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Ornamental Concrete Elevated Railway, New York City

By Maurice E. Griest*

One of the new rapid-transit lines now being built in New York City passes along the Queens Boulevard, a 200-ft. automobile thoroughfare, for more than three-fourths of a mile. To avoid disfiguring the boulevard—for which a large expenditure had already been made—with a steel elevated railway of the ordinary type, careful study was given to working out a practical type of elevated-railway construction which should be ornamental, and less noisy than a plain open-floor steel structure; and at the same time cheaper than subway construction. The final selection was a new form of structure, a reinforced-concrete arcade or series of vaulted concrete arches.

An ornamental steel structure with columns curving into the longitudinal and cross-girders by sweeping curves was considered. The details were unsatisfactory, however, and no appreciable reduction in noise could be expected. The estimated cost, with open tie floor, was about 1.2 times the cost of a plain elevated structure.

By making the steelwork of plain form and detail, securing the flowing lines by means of a concrete covering or veneer (Fig. 2), fairly satisfactory architectural results could be obtained. But with open floor construction and the ties delivering the impact of the train directly to the concrete covering of the stringers, it seemed doubtful that the concrete would remain intact. This objection was overcome by changing to a through girder construction with solid floor. For the latter design the estimated cost was 1.6 times the cost of a plain steel structure.

Substituting entire reinforced-concrete arch construction for the type just discussed, satisfactory results in point of solidity, grace, beauty, monumental effect and the reduction of noise and vibration were obtained. The estimated cost was about 1.8 times that of a plain steel structure.

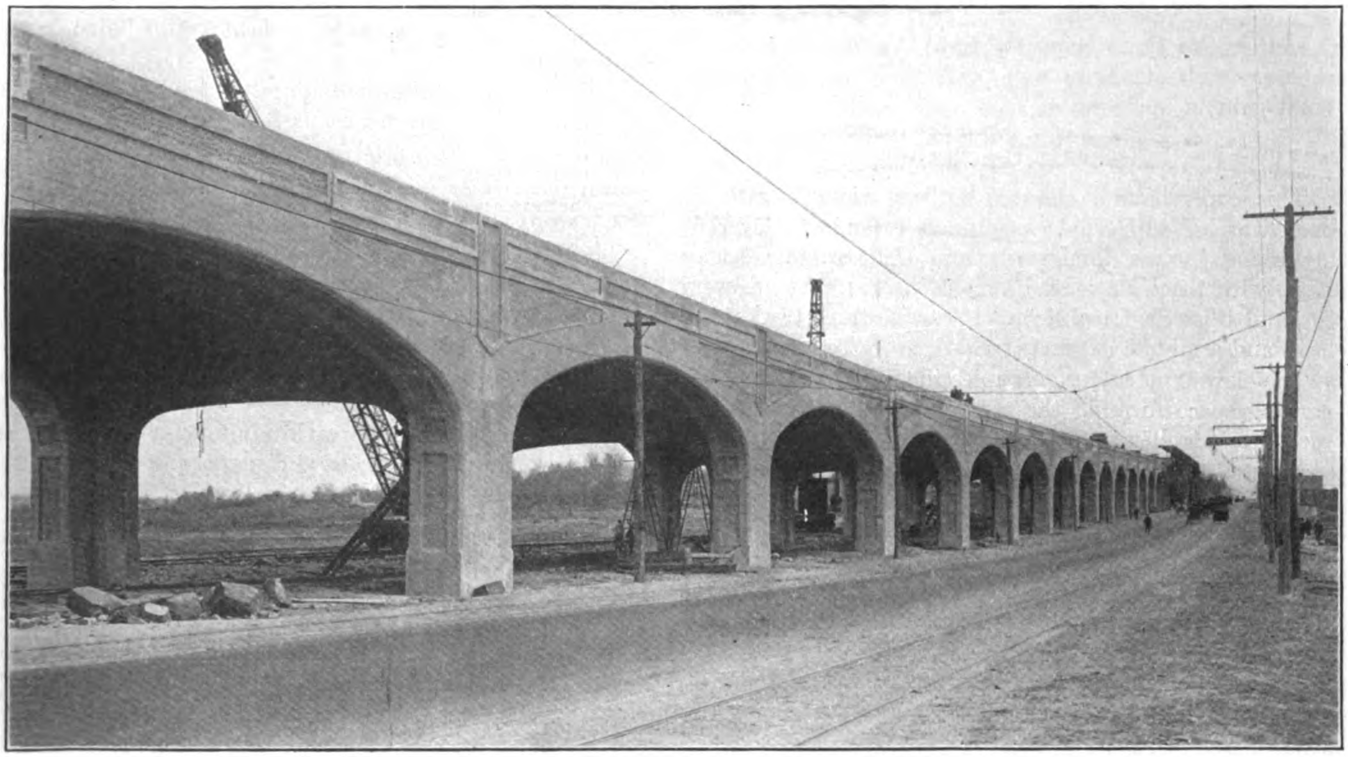

FIG. 1. COMPLETED STRETCH OF QUEENS BOULEVARD ELEVATED STRUCTURE

Colored tile is to be placed in recessed panels of the face. The station steel in the distance is to be cased in concrete

The substitution of a subway for the elevated structure would have been highly desirable except that the cost was estimated at four times that of an ordinary elevated structure.[1]

*Assistant Designing Engineer, Public Service Commission for the First District, 154 Nassau St., New York City.

