Design and Construction of Transversely Posttensioned Concrete Bulb Tee Beam Bridge

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A six-span prestressed concrete bulb tee beam bridge, designed to eliminate the need for a conventional cast-in-place concrete deck, was constructed in Minnesota in 1986. Two end spans of this 484.7-ft (147.8-m) long structure were 70 ft (21.3 m) clear, and four interior spans were 85 ft (25.9 m) clear. Bars 1 in. (2.54 cm) in diameter were used to transversely posttension 6-ft (1.8-m) wide top flanges of five 40-in. (100-cm) deep bulb tee beams. A slip-formed, cast-in-place, reinforced concrete railing limited required formwork to only the low-slump wearing course, expansion devices, and pier diaphragms. A 2-in. (5.1-cm) minimum concrete overlay provided a good riding surface. Construction of this structure was completed in just 5 months at a cost of \$450/yd^2 (\$538/m^2), with a savings of about 25 percent over a conventional design.

This discussion covers the design and construction of a six-span prestressed concrete bulb tee bridge that was built at the Northern States Power Company plant in Red Wing, Minnesota, in 1986. The bridge consisted of two 70-ft (21.3-m) spans and four 85-ft (25.9-m) spans, with an overall length of 484.7 ft (147.8 m). Five bulb tee beams, each 6 ft (1.8 m) wide, placed side by side constituted the deck width. Final deck width of 30.3 ft (9.2 m) included a 3-in. (7.6-cm) accumulation of horizontal beam total curvatures resulting from fabrication (Figure 1).

GEOMETRIC CONSTRAINTS

Several vertical constraints required grades of 6.05 and 5.38 percent with a sharp vertical curve of 140 ft (42.7 m) over a set of railroad tracks. One constraint was a minimum vertical clearance of 23 ft (7.0 m) over the railroad tracks. The railroad also required a minimum horizontal clearance of 22 ft (6.7 m). Another constraint was the elevation difference of 40 ft (12.2 m) between the power plant yard and Trunk Highway 61 (Figure 2).

Spans at all five piers were fixed, and expansion was taken up completely at the abutments because of steep grades. Strip-seal-type expansion devices were used, which allow a total expansion of 6 in. (15.2 cm).

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FOUNDATION CONSTRAINTS

The elevation differential of bedrock at the bridge site presented another design obstacle. At the south abutment and adjacent pier 1, footings were keyed directly into bedrock. At the other four piers and the north abutment, steel H-piles driven to bedrock were used.

A special hinge was placed in the column base of pier 1 to allow the pier to rotate longitudinally (Figure 3).

AESTHETICS OF PIERS

To minimize costs and achieve aesthetically pleasing piers, a common hammerhead cap on a 10- by 3.75-ft (3.0- by 1.1-m) shaft with rounded ends was chosen. Horizontal rustication was placed on the shaft at 4-ft (1.2-m) intervals to enhance the aesthetic appeal.

UNIQUE DESIGN FEATURES

In the conventional method for holding bulb tee beams together welded bar ties are spaced on 4-ft centers, which generally results in longitudinal joint cracks. The design of this bridge used transversely posttensioned bars instead. To accomplish this, bulb tee beams were fabricated with 3-in. (7.5-cm) diameter galvanized metal spiro ducts. The ducts were centered in the top flange 3.38 in. (8.6 cm) from the top of the beams.

The bulb tee beams were standard shapes, 40 in. (101.6 cm) deep with 6-in. (15.2-cm) wide webs (Figure 4). These beams were longitudinally pretensioned to the required loads with 0.5-in. (1.25-cm) diameter low relaxation strands.

Blockouts of 4.75 by 10.25 in. (12.0 by 26.0 cm) were placed in the outside edges of the facia beams for anchorage assemblies. Vertical 0.83-in. (2.1-cm) grout tubes were connected to spiro ducts in facia beams under the railing.

Actual spacing of the metal spiro ducts was 2.92 ft (88.9 cm). All the reinforcement, including tie wires in the bulb tee beams but not the low-relaxation strands, was epoxy coated.

