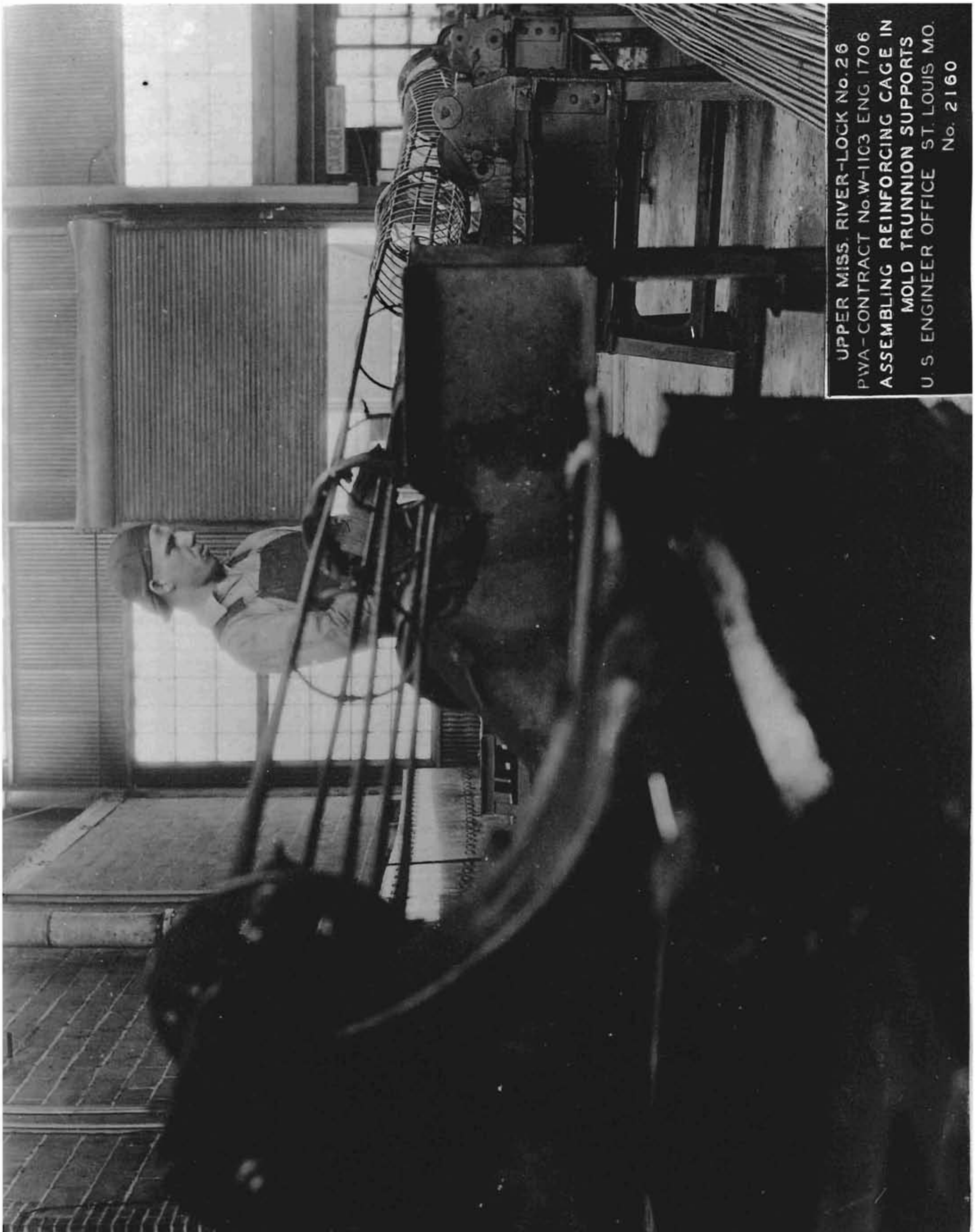

UPPER MISS. RIVER-LOCK No. 26
PWA-CONTRACT No. W-1103 ENG. 1706
ASSEMBLING REINFORCING CAGE IN
MOLD TRUNNION SUPPORTS
U.S. ENGINEER OFFICE ST. LOUIS MO.
No. 2160