[1]Topographical conditions made it necessary to extend a subway to the east beyond the point where the line leaves the Boulevard, increasing the relative cost for the length on the Boulevard from three times to four times that of the ordinary elevated.

In choosing between these different types the ornamental steel structure was abandoned because of its unsatisfactory detailing. For the remaining types the following tabulation shows the cost ratios:

RELATIVE COSTS OF DIFFERENT TYPES OF STRUCTURE

Ordinary steel elevated	1.0
Subway	4.0
Concrete-covered steel structure with solid floor	1.6
Concrete arch structure	1.8

Eliminating the subway because of its great excess of cost, it was decided that, the difference of cost between

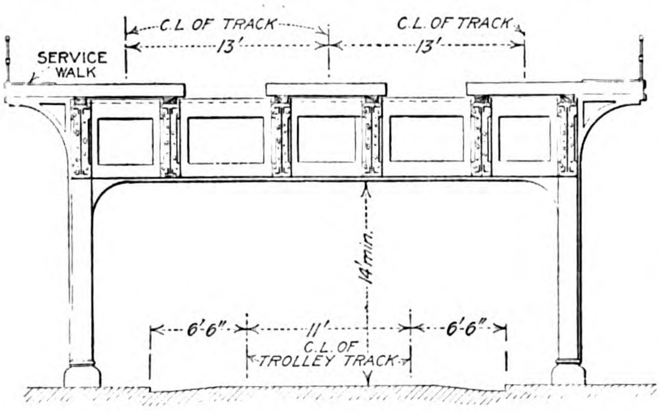

FIG. 2. SECTIONAL SKETCH OF VENEERED STEEL ELEVATED STRUCTURE

the other two types being so slight, the selection of the concrete arch structure was permissible because of the great gain in appearance.

General Description

This concrete-arch elevated railway viaduct with its domed arches supported on columns extends for over ¾ mi. along Queens Boulevard from Hill St. to Gosman Ave., with three stations, Rawson St.-Moore St., Lowery St., and Bliss St.-Carolin St. It carries three tracks, two local and a middle express track.

The elevated structure occupies the middle of the 74-ft parking space forming the central portion of the boulevard. Two trolley tracks occupy the center of the parking space and pass beneath the structure. Solid piers could not be used therefore either at the intermediate supports or at the abutments.

The over-all width of the structure is 44½ ft. A normal span length of 65 ft. was adopted, to fit both the 200-ft. block length and the 60-ft. width of cross-streets. Spans departing from the standard, in some cases down to a length of 52½ ft., were required at stations on account of the longer spans at the mezzanines.

A minimum clearance of 14 ft. over the trolley tracks is required. It was also desired that the height from street to station platforms should not exceed 30 ft., unless elevators or escalators were provided. This meant that the arches must be of very low rise; in fact with the flattest practicable arch the height to station platforms slightly exceeds 30 ft.

The general design of the structure is shown in Fig. 3.

Design of Arch

The ends of the arch are fixed, as the vaulted shape prevents any possibility of hinging. The investigation of stresses was made for a fixed arch by the strict method of elastic calculation, following closely the procedure given in Howe's "Symmetrical Masonry Arches." It was made on the assumption that each longitudinal strip acts independently of every other strip. Later, after the design was completed, the effect of the dome-shape was considered; the stresses were recalculated and proved to be not materially different from those found under the assumption that each longitudinal segment acts independently.

The design was made for a live-load consisting (for each track) of four 30,000-lb. axle loads, spaced 5, 10 and 5 ft.[2] Temperature change was taken as 30° F. rise or fall. Temperature stresses as well as the stresses due to shortening of the arch under load are included in the figures quoted later.

With a 4-ft. longitudinal rise on center line in the trial design and other dimensions as follows:

Span c. to c. of skewbacks	3 ft.
Rise transversely	2 ft. 4 in.
Crown thickness	2 ft.
Haunch thickness	4 ft.
Shape of arch	Circular segmental

The dead- and live-load stresses were found in general to fall within allowable limits, but the temperature stresses were excessive and the combined stresses prohibitive.