Two lifting cranes were needed to handle and place beams that weighed 34.5 and 41.4 tons (34.0 and 40.7 metric tons). Because these beams were placed at a slight slope to follow the final roadway transverse slope, special bearing plates were used.

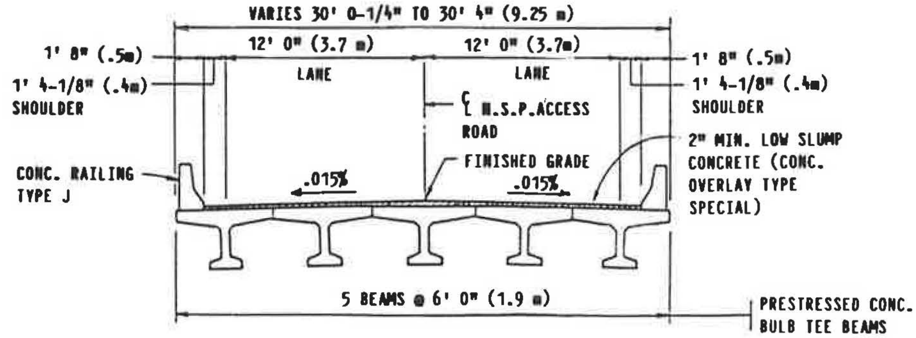

FIGURE 1 Cross section of bridge.

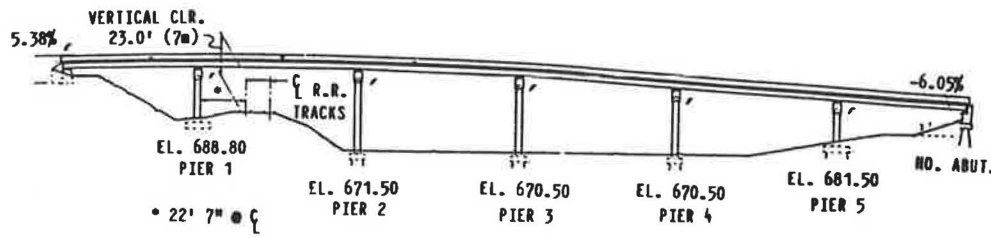

FIGURE 2 Elevation of bridge.

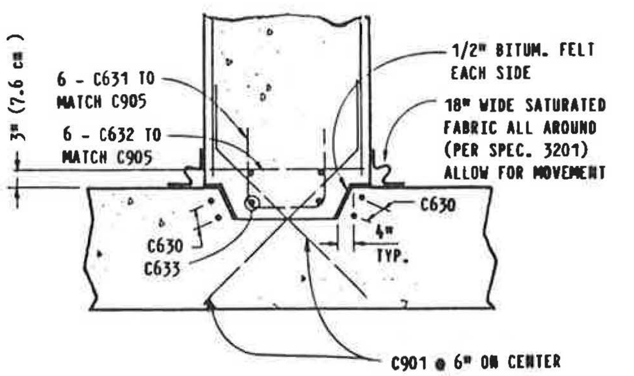

FIGURE 3 Hinge detail at base of pier 1.

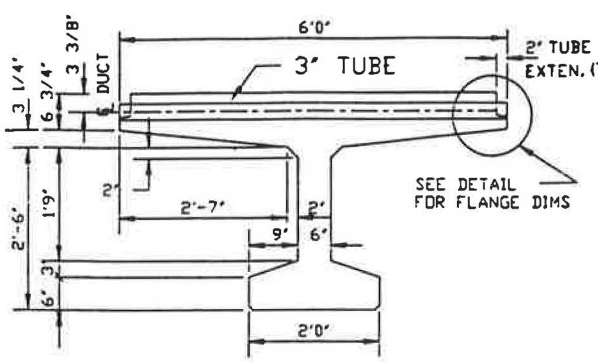

FIGURE 4 Typical beam section.