PLATE 27

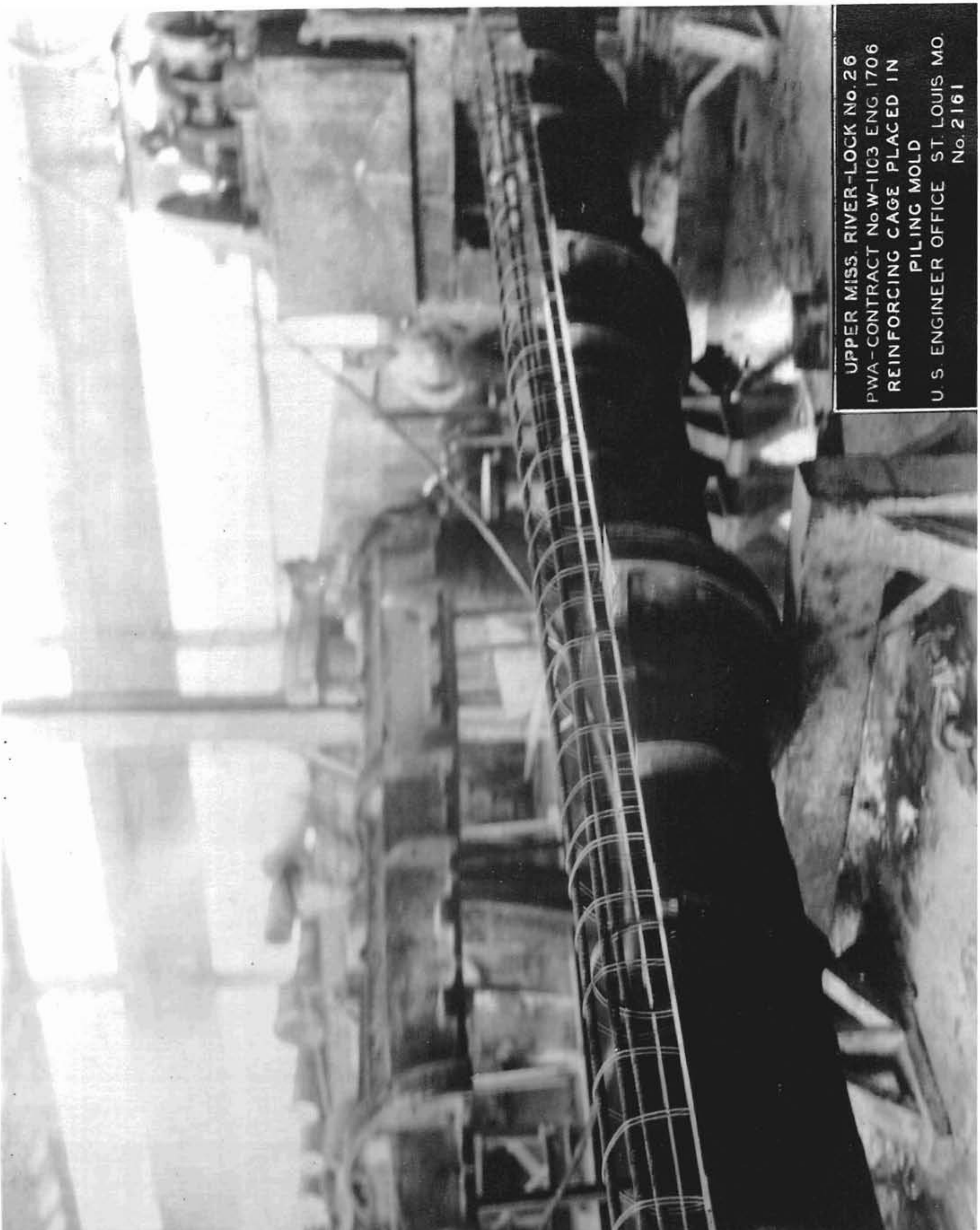
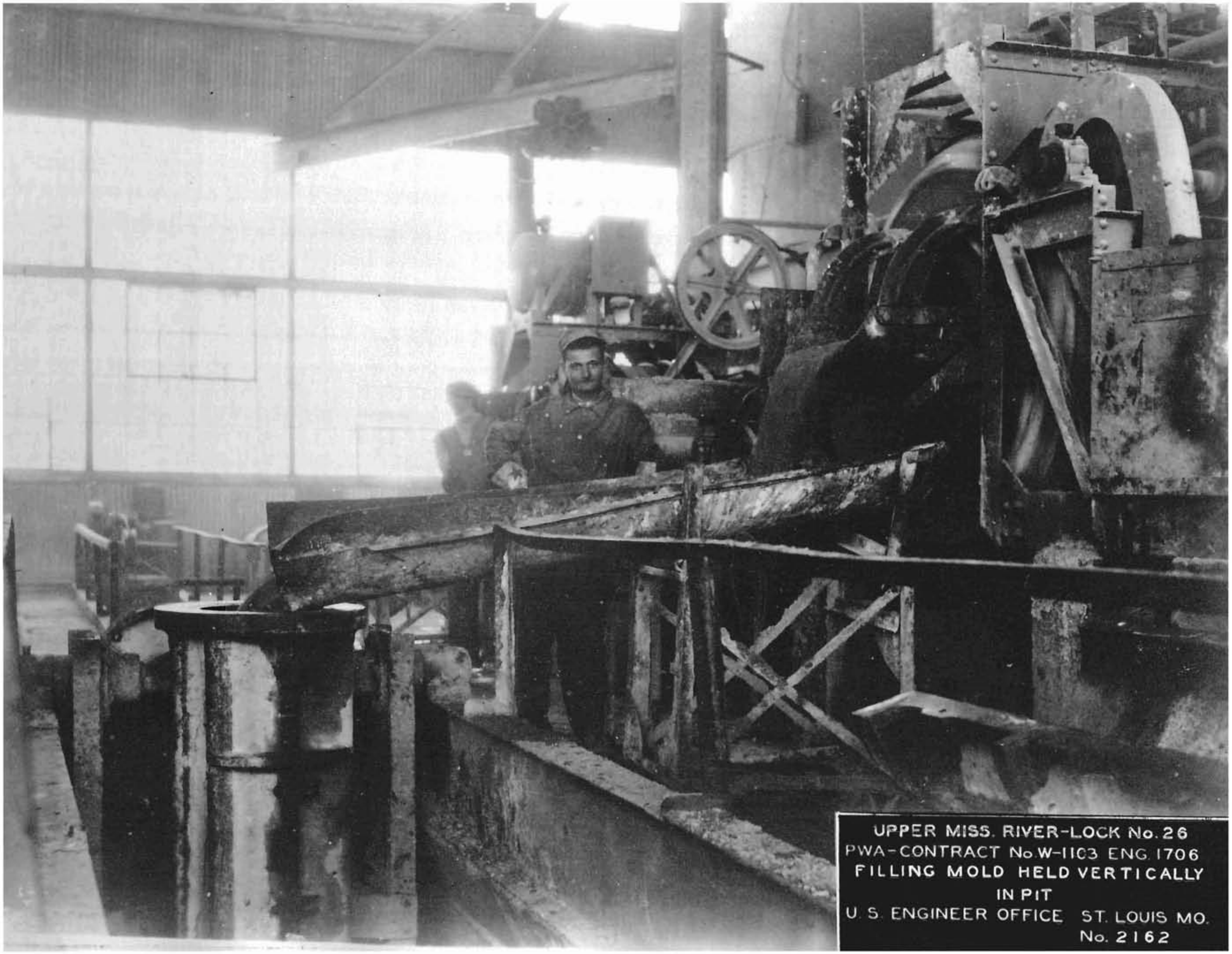