After some further rough trials it became evident that the rise of the arch and its flexibility must both be increased in order to reduce temperature stresses. The flexibility was increased by maintaining a constant section from crown to quarter point and increasing the thickness rapidly thence to the haunch. With a three-centered

[2]Dead-load includes weight of ballast at 100 lb. per cu.ft., concrete at 150 lb. per cu.ft., and the track. The stresses for a live-load of 2000 lb. per lin.ft. of track were in all cases less than those for the concentrated loads. The concentrated loads were assumed to distribute over 5 ft.—three ties—longitudinally and 12 ft. transversely. In spite of the use of ballast floor, impact was added, to the amount of 80% of the live-load, as computed from the Public Service Commission's formula: $I = 125 - \frac{1}{8}\sqrt{2000\,L - L^2}$; where L = loaded length of track. The stresses quoted further on include the impact effect.

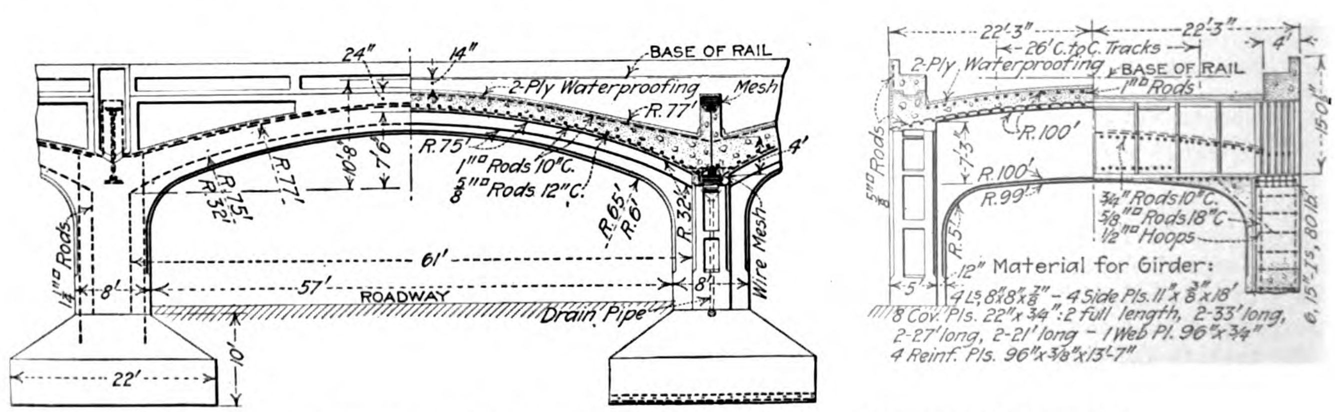

FIG. 3. TYPICAL ARCH SPAN OF QUEENS BOULEVARD ELEVATED STRUCTURE

segmental arch embodying these modifications, and with a longitudinal rise of 6½ ft. and a transverse rise of 18 in., the stresses seemed to come within practicable limits and the design was adopted. The reinforcing used comprises longitudinal rods 1 in. square spaced 10 in. c. to c., top and bottom; and transverse rods ⅝ in. square 18 in. c. to c., bottom only.

The stresses were first computed on the assumption of tension in the concrete, giving the following results:

	Crown		Quarter Point		Haunch	
	Top	Bottom	Top	Bottom	Top	Bottom
Concrete (max.)	+680	+500	+590	+150	+300	+430
Steel (max. comp.)	+10,000	+7,500	+9,000	2,200	+4,500	+6,500
Concrete (min.).	+40	—340	+230	—30	—260	0
Steel (min. compression or max. tension).	+600	—5,000	+3,400	—450	—3,900	0

As under these stresses the concrete would part in tension at the crown and the haunch under a combination of the most unfavorable conditions, the calculation was revised and the arch ring treated as a reinforced-concrete section subject to direct thrust and bending, with the usual assumption that tension is taken up entirely by the steel. On this basis the maximum stresses became:

	Crown		Quarter Point		Haunch	
	Top	Bottom	Top	Bottom	Top	Bottom
Concrete (max.)	+690	+500	+590	+150	+300	+560
Steel (max. comp.)	+10,000	+7,500	+9,000	+2,200	+4,500	+8,500
Concrete (min.).	+40	0	+230	0	0	0
Steel (min. compression or max. tension).	+600	—9,000	+3,400	—450	—7,000	0

All of these stresses are well within the elastic limits of steel and concrete, and, except for the maximum compression of the crown, within the Public Service Commission's working stresses (600 lb. concrete compression). As this case occurred only under the worst condition of load and temperature combination and with a large impact factor added to the live-load stress, it was considered entirely safe and no change was made on account of it.