The bearing plates were tapered in two directions to take into account longitudinal grade and transverse crown slopes of the roadway. Use of longitudinally curved plates in the bearing assemblies ensured good bearing on the elastomeric bearing pads. There were some difficulties in placing and tilting beams correctly on the bearing assemblies. These could have been greatly reduced by placing and using two additional lifting device points in adequately redesigned cantilever sections of the beam flange. Differential camber was not experienced as a problem. Joint openings between beams and vertical match-up of spiro ducts were negatively affected by the handling techniques of one pick point on each end of the beams. However, all transverse posttensioning bars were easily pushed through the spiro ducts.

In order to minimize rotational stresses at mid-span of the facia beams, steel intermediate transverse diaphragms were used at the middle of the spans. Concrete diaphragms were used at abutments and piers to restrain beams against lateral rotation at these locations. Design of each bulb tee beam for live load was based on Section 3.23 of the American Association of State Highway and Transportation Officials (AASHTO) specifications. Beams were longitudinally pretensioned using 22 low-relaxation 0.5-in. (1.27-cm) diameter strands for the short end spans and 34 low-relaxation 0.5-in. diameter strands in the longer interior spans. Six and 14 strands were draped in the end-span and center-span beams, respectively, to maintain compressive stresses in the concrete below an allowable stress value of 2,400 psi (16 540 kPa) based on $F'c$ of 6,000 psi (41 360 kPa).

JOINTS BETWEEN BEAMS

A 2-in. (1.5-cm) wide by 4-in. (10.2-cm) deep notch along the interior longitudinal edges of the top flange formed a joint key between the beams. This standard joint key accounted for the horizontal bowing of beams that results from fabrication and handling. During construction, a polystyrene rope was worked down into the joints between the beams to hold the grout above it. Because this opening between beams varied, it was recommended by construction personnel that a rubber tee or equivalent be used on future jobs (Figure 5). Couplings were used initially to reduce leakage and intrusions into the ducts

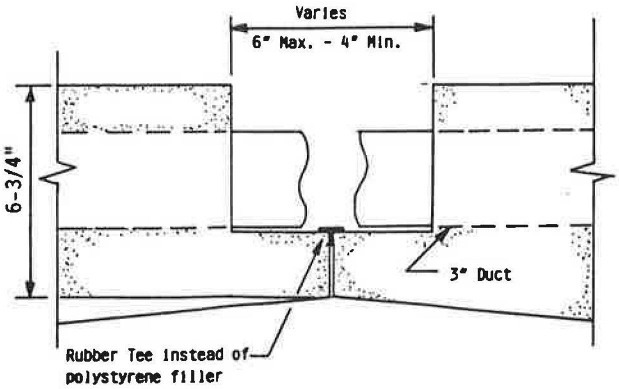

FIGURE 5 Beam joints.

during placement of mortar grout. On the final spans, the contractor simply secured the spiro-duct joints with tape. This procedure worked fairly well during pressure grouting; however, use of couplings would ensure tight joints.

The joint key between the beams was filled with concrete nonshrink grout with the appropriate admixtures. This grout was required to have 6,000-psi strength (based on a maximum water/cement ratio of 0.45) before transverse posttensioning.

TRANSVERSE POSTTENSIONING DESIGN AND DETAIL

The contractor used threadbars 29.67 ft (9.0 m) long with 1-in. diameter to posttension the beams transversely. These threadbars were epoxy coated, except for their heads. Steel anchor plates 4 by 8.25 by 1.5 in. (10.2 by 21.0 by 3.8 cm) thick were used to bear against the concrete and maintain the concrete stress of less than 3,000 psi (20 680 kPa). All contact surfaces and the hexagonal anchor nuts were uncoated to achieve uniform and consistent stresses in the threadbars. The threadbars, which were spaced at 2.92 ft (88.9 cm), were tensioned to 101,000 lbf (45 810 kgf) progressively across each span. This force was required to pull the beams together and prevent tensile stresses at the longitudinal joints. Threadbar elongation was calculated at 1.375 in. (3.5 cm), with an anchor set of 0.06 in. (1.52 mm). All exposed uncoated surfaces were epoxy coated after posttensioning. The actual elongations were measured at 1.375 in. (3.5 cm) when the jacking force of 101,000 lbf was achieved.

