HOW AN ELECTROSLAG WELD IS MADE

Russian engineers developed electroslag welding in the early 1950s for splicing heavy steel girders in a single pass, cutting manpower costs significantly. Though U.S. industry glimpsed its first machine for this type of weld in 1959, the process didn't catch on until [the early sixties].

Electroslag welding is done, usually in a shop, with the steel plates held vertically and with their faces separated. Water-cooled copper shoes placed on the sides of the joint contain the weld material as the splice is, in effect, cast.

Welding begins in a sump beneath the joint as one or more electrodes are fed continuously into the area through guide tubes, which may or may not be consumed during the process. Electric arcs from the electrodes initially heat the slag until molten; then the arc is extinguished by being submerged in the slag. The conductive slag is maintained in a molten condition by its resistance to current passing between electrodes and plates. Melted electrode and plate metal collect in a weld pool beneath the slag pool and slowly solidify to form the weld. The weld solidifies from the bottom upward, always with a molten metal covering the solidifying weld metal.

Welding parameters are critical. If amperage and voltage aren't carefully controlled, centerline cracking can occur.

Electroslag welds can be made from 0 to 20 in. thick with nonconsumable guide tubes and to an unlimited thickness with consumable guides, according to the American Welding Society. For larger weldments the electroslag process in a single pass can join plates that might require 150 passes with conventional submerged arc methods, producing cost savings on the order of 10:1

SOURCE: *Engineering News-Record,* Nov. 23, 1972.

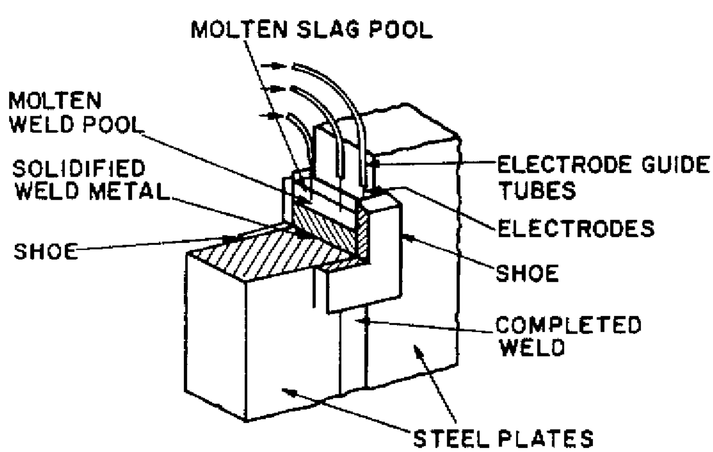

Figure 5-8 In electroslag welding, electrode wires become molten along with the ends of the members being joined. They solidify to weld the sections together. *(ENR.)*

Despite these problems with materials, however, most of the bridge disasters of the last century have been due to design methods that were not sophisticated enough to account for subtle effects—wind-induced reasonance at Tacoma Narrows, for example—or due to poor maintenance. This chapter discusses three doomed spans that illustrate the lessons learned by bridge designers over the past half century.

The Bridge as a Machine: Hackensack River Bascule, 1928

Dynamic as well as static forces must be taken into account in the design of a moving bridge. That seems obvious now, but it was not obvious before the bascule bridge carrying Lincoln Highway over the Hackensack River near Newark, New Jersey, failed on Saturday night, December 15, 1928. In a bascule, a heavy counterweight balances the weight of the bridge span itself, so the span can be swung upward for ships to pass. When the bridge is in its "closed" position, allowing cars to cross the span, the counterweight (in this case, one for each half span of the bridge) is suspended above the roadway.

Although the Hackensack's bascule counterweight weighed 770 tons, it was easily supported by thin latticework towers—or so the designers thought. As the editors of ENR noted 3 weeks after the failure, "This member apparently had only light service to perform; it was subject to small static stresses. But its actual service as a member of a moving machine was more severe. . . . The accident was a machine failure, resulting from stresses, distortions and oscillations arising under operation against the forces of friction and inertia. As such, it is one of the most important structural accidents of recent times."

The discussion which followed over the next year turned *Engineering News-Record* into an unusual forum in which the distinguished investigators of the disaster did not shrink from blaming the bridge's distinguished designer. The designer then replied that poor construction and maintenance was at fault—only to be brilliantly rebutted by the investigators. The affair also served to cement the profession's confidence in a brilliant consultant, D. B. Steinman.