UPPER MISS. RIVER-LOCK No.26
PWA-CONTRACT No.W-1103 ENG.1706
REINFORCING CAGE PLACED IN
PILING MOLD
U.S. ENGINEER OFFICE ST. LOUIS MO.
No.2161

PLATE 28


UPPER MISS. RIVER-LOCK No. 26
PWA-CONTRACT No. W-1103 ENG. 1706
FILLING MOLD HELD VERTICALLY
IN PIT
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No. 2162

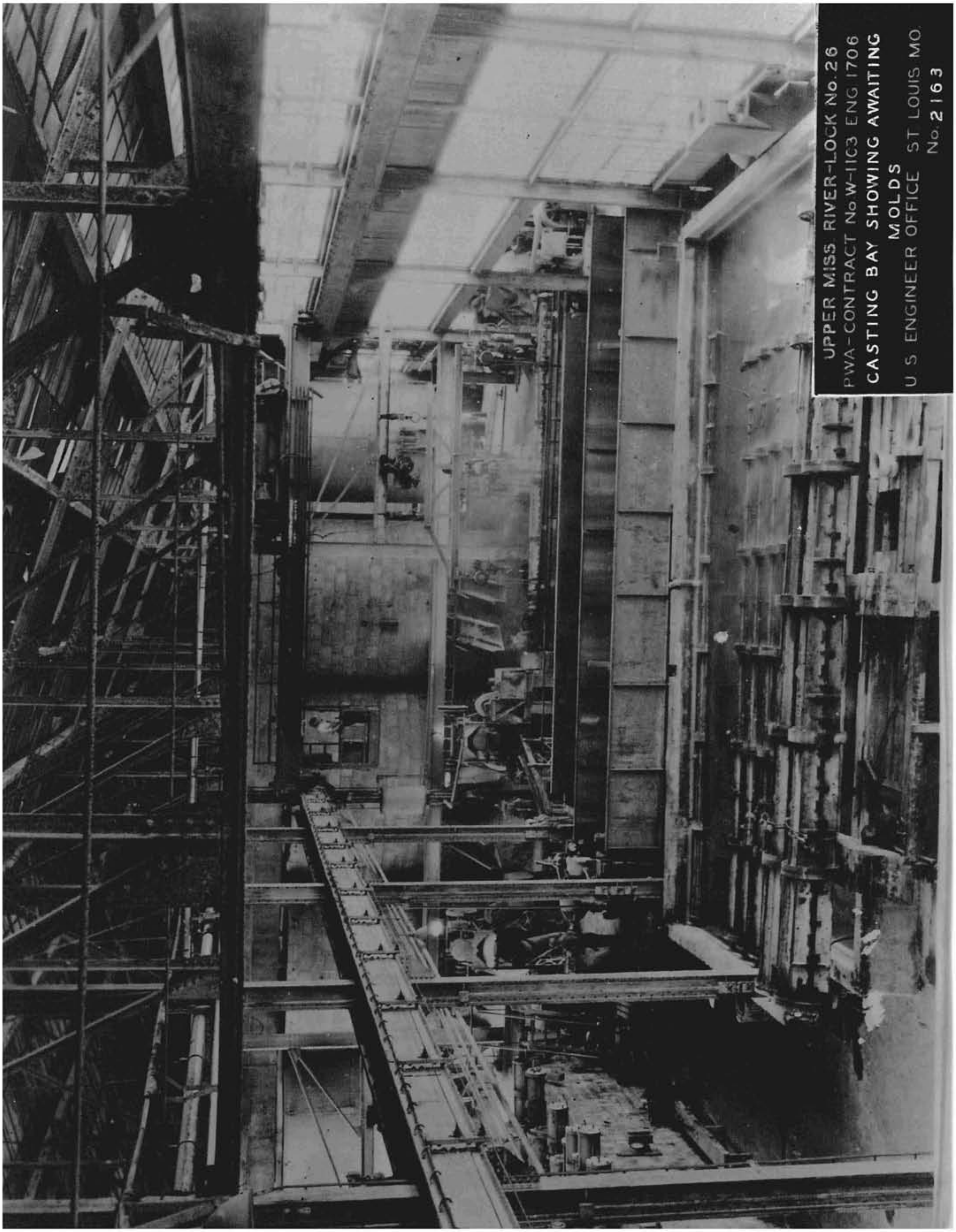
UPPER MISS. RIVER-LOCK No. 26
PWA-CONTRACT No. W-1163 ENG 1706
CASTING BAY SHOWING AWAITING
MOLDS
U S ENGINEER OFFICE ST LOUIS MO.
No. 2163