GROUTING OF SPIRO DUCTS

In the first attempts at grouting the spiro ducts with a cement-and-water mixture under 100 psi (698 kPa) pressure, the grout was blown out of the joints. A commercial grout mix pumped into the ducts at 80 psi (552 kPa) proved to be effective. Grout was pumped into one end of the spiro duct through one grout tube and air was let out of the other end of the spiro duct through the opposite grout tube.

RAILING AND WEARING COURSE

To further reduce construction time, Type J railings were placed by slip forming, which is a common practice in Minnesota. To take up the unevenness in the beam surfaces, a 2-in. (5.1-cm) low-slump dense concrete wearing course was applied as a final riding surface.

CONSTRUCTION ADVANTAGES

Use of the foregoing special time-saving construction techniques and absence of deck forming resulted in a construction time for the bridge of only 5 months. This is about half of the conventional construction time for a bridge of this size. Another significant advantage was that the actual construction cost of the bridge was \$450/yd^2 (\$538/m^2) of deck surface. A conventional bridge of this type in Minnesota would have cost about \$65/ft^2 (\$70/m^2), which amounted to about a 25 percent savings in the cost of construction.

CONCLUSIONS

- During the first year of service, no cracks appeared in the concrete wearing surface over the longitudinal joints.
- This type and size of bridge construction requires only one construction season of 6 months.
- Cost savings of 20 to 25 percent may be achieved using transversely posttensioned deck bulb tee beams.
- Maintenance costs would be lower because of the smaller amount of exposed steel and lack of joint weldments, which easily fatigue and break.
- Rubber tees, if used in the joints between beam flanges, would provide a tolerance of up to 1 in. to properly cover differential openings between the beams.
- Additional handling devices should be used in properly designed beam flanges to facilitate tilting and placement of the beams on their bearing supports.

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Transverse Load Distribution in a 536-ft Deck Arch Bridge

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An instrumentation and load-testing analysis of the 536-ft, two-span, open spandrel arch-rib Hennepin Avenue Bridge over the Mississippi River in Minneapolis, Minnesota, was conducted to measure the transverse load distribution among the six arches of the bridge and to determine whether the buckled webs of the arch ribs carry any load. When the structure was rated for Hennepin County in 1983, the load distribution and the ability of the buckled web plates to carry axial stress were questioned. Thus, an instrumentation analysis under static load was performed. The bridge was instrumented with 18 strain gauges and was loaded with three 27.5-ton tandem dump trucks positioned in nine different loading arrangements. Strain readings were averaged for each loading to determine the magnitude of load carried by each arch rib and then compared with a computer-modeled (linear) transverse distribution. It was found that the arch ribs carried not only axial stress, but also stress due to bending moments. It was concluded from the study that the floor beams and diaphragms do not transfer loads from one side of the bridge to the other. The results of the web testing demonstrated that a nominal axial load was being carried by the web, and that, although buckled, it was working effectively through a tension-field mechanism similar to that of a simple truss.

The Hennepin Avenue Bridge (S.B. 90589) over the Mississippi River in Minneapolis, Minnesota, is a historically significant, unique structure functioning as the second-longest solid arch-rib bridge span known to be in use today. Although this is the third bridge on this site, its earliest predecessor was the first recorded bridge across the Mississippi River. Constructed directly north of the Falls of St. Anthony and situated between the Burlington-Northern Railroad flats area and Nicollet Island, the steel arch bridge is now part of the St. Anthony Falls Historic District, birthplace of the City of Minneapolis. The bridge continues to function as a major transportation artery linking downtown Minneapolis with the northern and eastern metropolitan areas.

Of historic and technical significance, the steel bridge was constructed in longitudinal halves, with two 258-ft spans and supports, a 56-ft roadway, and two 12-ft sidewalks. The north half of the bridge was constructed in 1888 of 3 three-hinged arch ribs. In an attempt to reduce deflections and vibration, the design of the south half of the bridge was revised from a three-hinged to a two-hinged arch. The combination of three- and two-hinged solid arch ribs is the most unusual structural feature of the historic bridge.