The story begun unfolding in the issue of December 20, 1928:

Lincoln Highway Bascule Drops into River

East half of two-leaf drawspan over Hackensack River between Jersey City and Newark, N.J., fails while being lowered—Cause of failure as yet undetermined.

A remarkable bascule bridge failure occurred on Saturday night, Dec. 15, about 10 o'clock, at the crossing of the Lincoln Highway over the Hackensack River between Newark and Jersey City, N.J. The east leaf of the double-leaf bascule fell into the river channel as it was being lowered from open position. The west leaf, which had not yet started down, was undamaged. The bridge, which carries heavy vehicular traffic and also the main streetcar connection between Jersey City and Newark, was built less than two years ago, being placed in operation in November, 1927. It replaced a swing span in the same location which was wrecked by a steamer in 1922.

As soon as the wreckage of the east leaf is cleared away a timber pile trestle will be constructed to connect with the west leaf at the center of the river channel, thus providing a temporary crossing.

The failure is being investigated by a special committee appointed by W. G. Sloan, state highway engineer. The members of this committee are O. E. Hovey, assistant chief engineer of the American Bridge Company; Prof. George E. Beggs, of Princeton University; and D. B. Steinman, consulting engineer.

The bascule consists of two leaves 98 ft. long from trunnion to outer end and is about 48 ft. wide between trusses. It was designed by the Strauss Bascule Bridge Company, of Chicago, and is of its movable-counterweight type. It was fabricated by the American Bridge Company and erected under supervision of the New Jersey State Highway Department, which now maintains it, by the Stillman-Delehanty-Ferris Company, contractors, of Jersey City.

The counterweight system is in the form of a balanced and articulated parallelogram over the tail section of the leaf. The rear member of this parallelogram consists of a pair of heavy posts (each consisting of two built-up channels made of two angles and three plates and joined by angle lacing, one at each truss carrying its load down to pins in the ends of the tail sections of the trusses). The front member is a pair of fixed posts, one erected over the trunnion support of each truss and just back of the trunnion bearing: from a pin in the top of this fixed post a link extends to the top of the counterweight block, this link being parallel to the line connecting the trunnion pin with the tail pin. The fixed post and the upper link, which under ideal conditions carry no load stresses but serve only as a wind frame, were light members, the post being made of two 12-in. channels spread about 3 ft. at the base and connected by single angle diagonals.

The counterweight consisted of a block of concrete 20 ft. high, 11½ ft. wide and 48 ft. 10 in. long, weighing approximately 750 tons. In its down position (when the leaf was open) the counterweight cleared the roadway about 7 ft. As the leaf was lowered and the parallelogram assumed a more nearly rectangular shape, the counterweight block rose until in its highest position the bottom of the concrete block was 29 ft. above the roadway. It was in some intermediate position between low and high, and was rising, when the failure occurred, the bridge operator estimating that the leaf was between one-quarter and one-half down when he saw the north end of the counterweight move forward from its usual position followed by the crash of the structure.

A view of the wreckage [Figure 5-9] shows both the north and south trunnion pedestals in place. The north trunnion bearing, pin and curved rack all fell into the river, only the back end of the rack being visible above the water. The operating pinion is still in place on the pier and undamaged. The complete counterweight structure over this pedestal is also under water, the front channel of the fixed post breaking off at the top of the pedestal and the rear channel leaving a projection of about 18 in. above the top. The break of the front channel showed bright metal and indicated a clean tension break. The break of the rear channel, on the other hand, showed three different conditions of metal. The north flange and about an inch of adjacent web were covered with old rust, indicating a crack of some age. Next to this rust there was a section of web about 8 in. long upon which the break showed bright and coarse texture, while the remainder of the web and the south flange exhibited a somewhat duller color but a more silky texture. The break occurred through the rivet holes holding the lacing and almost straight across. The rivet hole on the north side was lined with old rust, while that on the south side was of the dull silky finish. In front of the trunnion pedestal the live-load support, a low

