Benton Street Bridge
Carrying Benton Street across the Iowa River
Iowa City
Johnson County, Iowa

HAER No. IA-30

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PHOTOGRAPHS

WRITTEN HISTORICAL AND DESCRIPTIVE DATA

Historic American Engineering Record
Rocky Mountain Regional Office
National Park Service
U.S. Department of the Interior
P.O. Box 25287
Denver, Colorado 80255

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HISTORIC AMERICAN ENGINEERING RECORD

BENTON STREET BRIDGE

Location:	Benton Street over the Iowa River in Iowa City, Iowa
UTM:	15: 621740: 4611780
Quad:	Iowa City
Date of Construction:	1949
Present Owner:	City of Iowa City
Present Use:	Highway bridge, scheduled for demolition in 1989
Significance:	Crossing the Iowa River in the south-central section of Iowa City, the Benton Street Bridge is an all-welded, continuous, five-span, deck, plate-girder highway bridge. Although all-welded bridges appeared in the United States as early as the 1920s, welding was not widely advocated for new bridge construction until immediately after World War II, when a number of state highway departments began preparing all-welded, deck, plate-girder plans. Designed in 1947 and erected in 1949, the Benton Street Bridge introduced all-welded bridge construction to the State of Iowa, and was nationally recognized as one of the most notable examples of the new genre. An early champion of welding, the bridge's designer, Edward (Ned) L. Ashton, had previously engineered several major Mississippi River crossings, and would subsequently be responsible for the world's first welded aluminum highway bridge, completed in Des Moines, Iowa in 1958. Ashton has justifiably been called "the most distinguished bridge engineer in the history of Iowa."
Historians:	Jeffrey A. Hess and Robert Hybben, October 1989

Introduction

Crossing the Iowa River in the south-central section of Iowa City, the Benton Street Bridge is an all-welded, continuous, five-span, deck, plate-girder highway bridge. Although all-welded bridges appeared in the United States as early as the 1920s, welding was not widely advocated for new bridge construction until immediately after World War II, when a number of state highway departments began preparing all-welded, deck, plate-girder plans. Designed in 1947 and erected in 1949, the Benton Street Bridge introduced all-welded bridge construction to the State of Iowa, and was nationally recognized as one of the most notable examples of the new genre. An early champion of welding, the bridge's designer, Edward (Ned) L. Ashton, had previously engineered several major Mississippi River crossings, and would subsequently be responsible for the world's first welded aluminum highway bridge, completed in Des Moines, Iowa in 1958. To appreciate properly the significance of the Benton Street Bridge in the history of bridge engineering and in the life of its designer, it is necessary to understand something of the general development of welding technology and its application to bridge construction.

History of Welding As Applied to Bridge Construction

Welding is the joining of materials through "a localized coalescence of metals or nonmetals produced either by heating the materials to suitable temperature with or without the application of pressure or by application of pressure alone and with or without the use of a filler material."[1] Its two most common styles are fusion and pressure welding. Fusion is the "process of welding metals in the molten or molten and vapor state, without any mechanical pressure or blows." In other words, the two items are melted together. Examples include the electric arc and oxy-acetylene techniques. Pressure welding, on the other hand, involves heating the weldments to malleability and then forcing them together. In its most important subtype, resistance welding, the welded items are softened by their resistance to an electric current. Resistance welding is divided into spot, projection, seam, flash, and butt welding. In the first three, lap joints are used, while in the latter two, the items are pressed together end-to-end in a butt joint.[2]

Welding's origins are unknown, but evidence shows that the Indians, Romans, and Egyptians all used a forge-welding technique during the Classical Age. Generally limited to gold, iron, and lead, it was, by today's standards, a relatively simple process: heating one piece of metal and then hammering it to another.[3] It was not until the nineteenth century, as a result of growing interest in electricity, that welding moved beyond the blacksmith's shop. Key to this was the discovery of the electric arc phenomenon in 1801 by England's Sir Humphrey Davy. Upon separating two contiguous, electrified carbon rods, Davy found that the electric current "jumped" the gap, forming a visible arc of incandescent vapor. Davy's discovery was first successfully applied to illumination. Although electric arc lighting was eventually supplanted for most purposes by incandescent filament lamps, the phenomenon found an enduring and widespread application in welding.[4]

Electric arc welding began in the early 1880s, with the work of France's Auguste De Meritens and his Russian pupil Nikolai N. Benardos. De Meritens found that if he replaced one of Davy's carbon electrodes with a stack of metal plates, the heat from the resultant arc welded the plates together. Later, Benardos added a metal rod as a source of "filler" material. He obtained the first welding patents in England in 1884, and in America in 1887.[5] Two other important methods of joining metal were also developed during this period. In 1886, the American inventor Elihu Thompson introduced resistance welding, and two years later, the first gas welder was built in England, although a practical torch did not appear on the market until about 1900.[6]

The next advance in arc welding occurred in 1889, when researchers in the United States and Germany simultaneously introduced the metallic arc process. In this technique, the carbon welding rod is replaced by a metal one. During welding, the electric current melts some of the rod and "arcs" it over to the weldment where it serves as filler. The next year, the Germans added an electromagnet to their machine to gain better control of the arc. This device, called the "electric blowpipe," received some use in Europe as a means of repairing broken castings.[7] In 1891, the world's first welding plant opened in England and, eleven years later, Baldwin Locomotive Works began the first commercial welding in the United States.[8]

During the early 1900s, the metallic arc process was improved by the introduction of chemically-coated electrodes. Bare metal rods tended to produce weak, oxidized welds and unstable arcs, but by using certain chemicals that vaporized during welding, an oxygen-free atmosphere was temporarily created, thereby eliminating oxidation problems. This is often called the "shielded arc" process. The high cost of such electrodes, however, restricted their use for many years.[9]

In the United States, welding first attracted national attention during World War I, when it was used to rehabilitate a large number of confiscated German ships, rapidly rendering them seaworthy for the Allied cause. At the same time, the government organized a welding committee to standardize techniques, helping to dispel several myths about the process while making it safer for general use. Although the federally funded committee disbanded at the end of the war, several of its members regrouped as the American Welding Society, whose Journal was to be instrumental in promoting welding technology.[10] Overall, however, welding was still considered "little more than an artisan's process for repairing broken castings and other metal parts." The technology faced two main obstacles to widespread acceptance, especially in new construction. First, there were few qualified welders and little proper equipment. Second, and perhaps most basic, welding had not yet passed the test of time. The few welded structures in existence simply had not been around for very long, so little was known of welding's long-term reliability.[11]

For those engineers who explored the matter, welding seemed to offer several advantages over riveting, not only in repair work, but in new construction as well. Welding did not weaken metal members with rivet holes, nor did it necessarily require the various connecting pieces such as angle sections and gusset plates which were a traditional part of built-up, riveted construction. As the noted welding advocate, J. F. Lincoln of Lincoln Electric Company of Cleveland, pointed out,"any structure which is made of riveted steel and depends on the strength of the joint for its efficiency must have from 30 to 100 per cent more steel in it than would be necessary were an arc-welded joint used. What this would amount to in the aggregate is a stupendous sum."[12] In addition to savings in material, welding also appeared to require less labor than riveting, and there was no question that it was far quieter, an important consideration in urban construction projects.

For the most part, test results from the 1920s supported welding's structural soundness. The American Bridge Company, the Carnegie Institute of Technology, and The American Architect all reported positive findings. Still, in a stress test sponsored by The Mechanical Engineer, unexpected breaks in weldment did occur, highlighting the importance of inspection. Although there were advocates of visual inspection, it was generally conceded that defects might occur beneath the surface of the weld drops. The most common field test of the period was to tap the welded item with a small hammer. If the weld were properly made, the ringing sounded identical to that of a solid piece of the same material. Less subjective inspection procedures included x-ray analysis and electric-resistance measurement of the welds. While such tests were more reliable, they were also more cumbersome and expensive.[13]

In contrast to industry, where welding found widespread acceptance in the manufacture of cars, pipelines, airplanes, and gas holders during the 1920s, the building professions treated the new technology with a great deal of caution. Beginning with England's solitary Olympia Stables in 1920, welded buildings numbered, nine years later, only 70 structures world-wide. In the United States, the most prominent examples were the 12-story, 550-ton Homestead Hotel in Hot Springs, Virginia, and General Electric's 987-ton West Philadelphia works. By the end of the decade, however, several American cities --most notably Atlantic City, Detroit, Providence, Youngstown, and Berkeley -- had modified their building codes to allow welded construction. And several more, including New York, Chicago, and Dallas, were considering the move. At the same time, there was an effort to include welding in the engineering curriculum of American universities and technical schools.[14]

The 1920s also saw the first applications of welding to bridge repair and construction. In 1920, a gas welder repaired the lower chord of a 60-foot, Pratt pony truss on an unidentified highway in Chester County, Pennsylvania. The county engineer had heard of welding's successful use in the automobile industry, and decided to give it a try. It held very well. Seven years later, arc welding made its debut in bridge repair on the Chicago Great Western's three-span, through truss over the Missouri River at Leavenworth, Kansas. New members were welded into the hip verticals, and cover plates were added to flanges of floor beams and stringers -- with no interruption to train service. Such repairs were notably cheaper than replacing an entire bridge.[15]

The first welded bridge ever built appears to be a footbridge, erected in 1921, between two buildings of the General Electric plant in Schenectady, New York. The first of any significance, though, was a 53-foot, arc-welded, twin-girder railroad bridge built by the Westinghouse Electric and Manufacturing Company over Thompsons Run in Turtle Creek, Pennsylvania. Completed on November 5, 1927, the bridge carried rail traffic between the company's Linhart Works and East Pittsburgh Works. Interested in promoting virtually any new technology allied with electricity, Westinghouse was a leading proponent of electric-arc welding. The company had showcased the technology in its all-welded factory completed in Sharon, Pennsylvania in 1926, and it considered the Turtle Creek Bridge to be "an excellent example of the confidence of our company in arc welded construction."[16]

In 1928, Westinghouse also sponsored the nation's second all-welded railway bridge, as a means of providing spur-track access to its plant in Chicopee Falls, Massachusetts. This, too, was an arc-welded affair. To develop comparative cost data on welding and riveting, Westinghouse based the 134-foot, single-span structure directly on the design of a riveted, Warren, through truss originally intended for the site, "the only interference with true comparison [being] the substitution of rolled shapes for the built-up sections shown in the riveted design." The welded bridge even employed conventional gusset plates for panel connections. At the completion of construction, Westinghouse claimed a "substantial" savings in cost: "The welded structure used about two-thirds of the quantity of steel required by the riveted equivalent, due to saving most of the connection materials, to avoidance of holes in tension members and to continuity of floor stringers."[17]

Arc welding also produced the world's first all-welded highway bridge, completed over the Sludwia River, near Lowicz, Poland in 1929. Like the Chicopee Falls Bridge, the Polish structure -- a 89-foot, Warren pony truss of camelback configuration -- was patterned after a conventional, riveted truss to permit cost comparison. According to the designing engineer, Stefan Bryla, welding saved about 21 percent in materials, although total project cost proved roughly equivalent to riveted construction --a result of having to include the expense of the welding equipment in the final bookkeeping. "In the course of time," Bryla affirmed, "the proportion of costs of the two types of bridges will be changed in favor of the arc-welded design."[18]

Although welding became a common repair technique for bridges during the 1930s, it charted only modest advances in new bridge construction, especially in the United States. By the end of the decade, there were approximately 1,000 welded bridges in the world, and almost all were in Europe. Recognizing that welding called into question certain assumptions based on riveting about the "shape and arrangement of joints," European engineers quickly moved beyond the truss-inspired design of Bryla's Sludwia River Bridge to investigate forms specifically suited for the new technology. The Vierendeel truss, named for the Belgian engineering professor who created it, was perhaps the most striking example. Featuring a tied-arch design "in which the verticals have a stiffness comparable with the chord," the Vierendeel truss reputedly placed joint welds "at the points carrying the smallest stress and where the distribution of stress could be calculated without ambiguity." Making its first appearance in the partially welded, 68-meter Lannaye Bridge in Belgium in 1931, the Vierendeel design was soon applied to several all-welded, Belgian bridges. The success of its basic design principles also encouraged European experimentation with similarly conceived, all-welded, rigid-frame construction in both buildings and bridges.[19]

During the 1930s, European engineers probably made their greatest contributions to welded bridge technology in the area of "solid web," or girder, design. Much of the work was done in Germany, which built over 300 all-welded, railroad girder bridges during the decade. When German State Railway engineers began designing welded plate-girder bridges in 1929, they originally used the conventional "web plate, flange plates and angles as employed in riveted plate girders." Deciding almost immediately that angle-section connectors were an anachronism in welded construction, they experimented with welding flange plates directly to the web. This innovation led to the development of new flange and web sections specially designed to alleviate stress concentrations in the welded plate-girder joints. Although the new components were available as standard rolled sections in Europe, they were slow in coming to the United States. Indeed, the 1949 Benton Street Bridge in Iowa City was apparently one of the first structures on the North American continent to utilize them.[20]

A handful of "European-style," all-welded, rigid-frame highway bridges were constructed in the United States prior to World War II, but, for the most part, American engineers ignored European experimentation with welded bridges. When Americans designed welded bridges at all, they generally followed "the beaten path of riveted construction, the thought being that welding was replacing riveting." The most publicized American welded bridge during the 1930s was notable primarily for its 400-foot length, which placed it first in the country's longest, welded bridge category. In most other respects, it was a highly conventional structure. Completed over Rancocos Creek in Burlington County, New Jersey, the highway structure comprised two fixed spans and one swing span, all of the Warren pony truss variety. By European standards, the bridge was not even interesting for its length. In 1932, German engineers had built an all-welded, 13-span, plate-girder highway bridge near Dresden that was two-and-one-half times as long.[21]

The Rancocos Creek Bridge was fabricated by the shielded-arc method, which was generally considered to produce the soundest welds. As measured by the sale of coated electrodes, the shielded arc method had advanced quite rapidly in the United States, increasing its share of the metallic electrode market from about 15 percent in 1927 to more than 75 percent in 1932. Indeed, the method became so dominant that, by 1937, it was "the process generally referred to when the word 'welding' is used." Confidence in shielded arc welding is one major reason why, by the end of the decade, New

York, Chicago, Boston, and San Francisco, along with well over 100 other American cities, had approved welded construction in their municipal building codes.[22]

Given welding's acceptance by the American engineering profession in general, the scarcity of all-welded bridges in the country before World War II should not be attributed to any lack of faith in the technology by bridge engineers in particular. Most bridge construction was controlled by state highway departments, which, typically, had spent the previous two decades preparing and promoting standardized plans for reinforced concrete and riveted steel bridges. These agencies had an understandable vested interest in maintaining their old designs, especially since few had any engineers on staff who were familiar enough with the new technology actually to design an all-welded bridge. When the Nebraska highway authority, for example, embarked on its first all-welded bridge project in 1934, it sent "a member of the designing force to a famous school of welding located in Cleveland where he completed with credit a course in both design and operation of welding details, operation and fabrication."[23]

If World War I ushered welding onto the American stage, World War II made the bit player a star. Everything that could be welded was, including 90 percent of American ships under construction. Despite popular images of Rosey the Riveter, the global conflict was essentially a "war of weldments." As one writer explained, "This war has been one of mechanization and movement. Lightness coupled with strength has been of paramount importance in weapons as well as transport equipment. It was to attain these features that welding has been so extensively used." By the early 1940s, welding could be described as "the most important method of joining steels."[24] Curiously, welding suffered a temporary setback immediately following the war, triggered, it seems, by the public's general weariness with "military" practices. But post-war shortages of construction materials encouraged the realistic appraisal "that welding had come to stay" as an appropriate peacetime technology. At the end of the 1940s, an informed observer noted that "practically all structural shops are presently equipped for services in welded construction. . . . In contrast, it is becoming increasingly more difficult to obtain facilities, equipment and skilled labor for riveted construction."[25]

All-welded bridge construction, primarily of the deck, plate-girder variety, made particularly strong gains in the United States in the immediate post-war period. Between 1946 and 1948, the State of Connecticut alone completed eight all-welded bridges, and placed another eight under contract or design. During these same years, the states of Kansas and Texas prepared plans and took bids for all-welded girder bridges, while the states of Ohio and New York apparently contemplated similar action. In Iowa, on the other hand, all-welded bridge construction was inaugurated by city rather than state authority, as is evidenced by the municipally-sponsored, plate-girder Benton Street Bridge, completed in Iowa City in 1949. These activities were encouraged by the federal Public Roads Administration, which, having studied the issue during the war years, had recently announced its approval of federal funding for welded bridge construction if "the latest methods of design and workmanship are employed." With an eye toward the possibility of future roadway widening, the federal agency also strongly promoted deck-style construction, decreeing that "new bridges should be designed to permit traffic to pass over rather than through the structure." It is therefore not surprising that when the federal government funded its first all-welded state highway bridge in 1946, it was a deck-girder structure.[26]

Whether the impetus for welded bridges originated on the federal, state, or local level, the thinking behind it seems to have been essentially the same. As a bridge engineer with the Texas Highway Department explained:

> Immediately following the war, we were of the opinion that welding had come of age. We believed that the great strides made during the war in its general use and the development of new techniques -- together with the many new facilities and the greatly increased number of qualified welders -- indicated that postwar construction would adopt welding as being more economical than riveting. Also, there were good chances that welding might prove superior to riveting from a structural standpoint. In an effort to keep abreast of developments . . ., we revised plans and specifications to permit welding as an alternate to riveting in several instance of standard construction, and we began the development of plans for particular welded jobs.[27]

The initial welded bridge projects of the post-war period were soon followed by a host of others throughout the country.[28] In 1953, the engineering press reported that "welded girder bridges are now generally used by most of our state highway departments" -- a clear sign that "the welded girder has advanced to a place of competitive importance in highway bridge construction." Despite chronic steel shortages in the 1950s -- a result of the Korean War, a nation-wide building boom, and a steel strike -- the all-welded highway girder increased in popularity throughout the decade. To a certain extent, the scarcity of steel actually promoted the use of welding. As the California State Highway Department explained, "Our staff has had difficulty for so many years in obtaining rolled shapes that we have widely adopted welding. Plates being more readily available, we have been able to design efficient sections for our needs."[29]

When state highway departments did explore alternate forms of bridge construction during the 1950s, they generally turned to prestressed concrete, especially for short spans. Although the country's first prestressed highway bridge only dated from 1949, there were more than 250 examples nationwide by 1958, the majority containing spans under 50 feet. The development of prestressed concrete bridge designs, however, was hampered by the fact that many states "have not as yet developed a flourishing market for commercial casting yards." When steel supply loosened toward the end of the decade, some states, such as South Dakota, cancelled their prestressed concrete programs and "restored steel as a purely economic choice." As a state bridge engineer for South Dakota explained in 1957, "with ready supply of plate . . . now assured, plate girders are again highly competitive."[30]

History and Significance of the Benton Street Bridge

Dry crossings of the Iowa River at Iowa City date back to Benjamin Miller's ferry service of 1839, and the Folsom Pontoon Bridge of 1853. Benton Street, however, was not bridged until about 1905, when the city erected a double-span, Parker, through truss with timber-trestle approaches to accommodate horse-drawn and pedestrian traffic. Known as the Ryerson Bridge, after the nearby Ryerson Mill, the structure was the city's southernmost river crossing, located in a relatively undeveloped area.[31] Perhaps because of its remote location, it received only minimal maintenance, and suffered accordingly. When Edward (Ned) L. Ashton (1903-1985) joined the engineering faculty at State University of Iowa in Iowa City in 1943, he found the bridge in distressed condition. As he later explained:

> When I came to Iowa in 1943 to teach. . . , I used the Bridge as an example of what to do and what not to do in bridge design. . . .Probably the . . . most serious thing that happened to the old bridge is the fact that birds nested in the recesses of the fancy latticed portals and caused . . . rusting away of portions of the main posts that hold up the whole span . . . . Some of the members have also been hit by traffic and, of two member sections, only one of them is carrying all of the load.[32]

In 1944, the City Council placed restrictions on automobile traffic over the bridge, eventually triggering public outcry for appropriate repair or replacement of the structure. The first petition to that effect was submitted in June 1946, prompting the council to obtain a repair estimate from the Des Moines Pittsburgh Bridge and Iron Company. In October of the same year, this company reported that the bridge was no longer safe for use and would require at least $30,000 for rehabilitation. The council responded by closing the bridge to all vehicular traffic and appointing a committee to investigate building a new structure. Although the council received another bridge-replacement petition in January 1947, it postponed action until late May, when it commissioned local engineer Ned Ashton to prepare preliminary plans and cost estimates for a new bridge. As Ashton later told the story in a public lecture, the council had originally considered hiring a consulting engineer from Des Moines, until "in some fashion, [they] found out that I also used to be a bridge engineer and designer . . . and came to see if I would be interested and able to do this work for them to keep the work in Iowa City."[33]

Ashton's self-effacing description of himself as a bridge designer was a kindness to the city politicians who had almost passed him by. When he addressed his peers in the engineering profession, as he did at a section meeting of the American Society of Civil Engineers in Davenport in 1944, he was more direct about his qualifications: "Since graduation in 1926 from the [State] University of Iowa, I have spent most of my time designing medium and long span bridges . . . ." After leaving the university with a master's degree in both structural and hydraulic engineering, Ashton found employment with the prestigious bridge engineering firm of Harrington, Howard and Ash in Kansas City, Missouri. His first major responsibility was as resident engineer of the combined highway and railroad bridge over the Mississippi River at Vicksburg, Mississippi. It was during this project that Ashton first began to consider the advantages of welding in bridge construction. As he recalled for an interviewer many years later, the stimulus was an offhand comment by the firm's senior partner, Louis R. Ash: "If we could only extend one plate through another -- welded together. How much better it would be."[34]

In 1929, Ashton moved to St. Louis to become an associate engineer in the firm of James A. Hooke. Although only 26-years-old, he was put in charge of the $20 million St. Louis Electric Terminal Railway Company project, which entailed the demolition of nine city blocks and the construction of a six-track subway. When this project terminated four years later, Ashton joined the United States Bureau of Reclamation in Denver where he worked on large concrete dams," designing such appertenances [sic] as the intake tower bridges, the hoist houses and the arch bridge over the Arizona Spillway at Boulder Dam and similar details on Grand Coulee and many others." Finally, in 1935, he returned "exclusively to bridge building," rejoining his original Kansas City firm, since renamed Howard, Needles, Tammen and Bergendoff (HNTB). During the next seven years, in which he rose to the position of chief designer, Ashton worked on approximately 20 bridges, including the main spans of such major Mississippi River crossings as the five-span, tied-arch, Rock Island Centennial Bridge at Rock Island, Illinois; the triple-span, continuous, through-truss Ben Humphrey's Bridge below Greensville, Mississippi; and the triple-span, continuous, through-truss Julien Dubuque Bridge at Dubuque, Iowa, with its highly unusual, through, tied-arch truss in the center span.[35]

In his work on these major bridges, as well as on many smaller ones, Ashton emphasized the reduction of what he considered unnecessary "detailing" -- a term generally used to describe the "smaller parts" of truss construction.[36] Ashton estimated that in conventional, heavy, highway trusses of the 1930s "the lacing bars and gusset plates constitute approximately 35% of the weight . . . and represent a considerable waste of materials." By fabricating truss bracing and chords out of solid web sections, instead of the customary channel and angle sections, he found that he could "eliminate all lacing bars" and cut the weight of detailing by almost half, effecting a considerable savings. The reduction of detailing was also one of the potential advantages of welded construction, which may explain why Ashton "used the welding process in bridge construction whenever he was free to do so," employing it, for example, on the open, steel, roadway curbing of the continuous-span Manchester Avenue over the Blue River in Kansas City, Missouri. During his stay with HNTB, Ashton also authored three papers on welding. Although no copies appear to have survived, the titles signal his areas of interest: "Arc Welded Steel Plate Floors Applied to Bridges and Viaducts," "Arc Welding in Design Manufacture and Construction," and most significantly -- "The All Welded Steel Bridge of Tomorrow."[37]

Ashton completed his preliminary design and cost estimates for the Benton Street Bridge in mid-June 1947. This initial effort gave no indication that the completed structure would indeed be Ashton's "all welded steel bridge of tomorrow." As published in the local press, the design called for a nine-span, deck-girder bridge, with a central, 348-foot-long, river-channel section consisting of three continuous, variable-depth steel girders. The bridge's overall length was calculated at 572 feet, its width at 33 feet (24-foot roadway plus flanking sidewalks), and its cost at $215,000. There is no mention of welding by the contemporary press, or by Ashton in his own description of the structure. To a certain extent, the design reflected Ashton's work on the Grand Avenue Viaduct in Sioux City, Iowa -- a HNTB project, which, according to Ashton, was responsible for "introducing and proving the economy and beauty of long span variable depth plate girder construction." The gently curved arch of the variable-depth girder not only presented an aesthetically pleasing profile, but also efficiently distributed the metal according to the actual lines of stress. The City Council accepted Ashton's preliminary study, awarded him a fee of $250, and commissioned him to prepare final plans and specifications for "a fee not to exceed five (5%) of the contract price of such bridge."[38]

For his preliminary work, Ashton had assumed that the new Benton Street Bridge would occupy the site of the old crossing. But after considerable debate, the City Council decided to swing the bridge diagonally across the river, directly linking East and West Benton Streets. Although the new route increased the estimated construction cost by about ten percent, it seemed to provide better traffic access to the state highway system. With site selection concluded, Ashton began final design work in August and delivered the completed plan in November. This plan retained the basic principle of continuous-span, variable-depth, plate-girder construction on concrete substructure, but it reduced the original nine-span layout to a symmetrical, five-span configuration, measuring approximately 480 feet in length between abutments, with span ratios of 78-100-120-100-78. The bridge's overall width remained the same. From the perspective of the local press, the most remarkable feature of the final design was that its estimated cost was about $30,000 less than the preliminary design, despite the route change. From an engineering standpoint, however, the design was most remarkable because it specified all-welded construction, the first example in Iowa, and a rarity even for the nation. At that time, there were probably less than 100 all-welded bridges of any type in the United States.[39]

Shortly after the completion of the Benton Street Bridge in the summer of 1949, Ashton would declare that "the decision to adopt a welded design was made in the beginning since it is the best modern method of joining metals and it offers a considerable economy of material." Since his preliminary design did not call for all-welded construction, Ashton was presumably referring to his work on the final design. The beginning of this project coincided with his attendance at a conference at Dartmouth College in August 1947, where he served "as one of twenty judges charged with the selection of the 400 or so winners of . . . cash prizes awarded in the latest $200,000 James F. Lincoln Arc Welding Foundation Award for [the] Progress [in Welding] Program." If his jury duty for the foundation did not actually inspire him to choose an all-welded design for the bridge, it certainly strengthened his resolve. As he later recalled, "On this assignment, I read papers about six hours a day and designed the Benton Street Bridge in between and obtained many useful and valuable ideas."[40]

Given his familiarity with the field, Ashton probably knew that several state highway departments were simultaneously preparing all-welded, plate-girder bridge designs. He did not attempt, however, to justify his own position in the vanguard by citing developments in other regions. Indeed, he did not allude to the innovative nature of his work at all during the early stages of the

Benton Street Bridge project. He apparently discussed the matter with his good friend Fred Gartzke, the city engineer, who advised against giving the impression that the bridge was in any way experimental. The new Benton Street Bridge was to be funded by city bonds, and, as Ashton's son-in-law Marshall McKusick later observed, the two men saw "no reason to make the bond holders nervous."[41]

After the Benton Street Bridge was up and open, Ashton was more than willing to discuss its innovative features. As he explained for Welding Journal, the structure was most notable simply for the fact that it was an all-welded, plate-girder design:

> Probably the greatest advancement in design in this structure is in the design of the girders themselves. Not only were the girders fabricated of three plate sections by [shielded-arc, shop] welding, but also all of the field connections and the main girder splices were made both in the shop and in the field as 100% penetration butt welds.

Ashton also emphasized that his welded girders introduced the European practice of employing specially shaped flange plate sections designed to reduce stress in the weldments:

> The author also wishes to call the attention of the American mills to the special flange plate sections that are rolled with a small projection of web attached for welding flange to web sections that have long been standard rolling mill sections in European practice. These sections were designed to help alleviate the shrinkage and warping problems in the flanges, and have been in use for a long time in foreign welded girder construction but they have not yet been made available from our own American mills.