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An instrumentation and structural analysis of the Hennepin Avenue Bridge was undertaken in mid-1983 to determine its load-carrying capacity and the feasibility of rehabilitating the bridge. During the field inspection, many arch-rib web plates were found to be bowed or buckled out of their vertical plane. Therefore, the ability of the buckled web plates to carry compressive loads was questioned. During the structural analysis, it became apparent that the methods (and effectiveness) of transferring live loads in the transverse direction between the arch ribs were ambiguous. As a means of establishing the transverse load distribution and determining the ability of the buckled web plates to carry compressive loads, the bridge was monitored with strain gauges under known loading conditions.

What follows is an overview of the structural nomenclature of the Hennepin Avenue Bridge, a cursory summary of the instrumentation procedure, and a discussion of conclusions reached as a result of the study. The basis of this paper is a report prepared in June 1984 (*1*).

BRIDGE DESCRIPTION

Figures 1 and 2 show the structural components of the Hennepin Avenue Bridge in typical section views and the nomenclature used throughout this paper to describe the components. A brief description of the individual components follows:

1. Batten plates: These plates (splice plates) are composed of $^3/_8$-in. steel and serve to keep the individual arch-rib panels in place.
2. Diaphragms: The diaphragms are located at each panel throughout the bridge between the arch ribs.
3. Sway bracing: The sway bracing (diagonals) runs diagonally between the top and bottom of adjacent spandrel columns. These are circular rods with turnbuckles for length adjustment. Because of their slenderness, they transmit only tension forces.
4. Wind bracing: The wind bracing consists of round bars, again with turnbuckles. This bracing is located in the same plane as the top and bottom flanges of the arch ribs and forms an X within each panel. Similar to the sway bracing, these members transmit only tension forces.
5. Web plates: These plates are riveted between the flange angles to form the web of the arch rib.
6. Floorbeams: These members carry the stringer loads to the spandrel columns. The floorbeams are hinged at the center of the bridge as a direct consequence of the original staged construction, and thus provide no moment transfer at that point.

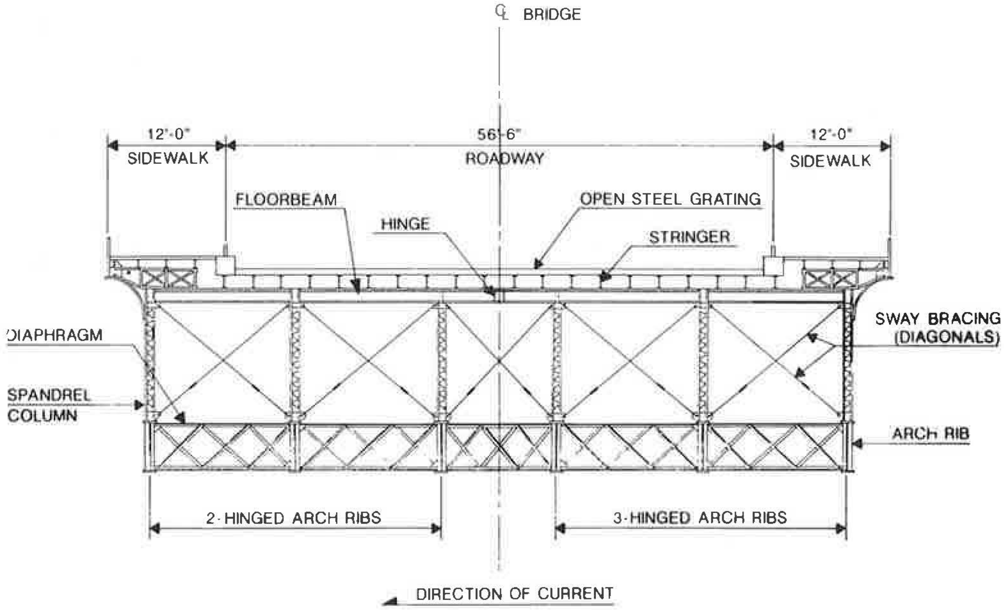

FIGURE 1 Typical section.