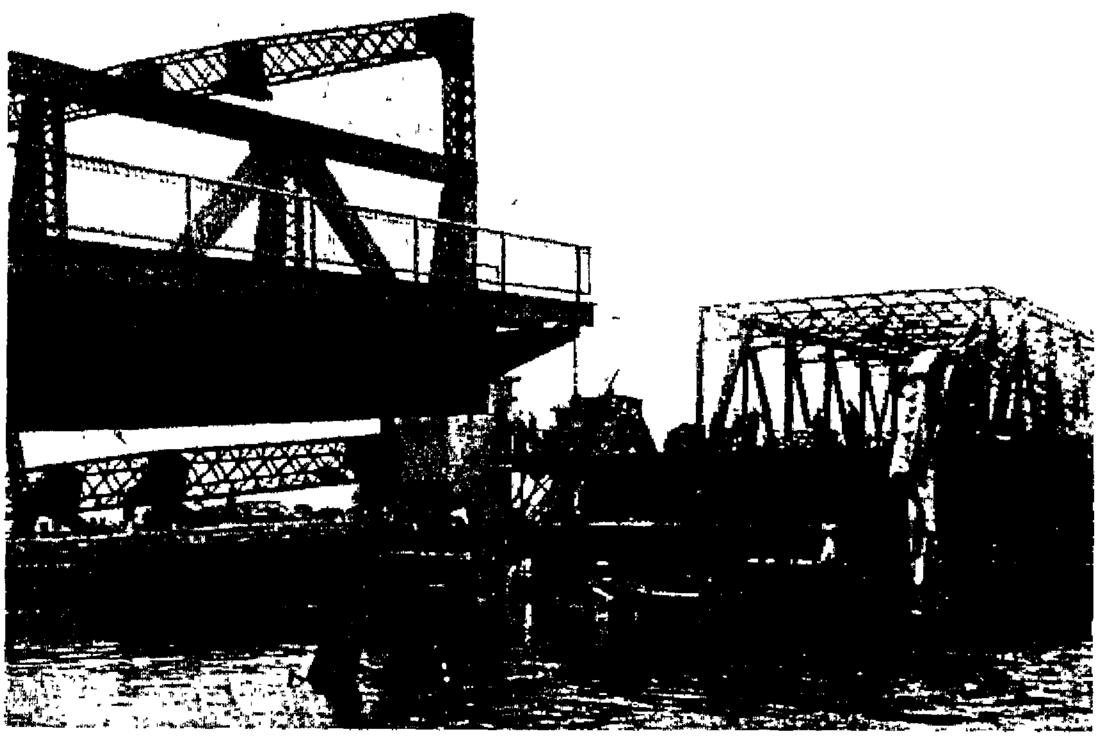

Figure 5-9 The wrecked east leaf of the Hackensack River bascule on the Lincoln Highway in New Jersey. In the left foreground the undamaged west leaf is shown in the closed position. At left (the north side of the wrecked leaf), the tailpiece and operating rack protrude from the waters of the Hackensack River. *(ENR.)*

