THE DESIGN AND CONSTRUCTION OF THE PAULIN'S KILL VIADUCT.

The Paulin's Kill Viaduct is a reinforced concrete structure, 1100 ft. long, composed of five 120-ft. and two 100-ft. arches carrying the new double track cut-off of the Delaware, Lackawanna & Western Railroad across the Paulin's Kill valley, in which runs the Paulin's Kill River, a private crossing, a public highway and a branch line of the New York, Susquehanna & Western Railroad. It is located in Northern New Jersey about 3 miles east of the Delaware River, near Portland, Pa. The completed structure will contain 42,000 cu. yd. of concrete and about 500 tons of reinforcement steel. A general description of it will be found in the Engineering Record of Aug. 15, 1908, and some details of the concrete plant and of the early construction methods, in the Engineering Record of Apr. 24, 1909.

The tracks on the structure will be on a rising grade eastward of 0.50 per cent. and the west end will be on a 1-deg. curve, the point of curve being at about the crown of the second arch from the end. Solid rock is found from 0 to 35 ft. below the surface and the piers and abutments are founded on rock.

in which L is the live load stress, D, the dead load stress, and I, the resulting impact stress. The arch when analyzed with these loads according to Professor Cain's theory shows that the resistance line follows very closely the center of gravity of the sections, and nowhere goes outside the middle third, thus eliminating tension in both the intrados and the extrados. The compression in the concrete has been kept below 400 lb. per square inch in both the 100 and 120-ft. arches, and throughout the rest of the structure.

The stress in the steel throughout the structure has been limited to 16,000 lb. per square inch.

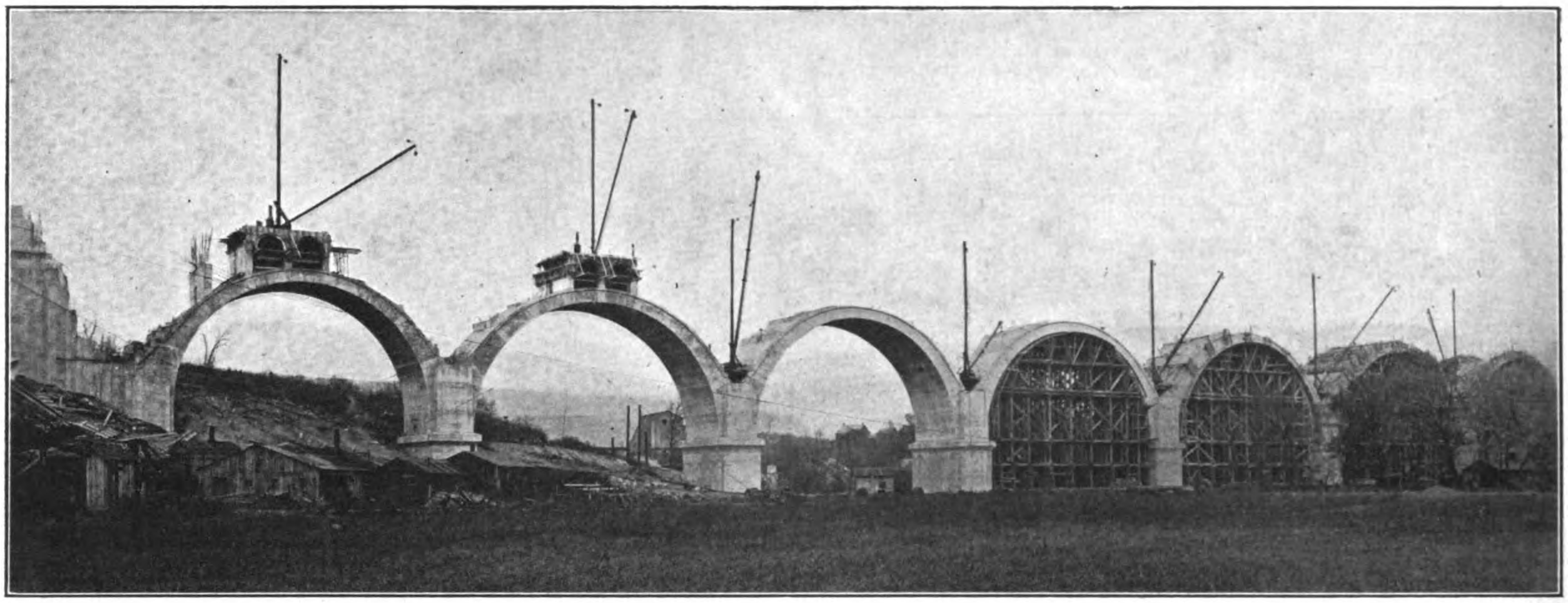
The spandrel arches are carried on reinforced concrete walls which vary in height from 5 ft. 9¼ in. to 38 ft. 8¾ in. The thickness is shown in the accompanying drawing. They are reinforced in each face by ¾-in. square corrugated bars placed vertically on 9-in. centers and by ½-in. bars horizontally on 2-ft. centers. The horizontal bars are continued around the intrados up to the crown, while the vertical bars are bent toward the curve of the arch at an angle of about 30 deg. and terminate in the reinforcement of the solid concrete floor.