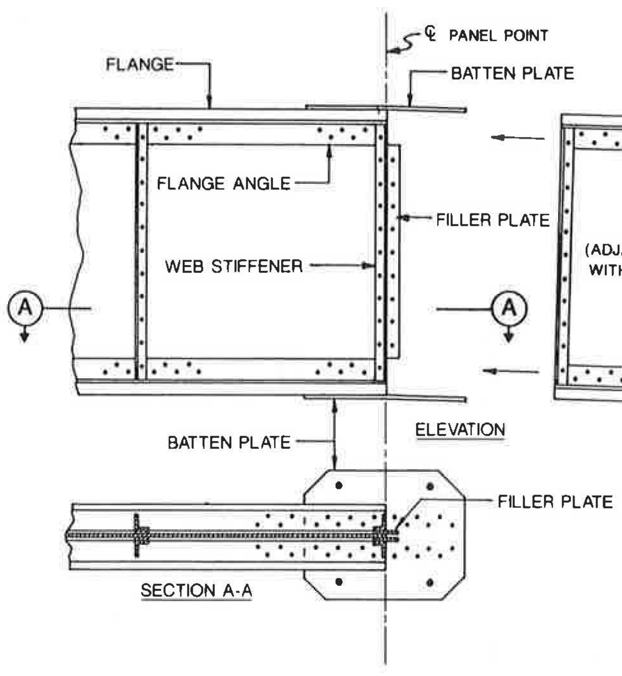

FIGURE 2 Typical arch section.

Figures 2 and 3 show a typical arch section and a plan and elevation view of the bridge, respectively. It should be noted that the arch ribs are not continuous members but a series of 42 panels constructed segmentally. This design feature limits the arch's ability to function as a beam (i.e., to carry bending stresses).

INSTRUMENTATION GOALS

The mechanism by which loads are transferred in the transverse direction and the effectiveness thereof were initially questioned after the following observations were made:

- Section properties of each arch are dissimilar by original design;
- Three of the arches are two-hinged, and three are three-hinged;
- Conventional analysis demonstrated that exterior arch ribs carried more load than design specifications would allow;
- A two-dimensional analysis could not accurately model all structural members, given the complicated three-dimensional aspects of the bridge; and
- Tension-only members, loose members, and the rigidity of connections are conditions that vary throughout the bridge.

