that the intrados of the jacketed arches at the spring lines maintained the existing curvature, thus retaining the original appearance of the arches.

Partial fixity at spandrel column base

The articulation of the structure by means of expansion joints in the deck or hinges, rollers, and sliding joints in the columns affects both the arch ribs and spandrel columns. One of the primary reasons for articulating the superstructure is to protect the superstructure from excessive forces that will otherwise be present. In general, the effect of the superstructure is beneficial to the arch proper. Any moment tending to flex the rib is resisted not only by the arch but by the connecting columns, with a consequent reduction in flexural demands in the arch. This reduction is accomplished at the expense of the superstructure in which certain forces are induced by means of its connection to the spandrel bents. In addition to the moments due to external loads, the deformation induced internal forces due to temperature changes, shrinkage, and creep tend to increase the rib moments that are in turn shared by the superstructure. These forces are kept within the permissible limits either by placing expansion joints in the deck as in the existing bridge or by inserting hinges at one or both ends of the spandrel columns (McCullough 1948).

In the current context, the longitudinal forces due to thermal movement in the deck and the elements of the arches, seismic forces, and other load effects were transmitted to the arch ribs through the spandrel columns in the form of in plane shear and bending moment. Therefore, the bases of the taller spandrel columns located between the arch spring lines and span third points were detailed to behave as a one-way (longitudinally) concrete hinge with partial fixity such that the moment transfer from the spandrel columns to the arch ribs was minimal. The spandrel columns located in the middle third of the arch spans on either side of the arch crowns were designed with fixed bases. Structural analyses on the proposed bridge indicated that this technique was one of the most effective means of reducing the flexural demands in the arch ribs. For Partial Moment Connection details, see Figure 9.

Use of longitudinal bracings

As explained earlier, the partial fixity at the base of spandrel columns located on either side of the crown in each arch span reduces the force demands in the arch ribs. The structural details in Figure 8 ensure a reduced yet finite value of rotational stiffness and flexural capacity of the hinge connection. Nevertheless, the longitudinal bracings were designed to connect adjacent spandrel bent caps to the arch pier bent caps in each span so that the longitudinal stability of the bents would be reinforced without jeopardizing the effectiveness of the partial moment connections. As shown in Figure 10, the bracings were designed as a set of four one-foot (300 mm) wide by one-foot six-inch (460 mm) deep precast concrete sections resting on one-inch (25 mm) thick reinforced elastomeric pads and connected to the respective bent caps by means of a pair of dowels. The longitudinal bracings could act both in tension and/or compression and were designed to have negligible rotational restraint at the respective ends.

Optimizing bearing types and location

The role of bearings is to transfer the vertical reaction from the superstructure to the substructure, fulfilling the design requirements concerning forces, displacements, and rotations. Bearing type selection for this bridge was optimized by considering the relative location of the spandrel columns with different base fixity. To reduce the longitudinal forces from the superstructure, 4-inch (100 mm) thick reinforced elastomeric bearings were designed for the spandrel bents with concrete hinges; whereas, 1-$^1/_2$-inch (38 mm) thick elastomeric bearings with PTFE (Teflon) coated rectangular sliders with a total bearing height of 3-inches (76 mm) were designed for the fixed spandrel bents, the arch piers and the abutments. Seismic analyses indicated that due to the lateral stiffness of the shorter spandrel columns located near the arch crowns, these "stubby" bents exhibit a tendency to attract high shear forces resulting in increased seismic demands at the spring lines of the arches. The sliders were selected for these spandrel bents to limit the longitudinal shear demands on the spandrel columns located near and on either side of the arch crowns. All bearings were designed to translate freely in the transverse direction. The arch and the approach pier caps and the abutments were designed with keeper blocks to prevent the deck from undergoing large transverse displacements that could result from a seismic event. For Location and Type of Bearings, see Figure 11.