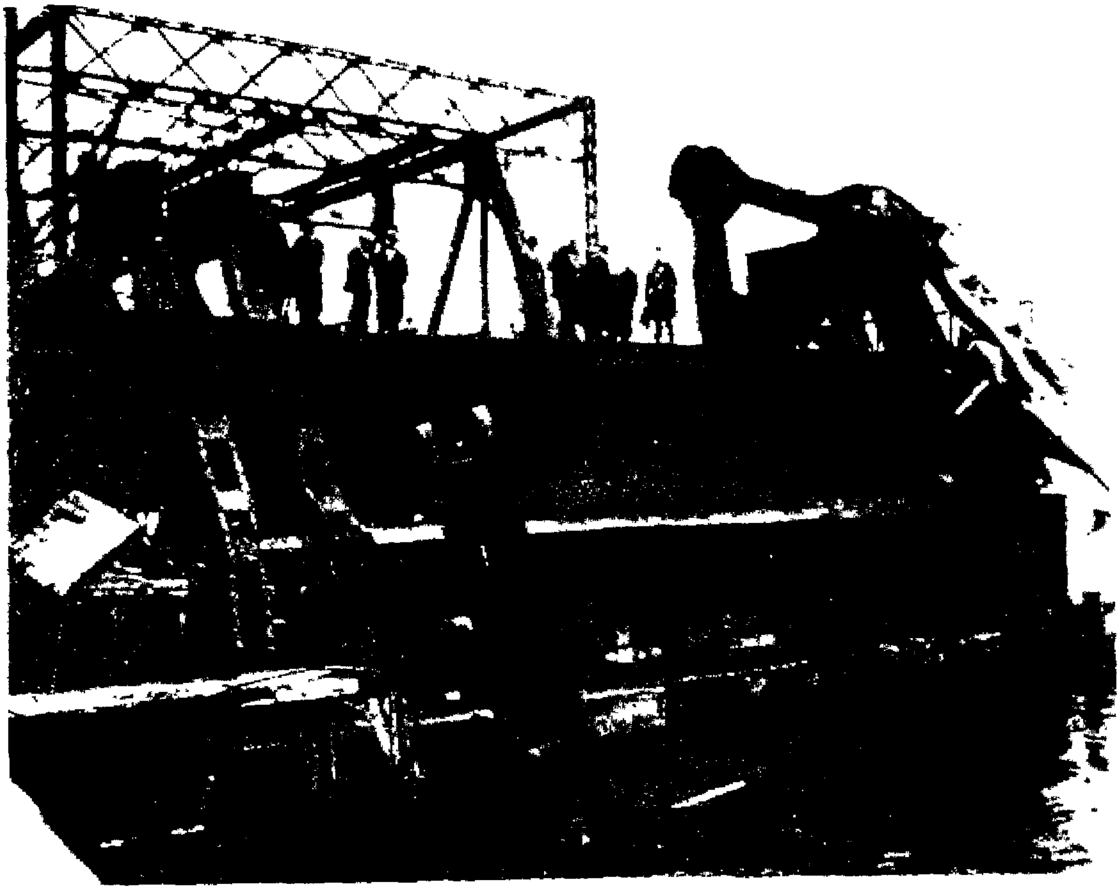

Figure 5-10 The Hackensack River bascule wreckage, looking south. Trunnion pedestals are in place. The north live-load support is smashed, and the counterweight post base can be seen upside down, hanging from the pin on the tailpiece in the foreground. *(ENR.)*

built-steel post on the pier, was distorted by a blow which appeared to have glanced off of the cap plate into the river [Figure 5-10].

The south trunnion pin and bearing did not fall into the river, but the whole assembly was pulled forward off of the trunnion pedestal [Figure 5-10], one of the bearings dropping off and landing on the bridge deck. Examination of the bronze bearing surface showed some evidence of wear. A portion of the top chord and the fist web vertical are still hanging to the south wreckage. The top chord of two built-up webs laced top and bottom and with batten plates on top at the panel points was broken off squarely about 8 ft. from the end. The first vertical was buckled nearly 180 deg. upon itself. The top chord was covered with concrete dust and was badly crushed directly over the buckled web member, as shown in [Figure 5-14]. The batten plate at this point was smashed by a blow that slid off of it toward the inside or north. A large group of rivets which connected the top chord gussets to the pin plates were clearly sheared and the north gusset of this member was cracked for nearly its full length. The fixed post of the parallelogram was bent sharply forward from its connection to the south trunnion pedestal, the angle lacing being buckled upon itself. The front channel of this member was badly buckled and was broken off about 4 ft. from the pedestal. The back channel was draped over the trunnion pin and extended nearly to the water, where it also was broken off. All of the wreckage on the south of the bridge was covered with dust from the counterweight and in position was inclined toward the north. The only wreckage protruding above the water on the south side is the end of a floor beam, evidently the one at the buckled first web vertical. The floor beams were fastened by kneebrace gussets extending practically the full height of the web verticals and all of the rivet holes through which this plate kneebrace of the visible floor beam was connected to the angles on the truss were torn free, apparently by a downward and northward pull. Neither the counterweight nor its two posts are visible.

Levels taken on the piers after the accident showed them to have been unaffected by previous dredging in the channel. According to engineers of the highway department, it is extremely unlikely that any of the pins bound because of lack of lubricant, since maintenance of the structure had been unusually thorough. The main trunnion and the tail pin were lubricated through pressure fittings, while the link pins at the top were of the automatic graphite type. Examination of the pins on the undamaged west leaf showed them to be well lubricated.