The floor has a thickness of 2 ft. above the soffits of the spandrel arches. The reinforcement of 1-in. corrugated bars 6 in. on centers, is

For the analysis of the pier various combinations of the thrusts from the adjacent spans, due to above loadings, were made and the resultants were each combined with the weight of the pier to determine the maximum pressure on foundations. This was obtained with the combination of the dead load thrust from one span and the full live plus dead load thrust from the adjoining span. The pier, 20 ft. wide, for this combination has maximum toe pressure of 25 tons per square foot on the rock foundation.

Each abutment consists of three vertical walls parallel to the axis of the viaduct, tied together by a solid floor system across the top and by horizontal struts at lower levels. These walls are 3 ft. thick and are reinforced vertically in both faces by ¾-in. square corrugated bars 12 in. apart for the full height. No horizontal reinforcement is provided below the line of the earth slope, for the walls will there be protected against severe temperature changes. Above that line, however, ¾-in. square corrugated bars are spaced on 24-in. centers to take the temperature stresses.

The east abutment has a maximum height from foundation to top of parapet wall of 58 ft., and the west abutment, a maximum height of 87 ft., with lengths of 70 and 90 ft., respectively.



General View of Viaduct, with Main Arch Rings Completed, and Spandrel Arches Started.

Both the 120 and the 100-ft. spans are semicircular arches with their springing lines approximately 25 ft. above high water level. The base of rail is about 110 ft. above high water and about 120 ft. above ordinary stages.

The main arches are solid rings of concrete, 34 ft. wide, carrying spandrel arches of 12½-ft. span, poured integral with the solid floor. The rings of the 120-ft. spans are 6 ft. thick at the crown and about 12 ft. thick at the pier skewbacks. Reinforcement of new style corrugated bars is placed near both the intrados and the extrados, the same size reinforcement being used in both planes. In the 120-ft. arches the longitudinal bars are 1-in. square, placed 9 in. center to center, and the transverse bars are ½ in. square, placed 2 ft. center to center. No bars are placed closer than 3 in. to the surface.

In the 100-ft. arches the crown thickness is 5 ft. The longitudinal bars are ¾ in. square, spaced 9 in. on centers, and the transverse bars ½ in. square, spaced 2-ft. on centers.

These main arch rings were designed for a dead load consisting of their own weight, the weight of the spandrel arches, solid concrete floor, 5 ft. of fill and ballast, and the tracks, together with a live load consisting of Cooper's E50 loading. Impact was allowed for in all parts of the structure according to the formula

$$I = L[L/(L+D)]$$

laid parallel to the axis of the bridge, and 3 in. above the plane of the spandrel arch crowns.

The main piers are 20 ft. wide and 42 ft. long and vary in height, from springing line to foundation, from 25 to 63 ft. The highest pier is on the east bank of the stream, it being 152½ ft. from base of masonry to top of parapet, while the first pier on the west bank, carrying the other end of the same arch, is 55 ft. high from rock to springing line, or 144½ ft. in total height. The piers are solid and without structural reinforcement from rock up to a point about 25 ft. above the springing line, where the extradoses of adjacent arches meet. Except for the intrados reinforcement, which is carried down to a point 8½ ft. below the coping, there is no reinforcement in the lower parts of the piers. Above the coping horizontal and vertical reinforcement of ½ in. square corrugated bars is placed in the faces of the pilasters, and above the junction of the extradoses the pier is hollow and consists of four vertical walls, reinforced in both faces, in the same manner as the walls carrying the spandrel arches. The floor over the hollow pier is 3 ft. thick and reinforced in the lower element.