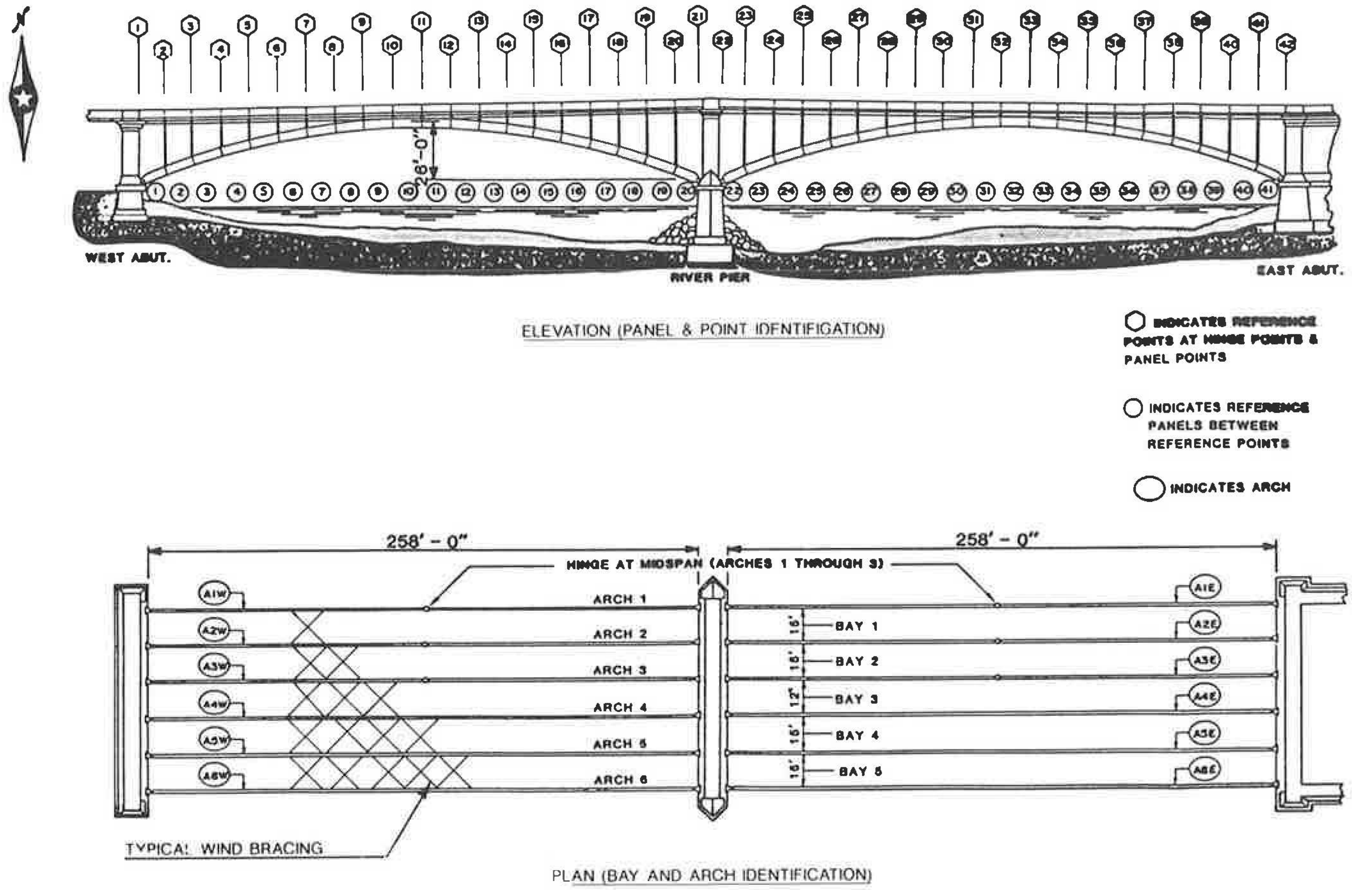

FIGURE 3 Bridge plan and elevation.

As previously mentioned, it was thought that the web plates would carry little, if any, compressive loads because of their buckled condition. It was speculated that the flange plates would accept additional loads and would therefore be stressed at a higher level than would otherwise be expected.

PURPOSE OF TESTING

The purpose of the testing was to monitor the bridge under a known loading condition. This would provide a correlation between the loads applied to the bridge and the corresponding strains (and hence stresses) in each of the arch ribs. This information would allow the prediction of the loads accepted by each of the arch ribs under a given loading condition and the documentation of the transverse distribution of loads. In addition, the web plates were monitored to determine whether they were contributing to the arch-rib section in a normal manner and accepting some of the compressive loads.

TESTING PROCEDURES

The Department of Civil and Mineral Engineering at the University of Minnesota was retained to provide the instrumentation for the load test. A total of 18 strain gauges were installed in a line near the east abutment, 18 in. from reference point 40 on panel 40 (see Figure 3). Twelve of the gauges were installed on the flanges of the main plate girders, with all the bottom flanges instrumented on the inside face and the top flanges on the outside face. The top flange gauge in Arch 4 was placed on the inside face because of the unevenness of the outside face. Three gauges were installed on two web faces at angles of approximately –45, 0, and +45 degrees to the horizontal (strain gauge rosette) in areas where the web was buckled. An external dummy gauge was also used to compensate for the temperature variation throughout the duration of the loading.

Three dump trucks, each with a gross weight of 27.5 tons, were used to produce nine different test load patterns. Some of the load patterns were mirror images. This was to see whether the load distribution from one longitudinal half of the bridge to the other was symmetric about the middle. The strain gauges were read twice during each load position and were then rezeroed before the beginning of the next loading.

TEST FINDINGS

The data obtained during the testing were recorded and interpreted as follows:

1. The live load distribution between the arch ribs could be approximated from the data provided. Careful analyses of the magnitude of forces and moments were made to ensure that these quantities were reasonable. It should be noted that given the buckled condition of many of the webs, the usual assumption of a linear strain distribution is questionable. The readings

obtained from the midweb horizontal gauges suggested a distribution very different from that expected by a straight line distribution between the top and bottom flange gauges. It was therefore concluded that calculations using linear-elastic theory and the measured strains would not be expected to correlate well with the results of a linear-elastic analysis of the structure using the entire section properties.