With mechanical defects tentatively eliminated, present indications point strongly to a structural failure of some sort. Although lacking the essential facts concerning the conditions of the counterweight posts, several deductions can be made from what is known. The inclination of the south wreckage toward the north, the battered south top chord showing unmistakable evidence that a terrific blow struck it and glanced off toward the north, and the torn condition of the floorbeam visible above the water indicate that the north side of the bridge failed first. These deductions are checked by the statement of the bridge operator, and by the fact that divers have found the north end of the counterweight to be about 18 ft. from the pier, while the south end is very close to the pier. As one looks to the north pier for some evidence of weakness, the cracked back channel of the fixed post commands immediate attention. Failure of this member during operation of the bridge would permit the counterweight to fall forward where it would be ineffective in checking the downward movement of the leaf. The smashed condition of the live-load support is evidence that the leaf fell upon it with great force. This also would tend to throw the counterweight and the tailpiece and operating rack some distance into the river. As stated, divers have located this end of the counterweight about 18 ft. from the pier.

Pending the report of the special commission and the raising and examination of the counterweight posts, the facts suggest that failure of the north fixed post occurred at an early stage, allowing this end of the counterweight to swing forward and thereby cause the north truss to fall.

The special commission appointed to examine the wreckage was at the site eight or nine hours after the collapse, viewing what could be seen, taking strain-gage measurements on the undamaged leaf and making preparations to

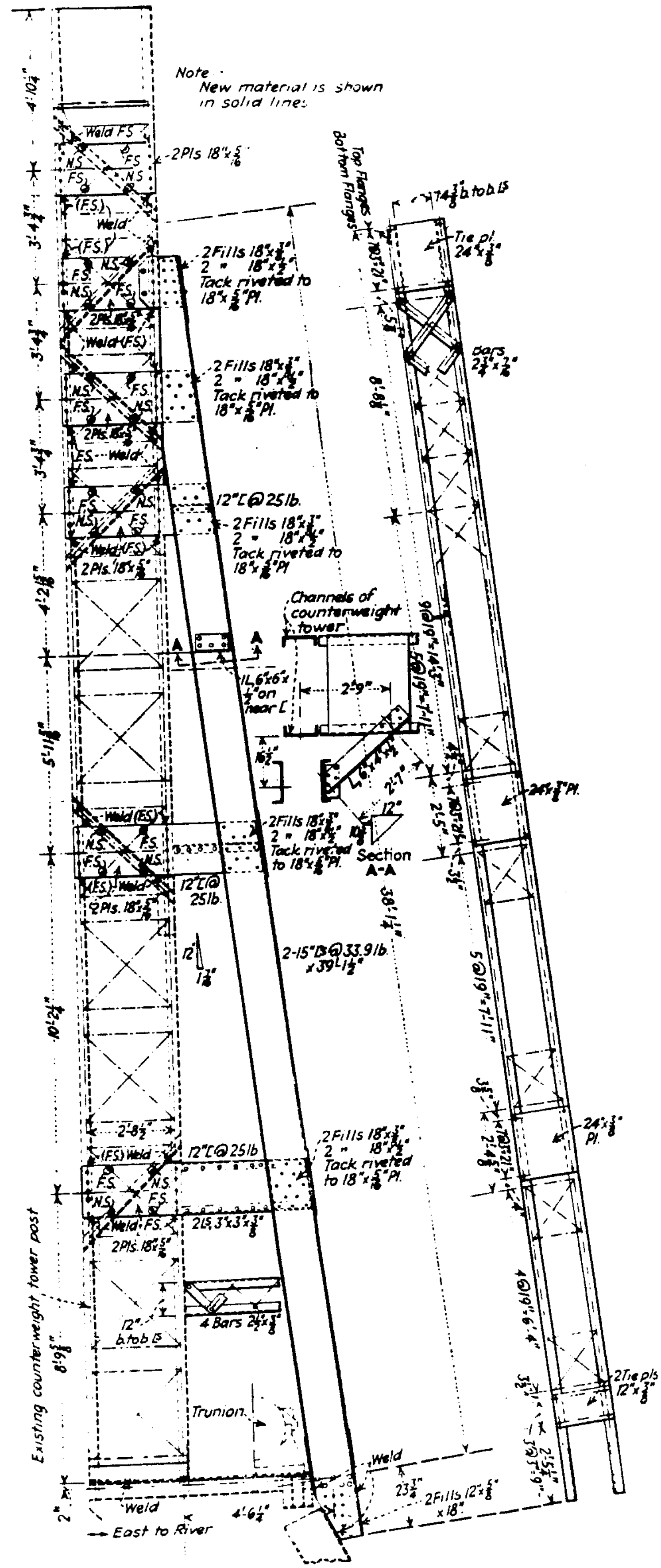

Figure 5-11 A new A-frame bracing system was added to stiffen the north and south legs of the counterweight tower of the Hackensack River bascule. *(ENR.)*

examine the submerged wreckage as soon as it could be cut into moderate-sized pieces which could be lifted by the salvaging derrick. The report of this commission is not yet available.