The arch rings were analyzed for three conditions of loading, namely, dead load only, live load on half of the span plus the dead load, and live load on the entire span plus the dead load.

In the east abutment struts 2 x 4-ft. in cross section, each reinforced with three 1-in. square bars in both the upper and lower surfaces are placed at about mid-height of the walls, 12 ft. apart on centers horizontally. The unsupported length of the struts is 12½ ft., which is the clear distance between these vertical walls.

In the west abutment these struts are required at two different elevations. The upper set is 20 ft. below the concrete track slab, and the second set 24 ft. below the first.

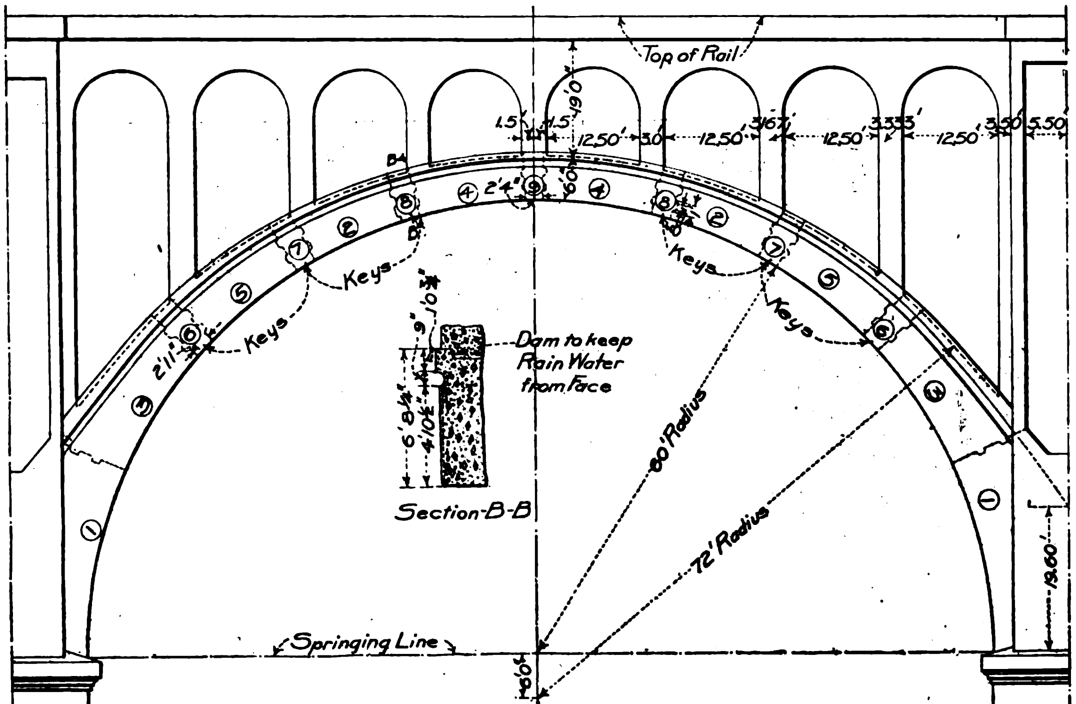
The design of the vertical walls of these abutments was based upon the assumption that the horizontal earth pressures on opposite sides of each would be equal and opposite, which would justify a wall designed for bearing only.

The horizontal struts were introduced to give lateral stability to the walls, which are very high as compared with their widths. They also serve to cut down the unsupported height of wall during construction before the fill is made, thus furnishing a factor against any unbalanced condition that might develop during construction or subsequent settlement of the fill. Owing to the indeterminate nature of these stresses likely to be set up during construction and filling, the struts had to be designed to take both compression and tension considerably in excess of any that might reasonably be expected in the completed structure. In addition they were made of sufficient depth to give them value as diagonal cross-bracing between the vertical walls. Enough steel was distributed in each member to make it a tie capable of taking approximately 100,000 lb. tension, using a unit stress of 16,000 lb. in steel having a specified elastic limit of 48,000 lb. per square inch. As a strut the design will safely take care of about 500,000 lb. in compression, while as a beam it safely supported the staging and centering for the heavy floor slab and the upper walls.