PLATE 30

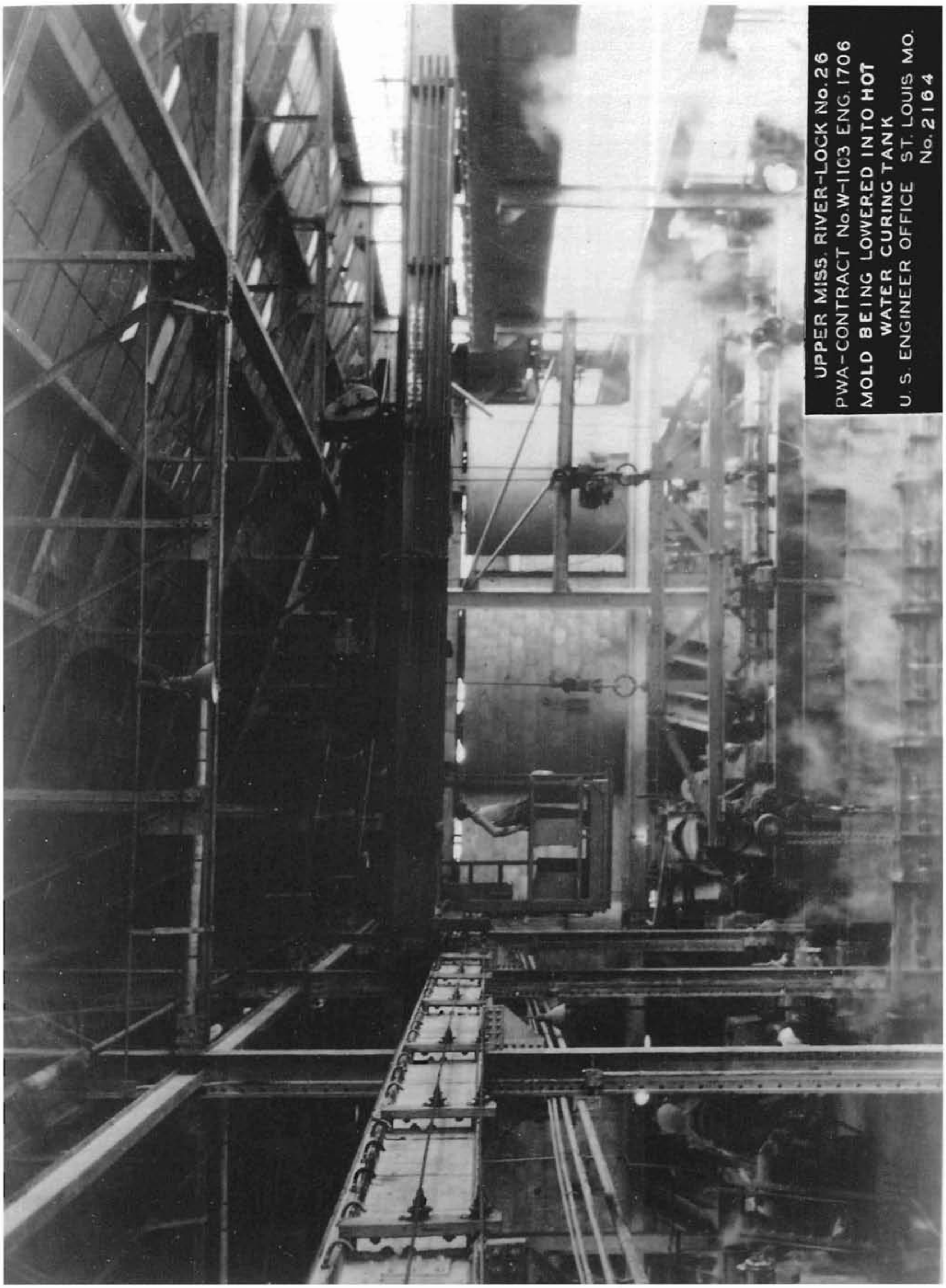
UPPER MISS. RIVER-LOCK No. 26
PWA-CONTRACT No. W-1103 ENG. 1706
MOLD BEING LOWERED INTO HOT
WATER CURING TANK
U.S. ENGINEER OFFICE ST. LOUIS MO.
No. 2164