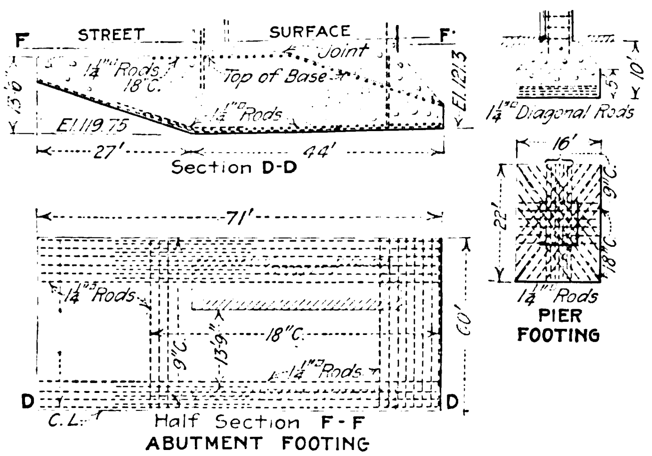

FIG. 4. NORMAL PIER FOOTING AND ABUTMENT FOOTING

Design of Columns

A cross-arch is sprung between the columns for appearance only, as an arch of sufficient strength to support the vertical component of the thrust at the haunch of the longitudinal arch proved entirely impracticable. A steel girder embedded in the concrete carries the load to the columns.

The dead-load thrust of adjacent arches balances at the top of the columns; the abutment of course is designed to resist the full thrust of the arch. The maximum live-load thrust of a single arch (adjacent arches unloaded) was assumed to distribute over *three bents,* so that each column was designed for one-sixth of the total live-load thrust from one arch with three tracks loaded. The highest column governed, and all columns, for appearance, were made of the same section.

A steel column, covered with concrete for architectural effect, was uneconomical, as the steel was concentrated near the center and therefore ineffective in resisting the bending stresses due to horizontal thrust, and the concrete added little strength. Furthermore, in order to prevent shrinkage, temperature and bending cracks, some reinforcement near the concrete faces would be required in any case. The steel columns would have been valuable during erection to support the girders so that they could be erected before any concrete was poured.

Type of Column Used

A concrete column with structural reinforcement in the form of a tower, with the verticals near the corners of the column, was economical. A detail, however, of a grillage to distribute the load of the completed structure over the concrete, and to support the weight of the girder during erection on the tower, proved unsatisfactory.

The only advantage of structural reinforcement over rods (which were used) was the support of the girder during construction. Practically the same results were obtained by pouring the concrete columns to the underside of the grillage, setting the steel, and then proceeding with the casting of the arch. In the end this proved to be an advantage to the contractor, for it was possible to pour the footings and columns while the girders were being detailed and fabricated.[3]

The columns are 5x8 ft. in section, with a rib 4 ft. wide projecting 1 ft. on the inside; they are reinforced with four 1¼-in. square rods at each end and ½-in. square hoops spaced 2 ft. apart vertically. On each side were placed four ⅝-in. square vertical rods to prevent temperature and shrinkage cracks. The reinforcement is proportioned for the bending with both ends of the column fixed.

On the basis of a maximum footing pressure of 2 tons per sq.ft. under direct load only, and a maximum edge pressure of 4 tons per sq.ft. (on sand and gravel), a footing 16 ft. wide and 22 ft. long of reinforced-concrete slab type with diagonal reinforcement was designed, as may be seen in Fig. 4. The original intention was to pour the entire footing as a monolith. This was found to be impracticable, however, so the footing was placed in two pourings, the top of the first lift being toothed in order to take care of horizontal shear.

Stations: Girder Construction and Mezzanine

At stations it was necessary to provide a mezzanine below the track for ticket booths and other facilities. A 65-ft. span was too short to give the required space, and the regular concrete-arch construction did not give sufficient clearance above the street to accommodate a mezzanine. Therefore the arch was replaced with steel girder construction 90 ft. in span.

Two serious problems arose in the station construction. The first was the support of the platforms along the sides of the regular concrete arch spans. In order to occupy as little as possible of the parkway, the structure was not widened, but the platforms were designed to

[3]About 75% of the columns were poured before any structural steel was delivered on the work.

overhang (Fig. 5). The outer edge of the platform was carried by longitudinal girders supported by brackets attached to the cross-girders, thus avoiding putting any additional load on the arches. Expansion in the arches produces no longitudinal movement, while the girders must expand and contract. The longitudinal girders and canopy columns are covered with concrete, thus maintaining a concrete exterior throughout. To prevent the concrete from cracking, expansion joints in the platforms were provided at the brackets. The expansions were so arranged that the joints in the concrete in general occur at the edge of a panel or pilaster.

The second problem was to provide for the arch thrust at the mezzanine spans. It was necessary to carry this thrust through the steel of the mezzanine and track floors. The average position of the thrust is slightly above the level of the former. The thrust is transmitted to the longitudinal struts by a vertical grillage of 20-in. I-beams attached at the top and bottom to horizontal girders in the track and mezzanine floors. It was first planned to attach the vertical grillage to the main cross-girders which support the longitudinal girders; the details, however, were very cumbersome and the actual stresses indeterminate. To avoid these difficulties the grillage is set clear of the cross-girders and its weight supported from the track floor. The thrust is carried into the beams by the concrete surrounding the grillage. The stress in the struts of the mezzanine floor is about 6000 lb. per sq.in. without temperature, and 14,000 lb. per sq.in. at maximum temperature; the shortening of the struts from the 8000 lb. per sq.in. temperature stress about equals the expansion of the steel due to the change in temperature, so that there will be no appreciable movement from temperature expansion.