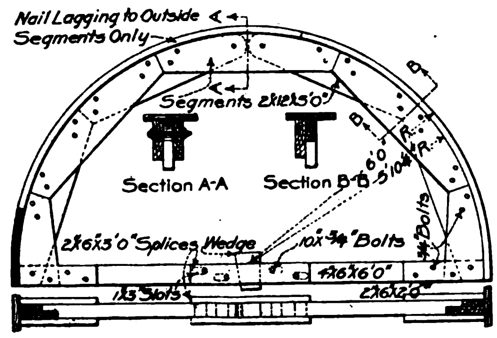
The track is carried on these abutments by floor slabs 3 ft. thick, reinforced with 1¼-sq. corrugated bars spaced 4-in. on centers spanning between the abutment walls. The abutment floor slab was made very heavy to prevent any deflection in the slab which might cause flexure in the supporting walls, which are connected to the slab as a monolithic unit.

In order that inspections may readily be made a 30-in. manhole is placed in the solid floor and spandrel arch above the crown of each of the main arches. Arched openings 2½ ft. wide and 4½ ft. high are placed in the walls carrying the spandrel arches so that the inspector can examine the entire structure. Openings are also provided into both chambers of the abutments. Where the extrados is steep, ladders made of pieces of bent reinforcing rods are built into the concrete.

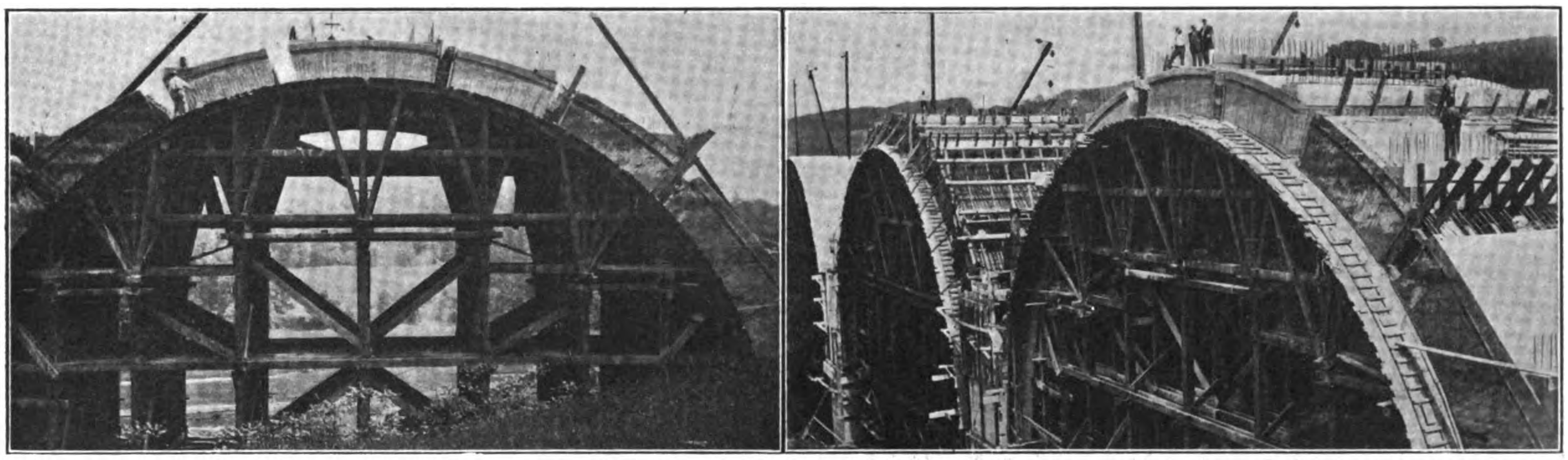
Provision has been made for carrying off rain water, not only from the solid floor, but also from the extradoses of the main arches. Water may find its way onto the latter by being driven in by wind during storms. An 8-in. tile drain is laid on the center line of the floor system of the viaduct from end to end, and summits are formed at alternate piers so that the drainage is collected and discharged through 6-in. vertical tile drains embedded in the concrete walls of the piers. Drainage from the extrados is carried to the inspection openings by proper crowning and finds its way down into the hollow abutments, from which it is discharged to the stream through 6-in. pipes. A 6-in. dike at the face of the voussoirs extends above the extrados, so that any water driven in by the rain cannot flow down and mark the face of the arch, but must find its way through the inspection openings.