Despite his use of the special sections, several of the girders buckled during shop fabrication, requiring careful heat treating to straighten them out. Ashton did not attribute this problem to the flange sections, but rather to his failure to specify a thicker web section for "more inherent stability."[42]

Since all-welded bridge construction was such an infrequent occurrence in the United States, few contractors had any experience in the area and, consequently, few were willing to take on the risk of the Benton Street Bridge project. When the city issued its call for bids in the spring of 1947, it received only two proposals: an offer of $257,814 from Jensen Construction Company of Des Moines, and one nearly 70% higher from L. Morris Mitchell, Inc. of New York. The city selected Jensen, a family-owned-and-operated company that had specialized in bridge building since its founding in

1912. Among the company's recent projects was a girder-bridge job involving some degree of welding. For its major subcontractors, Jensen selected the American Bridge Company (Gary Plant) for fabrication, and the Teleweld Corporation of Chicago for field welding. Jensen's handling of the contract would win high praise from Ashton:

> Their work on [the Benton Street Bridge] project was not only superior in quality but it was handled so expeditiously that there was not a single extra work order on the entire project. It was not only completed on time in full accordance with all requirements of the plans and specifications but also at a cost that was $6,500 less to the City than the amount of their original contract agreement. Of all the contractors I have ever had the privilege of working with, I regard the Jensen's as being the ones who were gentlemen of the highest character.[43]

Construction of the Benton Street Bridge commenced on October 2, 1948. Benefiting from unusually low water levels for Autumn, the concrete crew poured the entire substructure by the end of December. The project, however, significantly slowed during January and February, when the shop experienced difficulties both in procuring the steel and fabricating the girders. In late February, the first carloads of superstructure steel finally arrived at the bridge site, but floodwaters almost immediately put construction on hold for four weeks. This time was partly spent dismantling the old truss bridge. The erection of the superstructure resumed on April 4, and the last girder was put in place a week later, when final "closure was made . . . by jacking Span 5 horizontally from the west abutment." The ensuing installation of floor beams, stringers, cantilevered sidewalk brackets, railing sections, and light poles was all "more or less conventional." On May 12, the contractor poured the first concrete for the deck, and the bridge was ready for traffic eight weeks later, although it was not officially dedicated until July 28, 1949.[44]

The American engineering press was quick to recognize the significance of the Benton Street Bridge. In March 1949, while the superstructure was still under construction, Engineering News-Record hailed the bridge as one of three current projects representing "relatively new concepts of bridge design in the United States." As the article went on to explain, "More welding is being used on bridges, both in the field and in the shop, but fully-welded bridges are the exception." During the next few years, others writers would also point to the Benton Street Bridge as a notable example of the welded-bridge genre.[45]

The bridge's appeal, however, was not simply that it was an all-welded structure. Its construction also embodied several other important design trends of the post-war period. In 1947, Raymond Archibald, head of the bridge division of the Public Roads Administration, outlined the "best practices" in "modern bridge design" for future federally funded highway projects. In addition to advocating welded construction, he recommended deck-girder design, as a means of facilitating future roadway widening; "good architectural treatment," which was generally interpreted in terms of clean-lined, modernist aesthetics; and continuous-span construction, which "usually is more economical as it requires less material [and] has the further advantage of eliminating expansion joints -- the bane of the maintenance engineer -- and providing a smoother roadbed, thus contributing to the comforts of driving."[46] The Benton Street Bridge displayed all these characteristics, which is perhaps one reason why it was singled out for recognition by Engineering News-Record, instead of any of the half-dozen, multi-simple-span, all-welded girder bridges constructed by the Connecticut State Highway Commission during the same period. It was possible to praise the Benton Street Bridge for its all-welded construction. without apologizing for any other aspects of its design. Ashton himself seems to have understood this when he characterized his creation as a "modern" highway bridge."[47]

Ashton strongly promoted the Benton Street Bridge, preparing a paper on its design for Engineering News-Record, as well as a somewhat longer treatise for publication by the Lincoln Electric Company of Cleveland, a distributer of welding equipment. In return, the bridge promoted Ashton's consulting career, firmly establishing him as an authority on welded construction and garnering him additional bridge commissions. After Benton Street, his most notable bridge project was the design of the world's first all-welded, aluminum, highway bridge -- a continuous, four-span, deck-girder structure built in Des Moines by the Iowa State Highway Commission in 1958, a year after Ashton had officially retired from teaching. When one considers this structure in the light of Ashton's Benton Street Bridge and his earlier work on Mississippi River bridges for HNTB, there is good reason to concur with the assessment of Iowa bridge historian James C. Hippen that Ashton "in all likelihood was the most distinguished bridge engineer in the history of Iowa." Ashton himself hoped that his crowning achievement would be "to design the first long span aluminum bridge across the Mississippi River for Iowa," but the opportunity did not present itself.[48]

Ashton remained active in the bridge field throughout the 1960s, receiving national attention for his reconditioning of filled-spandrel concrete arches in Cedar Rapids, Iowa. During this period, he also pursued a long-standing interest in the structural engineering of radio telescopes, designing a major instrument built by the National Observatory in 1965 at Green Bank, West Virginia. As a result of failing health, Ashton gradually closed down his consulting practice during the 1970s. He died in Iowa City in 1985.[49]

The Benton Street Bridge outlived its designer by only a few years. In 1985, the City of Iowa City commissioned the local engineering firm of NNW, Inc. to study the feasibility of widening the two-lane bridge into four lanes. As part of their structural survey, the consultants noted that the bridge incorporated certain elements of design that might lead to catastrophic failure by virtue of metal fatigue:

> At the time of the design of this bridge, little was known about fatigue in bridges. While the 1948 design was in accordance with the then current specifications, it should be noted that the specifications did not include fatigue constraints. In fact, it was not until 1971 that fatigue design was included in A.A.S.H.T.O. (American Association of State Highway and Transportation Officials) specifications. . . . The detail which first attracts attention is [that] intermediate stiffeners are welded to the flange of the plate girder. This particular detail is no longer seen on welded plate girder bridges because it has been founded to induce fatigue problems into girders at rather low stress ranges . . . .If a fatigue failure should occur, would the effects on the structure be a serious matter? The answer is a simple yes.[50]

On the basis of these initial findings, the Benton Street Bridge was targeted for replacement. This decision was forcefully opposed by local historic preservationists who argued that the bridge was of sufficient historical significance to warrant more detailed study of its condition and possible retention. Subsequent investigation, however, supported the probability that the bridge was a fatigue-sensitive structure without realistic potential for rehabilitation and preservation. In consideration of the bridge's historical significance, the City of Iowa City agreed that, prior to demolition, it would document the bridge according to standards set down by the National Park Service, "including a written narrative placing the structure in engineering and historical perspective." The present study is intended to fulfill that obligation.[51]

Notes

1. W. L. Warner, "Welding Nomenclature and Definitions," Welding Journal 28 (May 1949): 428.

2. "Terms Used in Electric Welding Practice," Machinery 42 (October 1935): 110-111; H. Thomasson, "Recent Advances in Electric Welding," Engineering Journal 20 (January, 1947): 12.

3. A. J. Moses, "History and Development in the Art of Welding Ferrous Metals, Part I," Combustion 3 (February 1932): 38-40; P. W. Fassler, "Twenty Five Years of Electric Welding," Journal of the American Welding Society 10 (May 1931): 29; E. J. Blandford, "Welding from Antiquity to 1929," Canadian Welder 40 (December 26, 1929): 322. Surviving examples of early welding include small boxes, tools, and weapons. A wrought-iron column forge-welded in Delhi, India, during the fourth century A.D. apparently still stands today; see Howard B. Cary, Modern Welding Technology (Englewood Cliffs, New Jersey, 1979), p. 11.

4. A. J. Moses, "History and Development in the Art of Welding Ferrous Metals, Part III," Combustion 3 (April 1932): 31; David Boyd, "Modern Arc Welding," Engineering Journal 18 (November 1935): 481.

5. T. B. Jefferson, "A Quarter Century of Welding," Welding Engineer 26 (May 1941): 21; "Recent Progress in the Art of Electric Welding," General Electric Review 29 (March 1926): 151; J. C. Joublanc, "Welding Has Come of Age -- A Review of Technical Advances," Welding Engineer 24 (October 1939): 31; C.P. Croco, "Progress in Arc Welding," Westinghouse Engineer 9 (March 1949): 34; Moses, "History and Development in the Art of Welding Ferrous Metals, Part III," 31; Blandford, 324; Cary, 11. Benardos, also called de Benardo and Benardoz, worked with fellow Russians Slavianoff, Demetrus, and Olszewaski at different times during his career. Since very little is known of these men, their contributions cannot be accurately assessed.

6. Fassler, 29; Blandford, 332; Jefferson, "A Quarter Century of Welding," 21. Resistance welding, initially more popular than its rival arc technique, was quite well established by the early 1900s. Gas welding, however, was hindered by the high cost of oxygen and acetylene, as well as by the fact that its welds solidified more slowly. See R. E. Kinkhead, "First Principles of Application of Electric Arc Welding to Manufacturing and Repair Work," Journal of the American Welding Society 4 (March 1925): 334; Hoenisch, 114.

7. Joublanc, 31; Moses, "History and Development in the Art of Welding Ferrous Metals, Part III," 31. One of the great early mysteries of welding was how and why weld drops were carried across the arc. For a discussion of the physics involved, see Cary, 38-44.

8. Blandford, 326. There is some disagreement over who performed the first commercial welding in the United States. Blandford maintains that it was Baldwin, but another source holds that Baldwin merely investigated the possibilities, which were later realized on a commercial scale in 1910 by a company named Gray and Davies; see Joublanc, 31.

9. Louis A. Larson, "Arc Welding," Welding Journal 19 (May 1940): 360; Joublanc, 31-32; Moses, "History and Development in the Art of Welding Ferrous Metals, Part III," 31.

10. For information on the repair of the German ships, see A. G. Bissell, "The Adaptability of Electric Arc Welding to the Fabrication of Structural Steel," American Architect 128 (October 5, 1925): 327; "Discussion -- European Welding Practice and American Trends," Iron and Steel Engineer 10 (October 1938): 60; T. B. Jefferson, "A Quarter Century of Welding," 23. The welding committee was formed by a Harvard professor named Comfort A. Adams during the summer of 1917 and eventually came under the direction of the Emergency Fleet Corporation of the U. S. Government; see H.M. Hobart, "Some Reminiscence of the War-time Development of Electric Welding," General Electric Review 38 (August 1935): 358-365; Alexander Churchward, "Applications of Electric Arc Welding," Heat Treating and Forging 15 (October 1929): 1304; H.M. Hobart and W. Spraragen, "Research, the Beacon of Progress in Arc Welding," Journal of the American Welding Society 5 (May 1926): 13.

11. The quote is from E. A. Hurme, "Growth of the Electric Arc Welding Industry," Iron and Steel Engineer 10 (October 1933): 279. Edward's statement can be found in James H. Edwards, "Some Possibilities and Problems in Structural Welding," Engineering News-Record 97 (August 12, 1926): 265. See also E. Dacre Lacy, "Arc Welding in Europe," American Machinist 74 (March 19, 1931): 463; W.L. Warner, "Non-Destructive Test of the Reliability of Arc Welds," Journal of the American Welding Society 4 (April 1925): 47-55.

12. The quote is from J.F. Lincoln, "Arc Welding," Mechanical Engineering 49 (Mid May 1927): 558. See Kinkhead, 32, for advantages of welding over riveting. For information on welded structures, see W. L. Warner, "Arc-welded Trolley Stations," General Electric Review 29 (October 1926): 742; E. E. Thum, "Rapid Strides Made by Welding," Iron Age 125 (January 2, 1930): 52; Jefferson, "A Quarter Century of Welding," 23.

13. American Bridge's test results can be found in "Weld Test of Heavy Column Joints Shows Good Results," Engineering News-Record 102 (March 7, 1929): 385-386 and H.L. Whittemore, "Test of an Arc Welded Plate Girder by the American Bridge Company and the U.S. Bureau of Standards," Journal of the American Welding Society 6 (January 1927): 42. Carnegie's experiment is reported in "Welded Steel Members and Joints Tested at Pittsburgh," Engineering News-Record 97 (August 12, 1926): 263-264. The American Architect test is discussed in A.G. Bissel, "The Adaptability of Electric Arc Welding to the Fabrication of Structural Steel," 327-332. The Mechanical Engineering test is in R.R. Moore, "Fatigue of Welds," Mechanical Engineering 47 (October 1925): 794-794. For information on inspection, see W. L. Warner, 47-55.

14. An excellent article dealing with all the points raised in this paragraph, as well some in the previous one, is Frank P. McKibben, "One Way to Reduce Noise is to Weld," American City 41 (September 1929): 128-130. Additional information on welded buildings can be gleaned from the following: "Welding Developments in 1929," Journal of the American Welding Society 9 (January 1930): 41; "Electric Welding," American Institute of Electrical Engineers Journal 48 (July 1929): 526; Edward Dacre Lacy, "Welded Steel Structures Part II," Metallurgia 3 (February 1931): 15; Lacy, "Arc Welding in Europe," 465. For a discussion of building codes, see C.J. Holslag, "Modern Structural Industry Demands a Knowledge of Arc-Welding," Iron and Steel Engineer 5 (February 1928): 94 and the American Institute of Electrical Engineers article above. Welding's entrance into school curricula is discussed in Thomas P. Hughes, "Industrial Welding," Minnesota Techno-Log 8 (November 1927): 55-56.

15. For information on the Chester County Bridge, see Harry K. Ellis, "Repairing Small County Bridges by Arc Welding," Journal of the American Welding Society 12 (September 1933): 22. The Missouri River Bridge is discussed in "Use Electric Welding Process to Strengthen Bridge," Railway Age 82 (June 18, 1927): 1944-1946; "Electric Welding Reduces Cost of Strengthening Bridge," Railway Engineering and Maintenance 23 (July 1927): 279-281.

16. The Schenectady footbridge is referred to in Frank P. McKibben, "Welded Buildings and Bridges in the United States," Welding 2 (September 1931): 612. This article provides an extensive list of welded buildings and bridges built in the United States before September 1931. General Electric built another welded footbridge at the same plant in 1928; see Stuart L. Peebles, "A Steel, All Welded Foot Bridge Over the Delaware and Hudson R.R. Tracks at Schenectady, N.Y.," Cornell Civil Engineer 37 (April 1929): 187-188. Information on the Turtle Creek Bridge can be found in A. G. Bissell, "Plate Girder Railway Bridge Built by Welding," Engineering News-Record 100 (February 23, 1928): 322-323; Bissell, "An Arc Welded Railroad Bridge," Journal of the American Welding Society 7 (April 1928): 42-46; Gilbert D. Fish, "Arc Welded Buildings and Bridges," Journal of the American Welding Society 7 (June 1928): 57. Westinghouse's leadership in promoting arc welded construction is discussed in Carl W. Condit, American Building (Chicago: University of Chicago Press, 1982), pp. 192-193.

17. On the Chicopee Falls Bridge, see Gilbert D. Fish, "First Arc-Welded Railway Truss Bridge," Engineering News (July 26, 1928): 120-121; Fish, "Arc Welded Buildings and Bridges," 57-58 for information on the Chicopee Falls Bridge.

18. O. Bondy, "Ten Years of Progress in Structural Welding," Civil Engineering and Public Works Review 43 (August 1948): 408; LaMotte Grover, "Foreign Countries Lead U.S. in Welded Bridges," 708.

19. On the widespread acceptance of welding as a repair technique for bridges, see H.J.L. Bruff, "Strengthening Bridges by Welding," Civil Engineering 2 (November 1932): 701; Otto Bondy, "Strengthening Railway Bridges by Welding," The Railway Engineer 55 (October 1934): 313; A.G. Leake, "Repairs to Railroad Bridges by Electric Welding," Journal of the American Welding Society 12 (February 1933): 30-31; C.M. Taylor, "The Repair of Steel Bridges by Electric Arc Welding," Engineering and Contracting 69 (February 1930): 63-64; LaMotte Grover, "Foreign Countries Lead U. S. in Welded Bridges," Engineering News-Record 116 (May 14, 1936): 209; C. H. Jennings, "European Welding Practice and American Trends," Iron and Steel Engineer 15 (October 1938): 59. The quotes are from A. Spoliansky, "Welding Evolution in the Construction of Bridges and Steel Structures in Belgium," 293-295. See also Grover, "Welded Bridge Practice in Europe," Engineering News-Record 127 (July 31, 1941): 166-170; S.M. Reisser, " Welding of Structures--Past, Present, and Future," Metal Construction and British Welding Journal 4 (February 1972): 69; Fred L. Plummer, "Welded Rigid Frames, European Style," 55; Grover, "Welded Bridges," Welding Journal 27 (October 1948): 817-819.

20. On German all-welded plate-girder design during the 1930s, see O. Bondy, "Progress in Welding Large Railway Bridges," 492-496; O. Bondy, "Electrically-Welded Railway Bridges," The Railway Engineer 55 (September 1934): 280; Grover, "Foreign Countries Lead U.S. in Welded Bridges," 705-706. On the Benton Street Bridge, see Ned L. Ashton, "Welded Deck Girder Highway Bridge," The Welding Journal 28 (September 1949): 836-837.

21. The engineering literature identifies eight American all-welded, rigid-frame bridges constructed by 1940: two were built by the City of Cleveland; six by the Connecticut State Highway Commission; see Plummer, "Welded Rigid Frames, European Style"; LaMotte Grover, "Welded Bridges," Welding Journal 27 (October 1948): 812 The quote on American tendencies to follow riveted design is from Nathan W. Morgan, "Development of Welded Bridge Construction," Welding Journal 32 (October 1953): 925. For information on the Rancocas Creek Bridge, see Gilbert Roberts, "Welded Bridges," Journal of Electric Welding (February 1936): 91; A.F. Davis, "Largest All-Welded Bridge Completed in New Jersey," Welding Journal 14 (August 1935): 2-4; W.K. Greene and C.W. Wixom, "Welded Highway Bridge, Burlington County, New Jersey," Welding Journal 15 (April 1936): 12-16. California was occasionally credited with the having the country's longest welded bridge, by virtue of a 1,380-foot-long state highway bridge completed near Merced in 1932. This structure, however, contained

only "a relatively small amount of welding, since girders and floor beams are rolled sections." On the Merced bridge and the Dresden bridge in Germany, see Rene Leonhardt, "Welded Bridge in Germany Sets Records," Engineering News-Record 111 (November 16, 1933): 589; Grover, "Foreign Countries Lead U. S. in Welded Bridges," 206.

22. The quote is from Leon C. Bibber, "The Status of Welding in 1937," Metal Progress 32 (October 1937): 552; see also E. Steinert and W.W. Reddie, "Arc Welding Apparatus," Iron and Steel Engineer 13 (January 1936): 9; Wedenmeyer, "Modern Welding Methods in the Engineering Industry," Engineering Progress 6 (October 1925): 321; Thomasson, 11. On municipal building codes, see Frank P. McKibben, "Erecting Steel Buildings and Strengthening Steel Bridges by Welding," Journal of the American Welding Society 12 (May 1933): 6; C.H. Jennings, "European Welding Practice and American Trends, Iron and Steel Engineer 15 (October 1938): 59; and Stitt's article, page 812.

23. The Nebraska bridge, described only as "an all-welded fabrication of structural steel," was a grade separation project built in the City of Hastings with the collaboration of the C. B. & Q. Railroad; see Twentieth Biennial Report of the [Nebraska] Department of Roads and Irrigation, 1933-1934, (no pub., n.d.) pp. 85-85. The discussion of state highway departments and their reliance on standardized plans is based upon research compiled by Jeffrey A. Hess, one of the authors of the present study, as part of statewide historic bridge surveys of Minnesota, Wisconsin, South Dakota, North Dakota, and Nebraska; see, for example, Jeffrey A. Hess and Robert M. Frame, Wisconsin Stone-Arch and Concrete-Arch Bridges (Madison: Wisconsin Department of Transportation, 1986), pp. 100-102; 249-252.

24. The first quote is from Larson, 360. The second is from T.B. Jefferson, "1945--A Year of Triumph," Welding Engineer 31 (January 1946): 36. The last quote is from R.M. Gooderham, "Pre-War Thinking in Welding," Canadian Welder 56 (July 1945): 130. The 90% figure comes from Ted Jefferson, "1940--A Year of Welding Progress," Welding Engineer 26 (January 1941): 20. Other articles on the growth of welding during the war are: W.W. Reddie, "Present Trends in Arc Welding," Welding Journal 20 (February 1941): 102-104; Wendell F. Hess, "Adams Lecture for 1946--New Frontiers in Welding," Welding Journal 26 (February 1947): 114-120; T.B. Jefferson, "Facts and Figures; A Statistical Review of the Welding Industry," Welding Engineer 28 (March 1943): 39-52.

25. The first quote is from T.B. Jefferson, "1947 Found Welding Again Winning Rightful Place," The Welding Engineer 33 (January 1948): 34. The second is from Arsham Amirikian, "Welded Steel Structures," Welding Journal 28 (July 1949): 631. See also A. Ramsay Moon, "Welding During the War--And After," Structural Engineer 25 (January 1947): 213-218; T.B. Jefferson, "1945--Year of Triumph," 35-37, 47; Archibald, 57-59.

26. Randle B. Alexander, "Texas' Experience with Welded Bridges," Engineering News-Record 145 (July 20, 1950): 44. On the other states, see Grover, "Welded Bridges," Welding Journal 27 (October 1948): 812-813; George W. Lamb and E. S. Elcock, "Kansas' First All-Welded Bridge," Engineering News-Record 145 (September 14, 1950): 46-47. On the Benton Street Bridge, see Ashton, "Welded Deck Girder Highway Bridge," 832-840. On federal support for welding and deck-style design, see Raymond Archibald, "Significant Changes in Modern Bridge Design," Roads and Streets 90 (February 1947): 58. In 1946, the Connecticut State Highway Commission completed a deck-girder highway bridge that was apparently the first post-war welded structure built with federal aid; see Grover, "Welded Bridges," 812.

27. Alexander, 44.

28. For references to additional welded bridge projects in Iowa, California, Florida and Washington, see, respectively, Ned. L. Ashton, "If All-welded Railroad Cars Are Safe, Why Not Bridges?" Civil Engineering (October 1952): 861-862; L. C. Hollister, "California's All-Welded Viaduct Points Way to

Improved Design," Civil Engineering (September 1952): 764-822; F. L. Plummer "Design of Welded Steel Bridges and Buildings," Welding Journal 32 (June 1953): 496; H. M. Hadley, "Steel Box Girders Support Highway Bridge," Engineering News-Record 146 (April 12, 1951): 34-35.

29. The first quote is from Plummer, 496; the second from Morgan, "Development of Welded Bridge Construction." Welding Journal 32 (October 1953): 925. California's statement is found in "How States Are Adapting to Meet Material Shortages," Road and Streets (June 1957): 154; for an earlier discussion of the same subject, see Hollister, 180. On the increasing popularity of welded highway girders, see Robert W. Abbett, ed., American Civil Engineering Practice, vol. 3 (New York: John Wiley and Sons, 1957), p. 28-228.

30. The nation's first prestressed concrete highway bridge was a 308-foot-long, triple-span structure built by the City of Philadelphia in Fairmount Park; see "First Prestressed Bridge in the U.S.," Engineering News-Record 141 (December 30, 1948): 16-18. The statistical information on prestressed concrete bridges is from F. Walley, "Prestressed Concrete -- A Review," Reinforced Concrete Review 4 (June 1958): 622. On the interest of various state highway departments in prestressed concrete, including South Dakota, see "How States Are Adapting to Meet Material Shortages," 120-127.

31. Robert Jones Wheeler, Jr., "Erection Problems of a Modern Girder Highway Bridge," unpublished, MS thesis in Civil Engineering, State University of Iowa, 1949, pp. 8-9; "Bridge Served Rural Areas," Iowa City Press-Citizen, November 8, 1980.

32. Ned L. Ashton, "Benton Street Bridge," unpublished lecture, c. 1947, in Ashton Collection, State Historical Society of Iowa, Iowa City.

33. City Council Minutes of Iowa City, December 22, 1944; June 10, October 14, 28, 1946; January 13, May 21, 1947; "Name Ashton to Aid City's Bridge Project," Daily Iowan, May 22, 1947; Ashton, "Benton Street Bridge."

34. Ashton, "Comments on Design and Construction of Various Mississippi River Bridges," unpublished lecture delivered to Tri-City Section, American Society of Civil Engineers, Davenport, Iowa, April 6, 1944, typescript in Ashton Collection. Ashton's reminiscences are the basis of Richard F. FitzGerald, "Bridges are His Business," Iowa Alumni Review (December 1958): 13-17. For biographical information, see also Winfield Scott Downs, ed., Who's Who in Engineering (New York: Lewis Historical Publishing Company, 1941), pp. 54-55.

35. The quotes are from Ashton, "Comments on Design and Construction," p. 9. See also Fitzgerald, "Bridges are His Business," 16; Marshall McKusick, "Ashton House and the Engineer Who Built It," unpublished, 1986, pp. 51-58, in Ashton Collection.

36. On the nature of detailing, see J. A. L. Waddell, Bridge Engineering, vol. 2 (New York: John Wiley and Sons, 1916) p. 1947; Waddell, Economics of Bridgework (New York: John Wiley and Sons, 1921), pp. 199-201.

37. Ashton discusses his work in reducing detailing in "Comments on Design and Construction," pp. 11-13; the quote concerning his use of welding is from Fitzgerald, p. 15. Ashton is listed as "author" of the three papers on welding in Who's Who in Engineering, 1941, 55, but this source does not provide a publisher or publication. The papers evidently did not appear in a major engineering journal, since Ashton is not credited with a publication by the Engineering Index during the period 1930-1941. The papers are not included in the Ashton Collection at the Iowa State Historical Society in Iowa City.

38. Ashton's preliminary design sketch is found in Iowa City Press-Citizen, June 13, 1947; the design is described (albeit confusingly) in "Bridge Plans Go Forward: Estimate Cost at $215,000," Daily Iowan, June 10, 1947. Ashton himself describes the design in "The Benton Street Bridge." On the Grand Avenue Viaduct, see Ashton, "Comments on Design and Construction," p. 13. Ashton's selection for the final design is recorded in City Council Minutes, June 23, 1947. For a discussion of variable-depth girder design as applied to the final design of the Benton Street Bridge, see Ashton, A Modern Steel Deck Girder Highway Bridge (Cleveland: Lincoln Electric Company, 1952), pp. 6-7.

39. For the debate over site location, see Ashton, "The Benton Street Bridge"; "Ashton Shows Plan for Bridge Relocation," Iowa City Press-Citizen, July 1, 1947; City Council Minutes, July 14, August 11, 1947. Ashton's final design is briefly described in "Cost Estimate for Proposed Benton Street Bridge Drops $32,000," Iowa City Press-Citizen, November 29, 1947. The statistic on American all-welded bridges is from Grover's 1948 study: "Some 70 welded railway and highway bridges, involving fabricated plate girders, trusses or rigid frames, and with span lengths up to 180 ft., have been reported in the United States and Canada from 1926 to date. Many smaller, less important bridges have been constructed entirely by welding or with an extensive use of welding in their fabrication." Of the 70 larger bridges, 15 were Canadian; see Grover, "Welded Bridges," 814, 825.

40. The first quote is from Ashton, "Welded Deck Girder Highway Bridge," 833; the second from Ashton, "The Benton Street Bridge."

41. McKusick, in turn, heard the story from Ashton's nephew and future business associate, William Ashton, himself an engineer; Marshall McKusick, Letter to Lowell Soike, October 15, 1987, in Bureau of Historic Preservation, State Historical Society of Iowa.

42. All quotes are from Ashton, "Welded Deck Girder Highway Bridge," 836, 837. Presumably, the special flange sections were custom-made for the project by one of the mills of the United States Steel Corporation, which supplied the superstructure steel; see Walter R. Johnston, American Bridge Company, Letter to Ned L. Ashton, May 27, 1948, in Ashton Collection.

43. Ashton's commendation of Jensen work is dated February 5, 1951 ("To whom it may concern"), Ashton Collection. Background information on Jensen Construction Company is found in the company's statement of "History and Financial Condition" in Ashton Collection; the document was apparently submitted as part of their bid for the Benton Street project. Jensen's previous experience with welding in girder bridges is mentioned without elaboration in "Cost Estimate for Proposed Benton Street Bridge Drops $32,000," Iowa City Press-Citizen, November 29, 1947. On the selection of Jensen as general contractor, see "Low Bid on Benton Bridge is $257,814," Press-Citizen, April 19, 1948. Jensen's subcontractors are identified in Ashton, "Welded Deck Girder Highway Bridge," 840.

44. On the progress of construction, see Ashton, "Welded Deck Girder Highway Bridge," 832-840; Ashton, Modern Steel Deck Girder Highway Bridge, pp. 35-32; "Concrete Work on Benton Street Bridge Begins," Daily Iowan, May 12, 1949; "Bridge Completion Expected in Ten Days, Officials Say," Daily Iowan, July 6, 1949.

45. "Construction Today," Engineering News-Record (March 17, 1949): 87. See also F. L. Plummer, "Design of Welded Steel Bridges and Buildings," Welding Journal 32 (June 1953): 496; Robert Abbett, ed., American Civil Engineering Practice, vol. 3 (New York: John Wiley and Sons, 1957), p. 28-223.

46. Archibald, "Significant Changes in Modern Bridge Design," 57-59.

47. Ashton, A Modern Steel Deck Girder Highway Bridge. The Connecticut bridges are briefly noted in Grover, "Welded Bridges," 812.