Because of the increased load of the station span including the mezzanine, and the reduced section of the columns (due to stairways), it was necessary to provide heavy steel columns to support the main cross-girders, which receive all the load; the footings also were enlarged, to 21x26 ft.

The abutments at either end of the concrete structure, where the steel elevated railway adjoins, have to take a maximum arch thrust of 4,088,000 lb. As passage for the trolley cars had to be provided, each abutment is built as a set of three heavy reinforced-concrete piers above ground connected at the top by a heavy reinforced slab to carry the thrust into the piers, and at the bottom by a single massive spread footing. Sufficient tension steel is provided for the maximum bending stresses due to the arch thrust, and sufficient concrete area to take the shear. The toe of the footing is designed as a cantilever with a maximum toe pressure of 4 tons per sq.ft.

In arranging pouring joints the main objects considered were: (1) To secure stability, by making the joints

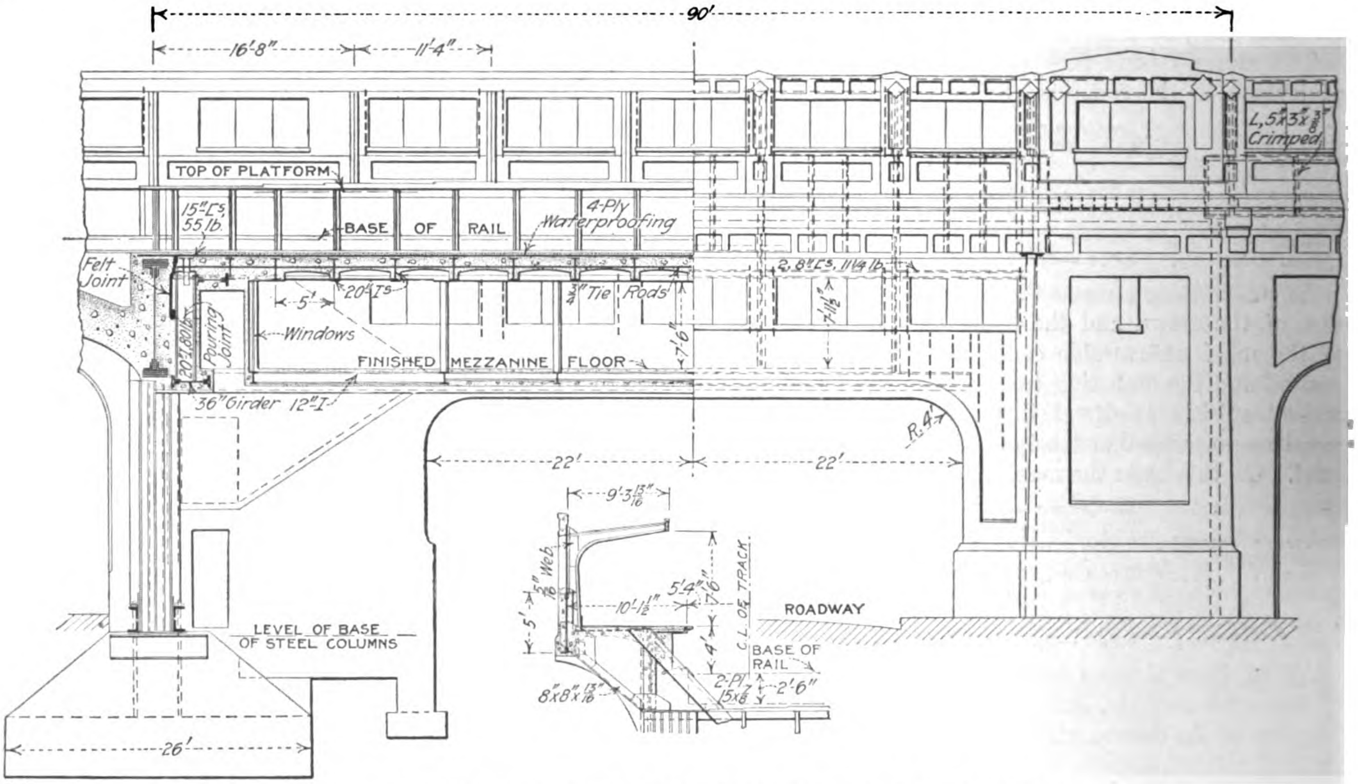

FIG. 5. STATION CONSTRUCTION ON CONCRETE ELEVATED RAILWAY

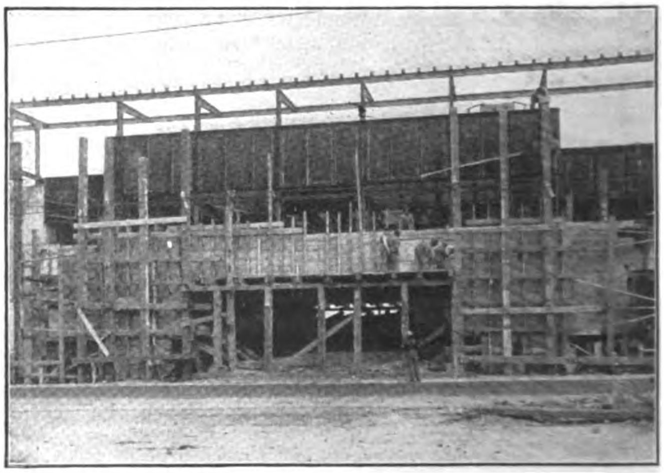

FIG. 6. STATION SPAN WITH FORMS FOR MEZZANINE PARTLY IN PLACE

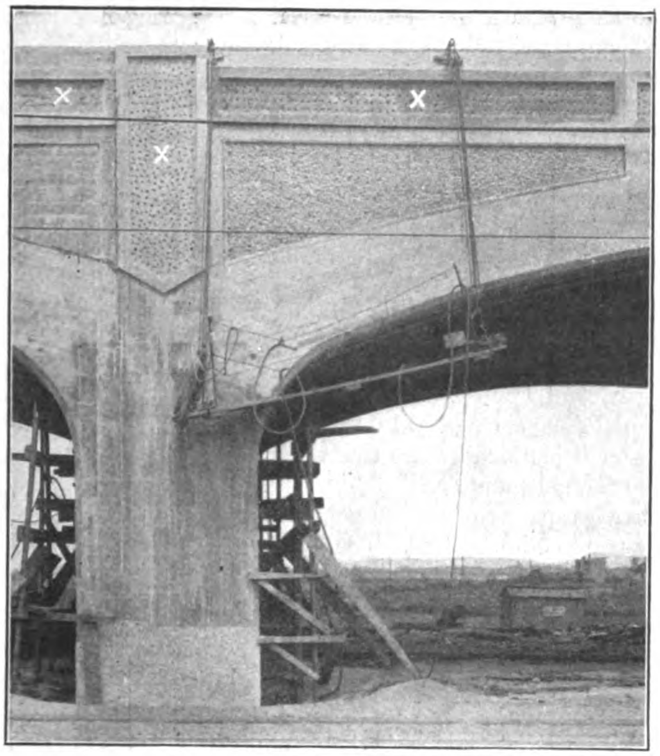

FIG. 7. FACE FINISH, QUEENS BOULEVARD ELEVATED RAILWAY

The arch at the right is tooled; the sunken panel is rough-point, the remainder of the face four-point work; the band around spaces "x" is rubbed. Colored tile is to be set in spaces "x." The arch at the left is untooled, showing the form marks.

normal to the line of strain, or by securing the sections against sliding through the use of dowel rods embedded about 3 ft. in each layer; (2) To arrange the joints where possible so that at the surface they came at edges of panels, etc., so as to be practically invisible.