2. The plates making up the arch ribs carried not only compressive stress, but also stress from bending moments. The trends through the test clearly showed that the compression produced by the dead loads on the bottom flange was relieved by the live loads. The results indicated that the arches were not acting as simple compression members.

3. The webs, although buckled, seemed to be working effectively through a tension-field mechanism. Thus, the webs were able to transfer forces by forming diagonal bands in tension, which resulted in a structural action similar to that of a truss. If this model is correct, the flanges are likely to be carrying higher stresses than the elastic theory would predict, whereas the webs are carrying lower stresses. This was supported by the experimental observations, which indicated that although very little or no force was present in the horizontal direction at the mid-depth of the web, the gauges at 45 degrees to this direction showed significant levels of stress.

4. The floor beams, diagonals, and diaphragms did not seem to transfer significant forces from one side of the bridge to the other; when the loads were placed on only one side of the bridge, very little load was transferred to the outside arch at the other side.

5. Investigation and analysis of the lack of rigidity of the panel point connections between the members making up the arches, although these connections do have a finite rotational stiffness, resulted in slightly lower moments and higher axial loads, but not by more than 10 to 15 percent. The main difference in these analyses came from the deflections, which began to increase rapidly as the rotational stiffness was diminished. As the stiffness was reduced to about 1,000,000 kips/radian, the centerline deflection of the arch with a truck (55-kip) load at joint 3 increased to about 5.5 in.

Inclement weather and technical problems were experienced during the instrumentation of the Hennepin Avenue Bridge. Ambient temperatures during the test dates in December 1983 averaged +20°F, which created problems with the attachment of strain gauges. Falling debris from the steel grating that served as the bridge deck made working conditions hazardous. The strain gauge placed on the bottom flange of Arch 2 failed just before the testing, preventing a complete independent measurement of the load distribution on one of the arch ribs. Financial considerations limited the level of instrumentation effort, and lack of equipment prevented dynamic load testing.

The recorded strain values were smaller than desirable to achieve precise analysis. The presence of utilities on the bridge also resulted in significant electrical noise. However, the values obtained were determined to be valid for the purposes of the test. Additional test load weight on the bridge would have provided larger strain values, but also presented the danger of permanent structural damage.

SUMMARY

The unique two-hinge and three-hinge arch design of this 100-year-old bridge made the determination of live load distribution an obvious issue. The limited bending capacity of the critical arch members because of their "segmental design," the badly deteriorated batten plates, the known decreases in structure dead load and increases in live load, and the buckled web plates all made the issue of load distribution a critical one. For a meaningful evaluation of the bridge, a determination of the load distribution was required.

The instrumentation of the bridge was successful in providing data for making better decisions about the key issues of transverse distribution and buckled web behavior.

The successful testing also played a major role in the rating of the bridge. It was determined that the entire section of the arch was not contributing to the section properties, thereby increasing stresses in the flanges of the arches. The interior two-hinged arch closest to the three-hinged arches was found to be carrying more than its share, making this arch critical over the more lightly designed exterior arches. The instrumentation revealed that the compression in the lower arch flange was being completely relieved and subjected to tension. This last finding revealed that the seriously deteriorated batten plates were controlling the rating. Immediate repairs to the batten plates were required to ensure safety of the bridge.

Finally, the test findings were very helpful in developing alternatives for possible rehabilitation of the bridge. They confirmed that any alternative would have to address improvements in relative load distribution to the arches. Alternatives studied included regrouping of the two- and three-hinged arches so that all the same designs would be placed in the same spans, introducing a center longitudinal joint separating the arch designs, and developing a stiffer deck to assist in load distribution.

ACKNOWLEDGMENTS

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REFERENCE

1. *Hennepin Avenue Bridge Structural Analysis Inspection Report.* Howard Needles Tammen & Bergendoff, Minneapolis, Minn., June 1984.

Publication of this paper sponsored by Committee on Dynamics and Field Testing of Bridges.