48. Ashton, "Iowa Tries a Welded Aluminum Bridge," Engineering News-Record 160 (February 20, 1958): 30-32; Ashton, "First Welded Aluminum Girder Bridge Spans Interstate Highway in Iowa," Civil Engineering 28 (October 1958): 79-80. James C. Hippen, a history professor at Luther College in Decorah, Iowa, has been a student of Iowa bridge engineering for over a decade; he summarized his early research on the subject at the Annual Meeting of the Society for Industrial Archeology at St. Paul, Minnesota in May 1983. For his assessment of Ashton's career, see Hippen, Letter to Lowell Soike, November 8, 1989, as quoted in Mckusick, "Ashton House and the Engineer Who Built it," p. 4. Ashton mentions his interest in an aluminum Mississippi River bridge in Fitzgerald, "Bridges are His Business," 17.

49. Ashton's later years are discussed in McKusick, "Ashton House and the Engineer Who Built It," pp. 63-69. On the Cedar Rapids bridges, see Ashton, "New Bridges Founded on Old," Civil Engineering 38 (November 1968): 44-48.

50. NNW, Inc. "Feasibility Study [of] Widening and Improvement Benton Street Bridge Corridor," unpublished, 1985, p. 19, 23, in Ashton Collection.

51. The preservation forces were led by Marshall Mckusick of Iowa City, who was instrumental in calling public attention to the bridge's historical significance; see, for example, McKusick, Letter to Lowell Soike, Office of Historic Preservation, Iowa State Historical Department, October 15, 1987, in Ashton Collection. On the issue of metal fatigue in the Benton Street Bridge, see S. T. Rolfe, "An Analysis of the Fracture and Fatigue Reliability of the Benton Street Bridge, Iowa City, Iowa," unpublished, 1988, in Ashton Collection. Requirements for historical documentation of the structure are set forth in Gregory D. Kendrick, National Park Service, Rocky Mountain Region, Letter to H. A. Willard, U.S. Department of Transportation, Federal Highway Administration, Iowa Division, May 14, 1989, in Office of Historic Preservation, Iowa State Historical Department.

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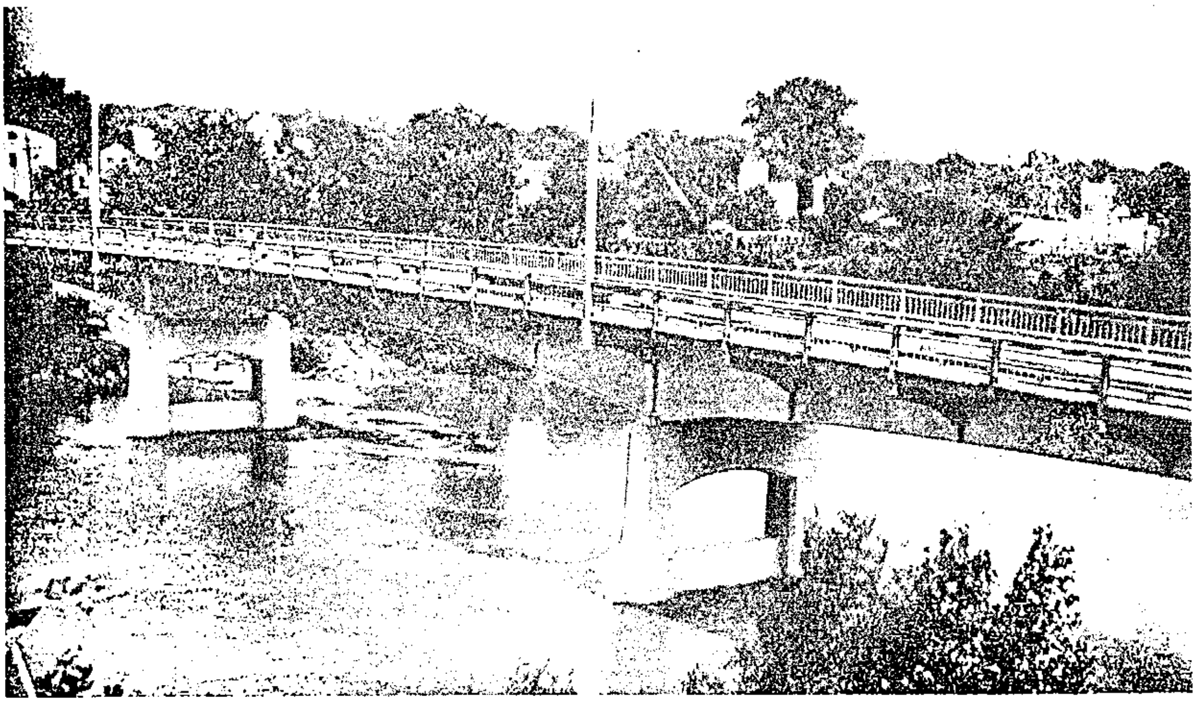
SUPPLEMENTARY DATA SECTION

Studies in
Structural Arc Welding

A MODERN STEEL DECK GIRDER HIGHWAY BRIDGE



Benton Street Bridge over the Iowa River, Iowa City, Iowa.

by NED L. ASHTON
Consulting Engineer
Iowa City, Iowa

Price 25c

Published by
THE LINCOLN ELECTRIC COMPANY
CLEVELAND, OHIO

A Modern Steel Deck Girder Highway Bridge

At right is a general view of a modern all welded steel deck girder highway bridge that has just recently been completed across the Iowa River in Iowa City, Iowa.

The design and construction of this bridge is being used as the subject of this series of Studies in Structural Arc Welding since it is typical of hundreds of other small highway bridges that are needed all over the country to bring our highway systems up to date for modern traffic. The simple graceful beauty of this bridge speaks for itself and the praise of its builders attest the wisdom of designing for welding in the fields of bridge engineering. Therefore it is believed that a detailed description of this structure will be of material use to others who have similar problems in this field of Civil Engineering.

A general cutaway cross sectional drawing of the bridge is shown in Fig. 2. The view shown is taken at one of the interior river span piers.

This drawing shows a bridge in which the two lane concrete roadway is supported directly on top of the main girders while the sidewalk is cantilevered on the outside of the girders on brackets extended from the floorbeams.

The sidewalk is a 3½ inch thick concrete slab that spans transversely from the fascia to the curb channel except that it is thickened to 5 inches near the curb so as to contain the electrical and Bell Telephone conduits.

The pedestrian traffic is separated from the roadway traffic by means of a safety curb and all of the curbs and handrails are made of steel.

A special feature of this design is that all of the curbs and handrails were prefabricated in the shop so as to erect as complete units in the field, each prefabricated unit being twenty feet long.

The girders also are three plate weldments made in convenient lengths for shipment and erection in the shop by means of automatic welding processes and then these pieces are butt welded together in the field to make continuous girders 476 feet long on each side of the roadway extending from end to end of the bridge.

Other advanced features of the design include the use of tee-shaped stiffeners and special high strength line bearings made of weld metal deposited on the surfaces of the structural weldments that form the bridge shoes.

The first steps in making a design such as this, of course, is to determine the volume of traffic and adopt suitable roadway and sidewalk widths to carry the traffic. Then locate the structure properly with respect to the stream and determine the span lengths and clearances to suit the requirements of the waterway.

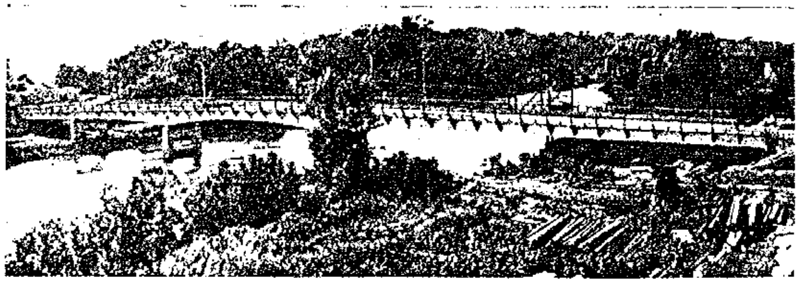


Fig. 1

In this respect the above bridge is considered to be one half of a future dual four lane highway bridge of which one half has been built presently to replace an older two lane highway bridge at the same site that had been condemned and closed to traffic and the other half can be added in the future whenever it is justified by the traffic.

The particular span arrangement that was adopted for this bridge is a symmetrical layout consisting of 78-100-120-100 and 78 foot continuous spans located so that the central 120 foot span is in the middle of the river.

The bridge is supported at the ends on the two abutments and runs continuously for 480 feet over four other intermediate piers. All of the piers and abutments are made of concrete and are founded on steel foundation piles driven to rock.

The structural framework was laid out as shown in Fig. 3. In this plan the girders are located under the roadway, spaced at 20'-3" centers, so as to act directly as stringer supports for the roadway slab in addition to framing with the floorbeams and cantilever brackets. This procedure saves two lines of roadway stringers, reduces the span length of the floorbeams and saves masonry in the foundations by reducing the lengths of the piers.

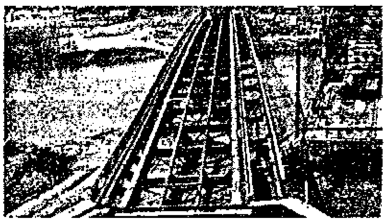


Fig. 3

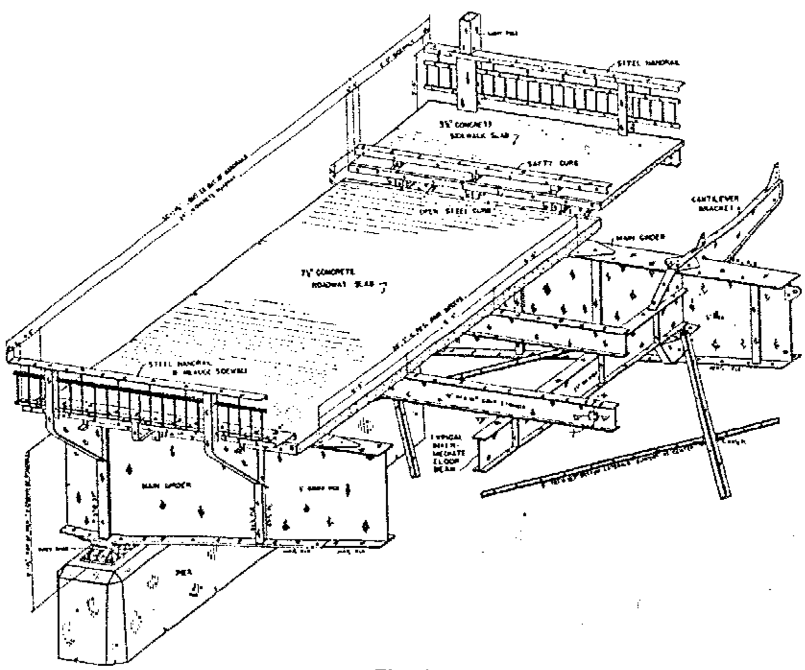


Fig. 2

Following the determinations of span lengths and floorbeam spacing the details of the cross section as shown in Fig. 2 are next worked out by the designer including the slabs, handrails and curbs.

Since the handrails are ordinarily shipped as a multitude of small pieces that consume a great deal of time for assembly and final alignment in the field it was decided to prefabricate these rails as complete units in the shop.

Figure 4 is a detailed trimetric projection of the sidewalk and sidewalk handrail with portions cut away to show the construction of the curbs and handrails.

This outside railing was pre-fabricated integral with the 12-inch 25-lb. fascia channel to erect in 20 foot units complete except for the end connections and the connections of the intermediate channel brace at the bottom of the interior posts.

tion pins or bolts through the plates in the ends of the channels.

These holes provide for holding and for the adjustment of the posts to vertical and horizontal alignment before welding. The holes in the web of the half channel end posts also insure that the ends of adjacent sections must be in the same alignment.

For the 16 middle panels of the handrail the top rails were increased 3/16 inch in length and cambered ¼ inch for the vertical curvature. And at the end of every third panel both the top rail and the intermediate rails were connected to the end posts with ½ inch round button head bolts in ¾ inch long slotted holes to provide for the differential expansion and contraction of the rail with respect to the rest of the deck.

Thus both of these curbs have open spaces between the posts that permits the wind to blow through under the horizontal members and helps facilitate the cleaning of the roadway surfaces of loose snow and dust and prevents large accumulations of debris on the roadway surfaces adjacent to the curbs.

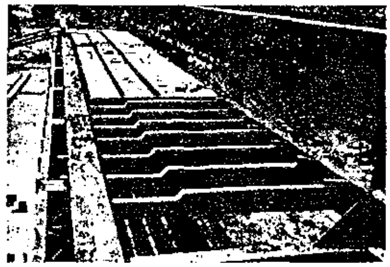


Fig. 5

The end posts of the curb sections are made as half post channel sections with plates in the ends of the 7-inch roadway channel to engage with bolts for holding it in alignment with the holes in the ⅝-inch supporting plates on the floorbeam brackets.

For design this type of roadway curb has the backing of the concrete sidewalk to resist lateral loads and needs only be designed for the gravity loading of the sidewalk.

The advantage of this construction for extending the stringers through the openings under the curb so that they can be supported on the edge of the roadway slab is shown on Fig. 6.

The other end of the form is blocked up from the bottom of the fascia stringer.

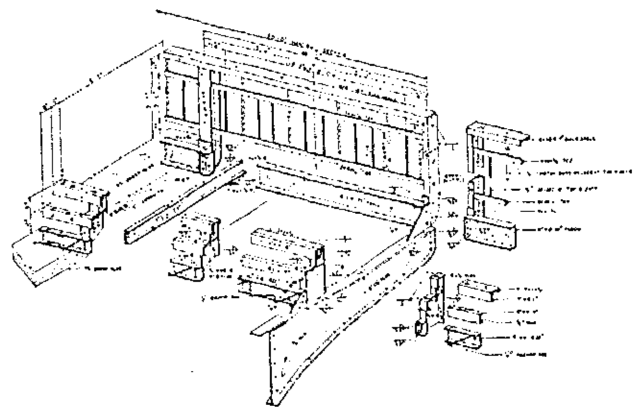


Fig. 4

The top member of the railing is designed to sustain a horizontal force of 150 lbs. per linear foot acting together with a simultaneous vertical load of 100 lbs. per linear foot applied at the top of the railing. It is supported by means of intermediate 6" x 6" x 15.5 lb. "H" section posts spaced at 10'-0" centers and by 6" x 3" x ¼" bent plate channel half post sections at the ends. Additional vertical support of the top rail is also provided by means of the alternate twisted spindles which are welded to it at 2'-2¼" intervals throughout its length. Hence the whole railing acts to sustain the vertical loads and the top rail has only the horizontal load to sustain as a beam while acting on the 10'-0" span between posts.

The ends of the prefabricated sections are held together with the half post channel sections. Thus the whole section erects as a single unit capable of spanning the twenty foot panel length between cantilever brackets. During erection it is held upright by the ⅝" plates that insert between the ends of the fascia channels on top of the brackets and engage three erec-

Figure 4 also shows the details of the sidewalk curb, the safety curb and the edge channels for the slabs that were all assembled together to make the second series of prefabricated units.

The regular curb consists of a standard 5" channel at 9 lbs. that is anchored to the edge of the sidewalk slab with ⅝ inch round anchor rods. This channel also supports the edge of the sidewalk and it in turn is supported vertically every 5'-0" by means of brackets from the 7" edge channel on the roadway slab.

The brackets from the roadway slab are made of ⅜ inch plates welded in the form of flanged sections that extend on up above the sidewalk to support the safety curb.

The safety curb extends nine inches above the surface of the sidewalk and is located six inches back from the face of the regular 10-inch roadway curb. It consists of another standard 5-inch channel welded to a 3" x 2" x 5/16" angle.

The weight of a typical 20'-0" section of the sidewalk handrail is 63.5 lbs. per lin. ft. of railing including the fascia channel and support for the edge of the sidewalk.

The sidewalk curbs and the structural metal in the edge of the roadway slab weighs 47.0 lbs. per linear foot including the anchor rods that are imbedded into the concrete.

The upper safety curb and the upper curb supports are designed for a load of 300 lbs. per linear foot applied horizontally at the top of the upper curb.

The diaphragm webs at the upper curb supports are clipped so as to simplify the connection and also so that the space inside the upper curb can be used for the location of an additional electrical conduit running the full length of the sidewalk if necessary.

Figure 6 is a cutaway trimetric projection showing the details of the steel refuge sidewalk curb and the handrail construction on the other side of the roadway.

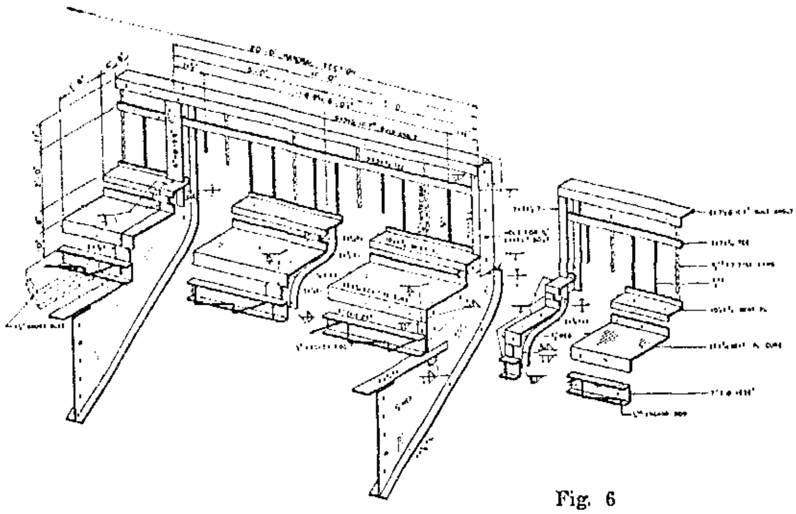


Fig. 6

The handrail on this side of the roadway is made of the same basic sections as the handrail on the sidewalk side except that it is all assembled together with both of the bent plate curb sections and with the cantilever brackets to form the prefabricated assemblies which are shown again in the pictures on Figs. 7 and 8.

On this detail the roadway curb is made of a 26" x 5/16" plate with the edges bent up and down into the form of an 18-inch wide zee bar.

The upper curb is also a 6-inch zee bar formed from a 10" x 5/16" plate. This member also forms the bottom rail of the handrail and supports the handrail spindles.

The six-inch width of the upper curb is designed to give arm clearance with the top of the handrail as a person uses the 18-inch lower curb as a sidewalk. This increases the effective width of the roadway surface by setting the face of the handrail farther away from the face of the curb barrier.

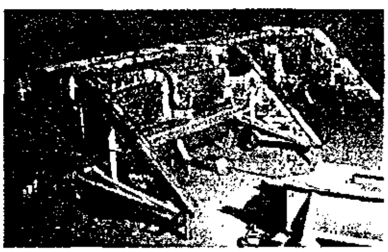


Fig. 7

Thus the useful width of the roadway is considered to be equivalent to the 26'-0" clearance between the faces of the upper curbs.

On the roadway side of the bridge the large cantilever brackets are provided at 10'-0" centers while smaller brackets are provided in between to support the curbs from the channel along the edge of the slab at 5'-0" intervals. These brackets were designed in the curved shape to enhance the general appearance of the bridge and the bottom flanges of the brackets are left open so that there is no place to accumulate debris from the roadway.

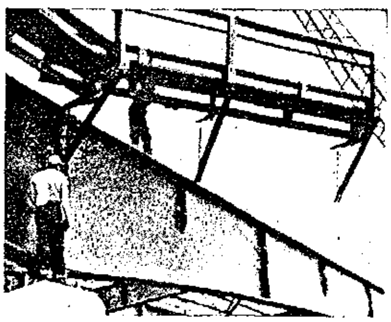


Fig. 8

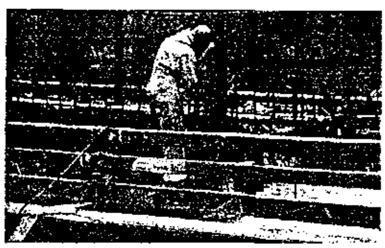


Fig. 9

Figure 7 shows a picture of one of these twenty-foot prefabricated sections assembled in a jig in the shop for the shop welding. In the assembly two main brackets go with each prefabricated unit and the other end is fitted with a regular half post channel section at the end and a bent plate channel half section of one of the smaller brackets for the field splice. This detail is also shown on the right of Fig. 6.

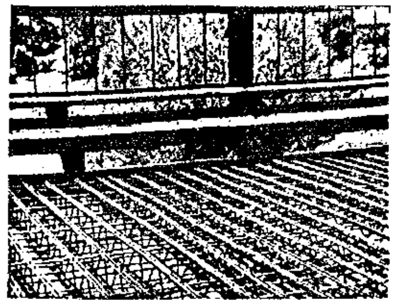


Fig. 10

Figure 8 also shows a picture of one of the typical 20-foot sections of the roadway curbs and handrail as it was being erected. The units were temporarily supported by means of a single rope sling and lowered into place so that the half post end section could be made endwise with the previously erected section and then the large brackets at the far end were engaged with bolts through the stiffeners on the girder by pulling them into place with a comealong ratchet. The bolts provided the necessary temporary support until it was field welded in final position. The erection of these units proved to be very easy and very fast and the shop work was so accurately done that the field alignment of the rail was automatic. As soon as full holes were made in the stiffeners no further adjustment was necessary.

The fully assembled weight of one section of rail as shown in Fig. 8 was 2,250 lbs. per 20-foot section or 113 lbs. per linear foot including the brackets and the edge channel on the roadway slab.

The final operation in the erection of the handrails was the final weld around the two half post sections to join the adjacent sections into a single post.

Figure 9 shows a welder at work making this weld on the sidewalk railing.

Figure 4 also shows the detail of a typical cantilever bracket under the sidewalk. These brackets were made by welding 6" x ⅝" flange plates to a ¾-inch web plate. The elevation of the top flange of the cantilever bracket was set so as to go over the top flange of the girder and butt weld to the top of the bracket on the floor beam. The bottom flange was curved for appearance.

These brackets were spaced at 20'-0" centers and are temporarily held in place during erection with bolts and drift pins engaging the web of the bracket with the 5" x ½" stiffener plates that are provided at connection points on the girders until field welding.

The roadway slab is designed as a typical transverse double reinforced concrete roadway slab with one half inch of extra thickness allowed on the top for wear.

Two lines of stringers are used to support the slab in between the girders making the interior slab spans 3 equal spans at 6'-9" 20'-3" center to center of girders. At both edges of the roadway the slab extends 1'-10½" beyond the center lines of the outside girders to the curbs where it is finished with a steel channel securely anchored to the edge of the roadway slab.

These channels function both as screeds for finishing the surface of the slab to proper longitudinal elevations and also as edge supports for the loads on the edge of the roadway slab.

The required area of steel is made up of welded reinforcing bar trusses spaced at 6-inch centers as shown in Fig. 10. Each truss consists of four one-half inch round longitudinal bars spaced properly by welding to a three-eighths inch recticuline web bar. The one-half inch wearing surface is neglected in the computations. The steel is transformed into an equivalent area of concrete and the stresses are computed as follows. All concrete in tension is neglected.

Section		A	y	Ay	Ay^2 +	I_o
Compression Concrete	12 x 2 =	24.0	1	24	24	8
Compression Steel	.78 x 9 =	7.02	1.25	8.77	10.97	
Tension Steel	.78 x 10 =	7.80	5.75	44.80	258.00	
		38.82		77.57		300.97

$$kd = \frac{77.57}{38.82} = 2 \qquad 2 \times 77.57 = -155.00$$

$$I = 145.97$$

$$f_c = \frac{5810 \times 12 \times 2}{146} = \underline{\underline{955}} \text{ lbs. / sq. inch}$$

$$f_s = \frac{5810 \times 12 \times 3.75 \times 10}{146} = \underline{\underline{17,900}} \text{ lbs. / sq. inch}$$

In the above computation the amount of concrete in compression is assumed at first and then is finally determined by the computation for "kd" which also is the location of the neutral axis and the center of gravity of the effective section.

The computation is made first about an axis through the compression edge and then it is transferred to the center of gravity axis after "kd" is determined. In this fashion the distances to the extreme fiber are obtained at the same time for use in the bending formula.

For estimating purposes each transverse reinforcing truss weighs 76 lbs. and there are 49 lines of ½-inch round longitudinal distribution bars running lengthwise of the bridge in the roadway slab.

The quantities of concrete and reinforcing steel in the slab per foot of bridge are summarized as follows:

	Concrete	Reinforcing
Roadway Slab	.565 c. yds.	190 lbs.
Sidewalk Slab	.065 c. yds.	10 lbs.
Total	.63 c. yds.	200 lbs.

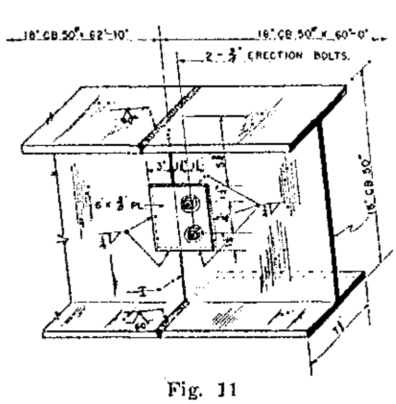


Fig. 11

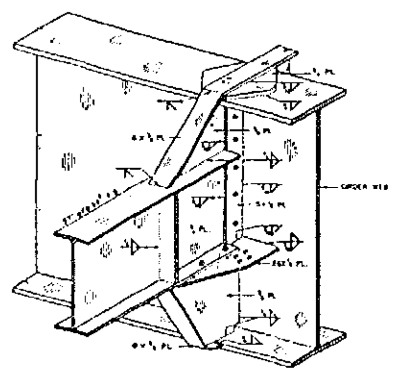


Fig. 12

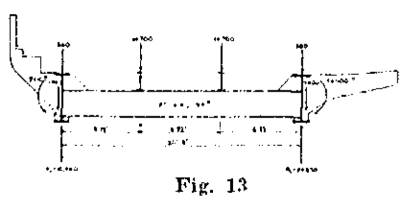


Fig. 13

The roadway stringers are designed as continuous beams for the full length of the bridge. All interior spans are 20'-0" while the two end spans are 18'-0" with short cantilever projections at each end of the bridge for the support of the expansion joints.

The stringers are spaced at 6'-9" centers and each stringer is designed to carry S/5 lines of wheels or 6.75/5 = 1.35 lines of wheels in accordance with the 1949 A.S.H.O. Specifications for two lanes of traffic on a concrete floor.

They are detailed to run continuously over the floorbeams and are fabricated in 60-foot lengths with field splices at the one-fifth span points in every third panel.

A detail of the field splice is shown on Fig. 11. The splice is located 4'-0" from the center line of the floorbeam and is made as a full penetration weld. The flanges of the beams are "Vee" beveled by flame cutting in preparation for welding and are held in place temporarily by means of 6" x ⅜" plates which are shop welded to the end of the cantilevered beam and engage the end of the other piece with two ¾-inch erection bolts.

Two 4" x ⅜" stiffener plates are provided on the web of the floorbeams under each stringer to reinforce the top flange of the floorbeam for the bearing loads. Thus the length of bearing area on the stringer web is increased to the full width of the floorbeam flange.

Figure 12 is the detail of the end of a typical 27" WF 94 lb. floorbeam at the north girder where it connects opposite the sidewalk cantilever brackets.

Figure 13 is a sketch of the floorbeam showing the distribution of the dead loads to the north and south girders.

These beams are designed for a maximum moment of 353,500 ft. lbs.

For maximum shear on the connection to the north girder the live load is added on the sidewalk. This increases the reaction on the girder by 47,500 ft. lbs./20.3 ft. or 2350 lbs.

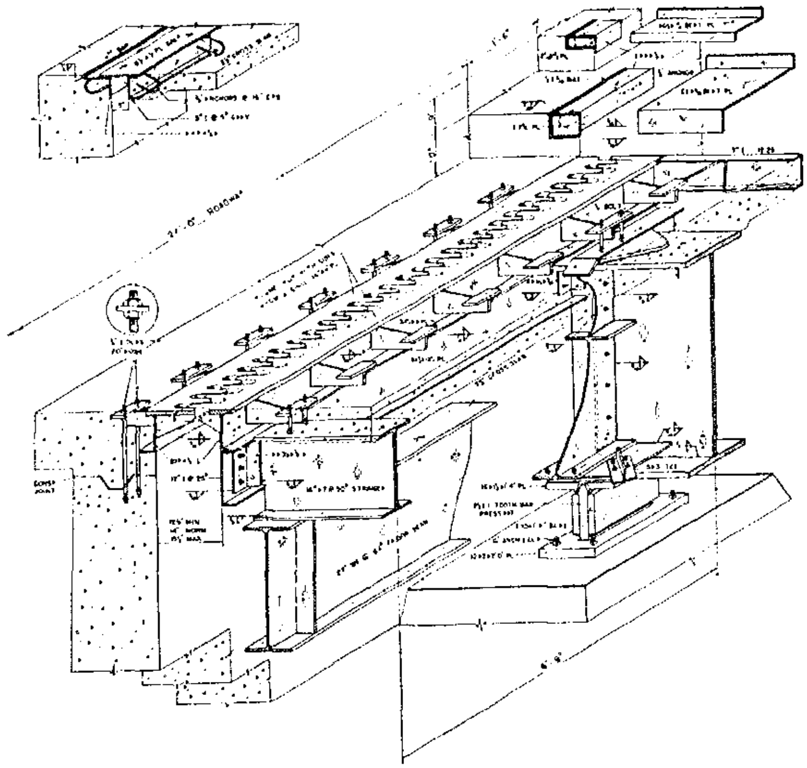


Fig. 14

The connections to the main girders are designed for 59,100 lbs. shear. See formula at top of page 7.