The ordinary pier footings were divided into two pourings, as previously mentioned. The large footings of the abutments required more divisions. Here the work had to be done in longitudinal sections on account of the cantilever action. The base was, therefore, divided into six longitudinal sections, each containing about 300 cu.yd., equal to the normal daily capacity of the contractor's concreting plant.

The columns were poured in all cases to the underside of the grillage beams under the main cross-girders. In this pouring, skewbacks were molded for both longitudinal and cross arches, as shown in the section in Fig. 8. The main arch was poured in three sections: first, the middle third, to bring the weight on the forms so that any settlement in them occurred before the arch was finally jointed; second, the haunches and the cross arch on both sides; third, the blocks covering the ends of the cross-girder. These blocks were poured before the arch centers were struck. The spandrel walls of the arch are separated from the arch vault by expansion joints made of a layer of elastic cement; these walls were poured last.

Finish of the Surface

In general the exposed concrete faces were dressed with a four-point tool used in a Boyer air hammer, with points set at the corners of a 1-in. square. Sunken panels on the underside of the ribs and in the spandrel walls are rough-pointed, using a single-point tool in an air hammer. Around the panels in the columns and arch ribs, and at other right-angle corners, a 6-in. draft margin is drawn with parallel cuts ⅝-in. apart. The band around the colored tile on the face of the structure is rubbed with carborundum brick.

Forms and Plant

Wooden centers and forms were used exclusively on the work. The arch centers, designed by the contractor, are shown in detail in Fig. 8. It should be noted particularly that the lagging was supported on transverse rather than longitudinal trusses. The transverse curvature is uniform for all spans, so that the trusses could be used for any span by respacing them longitudinally.