The three-man commission worked with admirable dispatch, rendering an initial verdict—that the design itself was at fault—only a week later. The final report was carried in the ENR issue of June 6, 1929:

Failure of Hackensack Bascule Bridge Found Due to Inadequate Design

Final report of board of engineers locates initial failure in counterweight tower and reveals existence of greatly excessive stresses in tower legs—Bearing friction not a contributing factor.

Collapse of the east leaf of the Hackensack River bascule bridge on the Lincoln Highway, near Jersey City, N.J., on Dec. 15 last, was caused by excessive stresses in the counterweight tower, according to the final report of the board of investigating engineers. The report, rendered on May 11, is summarized in two main conclusions:

1. The failure of the bridge occurred through the fracture of the north leg of the east counterweight tower, this fracture had been progressive for some time in the past.
2. The cause of the failure was the inadequacy of the design of the counterweight tower to withstand stresses readily calculable from forces known to exist during the normal operation of the bridge. . . .

The present final report is in general agreement with the preliminary report as to location and nature of the failure, but it is based on much more extended study of the actions involved, including stress measurements on the uninjured west leaf, thorough review and test of the electrical and mechanical equipment of this leaf, and analyses of the various kinds of stress that might be set up in the structure and machinery during operation.

THE BRIDGE AND ITS FAILURE

. . . The final report, declaring the breaks in the north counterweight tower to be the point of origin of the collapse, states that examination of the failed structure and of members salvaged from the river revealed no indications of failure originating elsewhere than in the counterweight tower. But it reasserts what was brought out in the preliminary report, namely, that the