The end connections of the floorbeams to the main girders were also investigated for a fixed end moment of —258,000 ft. lbs.

This was done because there is the possibility that the girder shoes and the roadway slab will react against the bracketed ends of the floorbeams and make them be fixed ended. The cantilever brackets also contribute to this and the roadway slab resists the thrust of this action.

Thus with the top flange butt welded to the cantilever bracket strap plate as shown on Fig. 12 the connection is good for either the simple beam or fixed end condition.

All of the floorbeams are made alike and they all carry the same loads excepting the end floorbeams.

Figure 14 shows the arrangement of the end floorbeams and the expansion joint details at the abutments.

The main girders rest on ten-inch diameter segmental rollers that are made of a single piece of 5" x 1'-4" structural slab nine inches high. This slab is built up to ten inches high by depositing high strength weld metal on the surfaces to form the line bearings. Similar bearing surfaces are also deposited on the top surface of the masonry plate and on the under side of the girder sole plate.

The roller is held upright in all positions by means of the 2½" x 1" toothed bars that are pressed into the end of the roller and engage both the masonry plate and the sole plate with standard stub teeth.

The masonry plate is set on a ground surface finished with red lead and canvas and these roller bearings support the whole end of the bridge.

Directly over the rollers there are two 6" x ½" end bearing stiffeners that transfer the reaction into the web of the main girder and which serve also as connection plates for the end floorbeam and the cantilever bracket.

The end floorbeams are made of the same section as the intermediate floorbeams and serve to support the ends of the intermediate stringers and all of the rest of the end of the bridge.

The break in the roadway slab is formed with an all-welded toothed plate expansion joint that provides for three inches of expansion and contraction at each end of the bridge.

This type of joint is most successfully made by building both sides of the entire joint, all welded to the under surface of the common 16" x 1" top plate before cutting the two halves of the joint apart. The flame cutting of the toothed profile is the last operation and by then the joint is strong enough to avoid all of the usual twisting and warpage. The teeth are spaced at four-inch centers and each tooth is six inches long. This provides for 3 inches of movement with a minimum of 1½ inches clearance and 1½ inches of lap in the extreme positions. The teeth are 1½ inches wide at the ends and taper to 2½ inches wide at the base. The flame cutting is done with the aid of an electronic tracing machine to guide the torch.

The understructure of the joint consists of two 8" x 4" x ½" standard structural angles that are first cut to conform to the curvature of the transverse crown of the roadway. Each tooth is supported by means of a 5" x ½" web plate and the anchor bolt brackets are spaced at 16-inch centers along the back of the joint. These brackets engage with jacking bolts in the abutment and are embedded into the roadway slab on the other side lapped with suitable reinforcing steel. The end of the roadway slab pours into the pocket formed by the eight-inch angle and the surface plate and both the end of the slab and half of the expansion joint are supported by means of the 12" channel that spans between the ends of the main girders and the roadway stringers across the end of the bridge.

The sidewalk expansion joint is shown in the upper left of Fig. 14. It is simply a sliding plate and angle joint supported with a 5-inch channel across the end of the slab and a 6" x 3" ⅜" angle on the edge of the abutment.

The roadway curbs are finished at the ends in the same fashion by letting the ends of the bent plate curb sections slide over the angle sections on the abutment and the end of the handrail is supported as a simple cantilever beyond the last steel end post.

Additional provision is also made for the differential expansion and contraction of the handrails at the end of every third panel by means of bolted connections in slotted holes to the handrail posts and at the light poles.

A temporary lateral system is now added in the plane of the bottom of the floorbeams to serve until the deck slab is poured and the designer is ready to summarize the loads and begin the design of the main girders.

The laterals were made of standard 5" x 5" structural tees designed for nominal "l" over "r" ratios not to exceed 200 for secondary tension members. They are supported once at the center and are connected to horizontal gussets at the ends with ribbed bolts for a nominal stress of 25,000 lbs.

	Shear		Moment
Dead Load	17550#		+ 89390 ft. lbs.
Live Load	30150#	x 6.75	203000 ft. lbs.
Imp. 30%	9050		61000 ft. lbs.
Total	56750#		+ 353390 ft. lbs.

The section modulus required is

$$\frac{353{,}500 \times 12}{18{,}000} = 237$$

and the 27" WF 94 floorbeam section provides 242.8.

Summary of Weights and Loads

Item	Total Weight	Unit Wt.	North Girder	South Girder
Sidewalk H.R. & fascia	29,478 lb.	61#/lf.		
Sidewalk Curb & Edge of Slab	22,963 lb.	48#/lf.	168#/lf.	—34#/lf.
Sidewalk brackets and H.R. Braces	11,948 lb.	25		
Roadway Curb, H.R. and Brackets	54,426 lb.	113	—18	131
Roadway Stringers	48,024 lb.	100	50	50
Floorbeams	50,802 lb.	106	53	53
Lateral Bracing	16,068 lb.	34	17	17
Expansion Joints	6,884 lb.	14		
Total Floor Metal	240,593 lb.	501	270	217
Concrete Sidewalk	134,300 lb.	280	349	—69
Concrete Roadway	1,070,000 lb.	2230	1115	1115
Total Except the Girders	1,444,893 lb.	3011#/lf.	1734#/lf.	1263#/lf.

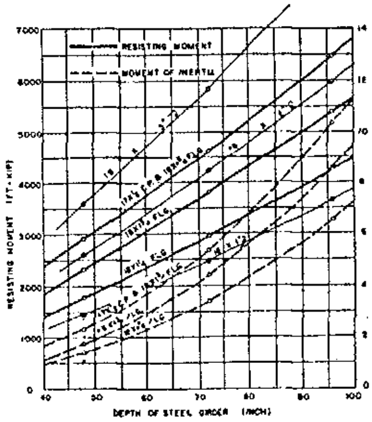


Fig. 16

As the next step in design the sizes and weight of the main girders have to be determined in some fashion.

This can be done in several ways such as by applying the theorem of three moments for the given loads and the live load, assuming a constant moment of inertia and a weight for the girders and then arbitrarily increasing the negative moment values by about 15% to allow for the variable moment of inertia over the interior piers sectioning the girders and figuring the weights. The corresponding positive moments will only change slightly.

It is at this point where it is believed that the record of this design will be of greatest value to anyone faced with a similar bridge problem since in the following analysis all preliminary trials are omitted and the design begins with the final design moment curves that were developed for the sidewalk side of the bridge.

Therefore, the *influence lines* developed herein and given on Figs. 23 and 24 should be especially useful for future preliminary designs since these curves are all determined from the elastic properties of the final sections and all trial computations have been eliminated.

The smooth curves on Fig. 15 are the design moment curves for the 120-foot middle span of this structure in which the variation of both the positive and negative moments have been plotted above the base line for convenience. The intersections of the negative curves at the ends with the positive curve in the central portion thus define the points of least moment

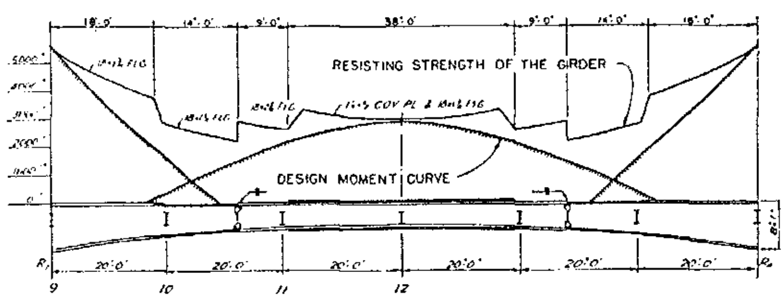


Fig. 15

and the general areas in which field splices should be located.

The upper curved and broken lines are the corresponding resisting moment strengths of the girder that have been provided in the girder sections.

The data for plotting the resisting moment of the girder is taken from a set of curves such as shown in Fig. 17.

These curves are made for the purpose by first calculating the moment of inertia of three different depths of the girder with two or three thicknesses of flange plates for each depth.

For these girders 4'-0", 6'-0" and 8'-0" depths were calculated with one-half-inch web plates and with 18" x 1", 18" x 2" and then 18" x 3" cover plates in each depth.

The results are then tabulated and plotted as the fine lines on Fig. 16. The moment of inertia is given in inches to the fourth power and the resisting moment is given in foot-pounds as shown below.

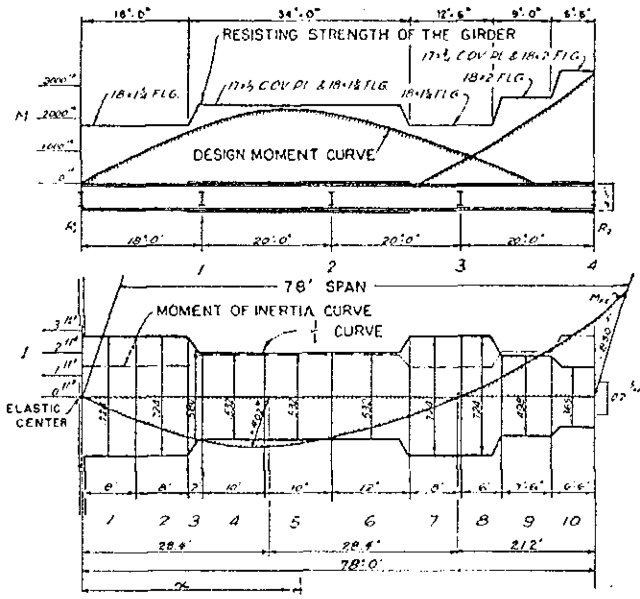
Then by the comparison of the design moment curve requirements with the *resisting strength* of various possible sections interpolated from these calculated values the designer selects a final tentative section for the girders.

Moment of Inertia (inches⁴)

½" Web Plates	4'-0"	6'-0"	8'-0"
18 x 1" Cov. pls.	23,936	59,692	115,800
18 x 2" Cov. pls.	41,649	101,300	191,450
18 x 3" Cov. pls.	57,887	140,780	264,375

Resisting Moment at 18,000#/sq. in.

	4'-0"	6'-0"	8'-0"
18 x 1" Cov. pls.	1,495,000	2,480,000	3,620,000
18 x 2" Cov. pls.	2,600,000	4,230,000	5,980,000
18 x 3" Cov. pls.	3,610,000	5,850,000	8,270,000'#s



PROPERTIES OF GIRDER								Uniform Load 1# per lin ft		Unit Concentrated Load 1# @ 1		1# @ 2		1# @ 3	
Div	ds	1/I	ds/I	x	xds/I	x²ds/I	+ I₀	M	Mxds/I	M	Mxds/I	M	Mxds/I	M	Mxds/I
1	8	.724	5.79	4	23.1	97	30.9	8	185						
2	8	.724	5.79	12	69.4	833	30.9	72	5000						
3	2	.580	1.16	17	19.7	335	.4	144	2840						
4	10	.532	5.32	23	122	2810	44.2	264	32200	5	610				
5	10	.532	5.32	33	176	5810	41.2	545	95900	15	2640				
6	12	.532	6.38	44	281	12370	76.6	967	272000	26	7300	6	1660		
7	8	.724	5.79	54	312	16830	30.9	1458	454000	36	11210	16	4990		
8	6	.724	4.34	61	265	16160	13.0	1860	493000	43	11400	23	6100	3	795
9	7.5	.498	3.74	67.75	254	17210	17.5	2290	582000	49.75	12670	29.75	7550	9.75	2480
10	6.5	.365	2.37	74.75	177	13710	8.3	2790	494000	56.75	10050	36.75	6510	16.75	2970
						85660	23.9		2431125		55830		26830		6245

HINGE @ $R_1 = \infty$
A = ∞ I = 85897

1# @ R_1
$\frac{1 \times 78 \times 78}{85897} = .0709$ Stiffness $R_1 = \frac{Mc}{I}$

Uniform Load	1# @ 1	1# @ 2	1# @ 3
$R_1 = \frac{2431125}{85897} = 28.4$	$R_1 = \frac{55830}{85897} = .650$	$R_1 = \frac{26830}{85897} = .313$	$R_1 = \frac{6245}{85897} = .0726$
R_2 = 49.6	R_2 = .350	R_2 = .687	R_2 = .9274
F E M @ R_2 = -830′#	F E M @ R_2 = -10.7	F E M @ R_2 = -4.56	F E M @ R_2 = -14.33
Pos M = 407′#	Pos M = +11.7	Pos M = +11.9	Pos M = 4.21

Fig. 17

At this same time the curvature of the girder flanges and the variation in the depth of the girders must be established in a definite manner before the final reisisting strength of the girders can be established. In both the 100-foot and in the 120-foot spans the top of the girders parallel the crown of the roadway on a 320-foot vertical curve. The top of the girder is thus established at Elevation 67.07 at the ends of the 120-foot span and at Elevation 67.35 at the center. The girder depths were then selected as 8'-0" over the piers and as 4'-0" back to back of flanges at the center of the 120-foot span so that approximately the same size flange is required in both the maximum negative and positive moment areas. These depths are also chosen so as to give a nicely proportioned appearance without increasing the roadway grade too much in the interests of the most economical depth which is somewhat deeper than four feet for the center and end spans.

The curvature of the bottom flange between these points is then taken as a parabola with a 4.28-ft. rise in a horizontal distance of 53 feet, beginning at a point of intersection 2'-0" from the center line of shoe and ending tangent to a 10'-0" straight section symmetrical about the center line of the span. Thus at ten-foot intervals from the center outward the girder depths are 4'-0", 4'-0⅜", 4'-3⅝", 4'-10½", 5'-9", 6'-10⅝" and 8'-1" respectively for the 120-foot span as measured back to back of the main flange plates of the girder. The resisting moments of the girder at these various points is then interpolated on the graph of Fig. 16 and used to construct the resisting stength of the girder as shown on Fig. 15.

The corresponding depths of the girders on the 100-foot spans at corresponding intervals are 8'-1", 6'-10⅝" and 5'-11¾" for the first twenty feet from the deep end to 4'-10" at 40', 4'-1" at 70', and 4'-0" at the shallow end. The shape of the bottom flange curve for the first 60' from the shallow end is circular with a middle ordinate of 4¾" while the remainder of the curve is parabolic from a common tangent with the circle.

The 4'-0" constant depth continues throughout the 78-foot end spans to the abutments.

Having thus assumed and established a tentative girder section to fit the preliminary design moment curves the designer is now ready to calculate the properties of these various girders, draw influence lines, determine the bending moments, shears and reactions and design the final sections.

The computations for determining the properties of the girders are shown on Figs. 17, 18, and 19 for each separate span.

The sidewalk girder of the 78-foot end spans is shown on Fig. 17. These girders are simply supported on rollers on the abutments at reaction R_1 and are continuous with the 100-foot spans over simple roller supports at R_2.

Figure 18 is similar for the sidewalk girder of the first interior 100-foot span. These girders are simply supported on rollers at R_2 and on fixed shoes at R_3 and are continuous with both the 78-foot end spans and with the girders of the 120-foot center span.

Figure 19 is for the 120-foot center span which is symmetrical about the center line of the bridge.

These three figures show the flange sections of the girders, the locations of the field splices, the final design moment curves, the resisting strength of the girder, and the general dimensions and locations of the floorbeams throughout the full length of the bridge, on the sketches at the top. The floorbeams are numbered from 1 to 12 to define their locations and the scale of the bending moment diagrams is indicated at the left using the top flange as the base of the diagrams.

Just below these figures the variation in the moment of inertia of the girder is plotted as another curve to a scale measured in foot units. The base line of the moment of inertia curve is also the centerline of another irregular figure that is plotted with a width of one divided by the corresponding value of the moment of inertia.

This figure is called the elastic area of the girder. Its width represents the relative stiffness of the various portions of the girder as measured graphically by the angle change per unit of length along the girder that is caused by applying a unit bending moment to each unit of length of the girder.

In reality, though, this area has no measurable width since all of these relative values have to be divided by "E" (the modulus of elasticity of the material) to obtain real angle changes, thus reducing the real width of the elastic area to the width of a single imaginary line for the full length.

The properties of these areas on Figs. 17, 18 and 19 are thus used only for visualizing the computations that are involved in the methods of the column analogy and of moment distribution.

They are also used, however, to determine the fixed end moments for each span for all conditions of loading by applying the corresponding statically determinate moment diagrams for the loads as a load on the imaginary elastic area.

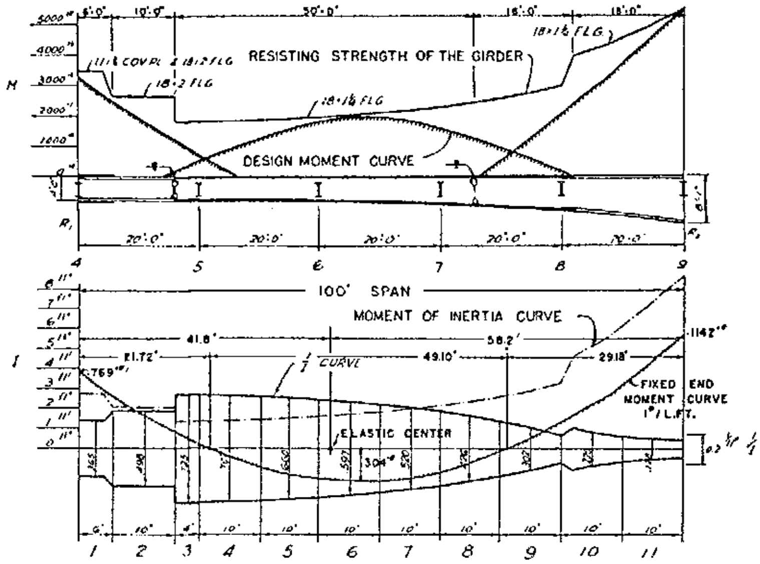
The relative stiffness and carry-over factors for each separate span and the common distribution factors for each joint are also determined from the properties of these areas.

The computations are shown in the tabulations at the bottom of Figs. 17, 18 and 19. These are explained as follows beginning with Fig. 17 for the 78-foot end span.

The area and moment of inertia of the elastic area are first computed under the heading of the properties of the girder. The divisions of this tabulation are so selected that divisions begin and end at load points and also so that the real values of "I" apply throughout each real "ds" length. Therefore the product of ds/I for each increment can be considered as an area and the distances X are measured from the centroid of the individual area to the center of gravity of the whole area.

In this case the center of gravity of the whole area is on a line through the left hinged support since the moment of inertia of a hinge is zero and one divided by zero is infinite. Thus the elastic area has an infinite width at the hinge and therefore its center of gravity must also be on a line through the hinge.

The last two columns under "properties of the girder" give the moment of inertia of the elastic area about the line of the hinge. The value of "I" of 85,897 thus calculated is also equal to the relative deflection of R_1 with respect to R_2 for a one pound reaction



Div.	ds	I	ds/I	x	x ds/I	x'	x' ds/I	x'² ds/I	+ I_o	M	M ds/I	M x' ds/I	M	M ds/I	M x' ds/I	M	M ds/I	M x' ds/I
										Uniform Load 1# per lin ft			Unit Concentrated Load $R_2 = 1$#			$R_1 = 1$#		
1	6	.365	2.19	3	6.6	-38.8	-84.9	3290	6.6	145.5	319	-17,400	3	6.6	-248			
2	10	.498	4.98	11	54.8	-30.8	-154	4750	49.8	489.5	2440	-75,100	11	54.8	-1685			
3	4	.725	2.90	18	52.2	-23.4	-69.0	1640	3.9	738.5	2100	-50,900	18	52.2	-1240			
4	10	.702	7.02	25	175	-16.8	-117.8	1980	58.5	937.5	6580	-110,400	25	175	-2940	75	527	-8850
5	10	.660	6.60	35	231	-6.8	-44.8	305	55.0	1137.5	7500	-51,000	35	231	-1570	65	429	-2920
6	10	.591	5.91	45	269	+3.2	+19.1	61	49.8	1237.5	7390	+23,600	45	269	+861	55	328	+1050
7	10	.520	5.20	55	286	+13.2	+68.7	907	43.3	1237.5	6430	+64,900	55	286	+3780	45	234	+3080
8	10	.426	4.26	65	277	+23.2	+98.9	2300	35.5	1137.5	4840	+112,100	65	277	+6430	35	149	+3460
9	10	.302	3.02	75	226	+33.2	+100.0	3300	25.3	937.5	2830	+94,000	75	226	+7510	25	75.5	+2510
10	10	.220	2.20	85	187	+43.2	+95.0	4100	18.3	637.5	1400	+60,500				15	33.0	+1430
11	10	.128	1.28	95	121	+53.2	+68.1	3620	10.7	337.5	431	+22,900				5	6.4	+340
			45.62		1985.6			26253 + 356.1 = 26610			42290	+99,200						

x̄ = 1985.6 / 45.62 = 41.8

1# @ R_1 Stiff @ R_1 CO @ R_1 — 1# @ R_2 Stiff @ R_2 CO @ R_2 — FEM

CO factor = .0691/.0916 = .795 — CO factor = .0697/.1490 = .467

FEM $_1$ = .769 — Pos M = 30

1# @ 5: R_2 = .8, R_1 = .2 — FEM R_2 = 4.57 — Pos M = 4.3

Fig. 18

applied upward at R_1 and resisted at R_2, since "x" equals "Ms" at the same point for a one pound upward reaction applied at the hinge, when the span is held against rotation at R_2,

and therefore $\frac{x^2 ds}{I}$ equals $\frac{M \times ds}{I}$ for one pound V.

Therefore the magnitude of R_1 can be determined for any loading condition by first determining the free deflection of R_1 with the load cantilevered from R_2 and then dividing this value by 85,897 to find out how many times the unit reaction would have to be applied to push the free deflected condition back to zero deflection at R_1.

Thus

$$R_1 \int \frac{x^2 ds}{EI} + \int \frac{Ms \times ds}{EI} = 0$$

$$R_1 = -\frac{\int \frac{Ms \times ds}{EI}}{\int \frac{x^2\, ds}{EI}} = -\frac{\int \frac{Ms \times ds}{I}}{\int \frac{x^2\, ds}{I}} \quad \text{when "E" is a constant}$$

The final moments for the beam as fixed, at R_2 are then determined by adding the value of R_1 to the cantilever moment diagram. Thus

$M = Ms + R_1x$ for the fixed end moment curve with the beam fixed at R_2 and hinged at R_1.

The next two columns on Fig. 17 are the determination of the free deflection of R_1 with respect to R_2 for a uniform load of one pound per linear foot using the free cantilever diagram with all the load cantilevered from R_2.

The calculated value of this deflection is 2,431,125 which if divided by "E" would be a real value of the deflection as measured in feet.

In the tabulation it is visualized as being obtained by using the cantilever moment diagram as a load on the analogous column area, writing moments about the elastic center of the area.

From these values R_1 is found to be 28.4 lbs., R_2 = 49.6 lbs. and the fixed end moment at R_2 is — 830 foot-lbs. The maximum positive moment occurs 28.4 ft. from R_1 at the point of zero shear and is equal to +402 foot-pounds.

The relative stiffness of the end span girder against rotation at R_2 is calculated as the fiber stress of .0709 lbs. per sq. ft. that would be caused by placing a unit load on the elastic area at R_2. This value is used in combination with a similar figure obtained from the second span to determine the distribution factors for the common joint at R_2, for the final distribution of any unbalanced fixed end moments.

The last six columns on Fig. 17 under the heading of "Unit Concentrated loads" similar data is given for three positions of a unit load, in positions over floorbeams 1, 2 and 3. In each of these cases the static moment diagram is taken as a free cantilever beam fixed and supported at R_2. Then R_1 is determined by making the free deflection of R_1 be equal to zero and the values of +M, —M and R_2 follow from statics as tabulated.

Figure 18 is similar to Fig. 17 except that it is for the 100-foot interior spans and the span is considered

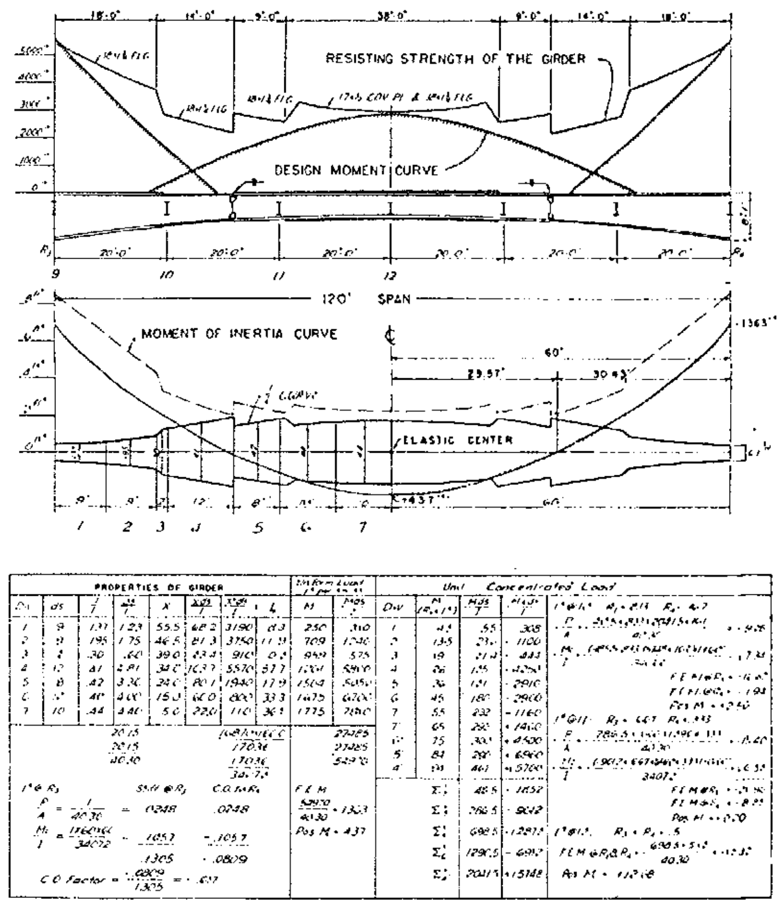


Fig. 19

as fixed at both ends. This assumption makes the moment of inertia infinite at each end and the width of the elastic area becomes zero at both ends.

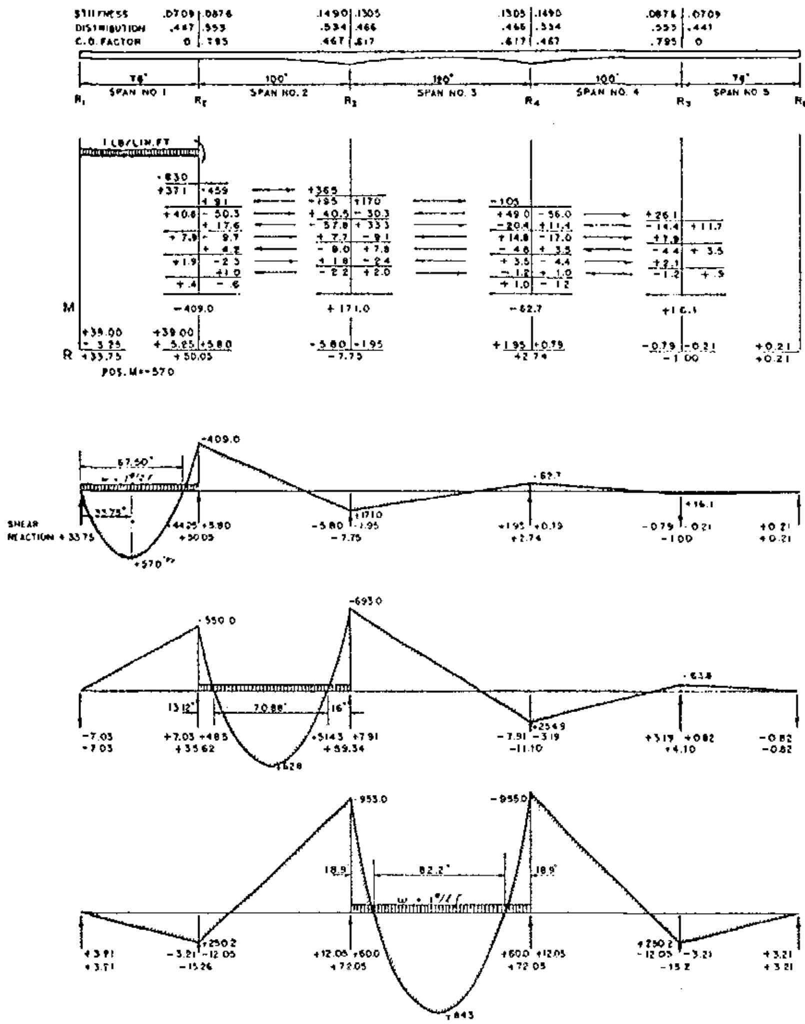
Therefore the area becomes definite and the elastic center or center of gravity of the figure is located 41.8 feet from the left end of the girder. The relative area of the 1/I figure is equal to 45.62 sq. ft. and the moment of inertia of the figure is 26,610 ft.[4] about the elastic center.