Elevation of 120-Foot Arch, Showing Sequence in Pouring Voussoirs.



Collapsible Centre for Spandrel Arches.



Two Views of Main Arch Ring, Showing the Key Spaces between Voussoir Blocks.

Construction.—The construction plant was described in the Engineering Record of April 24, 1909. The mixing plant is located close by, and the concrete is brought to the viaduct in 1½-yd. Shannon bottom-dump buckets, hauled on flat cars by Baldwin 12-ton locomotives.

In building the piers up to the springing lines, derricks were seated on the ground and transferred the concrete from the cars to place. A steel tower was then erected on each pier, and upon it was placed a guyed derrick with a 75-ft. mast and a 65 or 80-ft. boom, together with a 7¼ x 10-in. Mundy double-drum hoisting engine. Each derrick could place concrete up to the crowns of the arches on both sides of it.

Timber centering was employed for all arches, and did not differ materially from the design shown in the Engineering Record of Aug. 15, 1908. As only three 120-ft. centers and one 100-ft. center were provided, it was necessary to use the 100-ft. center and two of the 120-ft. centers a second time in constructing corresponding spans on opposite sides of the middle span. The center was left under this middle span while the others were being moved forward to complete the structure. In moving the centers they were taken apart, the sections lowered by derricks, transferred to cars, hauled to the new location and re-erected.

An accompanying diagram shows the order in which the concrete was placed in the main arch rings. The semi-arch, from springing line to crown, was divided into five main blocks and three keys, and an extra key was placed at the crown to close the work. The blocks and keys were cast in pairs extending the full width of the arch, and each pair was finished at a continuous operation. The maximum yardage in one of the main blocks is 250 cu. yd. The largest pair were cast in 30 hr., when the work was originally undertaken, but on account of the efficiency developed this time was cut down to 24 hr. on the last arch.

The side forms around these blocks were made of 2-in. tongued and grooved faced lagging, held by 3 x 6 and 3 x 8-in. studs, and securely braced against the lagging of the top of the arch by means of diagonal struts. An extra rib was placed on each side in the main centering, beyond the face of the concrete ring, in order to form a working platform and allow the voussoir forms to be readily braced. In addition, long rods and two 10 x 10-in. struts were run across the tops of the forms from face to face of ring for each block. The main blocks numbered 2 and 4 did not require any special precautions to prevent them sliding, but between blocks 3 and 5 it was necessary to introduce struts in order to hold number 5 in place until key 6 could be cast. These struts consisted of four concrete blocks, 2 ft. wide and 3 ft. deep, which were cast on block 3 when it was poured.

All reinforcement was securely wired at intersections. As soon as the lagging had been placed on the centers for a main arch the reinforcement was placed, that for the intrados being held the proper height above the lagging by 4 x 4 x 18-in. concrete blocks. The extrados reinforcement was held in place by tying it to the 10 x 10-in. braces across the top of each block form.

The specifications required that at least 7 days should elapse between placing the blocks and the keys, and that 28 days should elapse after the placing of the last key before striking centers.

Three classes of concrete were used. Class A, for the floor system and spandrel arches, was mixed in proportions of 1:2:4; Class B, used in the main arch rings, spandrel piers and abutment walls, was mixed in proportions of 1:3:5; Class C, used in the main piers and footings, was a 1:3:5 mixture, having two-man quarry stone embedded in it. The stone used was limestone, crushed to pass a 2-in. ring. Nazereth Portland cement was used.

Measurements showed a maximum crown deflection after the centers were struck of 3/16 in.

It will be noted from the accompanying photograph that when all of the main arch rings had been completed the centering was removed before the spandrel arches were placed.

After the completion of the main arches work was begun upon the two spandrel arches above the crown of each main arch. The centering and concrete was swung to place for these two spandrel arches by the derricks on the piers.

The derricks were then moved from the towers on the piers to the two completed spandrel arches, the centering of the latter being left in place. In this position each derrick could place the remaining spandrel arches on either side as far as the piers.