PLATE 31

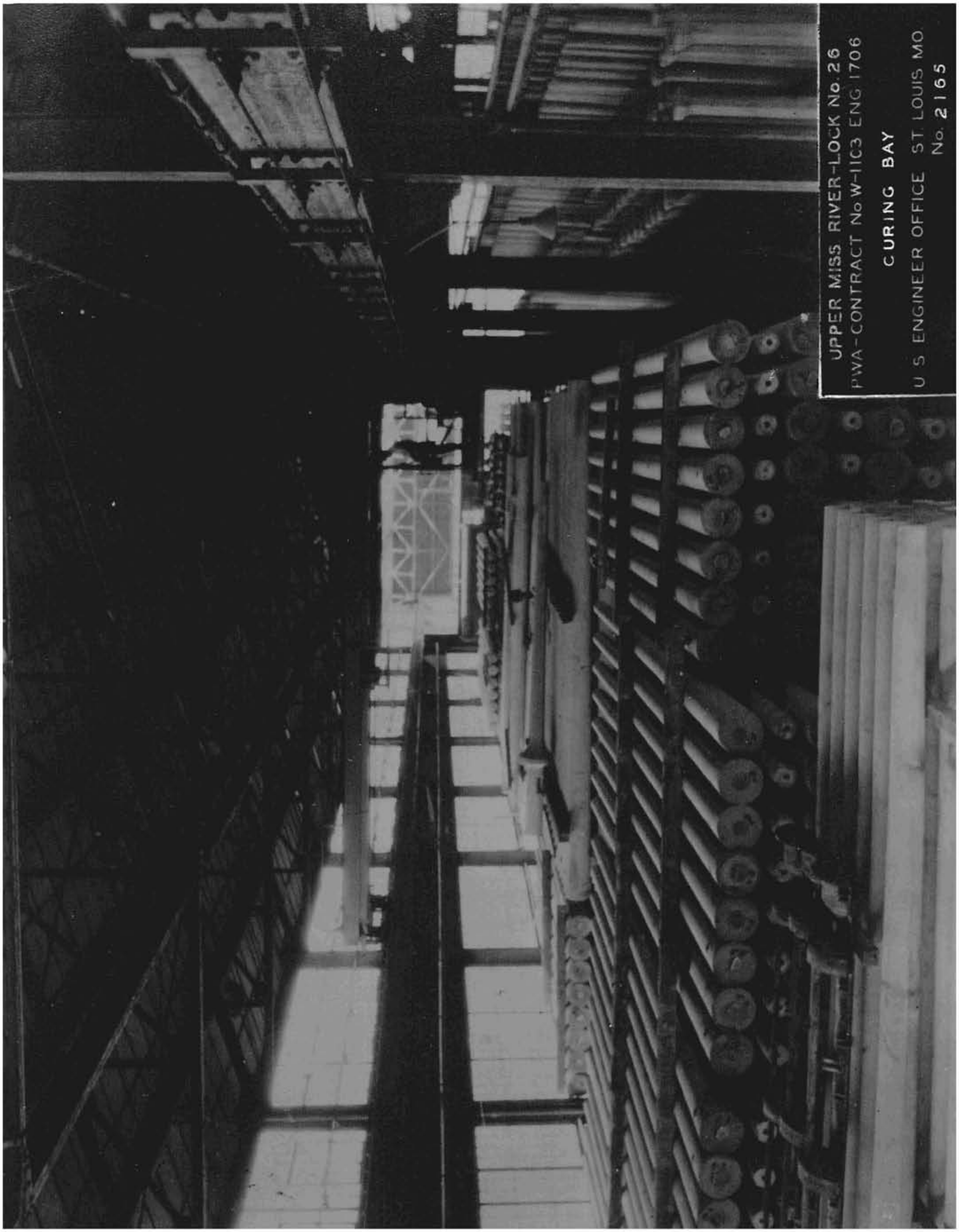
UPPER MISS RIVER-LOCK No. 26
PWA-CONTRACT No W-1103 ENG 1706
CURING BAY
U S ENGINEER OFFICE ST. LOUIS MO.
No. 2165