The value of 45.62 corresponds to the angle change that would be caused throughout the length of the girder by a unit moment applied at the ends and the value of 26,610 ft.[4] corresponds to the vertical deflection that would be caused by transferring a one-pound vertical reaction from R_1 to R_2 while the ends of the girder are fixed against rotation. They would be real values if they were divided by the modulus of elasticity of the material.

These properties of the girder are used to determine the relative stiffness and carry over factors for the girder. This is done by applying a unit rotation to the end of the girder and considering it as a unit eccentric load on the elastic area.

The stiffness at the end where the load is applied corresponds to the stress as computed from the formula $f = \frac{P}{A} + \frac{Mc}{I}$. The proportion of the applied moment that carries over to the far end is $f = \frac{P}{A} - \frac{Mc}{I}$. These values are given as .0876 for the stiffness at R_2 and —.0697 as the stress at the far end on Fig 18.

Therefore the carry over factor from R_2 to R_3 is —.795 of the distributed moment from the shallow to the deep end as computed on Fig. 18.



MOMENT CURVES, SHEARS, & REACTIONS FOR VARIOUS POSITIONS OF UNIT UNIFORM LOAD

Fig. 20

The corresponding stiffness factor of .149 for the 100-foot span at R_2 is found by placing the unit load at R_2 and the carry over factor is —.467 for moments distributed from the deep end at R_2 to the shallow end at R_2.

The computations for the fixed end moments for a uniform load of one pound per linear foot on the 100-foot span are shown on the next three columns of Fig. 18. The values of "M" are first calculated as if the girder is a simple beam. Then these simple beam angle changes $\frac{M\ ds}{I}$ are used as loads on the elastic area. The tabulation of $\frac{Mx'ds}{I}$ is thus visualized as the moment of these loads about the elastic center whereas in reality it is the vertical deflection of R_2 with respect to R_1 for the given moment diagram.

Then by applying

$$f\frac{P}{A} = \pm\frac{Mc}{I}$$

in the column analogy the fixed end moments at R_2 and R_3 are determined as —769 and —1142 foot-pounds for the given uniform load.

The values for the unit concentrated load in positions 5, 6, 7 and 8 are also determined from simple beam moment diagrams that are tabulated for unit reactions on Fig. 18.

Thus if a one-pound load is placed at point 5 the simple beam reactions are .8 lb. at R_2 and .2 lb. at R_3. Therefore the angle changes $\frac{M\ ds}{I}$ and deflections $\frac{M\ x\ ds}{I}$ are obtained by taking .8 of the values between R_2 and 5 for 1 lb. R_2 and adding them to .2 of the values between 5 and R_3 for one pound at R_3.

Likewise the values for point 6 are obtained by taking .6 of the values between R_2 and 6 and combining them with .4 of the values between 6 and R_3, since a unit load at point 6 causes a .6 lb. reaction at R_2 and .4 lb. at R_3.

Then by using these values as loads and moments on the elastic area the fixed end moments are obtained from $\frac{P}{A} \pm \frac{Mc}{I}$ as tabulated in the lower right hand corner of Fig. 18.

Likewise, on Fig. 19 the stiffness and carry over factors are given as .1305 and —.617 for the properties of the 120-foot span girders. The fixed end moment for a one-pound per linear foot uniform load is —1363 foot-pounds and the values for the unit concentrated load in positions 10, 11 and 12 are calculated.

In this case all values can be taken by proportion from the values of one pound reaction at R_3 due to symmetry. The required computations for the 120-foot span are shown on Fig. 19 to complete the determination of all the required properties and values of fixed end moments.

Then as the next step the final bending moment curve for the full length of the five span continuous girder is found for each condition of loading by the method of distributing the unbalanced moment at any joint in proportion to the stiffness factors of the joint beginning with the fixed end condition and continuing until all joints are balanced.

The method of moment distribution is illustrated at the top of Fig. 20 for a load of one pound per linear foot. The stiffness and carry over factors are taken from the previous computations on Figs. 17, 18 and 19. The distribution factors for each beam are obtained by dividing the stiffness factor of the beam by the sum of the stiffnesses of all members entering the joint. Thus the distribution factor for the 78-foot span at R_2 is

$$\frac{.0709}{.0709 + .0876} = .447.$$

Likewise it is $\frac{.0876}{.0709 + .0876} = .553$ for the 100-foot span at R_2.

The sum of all the distribution factors at any joint must always add up to one.

The process begins with the unbalanced fixed end moment of —830 ft. lbs. on the 78-foot span side of the joint at R_2. This 830 foot-pounds is then distributed in the ratio of .447 x 830 = 371 foot-pounds to the 78-foot span and .553 x 830 = 459 foot-pounds to the end of the 100-foot span. Thus the end moment of the 78-foot span reduces from —830 foot-pounds to —459 foot-pounds by being relieved of 371 foot-pounds of moment while the end moment of the 100-foot span builds up from zero to —459 ft. lbs. on the other side. Thus joint R_2 is balanced but —.795 x —459 ft. lbs. or 365 ft. lbs. of moment carries over to the other end of the 100-foot span at R_3 and causes joint R_3 to be unbalanced.

As the second step the 365 ft. lbs. at R_3 is distributed in the ratio of .534 x 365 = —195 ft. lbs. to the 100-foot span and .466 x 365 = +170 ft. lbs. to the 120-foot span. The end of the 100-foot span is thus relieved from 365 by —195 to +170 ft. lbs. while the end of the 120-foot span builds up from zero to 170 ft. lbs. and the joint at R_3 is balanced but in this process —.467 x —195 or +91 ft. lbs. carries back to the other end of the 100-foot span at R_2 while —.617 x 170 or —105 ft. lbs. carries on to the far end of the 120-foot span at R_4.

This causes both R_2 and R_4 to be unbalanced but in a much smaller amount than the original —830 ft. lbs. at R_2 and the process is converging toward a final answer.

As the third step joints R_2 and R_4 are balanced. At R_2 the 91 ft. lbs. reduces by 50.3 to +40.8 ft. lbs. while +40.5 carries back to R_3 and the —105 ft. lbs at R_4 reduces by 49 to —56.0 lbs. while —30.3 carries back to R_3 and +26.1 carries ahead to R_5.

As the fourth step joints R_3 and R_5 are balanced and this time only +17.6 ft. lbs. goes back to R_2 and so on the process continues until the carry over portions as indicated by the arrows on Fig. 20 become of insignificant amounts.

The final values of the balanced moments at the reactions are found to be —409 at R_2, +171. at R_3, —62.7 at R_4 and +16.1 ft. lbs. at R_5 by simple addition.

The 39-pound simple beam reactions on the end span are then modified by the moment at R_2 divided by the length of span (—409 ÷ 78' = —5.25 at R_1 and + 5.25 at R_2) to obtain the 33.75-pound reaction at R_1 and the 44.25 pound shear on the 78-foot span side of R_2.

The shear on the 100-foot span side is obtained by the general formula

$$\frac{M_2 - M_3}{l_2} \quad \text{or} \quad \frac{-409 - 171}{100} \quad \text{which}$$

equals —5.80 lbs. at R_2 and +5.80 lbs. at R_3. This shear is then combined with the 44.25 pounds shear on the other side to give the 50.05 lb. reaction at R_2.

As soon as the magnitude of the end reaction at R_1 is known the point of maximum positive moment in the end span is located at a point the same distance away from the end reaction as the magnitude of the reaction in pounds, since this is the point of zero shear for a uniform loading of one pound per linear foot.

The maximum value of the positive

$$\text{moment is } 33.75 \times \frac{33.75}{2} = +570 \text{ ft.}$$

lbs. at a point 33.75 feet from the end reaction. The inflection point in the end span is twice this distance from the end reaction or 67.50 feet from R_1 and the rest of the moment curve is a series of straight lines throughout the length of each consecutive span.

The final moment curve shown on Fig. 20 for span No. 2 is determined in the same way for a uniform load of one pound per linear foot. Beginning with the full fixed end moments of —769 ft. lbs. at R_2 and —1142 ft. lbs. at R_3 as determined on Fig. 18 and by following through this same process of distribution.

Again the point of maximum moment occurs 48.57 feet from R_2 and

$$\text{is equal to } (48.57 \times \frac{48.57}{2}) - 550 \text{ or}$$

+628 ft. lbs. Then setting this value

$$\text{equal to } \frac{w\,l^2}{8} \text{ for the simple beam}$$

moment between inflection points and solving for l the distance between inflection points is found to be 70.88 feet. Therefore the first inflection

$$\text{point is } 48.57 - \frac{70.88}{2} \text{ or } 13.12 \text{ feet}$$

from R_2 and the second one is 16 feet from R_3. Otherwise these same distances can be solved for as the point where $M = 0$ in terms of "x" beginning with the negative value of the bending moment at the reactions. Thus

$$-550 - \frac{w\,x^2}{2} + 48.57\,x = 0 \text{ from}$$

which x = 13.12 feet.

Likewise the final bending moment curve for one pound load per linear foot on the center span is determined as shown on Fig. 20.

From these three curves in combination with the first two curves reversed for loads on the fourth and fifth spans the following "Summary of Design Coefficents for Unit Uniform Loads" is tabulated for various possible combinations of uniformly distributed loads for both moment and shear at all critical sections and for the reactions.

The tabulated summary of design coefficients for unit uniform loads are all obtained from the three moment curves on Fig. 20 of plate 120. The summations of corresponding values of the same sign give the maximum positive and negative coefficients for equivalent uniform live loads while the total summations with all spans loaded simultaneously apply to the uniform dead loads.

When multiplied by the real values of the uniform loads these values give design values for the corresponding sections of the main girders at each floorbeam connection.

Figure 21 is a detail of the connection of a typical intermediate stiffener to the web and flanges of the main girders.

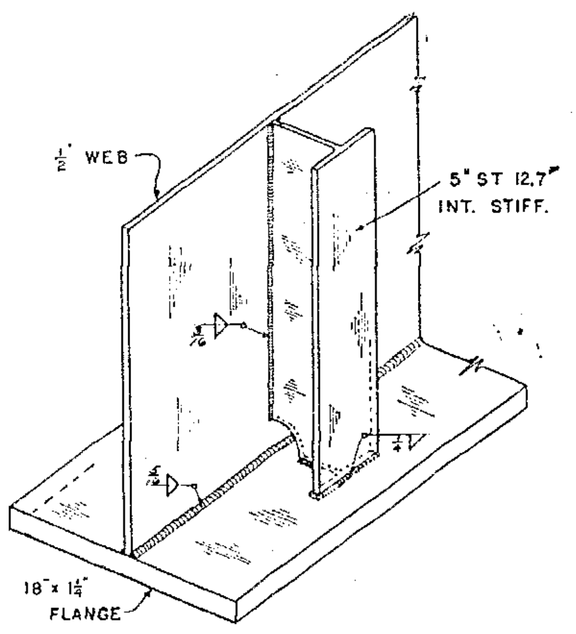


Fig. 21

SUMMARY OF DESIGN COEFFICIENTS FOR UNIT UNIFORM LOADS

Location of Load	R_1	M_1	M_2	M_3	V_{R_2L}	$M_4^{R_2}$	R_2	V_{R_2R}	M_5	M_6
Span 1	+ 33.75	+445	+561	+274	+ 44.25	—409.0	+ 50.05	+ 5.80	—293.0	—177.0
Span 2	— 7.05	—127.0	—268	—409	+ 7.05	—550.2	+ 55.62	+ 48.57	+222	+591.6
Span 3	+ 3.21	+ 57.8	+122	+186	— 3.21	+250.2	— 15.26	— 12.05	+ 9.2	—231.8
Span 4	— 0.82	— 15.7	— 31.1	— 47.5	+ 0.82	— 63.8	+ 4.01	+ 3.19	0	+ 63.7
Span 5	+ 0.21	+ 3.7	+ 7.84	+ 12.0	— 0.21	+ 16.1	— 1.00	— 0.79	+ .3	— 15.3
Max. +	+ 37.17	+506.5	+680.8	+472	+ 52.12	+266.3	+109.7	+ 57.56	+231.5	+655.3
Max. —	— 7.87	—141.7	—299.1	—456.5	— 3.42	—1023.	— 16.26	— 12.84	—293.0	—424.1
All Spans	+ 29.30	+364.8	+381.7	+ 15.5	+ 48.70	—756.7	+ 93.44	+ 44.72	— 61.5	+231.2

	M_7	M_8	V_{R_3L}	$M_9^{R_3}$	R_3	V_{R_3R}	M_{10}	M_{11}	M_{12}
Span 1	— 66.0	+ 55.0	— 5.80	+171.0	— 7.75	— 1.95	+131.9	+ 92.9	+ 53.8
Span 2	+561.5	+131.0	+ 51.43	—693.0	+ 59.34	+ 7.91	—535.0	—377.0	—219.0
Span 3	—472.9	—713.9	+ 12.05	—955.0	+ 72.05	+ 60.0	+ 45.0	+645.0	+845.0
Span 4	+127.5	+191.2	— 3.19	+254.9	— 11.10	— 7.91	+ 97.0	— 61.0	—219.0
Span 5	— 31.1	— 46.9	+ 0.79	— 62.7	+ 2.74	+ 1.95	— 23.7	+ 14.8	+ 53.8
Max. +	+689.0	+377.2	+ 64.27	+425	+134.13	+ 69.86	+273.9	+752.7	+952.6
Max. —	—570.0	—760.8	— 8.99	—1710.7	— 18.85	— 9.86	—558.7	—438.0	—438.0
All Spans	+119.0	—383.6	+ 55.28	—1285.7	+115.28	+ 60.00	+284.8	+314.7	+514.6

Figure 22 shows the results obtained by distributing the fixed end moments for ten separate positions of the unit concentrated load. Each curve is a series of straight lines between the points of maximum moment and the values of shear and reactions are also tabulated below each moment curve.

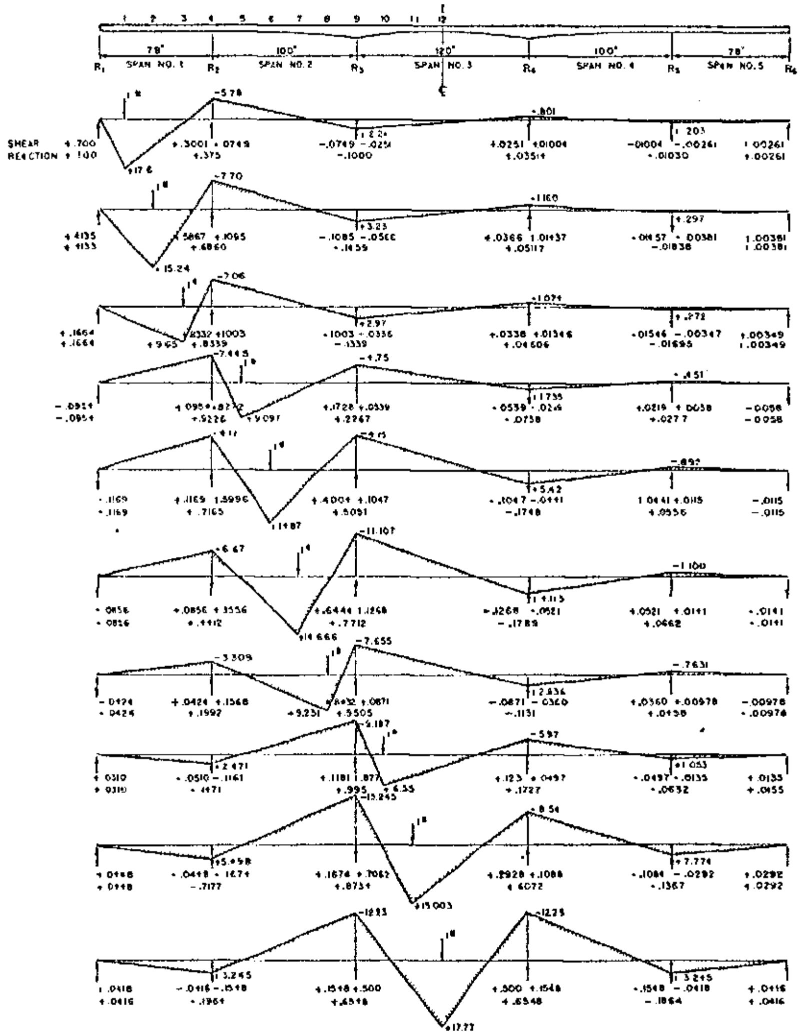
By reversing the values of Fig. 22 corresponding moment curves are obtained for all positions of the unit load on the other half of the bridge and thus complete data is obtained for the construction of any desired influence line.

The same data, of course, can be obtained from Fig. 22 without reversing the curve by simply reading the values down the loaded end of the bridge and back up the unloaded side for the reversed loading values at corresponding locations.

In this manner the following "Tabulation of Influence Line Coefficients for Concentrated Loads" is obtained for design purposes.

Partial graphs of the influence lines for the reactions are shown on Fig. 23 while the influence lines for shear adjacent to the interior reactions are indicated by the dotted lines.

Figure 24 shows a similar series of influence lines for moment at the various load points. These curves are useful for locating the load divide points for maximum positive and negative moment values at the floor beam connections nearest the reactions and otherwise indicate maximum loading conditions.



MOMENT CURVES, SHEARS, & REACTIONS FOR VARIOUS POSITIONS OF A UNIT LOAD

Fig. 22

TABULATION OF INFLUENCE LINES FOR CONCENTRATED LOADS

Load at	R	M	M	M	M	V L	R	V R	M	M	M	M	M	V L	R	V R	M	M	M
1	+.700	+12.6	+6.67	+.68	−5.28	−.3001	+.3550	−.0719	−3.58	−1.88	0	−1.52	−2.21	−.0519	−.1000	−.0251	+1.71	+1.21	+.71
2	+.4133	+8.21	+15.21	+3.77	−2.50	+.5867	+.0960	−.1093	−5.51	−3.32	−1.14	−1.65	−3.23	−.1093	−.1459	−.0366	+2.50	+1.75	+1.04
3	+.1664	+3.00	+6.32	+9.65	−7.06	+.8332	+.0339	+.1080	−5.06	−3.00	−1.62	−.97	−2.95	−.1080	−.1339	−.0306	+2.30	+1.02	+.95
4	0	0	0	0	0	0	+1.0000	0	0	0	0	0	0	0	0	0	0	0	0
5	−.0954	−1.720	−3.53	−5.51	−7.415	+.954	+.9226	+.8272	+9.997	−5.67	−2.19	−1.12	−1.73	+.1725	+.2267	+.0729	+3.65	+2.57	+1.15
6	−.1169	−2.105	−4.44	−6.78	−9.11	+.1169	+.7165	+.5996	+2.261	−11.855	−6.012	−1.551	−9.15	+.4001	+.5051	+.1017	+7.05	+4.96	+2.96
7	−.0856	−1.54	−3.25	−4.89	−6.67	+.0856	+.4112	+.3556	+.080	−6.55	−11.666	−1.78	−11.16	+.6411	+.7712	+.1268	+8.57	+6.01	+5.50
8	−.0124	−.764	−1.61	−2.46	−3.309	+.0124	+.1992	+.1568	−.130	+5.05	+6.23	+9.251	−7.633	+.8132	+.9306	+.0871	+5.89	+4.15	+2.10
9	0	0	0	0	0	0	0	0	0	0	0	0	0	0	+1.000	0	0	0	0
10	+.0310	+.559	+1.180	+1.800	+2.421	−.0310	−.1471	−.1161	+.099	−2.223	−4.511	−6.866	−9.187	+.1161	+.993	+.877	+8.35	+5.9	+3.14
11	+.0448	+.807	+1.71	+2.60	+3.498	−.0448	−.2122	−.1674	+.119	−3.395	−6.516	−9.85	−13.215	+.1674	+.8731	+.7062	+.88	+15.003	+9.115
12	+.0416	+.750	+1.58	+2.42	+3.245	−.0416	−.1964	−.1548	+.250	−2.515	−5.810	−8.955	−12.25	+.5518	+.6548	+.5000	−2.23	+7.77	+17.77
11'	+.0292	+.1525	+1.11	+1.69	+2.511	−.0292	−.1367	−.1084	+.111	−2.052	−4.915	−6.478	−8.51	+.1081	+.4022	+.2928	−2.66	+3.23	+9.115
10'	+.0135	+.243	+.513	+.781	+1.053	−.0135	−.0632	−.0195	+.058	−.935	−1.932	−2.925	−3.92	+.0197	+.1727	+.125	−1.47	+.98	+3.41
9'	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
8'	−.00978	−.176	−.372	−.566	−.7631	+.0098	+.0158	+.0060	−.013	+.677	+1.317	+2.117	+2.806	−.0360	−.1101	−.0871	−1.10	+.66	+2.16
7'	−.0141	−.254	−.535	−.816	−1.100	+.0141	+.0662	+.0521	−.057	+.986	+2.028	+3.061	+1.105	−.0521	−.1789	−.1268	−1.58	+.96	+3.50
6'	−.0115	−.206	−.435	−.664	−.892	+.0115	+.0556	+.0111	−.030	+.882	+1.600	+2.566	+3.12	−.0111	−.1747	−.1017	−1.33	+.75	2.86
5'	−.0058	−.104	−.220	−.335	−.451	+.0059	+.0275	+.0219	−.011	+.125	+.860	+1.751	+1.735	−.0219	−.0758	−.0559	−.67	+.41	1.49
4'	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
3'	+.00319	+.063	+.133	+.202	+.272	−.00315	−.01695	−.01516	+.005	−.265	−.535	−.801	−1.051	+.01516	+.01506	+.0536	−.10	+.28	+.95
2'	+.00281	+.069	+.145	+.221	+.295	−.00281	−.01038	−.01157	+.086	−.281	−.575	−.867	−1.160	+.01137	+.05137	+.0366	−.42	+.31	+1.01
1'	+.00261	+.049	+.100	+.151	+.205	−.00261	−.01090	−.01003	+.003	−.198	−.399	−.600	−.801	+.01001	+.00514	+.0251	−.30	+.21	+.71

They can also be used for the construction of a preliminary design moment curve for other similar girders of different span lengths. For concentrated loads the design coefficients for moment vary directly in proportion to the load and to the ratio of the span lengths and in the ratio of the area under the curve times the ratio of the span lengths squared for uniform loads.

For shears and reactions the ordinates of Fig. 23 can be used directly at corresponding points for any length of span to obtain approximate preliminary design values. The influence lines are simply stretched out or shortened up to fit the required lengths of the new series of spans.

As soon as the influence lines are determined the designer is ready to apply the design loads to the girder and determine the final values of the bending moment at all critical sections for which the girder must be designed.

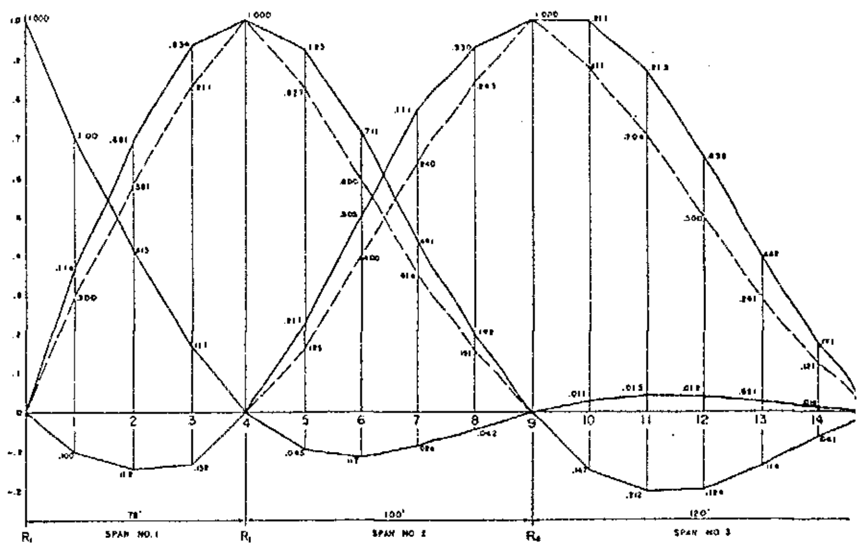
The design moment curves shown on Figs. 17, 18 and 19 are determined as follows:

First the average weight of the main girder is estimated for each span by multiplying the length times the weight per foot of each item and applying a small percentage for variations, such as 5% for welds and variations.

Thus the Estimated Weight of One Girder is:

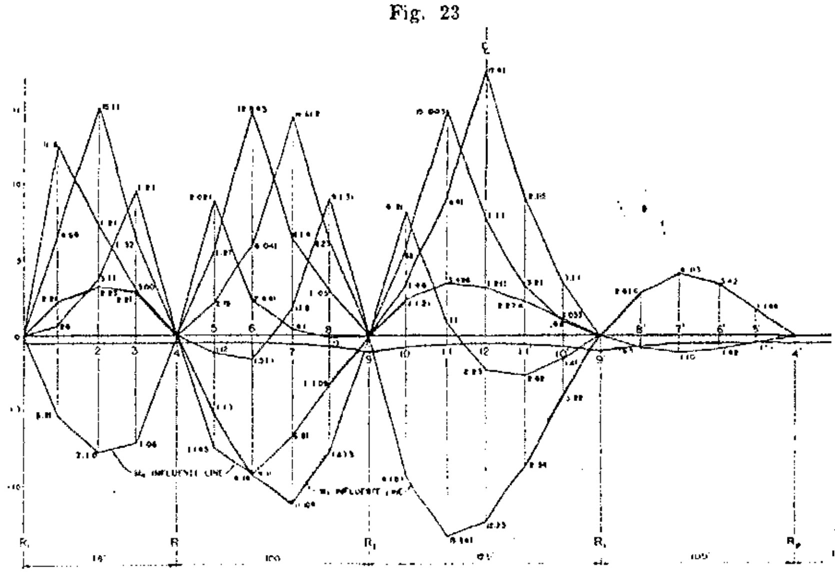
Two 78' End Spans at 24,160	=	48,320 lbs.
Two 100' Spans at 31,000	=	62,000 lbs.
One 120' Span at 43,630	=	43,630 lbs.
Total		153,950 lbs.

÷ 476 ft. = 323 lbs./lin. ft. of one girder



INFLUENCE LINES FOR REACTIONS

Fig. 23



MOMENT INFLUENCE LINES

Fig. 24

The average weight of the girder is then added to the previously calculated weight of 1734 lbs. per linear foot of girder to obtain a total dead load of 2,067 lbs. per lin. ft.

This figure multiplied by the panel length of 20 feet gives a total panel load of 41,320 lbs. per floorbeam whereas the actual figured bracket and floorbeam reaction is only 22,650 lbs. for dead load. Therefore, the difference of 18,670 lbs. is a uniformly distributed dead load of 933 lbs./lin. ft. which is applied directly to the girders by the slab.

The live loads on the girder consist of two ten-foot lanes of H2O-S16 equivalent uniform live loads placed eccentrically on the roadway adjacent to the curb, plus the effect of the sidewalk live loads and impact.

The two lanes of H20-S16 in the eccentric position are equivalent to 1.2 lanes per girder or 768 lbs. per lin. ft.

$$2 \times 640 \times \frac{12.125}{20.25'} = 768 \text{ lbs./lin. ft.}$$

The concentrated live load for moment therefore is 1.2 x 18,000 = 21,600 lbs. and for shear is 1.2 x 26,000 = 31,200 lbs.

A live load of 60 lbs./sq. ft. on the five-foot cantilevered sidewalk causes 380 lbs./lin. ft. on the girders for single span loading conditions.

$$60\# \times 5' \times \frac{25'\text{-}7\tfrac{1}{2}''}{20'\text{-}3''} = 380 \text{ lbs./lin. ft.}$$

When the loaded length is greater than 100 feet the following formula was used to determine the intensity of the live load on the sidewalk in pounds per square foot.

$$P = 30 + \frac{3000}{L}$$

in which "L" is the loaded length.

The formula $I = \frac{50}{L + 125}$ was used for impact to make an allowance for the dynamic effect of the moving live load. Thus the impact factors were 25%, 22.2% and 20.4% for the 78', 100' and 120' spans respectively.