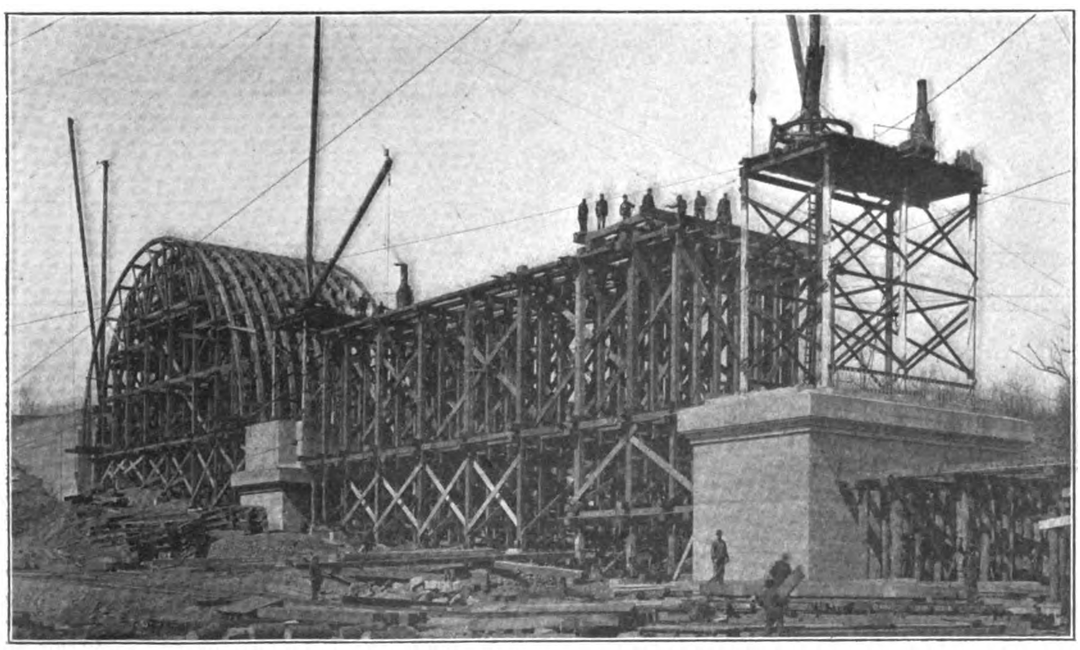
The forms for the spandrel arches are of the collapsible type, so that they may readily be removed from span to span. They are shown in detail in an accompanying cross section. It will be noted that they are hinged at the top and that the tie across the bottom at the springing line can be extended by means of a wedge.

All exposed surfaces of the concrete were carefully spaded during construction. No subsequent finishing has been necessary.

The viaduct is being built under the general direction of Mr. G. J. Ray, chief engineer of the Delaware, Lackawanna & Western Railroad, and under the immediate direction of Mr. F. L. Wheaton, engineer of construction. Mr. O. H. Kellogg is resident engineer and Mr. Galvin, inspector.

The viaduct was designed under the direction of Mr. Lincoln Bush, formerly chief engineer. The detailed design was made by Mr. B. H. Davis, formerly assistant engineer in charge of masonry design.

John Goll & Company, contractors, Philadelphia, are building the viaduct, under a sub-contract from Reiter, Curtis & Hill, of Philadelphia. Mr. John Feite is superintendent of construction.



Centering and Steel Tower for Derrick, Paulin's Kill Viaduct.

RAISING A HIGH-PRESSURE WATER PIPE.