PLATE 32

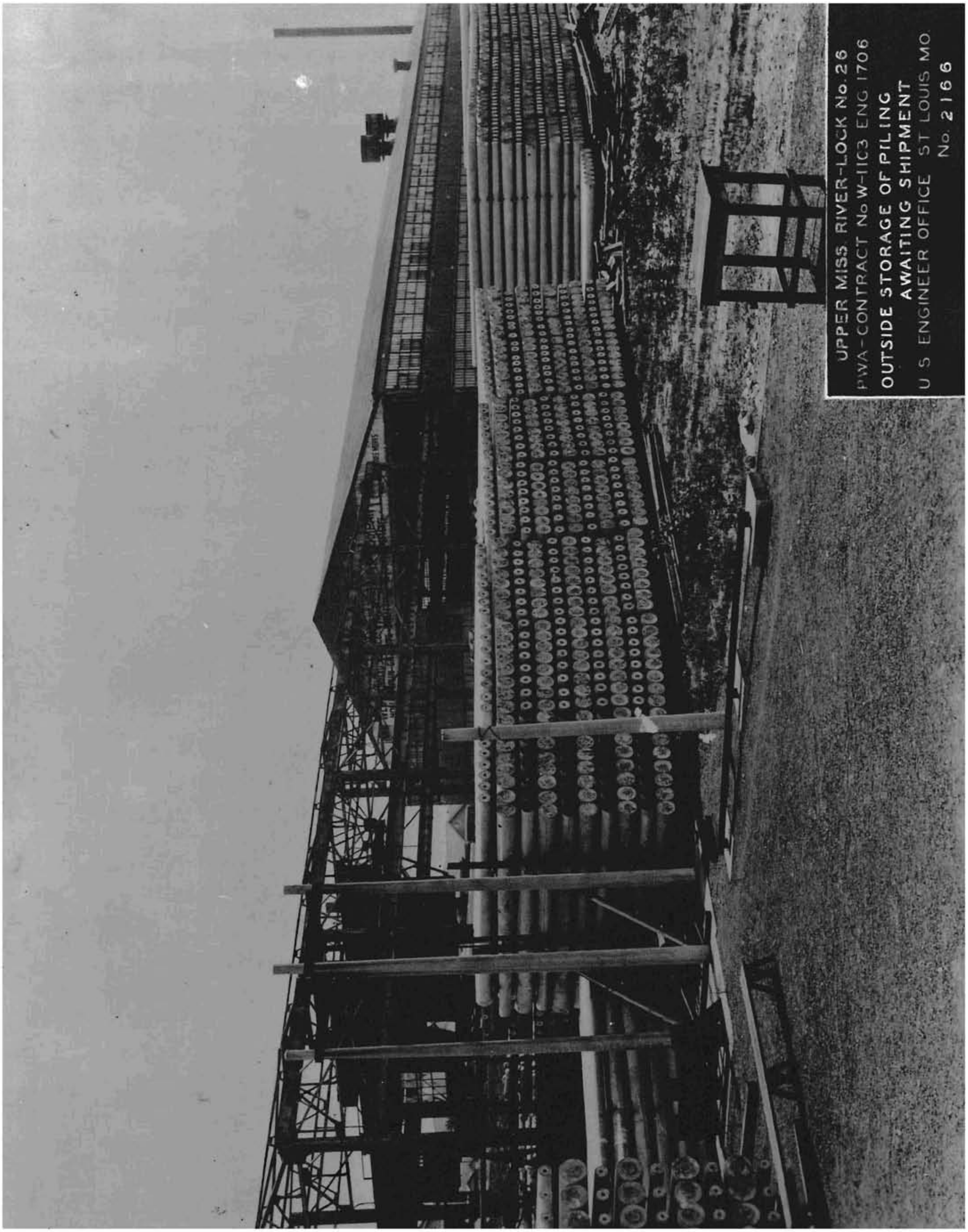
UPPER MISS. RIVER-LOCK No. 26
PWA-CONTRACT No W-11C3 ENG 1706
OUTSIDE STORAGE OF PILING
AWAITING SHIPMENT
U S ENGINEER OFFICE ST LOUIS MO.
No. 2166

PLATE 33


UPPER MISS. RIVER-LOCK No.26
PWA-CONTRACT No.W-1103 ENG 1706
PILING EN ROUTE TO LOCK No.26.
U.S. ENGINEER OFFICE ST. LOUIS MO.
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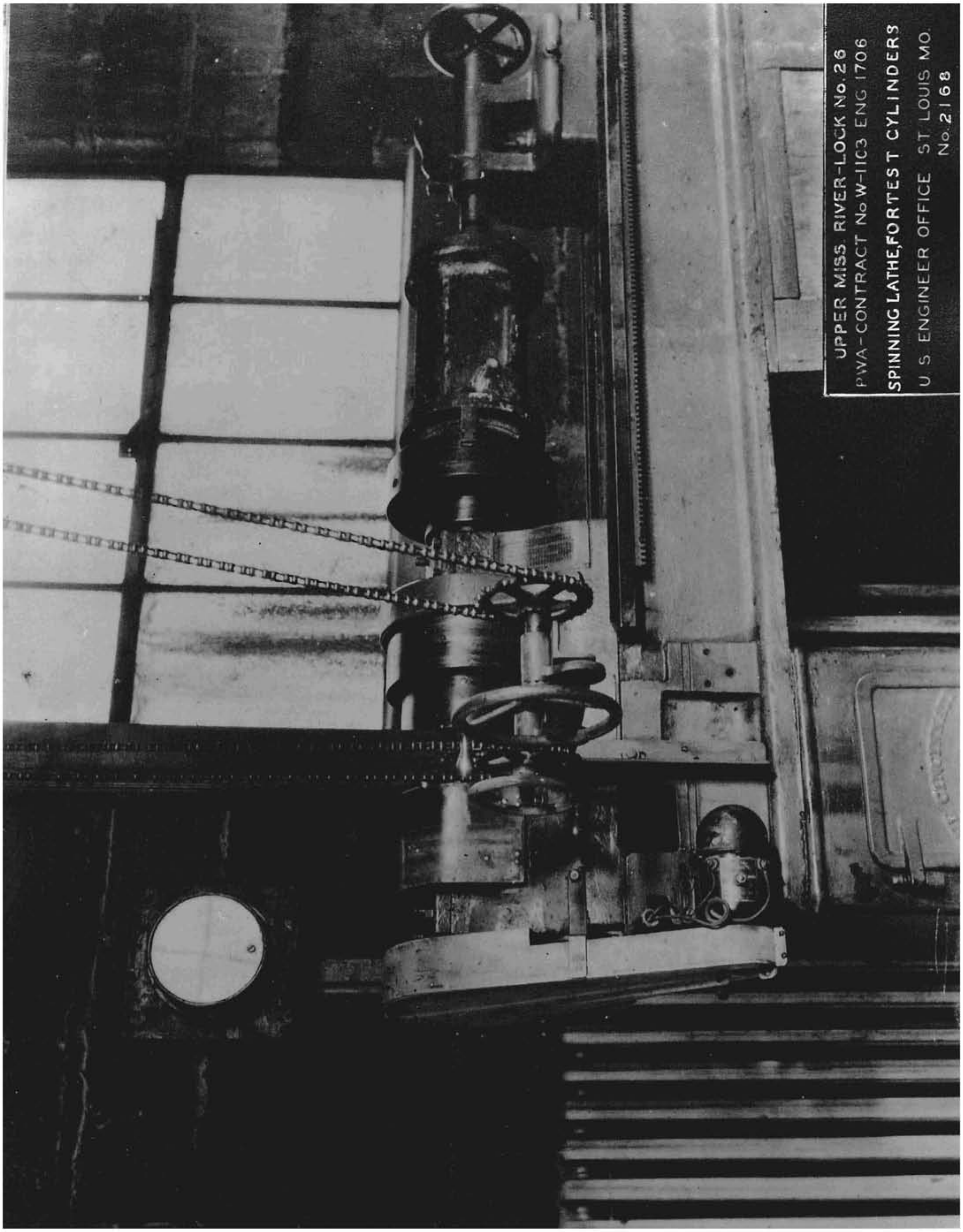
PLATE 34