Then with these loads and the influence lines or the tabulations of moment coefficients the values of the bending moments are calculated as follows:

The dead load weight factor of 2067 lbs./lin. ft. multiplies by the unit uniform load coefficients for all spans loaded for values in the 78-foot and 100-foot spans, while a weight factor of 2120 lbs./lin. ft. is used in the 120-foot span to allow for the greater weight of the girders.

DESIGN MOMENTS IN FOOT KIPS FOR THE MAIN SIDEWALK GIRDERS

Span No. 1 = 78' Span (Sidewalk side)

	$+M_1$	$+M_2$	$+M_3$	$-M_3$	$-M_4$	$+M_4$
Dead Load—2067#/l.f.	+758	+784	+38	+38	—1539	—1539
U.L.L. 768#/l.f.	389	522	368	—350	—786	+204
C.L.L. 21600/31200	272	329	208	—141	—363	+ 76
Impact 25%	165	213	143	—123	—288	+ 70
Sidewalk 380#/l.f.	192	258	179	—173	—389	+101
Design	+1776	+2106	+931	—749	—3365	—1088

Span No. 2 = 100' Span (Sidewalk side)

	$-M_5$	$+M_5$	$+M_6$	$+M_7$	$+M_8$	$-M_8$	$-M_9$	$+M_9$
D.L. 2066#/l.f.	—127	—127	+476	+245	—789	—789	—2650	—2650
Sidewalk 380#/l.f.	—111	+ 88	+248	+264	+143	—289	—650	+161
U.L.L. 768#/l.ft.	—225	+177	+503	+530	+290	—585	—1313	+326
C.L.L.= 21.600	—119	+196.5	+295	+317	+199	—213	—525	+ 89
Impact 22.2%	— 76	+ 83	+177	+188	+131	—177	—408	+ 92
	—658	+417.5	+1699	+1544	— 26	—2053	—5546	—1982

Span No. 3 = 120' Span

	$+M_{10}$	$-M_{10}$	$+M_{11}$	$+M_{12}$
D.L. 2120#/ft.	— 604	— 604	+ 665	+1090
Sidewalk 350#/l.f.	+ 96	— 195	+ 263	+ 333
U.L.L. 768	+ 210	— 428	+ 577	+ 731
C.L.L. 21600/31200	+ 181	— 152	+ 324	+ 383
Imp. 20.4%	+ 80	— 108	+ 184	+ 227
Total	+1171	—1387	+2013	+2764

Design Values for Reactions and Maximum Shears

	R_1	V_2L	R_2	V_2R	V_3L	R_3	V_3R
D.L. 2120#/ft.	+61.0	+99.6	+195.0	+93.3	+117.0	+244	+127.3
Sidewalk 380	14.1	19.8	41.6	21.9	24.4	47.0	24.4
U.L.L. 768	28.6	39.9	83.6	44.0	49.1	102.5	53.2
C.L.L. 31200	31.2	31.2	31.2	31.2	31.2	31.2	31.2
Imp. 20.4%	15.0	17.8	25.5	16.7	17.8	27.3	17.4
	+149.9	+208.3	+376.9	+207.1	+239.5	+452.0	+253.5

Both the 768 lb./lin. ft. uniform live load on the roadway and the 380 lb./lin. ft. on the sidewalk multiply by the maximum positive or negative uniform load coefficients while the influence line ordinates are used for the concentrated live load.

These values of moment are the ones that are plotted on Figs. 17, 18 and 19 and they are used in combination with the resisting moment and moment of inertia curves on Fig. 16 to determine the final sections of the girder.

The thickness of web plate is governed by the 208,300 lb. shear in the end span adjacent to the first interior reaction R_2. This requires 19 sq. inches of web at 11,000 lbs. per sq. in. and the 44 x ½" web plate at this section provides 22 sq. inches. At other critical sections the web is either much deeper or the shearing stress is smaller so the one-half-inch thickness of web plate is used throughout the full length of the bridge.

Intermediate 5" at 12.7 lb. tee section vertical stiffeners are provided on the inside surface of the web plate to prevent the web from buckling. These are needed the most where the web has the greatest depth and where the shear is greatest adjacent to the reactions over piers two and three. The maximum values of shear at these points are 253,500 lbs. on the 120-ft. span side and 239,500 lbs. on the 100-foot span side of R3. Since the girders are 8'-1" back to back of the 18" x 1¾" flanges, the clear distance between flanges is 93.5 inches and the one-half inch web plate meets the minimum requirement of $1/20 \sqrt{93.5} = .48''$.

The spacing of the intermediate stiffeners is determined from the formula $d = \dfrac{9000\ t}{\sqrt{v}}$ in which

d = the clear distance between stiffeners in inches

t = the thickness of the web plate in inches

v = the unit shearing stress in the web,

The value of the unit shearing stress is determined from the usual formula.

Section	A	y	Ay	Ay²	+ I_o
5" Tee at 12.7# =	3.69	3.8	14.05	53.5	7.81
10" x ½" web pl. =	5.00	—.25	—1.25	.31	
	8.69		12.80	61.42	

$$\text{c.g.} = \frac{12.80}{8.69} = 1.47'' \times 12.80 = -18.90$$

$$I = 42.52$$

$$\text{Section Modulus} = \frac{42.52}{3.53} = 12.1$$

$$v = \frac{V\ m}{I\ t} \text{ in which}$$

V = the external shear at the section

m = the statical moment about the neutral axis of the portion of the girder section that lies beyond the horizontal section that is being investigated for shear.

I = the moment of inertia of the whole girder.

At this point the maximum intensity of the shearing stress between the web and the 18 x 1¾" flange is

$$v = \frac{253{,}500 \times 1500}{176{,}800 \times .5} = \underline{\underline{4300}} \text{ lbs./sq. in.}$$

At the centerline of the web the unit shearing stress is

$$v = \frac{253{,}500 \times 2045}{176{,}800 \times .5} = \underline{\underline{5900}} \text{ lbs./sq. in.}$$

From this the maximum spacing of the intermediate stiffeners is found to be 58 inches at this point.

$$d = \frac{9000 \times .5}{\sqrt{5900}} = 58$$

The details of an intermediate stiffener are shown on Fig. 21. It consists of half of a standard 10" I at 25.4 lbs. welded to one side of the web plate. It acts as an I-beam in conjunction with part of the web plate and is about twice as strong as an ordinary two-angle pair of stiffeners.

Counting only ten inches of the web plate as flange the moment of inertia of one of these stiffeners is 42.5 calculated as shown above:

This compares to a section modulus of 5.8 for a pair of 2 angles 5 x 3 x 5/16 stiffeners and the Tee flange is much safer against a buckling failure.

The Tee flange can also be safely welded to the girder flange since it runs parallel to the flange stresses rather than only at right angles to the flange as in the case of ordinary plate or angle stiffeners. The inside corner of the tee is notched on a two-inch radius in the corner of the flange to avoid causing triaxial conditions of stress and the subsequent loss of ductility at the junction of the stiffener with the web and flange weld.

The main bearing stiffeners over piers two and three, as shown in Fig. 25, are also made of 7" at 37 lbs. structural tees. These tees are each half of a 14" x 10" WF at 74 lbs. H-beam column section so that when welded back together again on opposite sides of the web these two tees form a regular H-section column that bears against the bottom flange as the stiffener in combination with the web plate which is also in bearing over the reaction.

Fig. 25

The maximum required bearing area over pier two at 27,000 lbs. per sq. in. for the 452,000 lb. reaction is 16.8 sq. inches. The maximum section required as a column at 15,000 lbs./sq. in. is 30.2 sq. inches and the two stiffeners acting in combination with 18 inches of the ½-inch web plate provide 30.76 sq. inches.

At piers 1 and 4 and at the abutments 6" x 1" and 6" x ½" plates are used as bearing stiffeners on account of the shallow depth of the girders and for the floorbeam connections. Likewise 6" x ½" connection plates were used as stiffeners at all other floorbeam and bracket connections.

The design of the flange to web welding is also governed by shear and the minimum sizes of welds that are practical for the relative sizes and thicknesses of the parts.

The unit is also powered by a suitable tractor unit of its own that is set at the proper traveling speed that corresponds to the rate of welding and travels at about 36 inches a minute more or less.

As the unit moves along the flux adjacent to the arc melts and floats on the surface of the molten metal then solidifies as a slag on top of the weld. This blanket of flux protects the molten metal from contact with the air and assures the maximum quality of the welded metal.

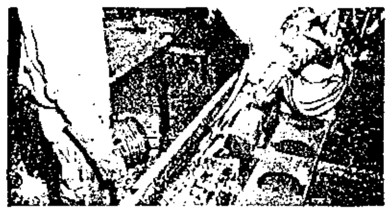


Fig. 26

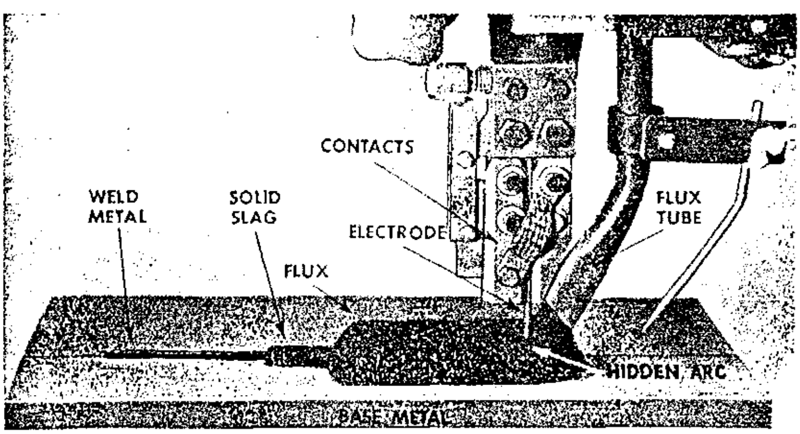


Fig. 27

These welds were all made by the automatic process as shown in Fig. 26. The essentials of the hidden arc process are as shown in Fig. 27. Granular flux is deposited on the joint deep enough to cover the completed weld. Then a bare metallic welding electrode is power-fed into the blanket of flux at a rate of feed controlled automatically by the proper arc length. Direct current, supplied by a motor generator welder, produces the arc between the electrode and the joint and the resultant arc heat fuses both the electrode and the parent metal to produce the welded joint.

To meet the maximum requirements for shear it is sufficient to determine the intensity of horizontal shear adjacent to the reactions and combine these values with the vertical load on the top flange.

At R_1 the value of external shear is 208,300 lbs., I = 56,750"⁴ and m = 1140 for the four foot girder with 18" x 2" flange plates and with one 17" x ¾" cover plate. Therefore the horizontal shearing force is 4200 lbs. per linear inch between the top flange and the web.

$$v = \frac{208{,}300 \times 1140}{56{,}750} = 4200$$

The direct load on the top flange is approximately 1390 lbs. per inch consisting of dead load, live load and impact.

D.L. = 650#/lin. ft.	÷ 12	=	54
H-20 Live Load 16,000#	÷ 18"	=	890
Impact 50%		=	446
			1390# lin. inch

Resultant $= \sqrt{4200^2 + 1390^2} = \underline{\underline{4400}}$ lbs. per lin. inch

This requires a minimum of two ¼-inch fillet welds with a design capacity of 4800 lbs./lin. in. for the stresses but this size was increased to two 5/16" fillet welds for the 18 x 1¼" flanges and to two ⅜" fillet welds for the 1¾" and 2" thick flange plates to meet the minimum size of weld specifications. Therefore the connections of web to flange were the equivalent of 100% penetration welds throughout the entire length of the girders and all of these welds were by the automatic process.

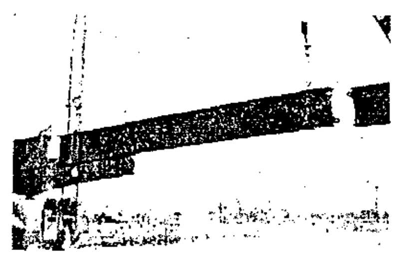


Fig. 28

During fabrication these welds cause a transverse warpage or bending of the flange plates which amounted to an angle change of approximately ⅜" in 9 inches for the 18" x 1¼" flange plates.

This in turn was compensated for by postheating the opposite side of the flange plate with another longitudinal bead of heat to shrink it back into the flat position.

The combined effect of both the flange to web welding heat and the compensating heat causes a longitudinal shrinkage of the girder flanges with respect to the web which amounts to approximately one inch of length in about 50 or 60 feet of length of the girder. This can be compensated for by either preheating the central portions of the web during the welding process or by postheating the web to shrink any undesirable buckles back into alignment after welding.

Figure 28 shows one of the completed sections of the main girders and the details of a typical field splice. The girder shown is the center section of the 100-foot span as it was being erected between pier 3 and pier 4.

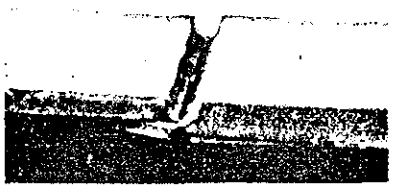


Fig. 29

The splice at the right clearly shows how two 2½-inch diameter pins were used to hold the ends of the girder in alignment for welding. These pins engage the web of the adjacent girder section between two ½-inch pin plates that are welded to the web on the other side of the joint. The pin holes were made 2$\frac{9}{16}$ inches in diameter to give a small amount of adjustment at the time of erection and for ease in the entering of the pins.

Figure 29 is a picture of a typical field splice after erection and before welding. The joint shown is an 18 x 1¼ flange prepared for butt welding to an 18 x 1¾ flange. The 1¼" side is built up to the same thickness as the 1¾" plate by fillet welding tapered end sections of cover plate to the under side of the flange so that the weld can be made of the full thickness of the thicker plate. Thus a considerable margin of safety is given to the joint since the actual stress in the weld must originate in the thinner plate and will pass through the weld at intensities that are in proportion to the area of the thicker plate.

In the field welding procedure two welders began simultaneously welding on the webs of both girders as shown in Fig. 30 attaching the pin plates permanently first and then working outward from the center of the web toward the flanges.

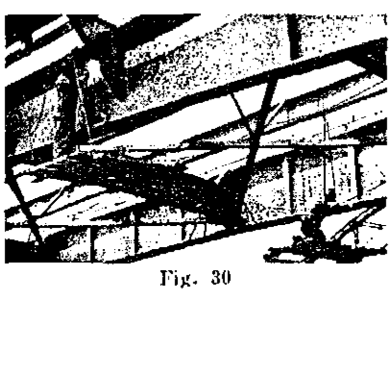


Fig. 30

The root of the flange welds were made in the overhead position and finally the bulk of the U-groove filled in in the downhand position from the top after chipping and rewelding the root as required. The downhand welding being shown in Fig. 31 and the completed weld being shown for one end of the center girder section G8 in the 120-foot span on Fig. 32.

Thus the field splices were made in this bridge without the aid of auxiliary splice plates and for 100% of the strength of the girder. A typical splice such as this required approximately from 14 to 16 man hours of welders time to make the complete splice.

The entire project required only 528 man hours of welding time, 320 man hours of riggers time, 160 man hours of foreman's time and 1485 lbs. of electrodes for all of the field welding. The field welding was done as a separate sub-contract by the Teleweld Corporation of Chicago and all welders were certified and qualified under the American Welding Society Code.

As an alternate procedure for welding the girder splices Mr. Gordon Cape of the Dominion Bridge Co. of Canada recommends the following sequence of welding for deep girders from their experiences in making welded plate girder field splices.

1. Weld the top and bottom flanges for one third to one half of their thickness. The amount of welding on the flanges at this time is intended to result in a partial contraction of the joint to such an extent that the remainder of the flange welding will cause about the same amount of contraction as the web welding. It has been found that the total contraction of an open-gap butt weld made in the downhand position with backing bars is about $\frac{3}{16}$ inch.

2. Weld 12 to 15 inches at the top and bottom of the web splice for the full thickness. These portions of the weld will then be under tension due to the weld shrinkage.

3. Complete the welding of the top and bottom flange joints. This reduces the tension in the web and results in some web buckling.

4. Complete the welding of the remaining mid-height portion of the web joint. The shrinkage of this welding removes the buckling in the web which occurred during step 3.

The measured amount of weld contraction on the Benton Street girder splices was found to vary from 0.10" to 0.17" total contraction in five inches gage length across the splice.

Following the design of the sidewalk girder the same influence lines and other data was used for sectioning the girder on the other side of the roadway. This girder carries exactly the same loading from the roadway but has a lighter dead load and only negative loads from the sidewalk.

A dead load of 1600 lbs. per lin. ft. of girder was used for design purposes neglecting the effect of the sidewalk. The results of this tabulation follows for the Main Girders on the Roadway side. 78' Span.

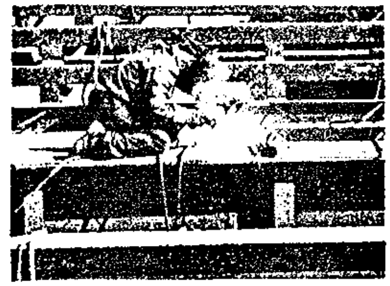


Fig. 31

Figure 33 is similar to Fig. 16 except that it gives the resisting moment and moment of inertia for various depths of girders with various thicknesses of 14-inch wide flange plates.

It is used with the above table of design moments to select the proper

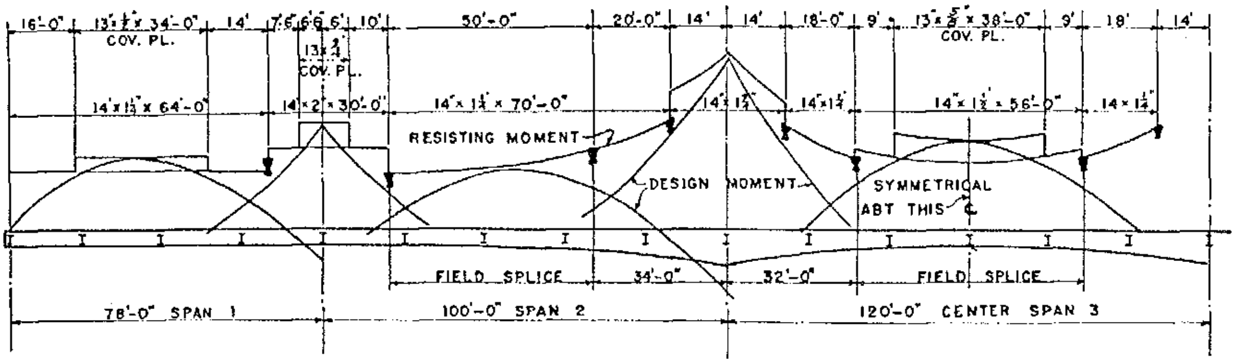
Fig. 32

sizes of girder sections as shown in Fig. 34, using exactly the same depths of girder at all points as were previously used for the sidewalk girder. Thus the two girders are exactly the same in all details except for the width and slight variations in the thickness of the flange plates.

This completes the design of all the superstructure metalwork except for camber, the temporary lateral system and the bridge shoes.

The final shop weights of the main girders are tabulated as follows, girders G1 to G4 are the roadway girders and G5 to G8 are on the sidewalk side.

Mark	Weight	Mark	Weight
G1L	25364	G5R	30135
G2L	11658	G6R	13305
G3L	20750	G7R	23598
G4	15680	G8	19820
G3R	20750	G7L	23741
G2R	11367	G6L	13350
G1R	25465	G5L	29985
	131,034 #		153,934 #
÷ 480' =	275 lbs./lin.ft.		320 lbs./lin.ft.



ROADWAY GIRDER

Fig. 34

78' Span							
Loads	R_1	$+M_1$	$+M_2$	$+M_3$	$-M_3$	$-M_4$	$+M_4$
Dead Load 1600#/lin. ft.	48.0	+ 595	+ 640	+ 45	+ 45	—1180	—1180
U.Live Load 768#/lin. ft.	28.6	+ 389	+ 522	+ 363	— 350	— 786	+ 204
C.L.L. 21600	31.2	+ 272	+ 329	+ 208	— 141	— 363	+ 76
Impact	15.0	+ 165	+ 213	+ 143	— 123	— 288	+ 70
Total	122.8	+1421	+1704	+ 759	— 569	—2617	— 830

100' Span							
	$-M_5$	$+M_5$	$+M_6$	$+M_7$	$+M_8$	$-M_8$	$-M_9$
Dead Load 1600#	— 101	— 101	+ 368	+ 201	— 607	— 607	—2050
Unif.Live Load 768	— 225	+ 177	+ 503	+ 530	+ 290	— 585	—1313
Conc. Live Load	— 119	+ 196.5	+ 295	+ 317	+ 199	— 213	— 525
Impact	— 76	+ 83	+ 177	+ 188	+ 131	— 177	— 408
Total	— 521	+ 355.5	+1343	+1236	+ 13	—1582	—4296

120' Span							
Loads	$+M_9$	$-M_{10}$	$+M_{10}$	M_{11}	M_{12}	R_2	R_3
Dead Load	—2050	— 454	— 454	+ 510	+ 823	150	184
U.L.L.	+ 326	— 428	+ 210	+ 577	+ 731	83.6	102.5
C.L.L.	+ 89	— 152	+ 181	+ 324	+ 383	31.2	31.2
Impact	+ 92	— 108	+ 80	+ 184	+ 227	25.5	27.3
Total	—1543	—1142	+ 17	+1595	+2164'K	290.3K	345.0K

The table illustrates the practical use of influence lines and the great saving in time that is made possible by having this data available.

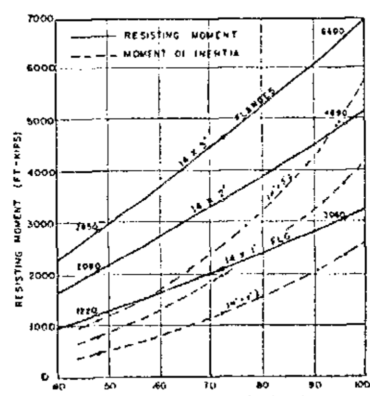


Fig. 33

Figure 35 shows the erection of one of the typical fixed bridge shoes on top of Pier 3. All of the shoes were designed as structural weldments and were set on ground bearing areas on three plys of red lead and canvas.

These shoes are 24" x 29" in plan and 10 inches high and are designed to support a direct load of 453,000 lbs. on a hardened steel line bearing surface 17 inches long acting in combination with a longitudinal friction force of 61,000 lbs. which is 25% of the dead load reaction.

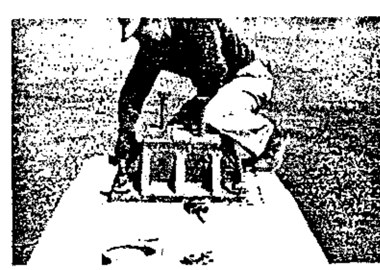


Fig. 35

Fig. 36

These forces cause a maximum pressure of 870 lbs. per sq. in. on the masonry computed as follows:

$$f_d = \frac{453000}{24 \times 29} = +650 \text{ lb. per sq. in.}$$

$$f_b = \frac{61000 \times 10 \times 6}{29 \times 24 \times 24} = \pm 220 \text{ lbs. per sq. in.}$$

$$\text{Total} \quad +870 \text{ lbs. per sq. in.}$$

A one and one-half inch thick base plate is provided in the shoe to distribute this pressure to the masonry. The plate is supported with two 1¼ inch web plates and one 1½ inch web plate each way and the shoe is capped with a 14" x 2" x 1'-5" top plate.

The crossed web plates were half dapped, as shown in Fig. 36, before being assembled together with the top plate in the upside down position. Then after all interior welds were made at the crossed intersections of the webs and to the top plate including the anchor bolt nuts the whole assembly is turned over and welded all around the outside of the webs to the base plate as shown in Fig. 37.

This figure also shows the pad of high strength weld metal that was deposited on the top of the shoe by manual welding to give high yield point characteristics in the line bearings. This pad was built up ⅝ inch and finished flat while a similar pad deposited on the sole plate on the girder was built up and finished on a 10-inch bearing radius for the line bearing contact.

The diameter of the bearing circle is determined by equating the required intensity of bearing pressure per lineal inch to the formula

Therefore a radius of 10 inches was used for the bearing surface.

On top of the shoe a three-inch thick sole plate is used on the bottom flange of the girder to distribute the load from the line bearing hinge to the area of the H-beam stiffener.

The thickness of this plate is determined by cantilevering a proportional part of the bearing load out from the line bearing to the bearing area under the flanges of the stiffeners.

Thus an 18" x 17" x 3" sole plate with a one-half inch hardened steel line bearing surface was provided on the bottom flange butt welded in between two pieces of a 17" x 1" bottom cover plate.

These plates are also welded to the girder flange sufficiently to compensate for the loss of metal caused by the two 2" round anchor bolt holes through the bottom flange for the 1½" round anchor bolts and they also provide substantial reinforcement for the reversed curvature in the bottom flange plate of the girder.

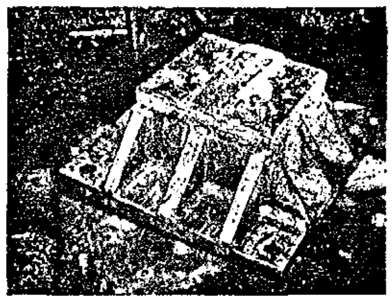


Fig. 37

Fixed shoes of this type were used at both ends of the 120-foot center span on both piers 2 and 3 for symmetry and one inch of expansion and contraction movement in this span is provided for by sliding a limited amount on top of the shoes in the oversize bolt holes.

The fixed shoes under the roadway girders were made in exactly the same manner excepting that they are made four inches narrower to fit the 14-inch wide girder flange in place of an 18-inch flange. Thus the roadway side shoes are 24" x 25" x 10" high designed for a 345,000 lb. reaction.

All fixed shoes are anchored to the masonry with four 1¼-inch round anchor bolts cast with an embedment of 18 inches into the masonry.

Expansion rocker shoes were provided over Piers 1 and 4 for the common reactions between the 100-foot and 120-foot spans and single segmental rolller shoes were provided on the abutments as shown in Fig. 14.

Figure 38 is a photographic study of the rocker shoe on top of Pier No. 1 including the sidewalk cantilever bracket, handrail post and fascia.

$$\frac{P - 13000}{20000} \times 3000\sqrt{d}$$

in which p = the yield point of the material in tension in the roller and d = the diameter of the roller.

Thus assuming p = 55,000 lbs. per sq. inch

$$\frac{453{,}000\#}{17 \text{ inches}} = \frac{55{,}000 - 13{,}000}{20{,}000} \times 3000\sqrt{d} \qquad \text{from which}$$

$$\sqrt{d} = \frac{27{,}000 \text{ lbs./lin. in.}}{6300} \qquad \text{and } d = 18.5'' \text{ diameter.}$$

The total bearing area is 30.76 sq. inches.

14 WF at 74 lbs.	=	21.76 sq. in.
18 x ½" Web	=	9.00 sq. in.
		30.76 sq. in.

453,000 lbs. ÷ 30.76 = 14,800 lbs./sq. in.

The half flange and web areas equal 12.0 sq. inches and therefore carry 178,000 lbs. of the reaction.

Two half flanges	5 x ¾	= 7.5
Web	9 x ½	= 4.5
		12.0 x 14.800 = 178,000 lbs.

Cantilever Moment = 178000 x 2.5" = 445,000 inch lbs.

$$\text{Therefore from } f = \frac{6\text{ M}}{bd^2} \text{ at } 18{,}000 \text{ lbs./sq. inch}$$

$$d = \sqrt{\frac{6 \times 445{,}000}{18000 \times 17}} = 2.95 \text{ inches}$$

This rocker shoe is a structural weldment made entirely of plates as shown in Fig. 39.

This shoe is designed to support a total reaction of 380,000 lbs. while providing for a longitudinal movement of two inches plus or minus one inch each way from a normal vertical position.

A ten-inch radius was used for the high strength line bearing surfaces on both the top and bottom surfaces of the rocker so that

$$\frac{55000 - 13000}{20000} \quad 3000 \sqrt{d} = 28,000$$

lbs. per lin. inch pressure is permissible in the design of the rollers. This requires only 13.6 inches of length of linebearing surface whereas 18 inches of length is provided at the top and 21 inches net length is provided on the masonry plate after deducting for the two three-inch round dowel pins. The roller is eighteen inches high and the two ten-inch radii overlap two inches at the center to save height.

The shoe rests on a 20″ x 3″ x 3′-2″ masonry plate erected on three ply red lead and canvas on a ground concrete bearing surface. This plate is held in position by means of four 1¼-inch anchor bolts engaged in the masonry. This plate is designed as a double cantilevered beam from the line bearing to support a masonry pressure of 500 lbs. per sq. in.

Two 3-inch round dowel pins hold the shoe in place on top of the masonry plate and two 1½″ x 2½″ standard involute stub tooth bar guides engaged with the sole plate on the girder hold the shoe in place at the top.