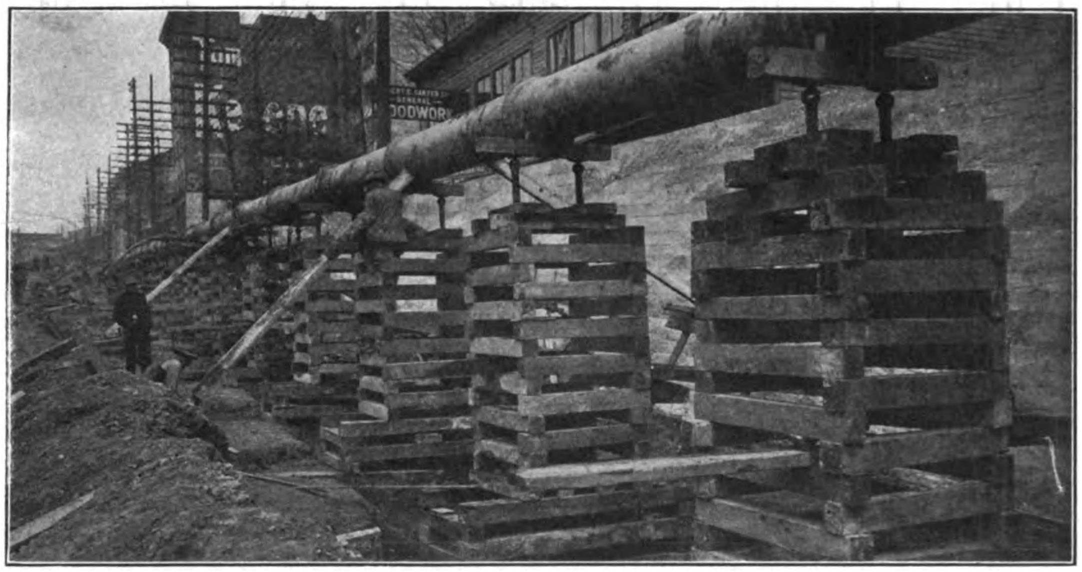
In regrading one of the streets in the business section of Seattle, it was necessary to raise an 1800-ft. length of 20-in. cast-iron water main operating under a pressure of 130 lb. to the square inch. The location of the main was such that it was impracticable to cut it out of service while the raising was in progress. The decision therefore was made to undertake the work without interfering with the operation of the pipe.

The street in which the pipe was laid had a dip in grade about midway on the 1800-ft. length that it was necessary to raise. The street improvement was made to remove this dip and to secure a uniform grade of 4.9 per cent. The pipe was laid at a depth of 4 ft. below the original street surface and the maximum raise required was to be about 13 ft. above that surface, making a total of 17 ft. From this maximum the amount of change tapered off gradually toward both ends of the length of the main affected.

The scheme of raising adopted was to carry up the whole length of the pipe by means of jack-screws and cribbing. The trench was first opened and the pipe brought to the surface in this manner. Then shoring was placed across the trench and the raising continued until the final position was reached. A cross-timber was placed near the bell end of each section of the pipe, to provide a bearing for a pair of jack-screws that were seated on a crib of 6 x 6-in. timbers built up in cob-house fashion. Side struts were used at intervals to maintain the alignment. Extra cribs were placed under valves and specials.

Enough men were employed to turn the jacks so that 60 ft. of the pipe could be raised simultaneously. The work was started at one end, by raising about 60 ft. of the pipe 1 in. Then the next 60 ft. was raised that amount and so on continuously to the opposite end of the work. As soon as that end was reached, the men worked right back again, and so on until the pipe was up to the proper height.

One man was stationed at each jack and all of the men worked to a signal from the foreman so the jacks under the 60-ft. lengths in which the raising was handled were turned at a uniform rate. As soon as the jacks were extended a 6 x 6-in. timber was slipped under the pipe at each crib to take the load until the jacks could be removed and another story added to the crib. The jacks and timbers thus were manipulated so the pipe was never unsupported at any point.



Method of Raising High-Pressure Water Pipe.

After the pipe had been raised to the final grade a framed timber bent was built under each section to carry it until the fill was completed. While the work was in progress practically no leakage occurred at the joints although some of the latter had to be caulked occasionally. After the pipe was out of the trench the balance of the raising was accomplished in 14 days.

The work was done under the immediate direction of Mr. T. H. Carver, assistant city engineer of Seattle, by Paul Steenstrup, contractor.

THE EXTRACTION OF OIL FROM DEEP WELLS by the Leinweber method is accomplished by means of an endless cable carrying absorbent material. As described by Mr. F. A. Talbot in the "Engineering and Mining Journal," a belt faced on one side with carpet shag, is lowered into the bore hole and absorbs oil as the band trails through the pit; the band is subsequently passed through mangling rollers above ground and the oil which it contains is squeezed out, the dry portion of the band descending continuously into the pit. The equipment was put into actual operation at a bore hole 3838 ft. deep in the Galician oil fields. The cable used was built up of a series of steel wires each 0.098 in. in diameter. The band was 2.2 in. wide and covered with shag. It was found that the band possessed an absorbing and hauling capacity of 9.16 oz. of crude oil per foot of cable, and the net yield at the rollers was 7 oz. per foot of band. The cable speed is given as 2½ ft. per second. A plunger system previously used at this hole raised 44,000 lb. of oil per day, according to Mr. Talbot. With the Leinweber band the yield was about 50 per cent greater, the aggregate being 66,000 lb.