UPPER MISS. RIVER-LOCK No. 26
PWA-CONTRACT No W-11C3 ENG 1706
SPINNING LATHE, FOR TEST CYLINDERS
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No. 2168

PLATE 35

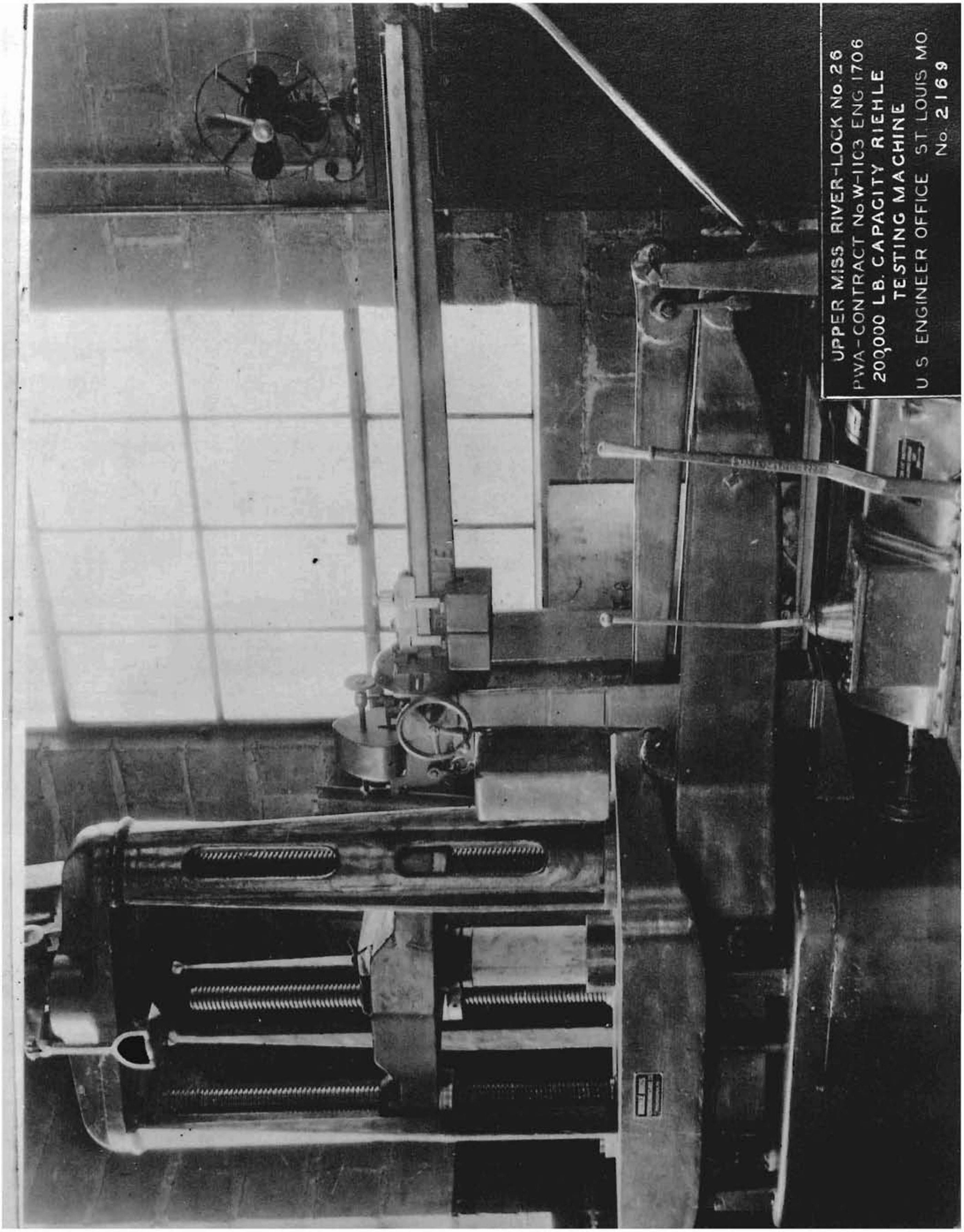
UPPER MISS. RIVER-LOCK No. 26
PWA-CONTRACT No W-1103 ENG 1706
200,000 LB. CAPACITY RIEHLE
TESTING MACHINE
U S ENGINEER OFFICE ST LOUIS MO
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PLATE 36