The rocker itself consists of two 8¾″ x 2″ top and bottom flange plates welded to a 14″ x 1¾″ x 2′-7″ web plate which is in turn reinforced with eight 3½″ x 1″ vertical stiffeners as shown on Fig. 39. The web plates and stiffeners are finished to bear on the flange plates and the welds are standard fillet welds as indicated.

The other rocker shoes on the roadway side of the bridge are made in a similar fashion except four inches narrower to fit under the 14-inch girder flanges. They are designed for a load of 290,000 lbs. per shoe, the rockers are only 2′-3″ long and the masonry plates 2′-10″ in place of 3′-2″.

At the abutments the segmental roller shoes are cut from a piece of 5″ x 9″ x 1′-8″ structural slab finished to 10 inches in diameter by adding one-half inch of weld metal on each edge for the high strength line bearing. They are held in place by means of 2½″ x 1″ tooth bars that are pressed into the ends of the roller. These bars engage with tooth slots in both the masonry plate and the girder sole plate.

These rollers are designed to provide for three inches of expansion and contraction movement while supporting a 150,000 lb. girder reactions on the sidewalk side or 122,000 lbs. on the roadway side. The masonry plates are 12″ x 2″ x either 2′-0″ or 2′-4″ long.

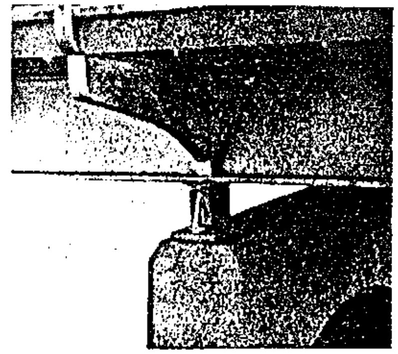


Fig. 38

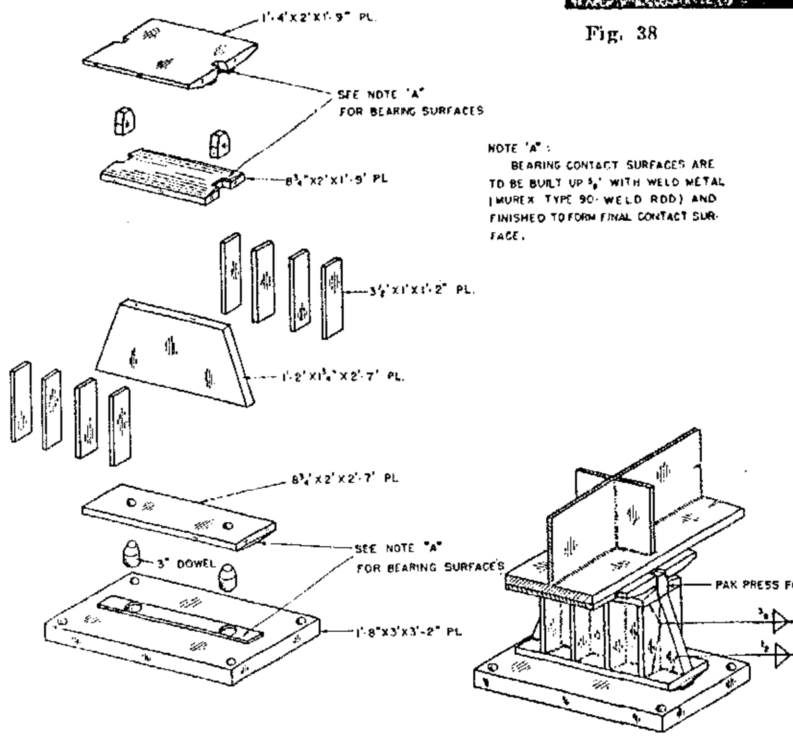


Fig. 39

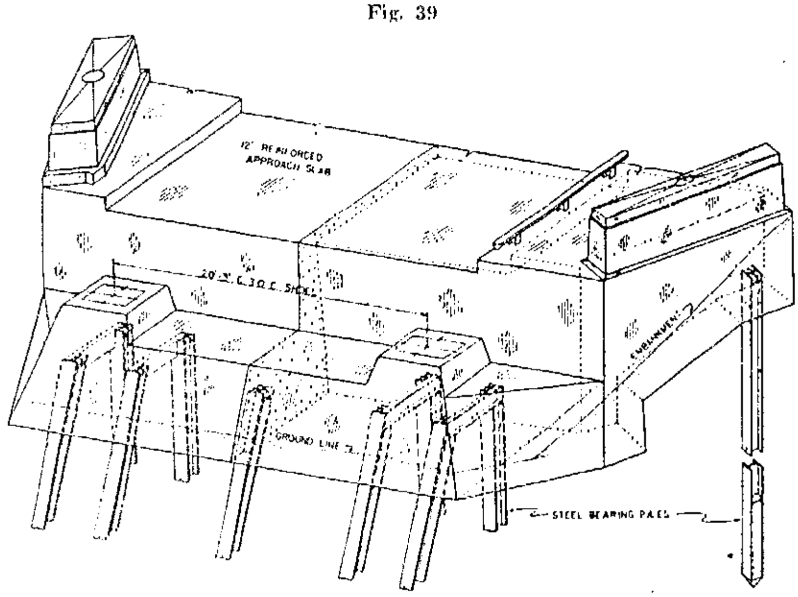


Fig. 40

This completes the design of the superstructure and establishes the final dimensions from crown of roadway to masonry for all portions of the substructure as follows:

	At Abutments	At Piers 1 & 4	At Piers 2 & 3
Elev. C./R.	64.15	66.10	67.82
C./R. to bk. of Girder	9¾"	9¾"	9¼"
Bk. to bk. of Girder	4'-0"	4'-0"	8'-1"
Sole plate	2"	2½"	3½"
Roller	10"	1'-6"	
Masonry plate	2½"	3½"	10"
Total	6'-0¼"	6'-9¾"	9'-11¾"
Elev. of Masonry	58.13	59.29	57.84

These elevations were all considered satisfactory since the recorded elevation of extreme high water is Elev. 57.7 as recorded from the existing old bridge and a flood control dam is already under construction just nine miles further upstream for the control of future floods.

Fig. 40 is a sketch of one of the typical buried pier abutments.

Each abutment is founded on eleven 10" x 10" at 42 lbs. per foot steel bearing piles that were driven approximately sixty feet through sand and gravel to bear on solid limestone rock.

The piles were fitted with steel armored points filled with concrete and have channel beam caps welded to the tops as shown in Fig. 40.

Figure 41 is a picture of the reinforcing steel in the base of the East Abutment. This heavily reinforced beam is founded in natural ground and retains the end of the embankment that is filled in behind the backwall after construction. These beams are also designed to distribute the bridge and embankment loads to the piling.

The bridge shoes are each supported directly over four of the pilings on the two small pedestals that project eighteen inches above the rest of the bridge seat. This projection is intended to keep the shoes clear of any future accumulation of debris that may occur on the rest of the bridge seat.

The approach slab over the backfill is also reinforced to prevent settlement of the approach paving.

The abutments are designed both as retaining walls and as piers. The volume of concrete, its weight and the location of its center of gravity are calculated first and then the pile loads are determined for various stages of construction.

The plan arrangement of the piling is shown in Fig. 42 upon which the corner piles are designated as A, B, C and D. The loads on these piles are summarized as follows for the various stages of construction.

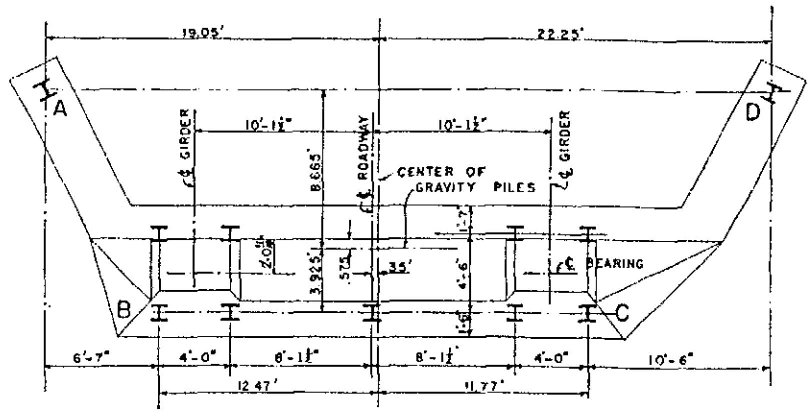


Fig. 42

	A	B	C	D
1. Dead Load of Abutment	42,500	19,600	25,080	51,710
2. Earth Pressure	—24,400	11,000	11,000	—24,400
3. Dead Load of Bridge	1,100	12,650	14,250	3,750
4. H-20-S16 Live Load	5,830	17,280	16,710	5,450
5. Sidewalk Live Load			2,550	2,370
Total	24,930	60,530	69,590	38,570#

For safety against sliding the base of the abutment extends three feet below the natural surface of the ground in front of the abutment for passive resistance, the top of the abutment is anchored to the approach pavement and the front row of piles is battered 4" in 12 inches.

The Final Cost of Two Abutments

Item	East Abut.	West Abut.	Total	Unit	Cost
Steel Piles	26,400 lbs.	22,400	48,800 lbs.	$ 0.138	$ 6,734.40
Concrete	80.5 c.y.	81. c.y.	161.5 c.y.	52.50	8,478.75
Reinforcing	6,000 lbs.	6,000 lbs.	12,000 lbs.	.154	1,848.00
Total					$17,061.15

The Man and Machine Hours

	East Abut.	West Abut.	Total
Foreman	124 hrs.	105 hrs.	229 hrs.
Carpenters	70 hrs.	38 hrs.	108 hrs.
Laborers	233 hrs.	278 hrs.	511 hrs.
Crane Operators	181 hrs.	107 hrs.	288 hrs.
Welder	30 hrs.	24 hrs.	54 hrs.
			1190 man hrs.
Cranes	69 hrs.	65 hrs.	134 hrs.
Steam Hammer	11 hrs.	5 hrs.	16 hrs.
Pump	6 hrs.	5 hrs.	11 hrs.
			161 machine hrs.

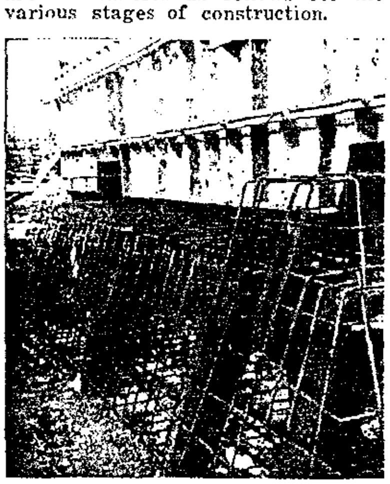


Fig. 41

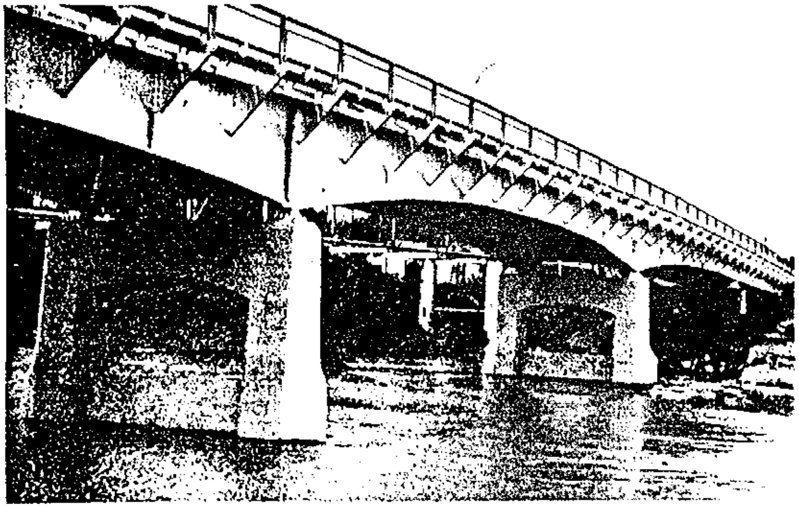


Fig. 43

Figure 44 is a sketch of the intermediate bank pier #4 located between the 78 ft. and 100 ft. spans.

This pier was designed with a split base so as to straddle the twin 14-inch cast iron sewer siphons that cross the river at this point.

Pier #1 in this same position on the east bank was made the same as pier 4 for symmetry and duplication.

These piers are founded on twelve 12" at 53# steel bearing piles that were driven to rock under each pier. The piers are protected against scour by riprap and the bottom of the bases were set at elevation 35. Since this was only three feet below the low water surface in the river these excavations were unwatered by pumping and the bases were poured in the dry as shown in Fig. 45 without the use of sealing courses.

The loads are delivered to the tops of these piers through expansion roller shoes as shown in Fig. 39.

Since these are expansion shoes and since they are located directly over the center lines of the pier shafts these piers are designed only for the direct gravity loads and the cap beam functions as a rigid frame only for its own weight and for transverse winds. These loads were determined as Okay by inspection and by comparison with the previously figured loads on piers 2 and 3.

A maximum direct load of 46 tons per pile was computed as follows for the shaft on the sidewalk side.

Also 2820 bd. ft. of lumber was required for the construction of the forms for pier 1 and the same forms were reused for pier 4.

The simplicity of the form construction is shown for Pier 1 in Fig. 46.

Figure 47 shows the construction of Pier 2 within a temporary cofferdam in the river just as the nosing forms had been removed. Thus by dredging up the sand island around the pier everything above the sealing course was built in the dry just the same as if these piers had been on dry land.

This picture also shows the nosing forms in the foreground. The large chamfered corners are simply extended to form the point and a large slice is blocked out of the form for the upper part of the pier shaft.

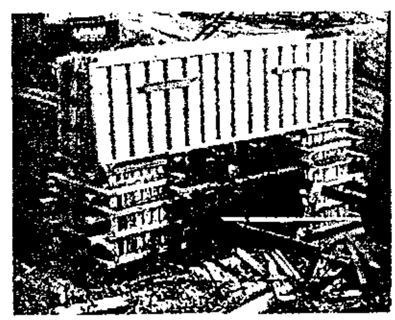


Fig. 46

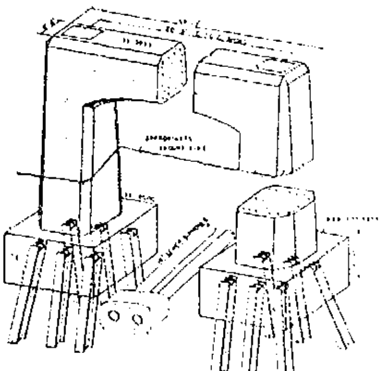


Fig. 44

Fig. 45

Fig. 47

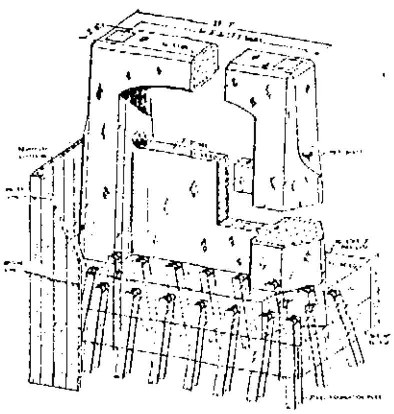


Fig. 48

The loads on the piles are summarized as follows for the various conditions of loading:

	Direct Loads		
Concrete base	13.3 c.yds.		
Concrete Shaft	16.0 c.yds.		
½ Concrete Capbeam	6.0 c.yds.		
	35.3 c.y. at 4050 =	143,000	lbs.
Earth on top of base		30,800	lbs.
Dead Load Sidewalk Girder		195,000	lbs.
Live Load on Sidewalk		41,600	lbs.
H-20-S-16 Live Load		114,800	lbs.
Impact		25,500	lbs.
Total load		550,700	lbs.
Divided by six piles	=	91,800	lbs. per pile

or 46 tons per pile.

The piles are battered 1½" in 12" for lateral stability against possible unbalanced bank loads against the pier and the end piles are also battered 1½" in 12" up and down the river to resist the thrust from frame action and the transverse wind loads on the bridge.

Figure 48 is a typical sketch of the two river Piers 2 and 3.

These piers are founded on twenty 12" WF 53# steel bearing piles under each pier that were driven to rock within temporary timber sheet pile coffer dams. The material from within the cofferdam was excavated to a depth of fourteen and a half feet below the low water surface and then a four-foot thickness of tremie concrete was placed all over the bottom of the excavation to seal off the water. The interior of the cofferdam was then pumped out and the rest of the pier construction was in the dry.

Piers 2 and 3 are designed to support the combined reactions of the 100 and 120-foot spans and also to function as the longitudinal fixed points for the whole bridge. The pier shafts are battered from 4'-0" wide at the top to 6'-1" wide at the top of the base and stand 25'-10" high above the base. The pier base is 10'-6" wide by 31'-6" long by 8'-6" thick including the thickness of the seal. The piles are spaced at 4'-0" centers longitudinally and at 3'-9" centers transversely in the pier base. An edge distance of 1'-9" was maintained on the piling from the edge of the concrete to the center of the piles at the bottom of the sealing course. The piles are also battered 2" in 12" for stability as shown on Fig. 48.

The actual cost of these two piers was $15,186.40
The Cost of Two Bank Piers #1 and #4

Item	Pier #1	Pier #4	Total	Unit	Cost
Steel Piles	17,600#	12,000#	29,600#	$0.138	$ 4,084.80
Concrete Bases	27 c.y.	33 c.y.	60 c.y.	83.90	5,034.00
Concrete Shafts	43.5 c.y.	44.5 c.y.	88 c.y.	52.50	4,620.00
Reinforcing Steel	4,700#	4,700#	9,400#	0.154	1,447.60
Total cost of both piers			=		$15,186.40

The actual labor and machine hours:

Man Hours	One Pier Base	One Shaft	Total Pier 1	Total Pier 4	Total Two Piers
Foremen	73	45	118	109	227 hrs.
Carpenters	139	77	216	15	231
Laborers	169	118	287	354	641
Crane Operators	76	41	117	122	239
Welder	36		36	34	70
					1408 Man hours
Cranes	68	40	108	70	178
Steam Hammer	14		14	10	24
Pump	30		30	30	60
					262 Machine hours

Summary of Pile Loads

	Upstream Piles			Downstream Pile	
1. Dead Load	55,600 lbs./pile*			47,600 lbs./pile*	
2. H-20-S16 Live Load	11,200	*		11,200	*
3. Live Load Sidewalk	5,500	*		— 1,800	
4. 30# Transverse Wind	±23,600	*		∓23,600	*
5. 50# Transverse Wind	±23,000			∓23,000	
6. River Current	— 720			+ 720	*
7. Ice Load	—22,800	*		+22,800	*
8. Traction	±10,200	*		∓10,200	*
Max. Load	106,100 lbs./pile*		+	116,120 lbs./pile*	
Min. Load			+	12,000 lbs./pile	

Say 60 tons per pile maximum.

The piles are 12" x 12" W.F. 53# bearing pile sections with seven sixteenth-inch thick web and flange. The area of the section is 15.58 sq. inches and the average stress in the pile for the maximum load is 7,500 lbs. per sq. inch.

In addition to the above forces these piers are subjected to some variations in stress due to temperature changes in the length of the 120-foot span since fixed shoes are used on the tops of both piers at both ends of the span.

Provision is made in the size of the bolt holes in the girder flanges so that the girder can slide back and forth a limited amount on top of the fixed shoes. The bolts are one and one half inches in diameter in two-inch round holes thus providing for one-half inch of movement at each end of the span.

This provision limits the temperature force to that which will cause sliding under dead load or to that which will cause a plus or minus ¼ to ⅜-inch deflection of the top of the pier.

In this case the deflection force is the smaller and since it is about the same size as the longitudinal tractive force it has been neglected in the tabulation.

Referring again to the sketch of the pier on Fig. 48 the following is a series of actual photographs that were taken during construction.

Figure 49 shows the steel bracing frame for the cofferdam for pier 3 set up in place on the sand island.

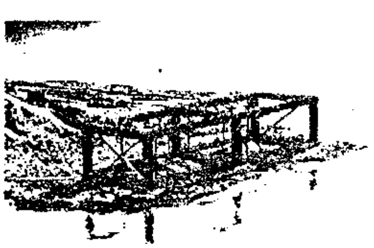


Fig. 49

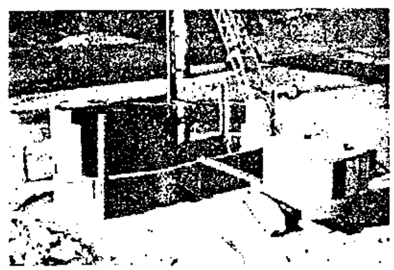


Fig. 50

Figure 50 was taken during the process of the second stage of driving down the wakefield sheet piles. The piles are each made of three 2 x 12 x 22'-0" timber planks bolted together with tongue and groove edges and special angle shaped corner piles. The corner piles were set up first and driven partially down as shown in this view, then all the side and end piles were set up in between and after a whole side was in place they were jetted and pushed down to the same elevation as the corner piles with a drop hammer.

The interior of the cofferdam is partially excavated as the next stage sufficiently so that the interior bracing frame can be driven down or allowed to sink inside until the top of the frame is about level with the sand outside.

Then all of the intermediate piles were driven and jetted down to final elevation as shown in Fig. 50 and finally the corner piles were driven down to the final elevation to complete the cofferdam.

Figure 51 shows the method of excavating the material from the inside by means of an ordinary clamshell bucket. This process continues until both the frame and the excavation are to final grades as measured by means of a sounding pole.

Fig. 51

The armor pointed steel piles are then driven as shown in Fig. 52. All of the piles are first stacked into one end of the cofferdam and then they were driven with a number 1 Vulcan single acting steam hammer to refusal on rock until the average penetration was only ⅝ of an inch under the last ten blows. The battered piles are spotted on the bottom in the vertical position and then leaned over to the proper batter as measured with a steel square and hand level. The driving leads and the pile are then held in this position throughout the rest of the driving.

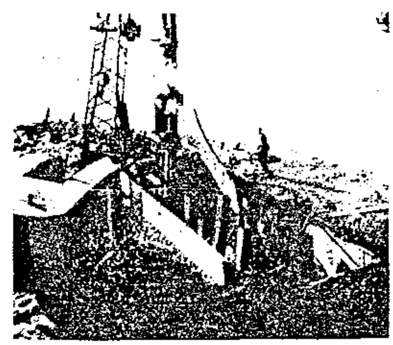


Fig. 52

After the pile driving is completed the excavation is checked again for grade and the sealing course of concrete is poured over the area of the bottom of the cofferdam. Figure 53 shows this operation with a ready mixed concrete truck backed right up to the edge of the cofferdam. The lower end of the tremie pipe is kept in contact at all times and both the pipe and the hopper are kept full so that the concrete is placed with as little disturbance and mixing with the water as possible.

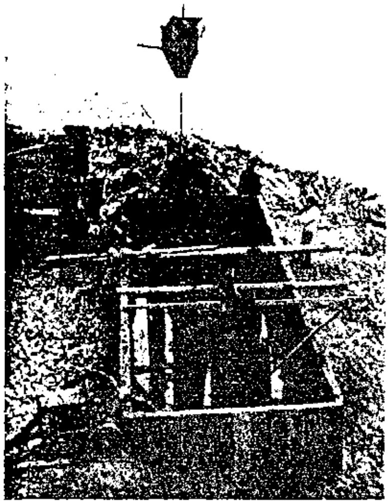


Fig. 53

Figure 54 shows a welder at work during the next stage capping the piles. This picture shows the pattern of the piles at the cutoff elevation and the layer of reinforcing steel around the pile caps. After the water is pumped out the excess lengths of piles are burned off and portions of the cutoffs were used to form the pile caps. The remaining lengths of cutoff pilings were butt welded together end to end to form stringers for a temporary erection trestle and other erection purposes.

The roll of building paper in the background is for lining the sides of the cofferdam against the inside surface of the wood to confine the leakage into a channel around the edge of the base to a sump in one corner from which it is pumped until after the next layer of the base concrete is in place.

Fig. 54

The base of the pier is designed as a large concrete beam capable of delivering the pile reactions to the concrete pier shafts and the pier web. The shafts and web are fastened to the base with heavy steel reinforcing dowels that are designed to lap with the main vertical bars in the pier shafts.

Above the top of the base the rest of the pier is built as shown in Figs. 46 and 47.

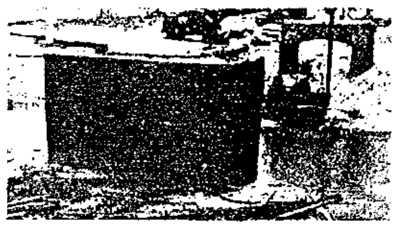


Fig. 55

Figure 55 shows the canvas enclosure that was used during cold weather for protecting the concrete from freezing during the curing period. The tarpaulins were nailed to a light framework of two by fours and salamanders were used inside to furnish the heat.

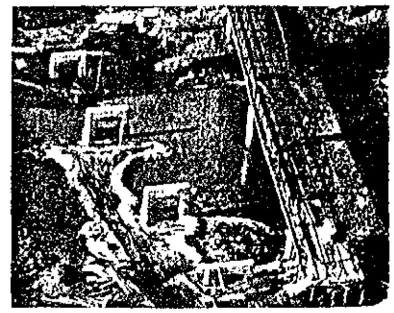


Fig. 56

Figure 56 is an aerial view of the project as of December 11, 1948, showing the progress of the work. The bridge was built by the Jensen Construction Company of Des Moines, Iowa, under a general contract for the whole project. They started work on October 2, and their work was so well organized and managed that by December 20, 1948, all of the substructure was complete and ready for steel. This photograph shows the old bridge on the right and the east abutment under the tarpaulins in the foreground.

The actual cost of the two river piers was $34,718.14.

Cost of Two River Piers No. 2 and No. 3

Item	Pier No. 2	Pier No. 3	Total	Unit		Cost
Steel Piles	18,100#	25,500#	43,600#	$ .138	=	$ 6,016.80
Concrete Bases	102.1 c.y.	101.5 c.y.	203.6 c.y.	83.90	=	17,082.04
Concrete Shafts	78 c.y.	77 c.y.	165 c.y.	52.50	=	8,662.50
Reinforcing Steel	9,600#	9,600#	19,200#	.154	=	2,956.80
Total Contract Cost						$34,718.14

The Actual Labor and Machine Hours

Man Hours	One Pier Base	One Shaft	Total Pier 2	Total Pier 3	Total Two Piers
Foremen	111	82	193	144	337 Man hrs.
Carpenters	166	148	314	165	479
Laborers	188	171	359	439	798
Crane Operator	116	36	152	180	332
Welder	28		28	30	58
Total					2004 Man hrs.
Machine Hours					
Cranes	91	36	127	132	259
Steam Hammer	14		14	18	32
Pump	39		39	57	96
Total					387 Machine hours

Also 3,250 board feet of lumber was required for the forms for Pier 2 and the same forms were reused for Pier 3.

12,000 board feet of lumber was required for one cofferdam and the same sheeting was reused for Pier 3.

Fig. 57

This fill was still available at the beginning but was lost during the spring flood and erection was delayed again from March 4 to April 4 by the melting of the winter's accumulation of snow and ice.

During this time the rest of the steel was received from the fabricator and the contractor dismantled the floor of the old bridge and built a new temporary erection trestle out of the salvaged material.

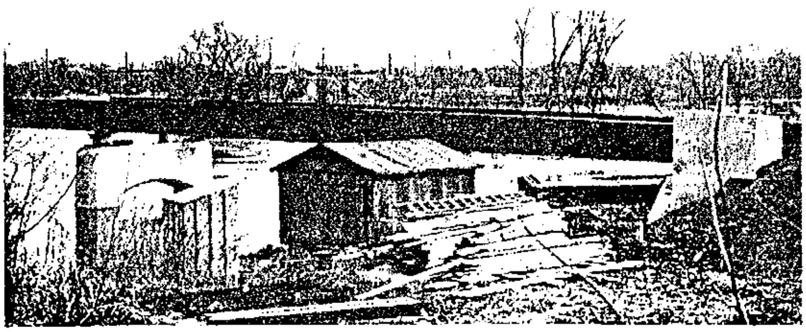


Fig. 58

Erection began again on April 4, 1949, with the erection of the variable depth girders balanced over Pier 1 as shown on Fig. 60. These girders project only 32 feet on the far side of the pier while they are 34 feet on this side so they were easily balanced with only a 4 x 12 bridge plank standing on end as the post.