The concrete was mixed at a central plant consisting of a 1-yd. Chain Belt rotary mixer driven by a 30-hp. a.c. motor. Cement was moved from the cement house to the platform of the mixer by a Robins belt conveyor driven by a 2-hp. a.c. motor, while the sand and gravel were handled with a 1-yd. Hayward clamshell bucket operated by a six-drum 52-hp. Maine Electric Co. hoist. The maximum capacity of the plant was 450 cu.yd. per day; the average capacity, about 300 cu.yd.

The concrete was handled for the most part in 1-yd. bottom-dump buckets and distributed on Koppel side-dump cars rebuilt for the purpose. These cars were moved in 5-car trains by three 24-in.-gage locomotives.

The concrete was handled and the forms moved and reset by two 15-ton Brownhoist locomotive cranes with 70-ft. booms. A derrick-car of 40 tons' capacity was used in setting the cross-girders. The heavy longitudinal girders at the station were handled by two 30-ton stiff-leg

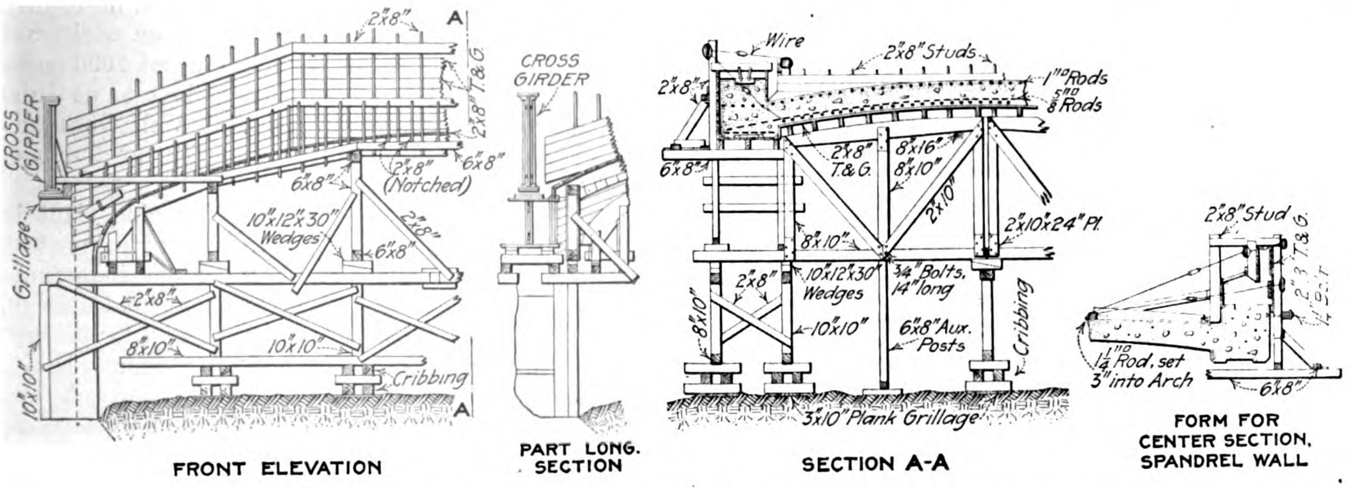

FIG. 8. FORMS FOR ARCH CONCRETING, QUEENS BOULEVARD ELEVATED

derricks. One stiff-leg derrick of the same capacity was used on the superstructure for erecting platform girders and light canopy material.

The plant for finishing the concrete consists of one 50-hp. locomotive boiler and one Ingersoll-Rand straight-line compressor with a capacity of 175 cu.ft. of free air per min. This supplies air for 23 hammers although usually only about 15 are working at one time. The staging for finishing the under side of arches is hung from the spandrel walls.

Progress

The first concrete in the footings was poured on July 1, 1913, and the first arch poured on June 19, 1914. The center of this arch was struck on Aug. 8. The contract provides that the concrete in the arches shall set 28 days before the centers are struck.

The concrete of the arches has all been placed and the concrete covering the steel of the stations is now being poured. About one-half of the surface has been finished. The daily rate for four-point work is from 75 to 90 sq.ft. per man per 8-hr. day, for rough-point work, about 60 sq.ft. The drafted margin is cut at the rate of about 30 lin.ft. per man per day. The above rates include the time required for shifting staging, etc.