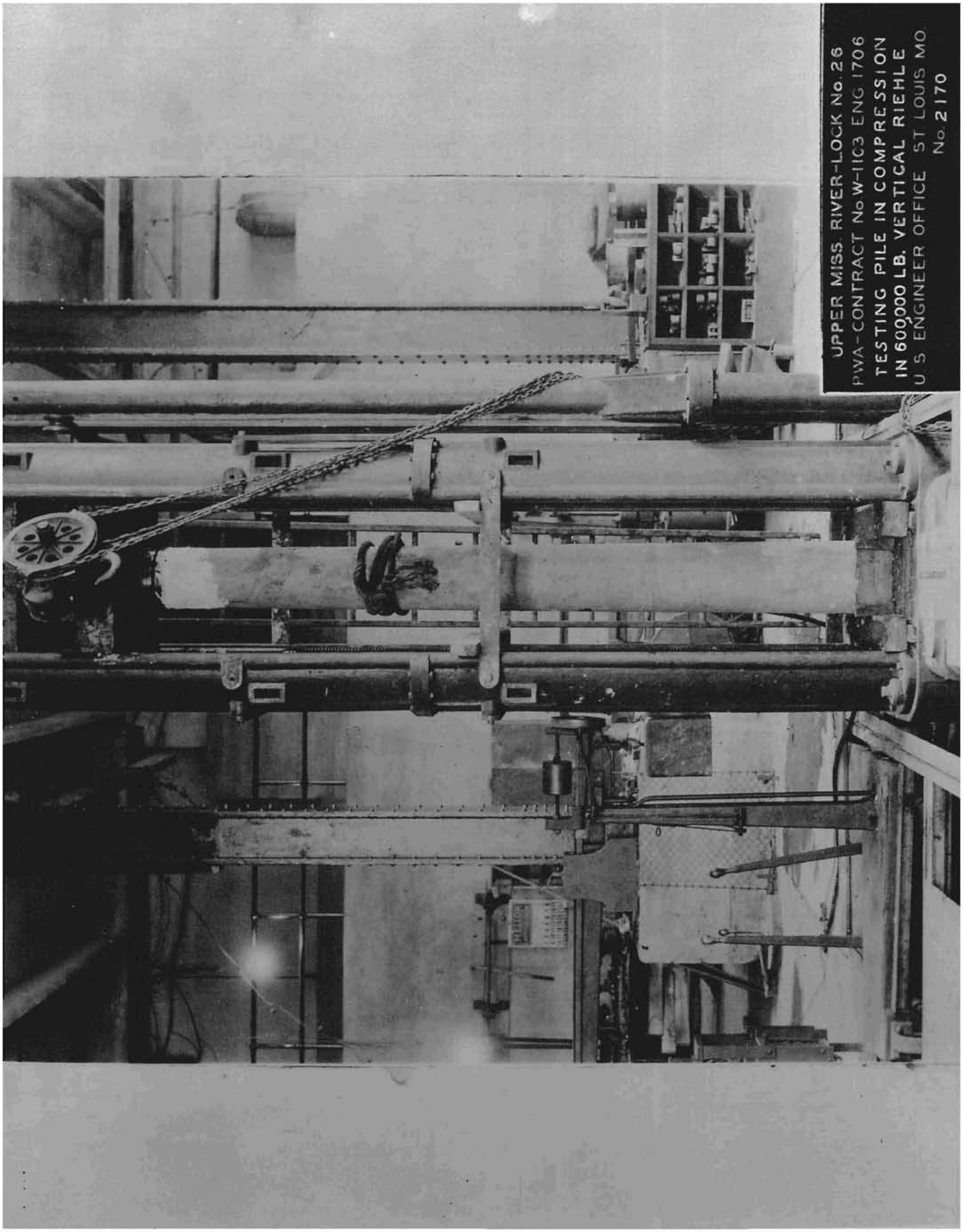
UPPER MISS. RIVER-LOCK No. 26
PWA-CONTRACT No. W-1103 ENG 1706
TESTING PILE IN COMPRESSION
IN 600,000 LB. VERTICAL RIEHLE
U.S. ENGINEER OFFICE ST LOUIS MO
No. 2170

PLATE 37