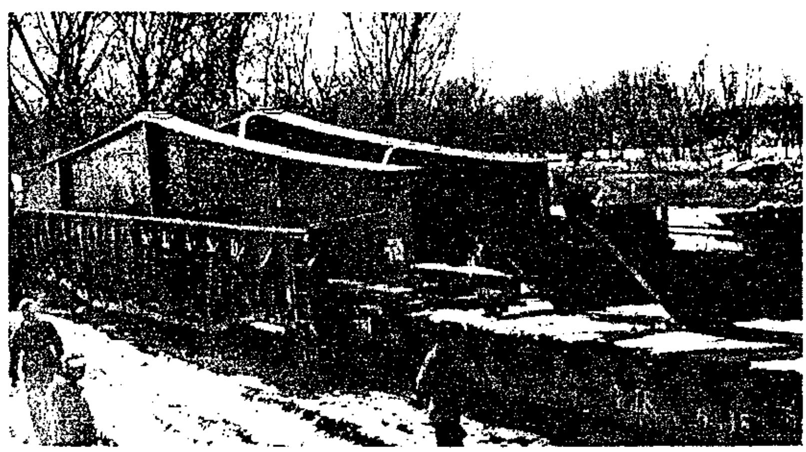


Fig. 59

Due to the difficulties of procuring steel the shop fabrication of the superstructure did not progress as rapidly as first expected and the first shipment of steel from the American Bridge Co. did not arrive in Iowa City until February 25, 1949.

Figure 57 shows the first double car shipment of the four end span girders. These girders were fabricated in 94-foot sections and were unloaded and erected immediately on temporary blocking spanning from the abutments to Piers 1 and 4, cantilivering beyond the piers to the field splice as shown on Fig. 58. Two of the girders were unloaded first for erection on the east side of the river and the other two were left on the cars and reswitched to another railroad siding on the west side of the river and then transported to the bridge site by means of a motor crane and truck. Otherwise all of the steel was received for unloading and erection from the east side of the river.

Another typical shipment of steel is shown on Fig. 59. The variable depth sections of the girders were shipped in 66-foot sections loaded upside down on a single car with the end gates down. The idle car being loaded with a section of prefabricated handrail.

The original plan was to erect the superstructure during January and February from the same sand fill as used for the pier construction.

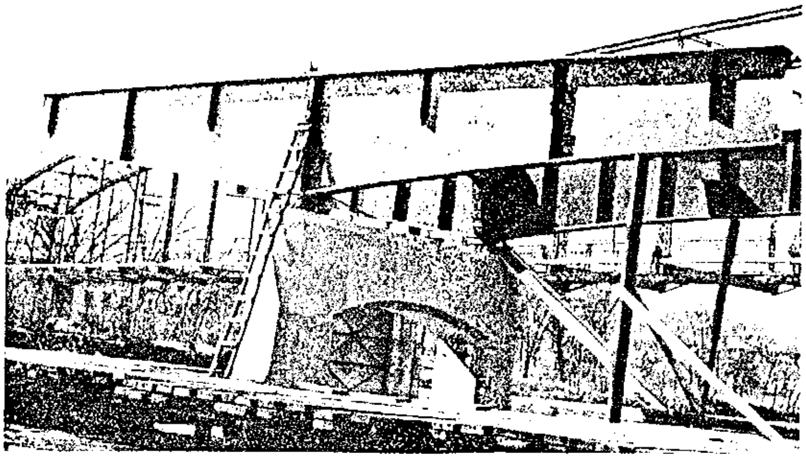


Fig. 60

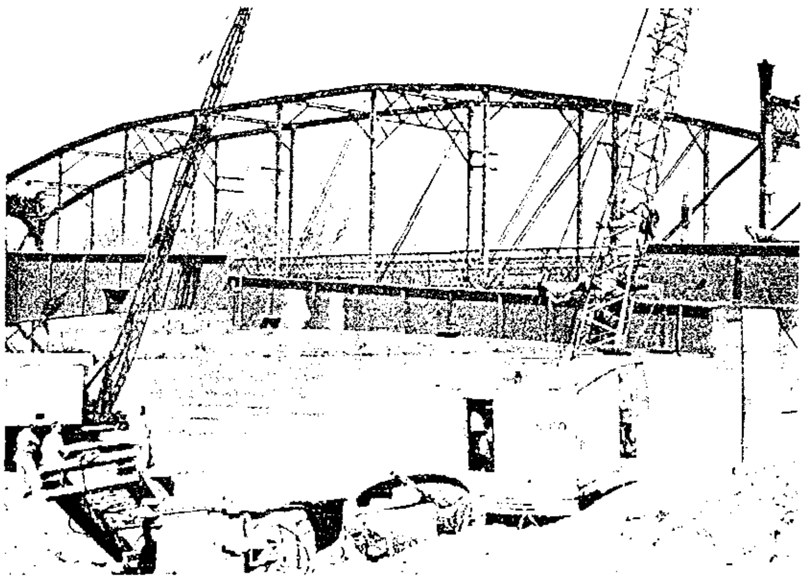


Fig. 61

During the third stage the 50-foot central section of the girders of span two were filled in to complete the erection of the girders back to the abutment. This was done first by connecting on to the end of the balanced section and then jacking the end span sections horizontaly to close the other end as shown in Figs. 61 and 62.

All temporary girder connections were made by means of two 2½-inch pins through the webs and projecting pin plates as previously shown on Figs. 28 to 32 inclusive.

At this time the end sections of the girders were also lifted and placed on their permanent roller shoes on the abutment and over Pier 1 since the steelwork was now fastened to the fixed shoes on Pier 2. The floorbeams and bracing members were then filled in so that by the end of the second day the steel work was complete clear back to the abutment as shown on Fig. 63 except for the handrails, curbs and stringers.

The contractor then extended the trestle to Pier 3 and erected the two central portions of span 3 on temporary supports as shown on Fig. 64 on April 9.

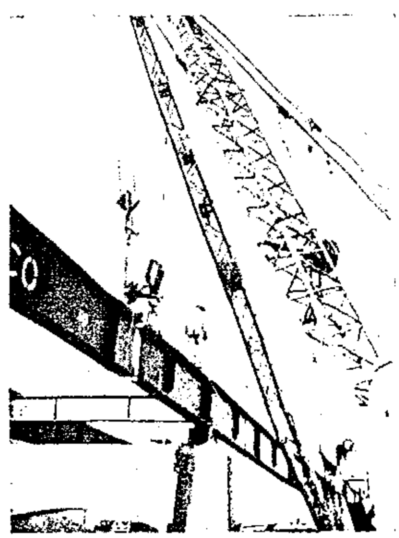


Fig. 62

The most difficult phase of erection was accomplished on April 1 in the placing of the north girder over Pier 3. It was first carried out to the pier between the two cranes as shown in Fig. 65 and landed temporarily on top of the pier while the rear crane maneuvered into the new position under the end of the south girder. Then the girder was lifted again from the top of the pier into its final position as shown on Fig. 66.

Figure 67 shows the placing of the south girder over Pier 3. The light colored areas on the girder web are newly painted areas where some additional flame straightening of the girder webs was done in the field to flatten out some slightly buckled panels of web that were discovered during the unloading operations.

The buckles were apparently caused by the longitudinal shrinkage of the

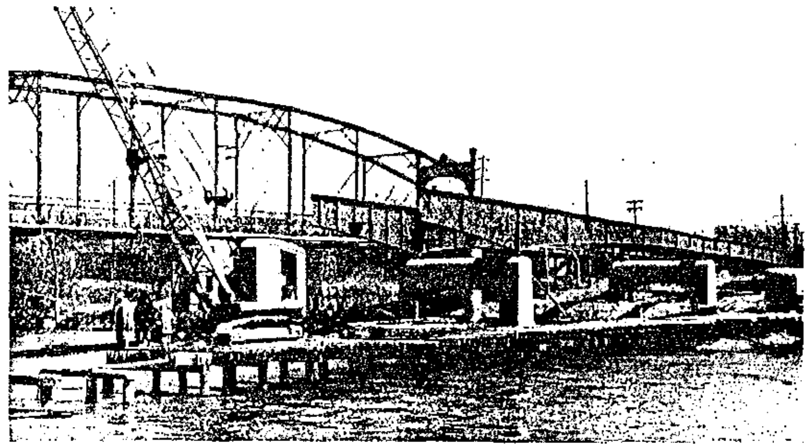


Fig. 63

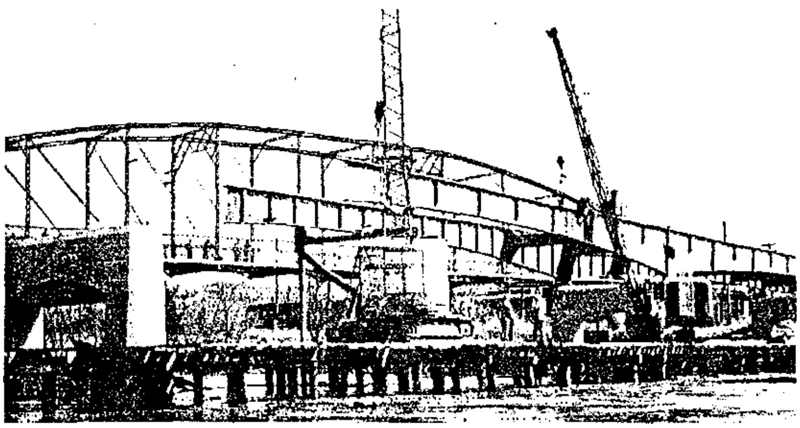


Fig. 64

flange welds and were remedied by applying vertical beads of heat on the convex side of the bulge to shrink the metal back into a plane surface. Several buckles that measured as much as ⅝ of an inch at the middle were corrected in this manner until the entire surface did not deviate more than ⅛ of an inch from a plane surface.

Figure 68 is the begining of the final stage of the erection of the main girders. The last section of the north girder is being carried out and is about to be handed across the gap to another crane on the west bank.

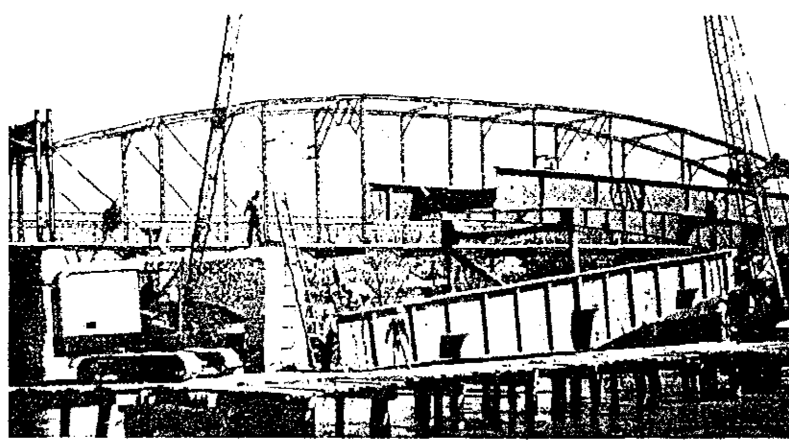


Fig. 65

Figure 28 shows the erection of this girder and Fig. 69 shows the placing of the last section of the south girder to complete the erection of the main girders on April 11, 1949. The final closure was made by jacking Span 5 horizontally from the west abutment in the same manner as previously used on Span 1 including the lifting of the girders and placing of the roller bearings on the west abutment and over Pier 4.

The final operations consisted of the erection of the curbs and hand-

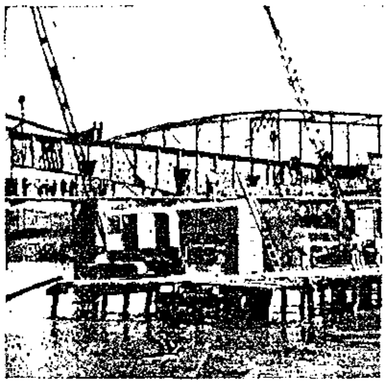


Fig. 66

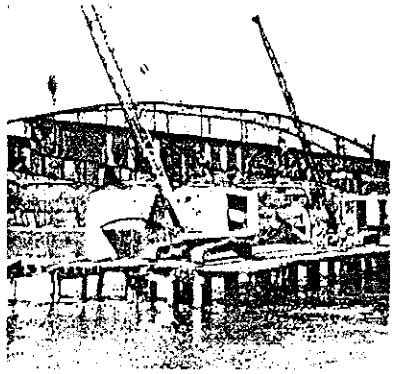


Fig. 67

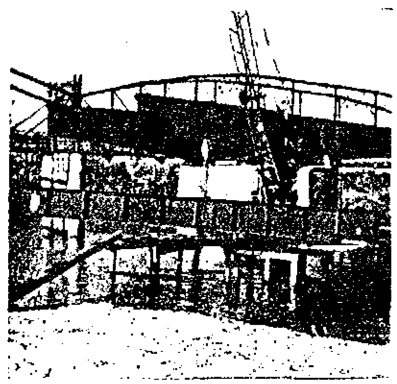


Fig. 68

rails and stringers as shown on Fig. 70 and previously on Fig. 8 to complete the bridge ready for welding as previously shown on Fig. 3.

The field welding for the project was done by the Teleweld Corporation of Chicago under a lump sum subcontract for $4100.00.

They assigned four certified welders to the project and began work on April 18, 1949, and it required approximately 4 weeks of time to complete the bridge ready for painting. Approximately 2600 lin. ft. of average ⅜-inch fillet welds were made in the field in addition to the main girder splices.

The field welding required 528 man-hours of welders time, 320 man-hours of riggers time, 160 man-hours of foreman's time and 1485 pounds of electrodes of which approximately 10% was lost as stub ends and scrap.

In addition to this the Jensen Construction Co. devoted the following man and machine-hours on this project in building the temporary bridge and erecting the superstructure structural steel.

	River Fill	Temporary Trestle	Erection of Steelwork	Total	
Foremen		135	462	597	Man hrs.
Crane Operators	60	199	436	695	
Carpenters		99		99	
Laborers		1137	1204	2341	
	60	1570	2102	3732	Man hrs.
Cranes	60	178	311	549	Machine hrs.
Jet Pump		16		16	Machine hrs.
				565	Machine hrs.

The field painting of the superstructure metalwork was also by subcontract for $1250.00 with the Jensen Construction Co. furnishing all of the paint.

The welders finished work on May 16, and since they started from the west side the form construction for the concrete roadway also started from the west side as soon as all welding was complete to Pier 3.

The forms were made of ¾-inch plywood supported on four 2 x 12 x 22' planks laid flat between each pair of stringers. The planks in turn were supported on short pieces of salvaged 4 x 12 bridge planks laid crossways between the steel stringers at 5'-0" centers.

Fig. 68A

The cross planks were all prefabricated in one legged jack shape so that they could be carried into position, supported directly from the bottom flanges. The 22-foot stringer planks were salvaged material from the timber cofferdam and were still in excellent condition even after their use as form lumber.

The girders were cambered for full dead load assuming the steel girders to carry the entire load without any composite deck action.

Therefore the sequence of pours was arranged so that in general the positive moment areas were loaded first and the negative areas last. The amount of camber was checked by level before and after each pour.

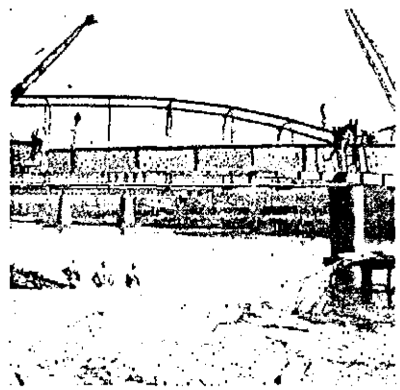


Fig. 69

Pour No. 1 began 20 feet west of Pier 3 and extended west toward Pier 4 to include the central 60-foot section of the 100-foot span No. 4.

Pour No. 2 began at the west abutment and extended east to join Pour No. 1.

Pour No. 3 began at the center line of bridge and extended west to join Pour 1. Pours 4, 5 and 6 were made in the same sequence over the corresponding areas on the eastern half of the bridge to complete the deck.

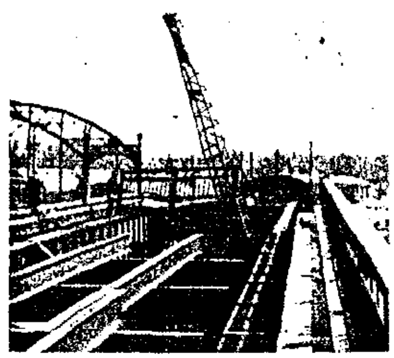


Fig. 70

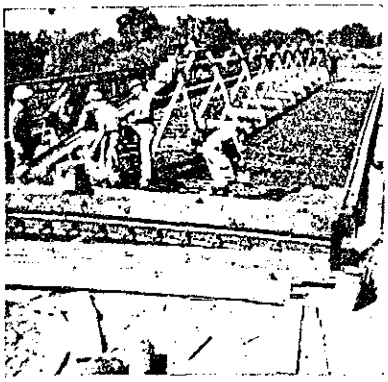


Fig. 71

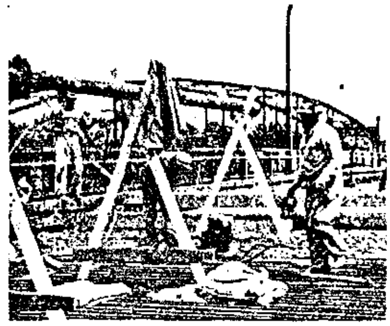


Fig. 72

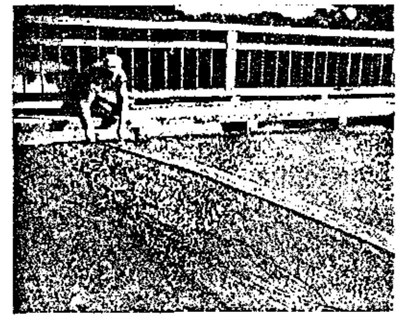


Fig. 73

All concrete was placed on the deck through a pipe from a pump-crete machine located just back of the abutment at either end of the bridge as shown in Fig. 71. The pipe was carried out to the far end of Pour 1 first and then extended back to the abutment for Pour 2, etc., with the pipe being dismantled as the concrete was poured.

A mechanical gasoline driven vibrator was used to compact and settle the concrete and the slab profile was struck off transversely with an electric vibrating screed as shown in Fig. 72. Then it was smoothed off longitudinally with a wooden plank float and finally finished transversely with a canvas belt as shown in Fig. 73.

Following these operations the slab was covered with wet burlap and kept moist throughout the daylight period of the second day after the concrete was poured.

The final check on the finished grade of the hridge revealed that all spans were very close to their computed final elevations. Span 1 was ⅛-inch low, Span 2 was ⅞-inch low, Span 3 was ½-inch high, Span 4 was ⅛-inch low and Span 5 was right on grade, all measurements being at the center of each respective span. This indicates that the dead load stresses are in close agreement with the design values and also seems to indicate that the additional stiffness of the concrete deck over the area of the initial pour prevented the second and fourth spans from recovering the full amount when the center and end spans were finally loaded.

The sidewalk slab was poured from concrete trucks running on the roadway slab. The open type curb is advantageous for supporting the sidewalk forms and otherwise the construction of the sidewalk was conventional.

In terms of labor and materials the cost of the deck slabs is divided as follows:

Cost of the Concrete Deck

Man hours	Roadway Slab	Sidewalk	Total
Foremen	361	84	445
Carpenters	591	320	911
Laborers	1,353	237	1,590
Total	2,305	641	2,946 Man hrs.
Materials			
Forms	31,000 bd. ft.	5,500 bd. ft.	36,500 bd. ft.
Reinforcing	90,600 lbs.	3,400 lbs.	94,000 lbs.
Concrete	295.5 c. yds.	37.7 c. yds.	333.2 c. yds.

The concrete was delivered ready mixed and the Pump-Crete machine operated for 20 hours during the pouring of the roadway deck.

The following is a summary of the actual cost of the whole bridge in terms of labor and materials and machine-hours as purchased by the General Contractor.

Actual Costs

I. Substructure					
1. Labor					
Foremen	753 hrs.	at $2.00	=	$ 1,506	
Carpenters	818 hrs.	at 1.60	=	1,308	
Laborers	1950 hrs.	at 1.35	=	2,635	
Welders	600 hrs.	at 1.75	=	1,050	
Crane Operators	859 hrs.	at 1.75	=	1,505	
	4980 hrs.			$ 8,004	
25% for overtime, taxes and insurance				2,000	
					$10,004
2. Materials					
Steel Piling	159,000 lbs.	at $0.04	=	$ 6,360	
Concrete	667.5 c.yds.	at 12.00	=	8,020	
Lumber	20,840 bd. ft.	at 0.13	=	2,710	
Reinforcing	40,600 lbs.	at 0.07	=	2,840	
					$19,930
3. Machines					
Cranes	8 months	at $375.00	=	$ 3,000	
Steam Hammer	2¼ months	at 500.00	=	1,125	
Jet Pump	4 months	at 60.00	=	240	
Pump	3 months	at 15.00	=	45	
Compressor	4 months	at 100.00	=	400	
Boiler	3 months	at 25.00	=	75	
Welding Machine	5 months	at 50.00	=	250	
					$ 5,135
Sub-total					$ 35,069
10% for supplies and incidentals					3,500
Total substructure					$ 38,569

II. Superstructure

1. Labor

Foremen	1042 hrs. at $2.00	=	$2,084		
Carpenters	1010 hrs. at 1.60	=	1,616		
Laborers	3331 hrs. at 1.35	=	4,500		
Welders	1008 hrs. Lump sum	=	4,100		
Crane Operators	635 hrs. at 1.75	=	1,120		
Salvage Labor	600 hrs. at 1.35	=	810		
	7626 hrs.		$14,230		
25% for overtime, taxes and insurance			3,500		
					$17,730

2. Material

Timber Piling	1,620 lin. ft. at $ 0.25	=	$ 400		
Concrete	333.2 c.y. at 12.00	=	4,000		
Lumber	10,500 bd.ft. at 0.13	=	1,500		
Structural Steel	545,100 lbs. at 0.142	=	77,500		
Reinforcing Steel	21,200 lbs. at 0.07	=	1,490		
Reinforcing Trusses	73,800 lbs. at 0.12	=	8,850		
					$93,740

3. Machines

Cranes	5 months at $ 375.00	=	$1,875		
Pump	½ month at 60.00	=	30		
Pump-Crete	1 month at 100.00	=	100		
Compressor	4 months at 100.00	=	400		
					$ 2,405

Sub Total	$113,875
10% for supplies and incidentals	11,400
Total Superstructure	$125,275
Total Substructure	38,569
Total for Bridge	$163,844

or say $165,000 for round figures of the actual costs to the contractor.

The contractor then adds his allowances for the hazards of erection, floods, etc., and for a margin of profit to these figures and bids the job. It should also be noted that margins of profit to the subcontractors are already included in the items of structural steel, reinforcing trusses, lumber and concrete.

The actual cost of the bridge to the City of Iowa City including the removal of the old bridge was $221,928.66. The highest bid for the same portion of the work was $375,000.00.

The Contract Cost is summarized as follows:

Item 1 Steel foundation piles	126,441 lbs. at	$ 0.138	=	$ 17,448.86
Item 2 Concrete in pier bases	261.6 cu. yds. at	83.90	=	21,948.24
Item 3 Concrete—pier shafts and abutments	399.9 cu. yds. at	52.50	=	20,989.50
Item 4 Concrete in floor	273.2 cu. yds. at	52.50	=	14,343.00
Item 5 Reinforcing steel	141,768 lbs. at	0.154	=	21,832.27
Item 6 Structural steel	545,073 lbs. at	0.23	=	125,366.79
Total contract cost of main bridge				$221,928.66

The entire project also included the construction of the approaches leading to existing highways at both ends of the bridge and a mercury vapor bridge lighting system.

The fills for these portions of the work were placed in the fall and the paving was completed during the month of June.

The new bridge was formally dedicated and opened to traffic on July 28, 1949. The whole project required just a little less than ten months for completion, at a contract cost of $252,000.00.

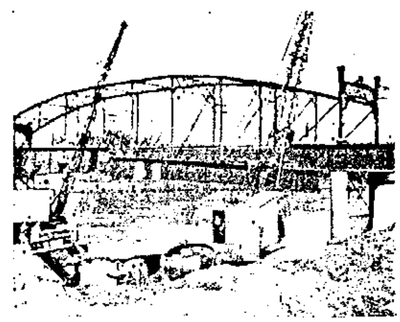


Fig. 74

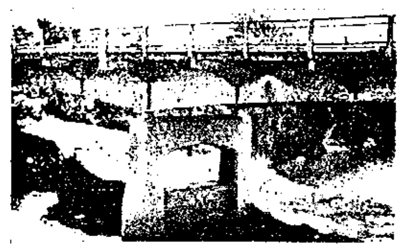
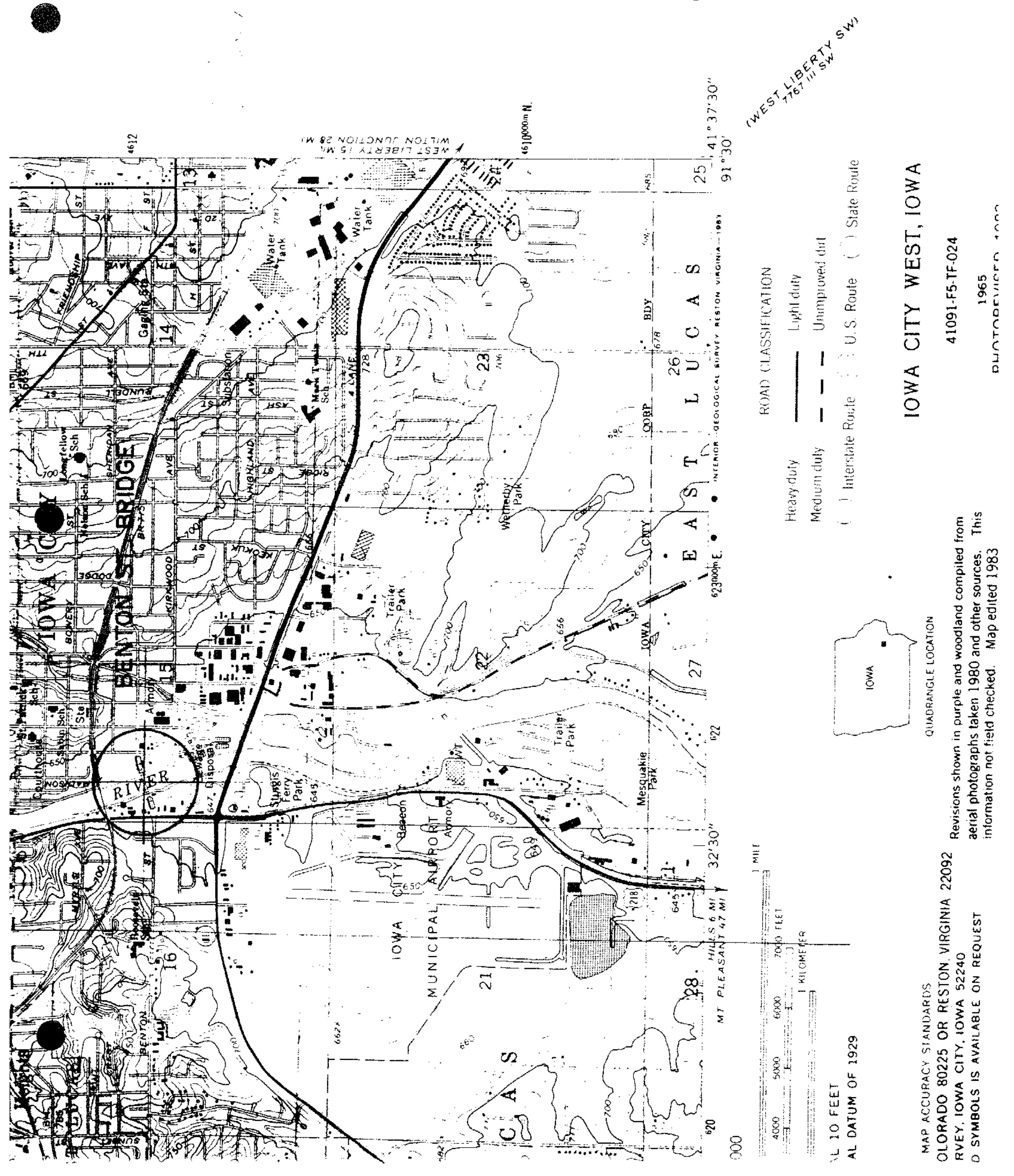


Fig. 75

Fig. 76


IOWA CITY WEST, IOWA
41091-F5-TF-024
1965
ROAD CLASSIFICATION
Heavy-duty
Light duty
Medium duty
Unimproved dirt
Interstate Route
U.S. Route
State Route
QUADRANGLE LOCATION
IOWA
Revisions shown in purple and woodland compiled from
aerial photographs taken 1980 and other sources. This
information not field checked. Map edited 1983
MAP ACCURACY STANDARDS
(WEST LIBERTY SW)
7767 III SW
41°37'30"
91°30'
32'30"
BENTON ST. BRIDGE
IOWA CITY
RIVER
EAST LUCAS
IOWA CITY MUNICIPAL AIRPORT
Mesquakie Park
Sturgis Ferry Park
Trailer Park
Wetherby Park
Water Tank
Courthouse
Longfellow Sch
WEST LIBERTY 15 MI
WILTON JUNCTION 28 MI
HILLS 6 MI
MT PLEASANT 47 MI
1 MILE
1 KILOMETER