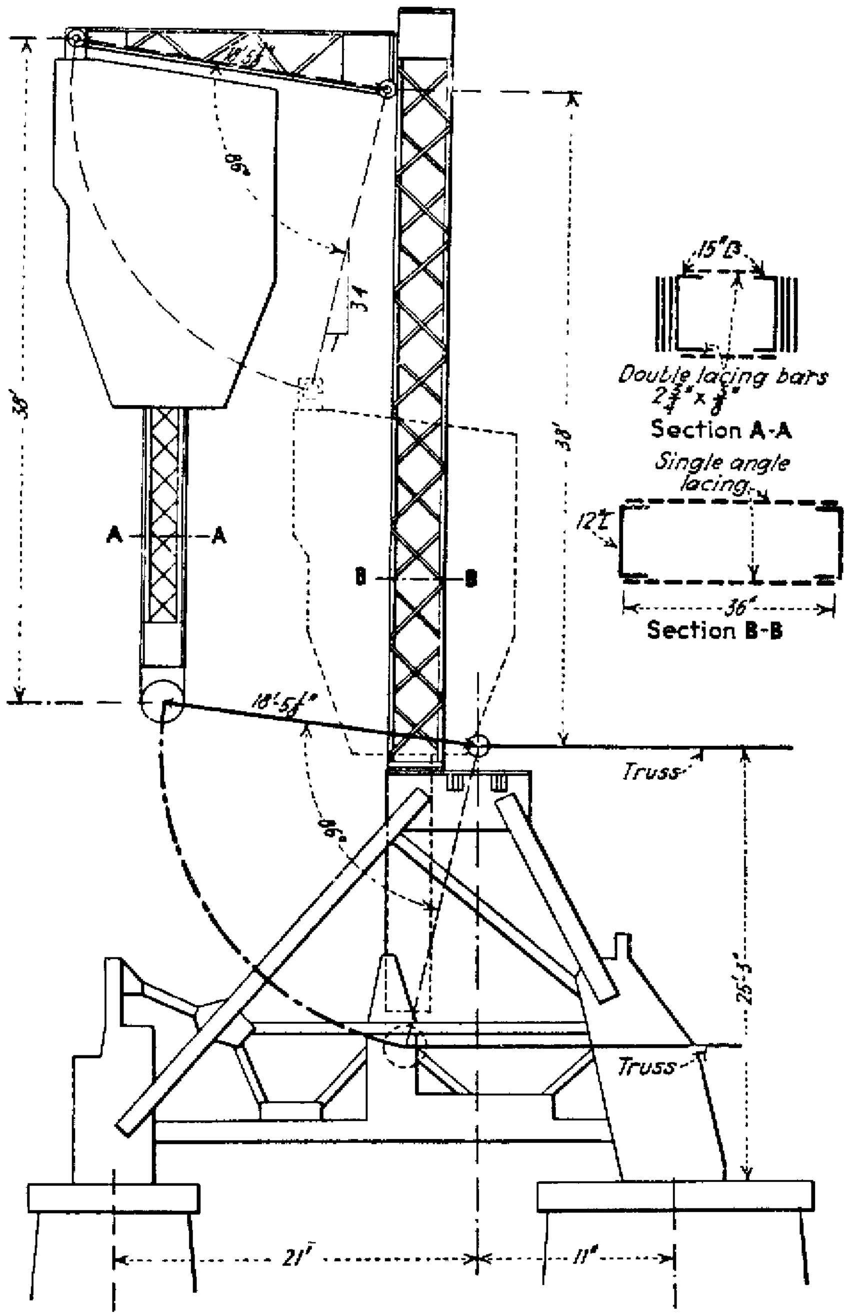

Figure 5-12 Diagram showing the counterweight system of the Hackensack River bascule. The heavy concrete block at top left was supported on slender latticework. *(ENR.)*

counterweight trunnion posts, which were heavy latticed members, were near failure through buckling of their latticing and batten plates. One batten plate of the trunnion posts of the west leaf showed a dishing of 21/32 in., and the lattice bars were buckled as much as 1 in. from their original plane.

Deflections of Counterweight Tower. The board reports that while operating the west leaf of the bascule at about one-third normal operating speed, using the auxiliary gasoline engine, it observed large oscillatory deflections of the top of the tower forward and back, with a period of 4.13 sec. and a maximum amplitude of 0.48 ft., as measured by a large number of transit observations. Some 600 extensometer readings were taken on the channels of the tower near their base during the operation of the leaf, and these indicated a maximum variation of stress of 25,000 lb. per square inch, which proved to be reasonably consistent with the observed deflections.

STRESSES IN THE STRUCTURE

From study of the calculated stresses in the various parts of the bridge the board finds that:

> The critical stresses occur at the base of the counterweight tower (as designed), and these stresses are a maximum when the moving leaf is reaching its highest position at the end of the opening operation. At that position the counterweight is moving in a very nearly horizontal direction, which imposes the maximum possible inertia forces on the counterweight tower due to deceleration of the counterweight as brakes are applied. When the leaf is at or near its highest position, assumed in the design and in our computations at an angle of 86 deg. above the horizontal, any horizontal force acting on the counterweight produces a proportional horizontal component in the top link and a simultaneous vertical component amounting to 3.4 times the horizontal component, due to the obliquity of this link. The connection details are such that this vertical component acts with an eccentricity of 27 in. from the center of the counterweight tower leg.

For the just stated position of the leaf and counterweight, the board computes the separate items of stress action affecting the counterweight tower as follows:

a. The deceleration due to application of the motor brakes (service brakes) at the specified setting of 250 lb.-ft. torque at motor shaft per brake is 0.700 ft./sec.^{2} at the counterweight trunnion, causing a maximum tension of 27,000 lb. per sq.in. of net section at base of counterweight tower channels.

b. The deceleration due to application of the emergency brakes at their specified setting, which was equivalent to 350 lb.ft. at motor shaft for each of the two brakes, is 0.97 ft./sec.^{2}, causing a maximum tension of 37,600.

c. Friction in the counterweight trunnion bearings, when taken at the designer's value of 0.18, produces a tension of 9,000 lb. per sq.in., if this friction acted alone, the deceleration of the counterweight would cause an opposing stress of 5,000 lb. per sq.in., leaving a resultant stress of 4,000. The observed value of the friction, however, was only 0.13, which reduces the resultant stress to 2,900.

d. Wind on the counterweight and tower at 15 lb. per sq,ft., causes a tension of 14,000 lb. per sq.in. and 30-lb. wind a tension of 28,000.

e. Vibration of the tower as already mentioned produces a tension of 14,100 lb. per sq.in. of net section. With the higher speed of the electric motor drive much larger vibrations and hence larger stresses would probably result, but for the purpose of the report this increase was disregarded.