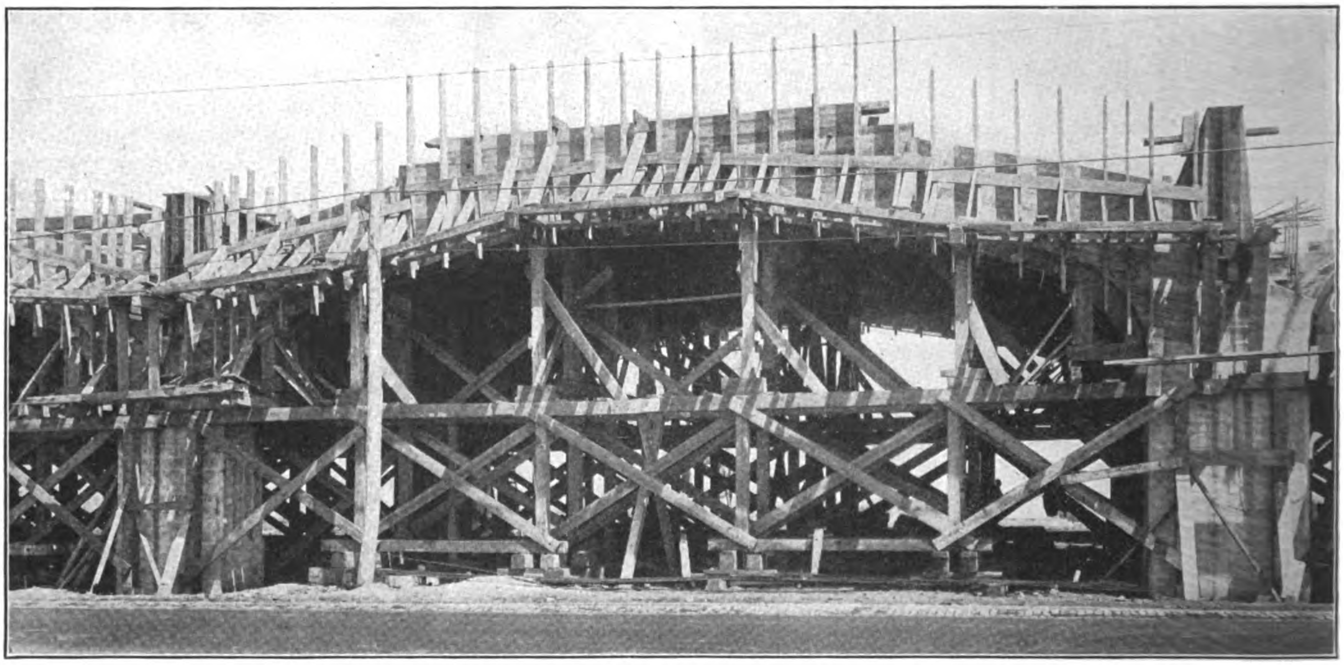

FIG. 9. ONE OF THE ARCHES OF THE QUEENS BOULEVARD WITH THE CENTERS IN PLACE

Personnel

The concrete viaduct is a part of the Dual System of rapid transit which is being constructed by the City of New York, acting by the Public Service Commission for the First District. The design was made and the construction supervised by the Commission's engineering staff, Alfred Craven, Chief Engineer; Robert Ridgway, Engineer of Subway Construction; D. L. Turner, Deputy Engineer of Subway Construction. The construction was done under the supervision of John H. Myers, Division Engineer, and Robert H. Jacobs, Senior Assistant Division Engineer. The design was made under the personal supervision of the writer under the general direction of Sverre Dahm, Principal Assistant Engineer in charge of design, and A. I. Raisman, Senior Designing Engineer. The architectural treatment was designed under the direction of Squire J. Vickers, Designing Architect. The E. E. Smith Contracting Co. is the contractor for the elevated line to Corona, which includes the concrete-arch viaduct.

Variable-Speed Pumping Plant for a Drainage District

The pumping plant for the South Quincy Drainage and Levee District, in Adams County, Illinois, is designed to pump the drainage and seepage waters of a drainage district of approximately 6000 acres, to which an additional 4000 acres of hill drainage have to be added. This water is handled by two electrically driven double-suction centrifugal pumps with 36-in. discharge openings. They have a capacity of 60,000 gal. per min., and are operated by two 200-hp. motors. The layout of the plant is shown in Fig. 1. The current is supplied by an 11,000-volt line from the Mississippi River Power Co., which is transformed to 440 volts.

This plant has to pump all water falling in the district, as no sluiceway is provided. The topography of the county is such that many lakes can be drained by an additional 2 ft. of lift, and since these would add about 1000 acres of agricultural land it was considered best to do this. The pumps were built by the American Well Works, of Aurora, Ill., and were installed by the Farrar Pump & Machinery Co., of St. Louis, Mo.

One of the special features of this pumping plant is the chain-drive arrangement. In order to get the greatest efficiency the manufacturer recommended that the pumps be driven at 219 and 244 r.p.m. for different stages of water. To accomplish this change of speed, sprocket wheels of two different sizes are fitted to the motor shaft.

In some other plants of this kind the sprockets are slipped on and off the shaft as required. In this case, however, the two sprockets (with 30 and 27 teeth) are mounted permanently on the motor shaft, while the 59-