Use of lock-up device

One of the primary objectives of the seismic design of this bridge was to keep the forces in the elements of the arches within the linear elastic range. Under seismic excitation without the lock-up devices, the continuity of the superstructure deck would induce dynamic interaction of the multi-span arches and higher mode participation in the longitudinal modes of vibration resulting in excessive force demands at specific locations of the arch ribs, arch piers, and the abutments. To protect these elements, suitability of lock-up devices (LUD) were evaluated at the approach bents. These devices are components that allow unrestricted motion at low translational speed (thermal movements); however, under seismic or wind transients, the LUD activate, and dynamically form a rigid brace connection between the superstructure deck and the approach piers. After the transient event ends, the LUD revert to low force output, permitting structural sections to thermally expand or contract without added stress. For the proposed rehabilitation, lock-up devices each with a capacity of 170 kips (756 kN) were designed along the interior beam lines at the approach bents. For Lock-up Devices at Approach Piers, see Figure 12.

The approach bents were designed as "super bents" capable of carrying the loads under extreme events. Each bent was designed on three 6-foot (1.8 m) diameter drilled shafts socketed into the rock.

SUMMARY AND CONCLUSIONS

In addition to addressing issues relating to sustainability as noted earlier, the following innovative design and detailing techniques were used to provide engineering solutions to the rehabilitation of the Henley Street Bridge:

1. A hybrid system comprised of a superstructure deck acting together with the arch ribs was used in the design. The replacement superstructure would be 3.5 times stiffer than the existing deck. The interaction between the arch ribs and the deck contributed to a reduction in forces in the arches.
2. Concrete jacketing of the existing arch ribs designed at the spring lines in spans 1 and 6 provided a means to increase the flexural capacity of the arch ribs at these locations.
3. Partial moment connections designed at the bases of the taller spandrel columns located between the arch spring lines and span third points proved to be most effective in reducing the force transfer to the existing arch ribs.
4. The longitudinal bracings were designed to connect adjacent spandrel bent caps to the arch pier bent caps in each arch span so that the longitudinal stability of the bents would be reinforced without jeopardizing the effectiveness of the partial moment connections.
5. Bearing type selection was optimized by considering the relative location of the spandrel columns with different base fixity. Reinforced elastomeric bearings were designed for the spandrel bents with concrete hinges; whereas, elastomeric bearings with PTFE (Teflon) coated rectangular sliders were designed for the fixed spandrel bents, the arch piers and the abutments. All bearings were designed to translate freely in the transverse direction. The arch and the approach pier caps, and the abutments were designed with keeper blocks to prevent the deck from undergoing large transverse displacements that could result from a seismic event.
6. Lock-up devices were designed along the interior beam lines at approach bents. The approach bents were designed as "super bents" capable of carrying the loads under extreme events.

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Table 1— Minimum capacity-to-demand (C/D) ratios in selected members of the proposed bridge due to non-seismic loads

Member Group	Bending-Y			Bending-Z			Shear-Y			Shear-Z			Torsion		
	C/D	Location[1]	Loading[2]	C/D	Location[1]	Loading[2]	C/D	Location[1]	Loading[2]	C/D	Location[1]	Loading[2]	C/D	Location[1]	Loading[2]
Arch Rib	2.75	3 (B)	V	1.01	1 (A)	VI	1.11	5 (B)	IV	1.50	3 (A)	VC	1.44	6 (7)	V
Arch Pier	1.03	Pier 4	VC	1.06	Pier 3	VC	1.03	Pier 3	I	1.09	Pier 3	VC	1.76	Pier 3	VC
Arch Rib Strut	1.42	3 (2)	V	1.43	3 (2)	V	1.21	3 (2)	I	2.76	3 (4)	II	17.00	3 (4)	II
Web Wall	1.66	Pier 4	VC	1.00	Pier 1	V	1.16	Pier 4	VC	13.29	Pier 4	VC	6.67	Pier 4	VC

Table 2—Minimum capacity-to-demand (C/D) ratios in selected members of the proposed bridge due to seismic loads

Member Group	Bending-Y		Bending-Z		Shear-Y		Shear-Z		Torsion	
	C/D	Location[1]	C/D	Location[1]	C/D	Location[1]	C/D	Location[1]	C/D	Location[1]
Arch Rib	1.10	3 (A)	1.45	1 (A)	1.61	1 (A)	2.10	4 (B)	1.00	4 (8)
Arch Pier	1.23	Pier 3	1.12	Pier 4	1.09	7	1.19	Pier 3	1.16	Pier 3
Arch Rib Strut[3]	13.45	3 (2)	2.00	3 (2)	1.49	3 (2)	10.56	3 (2)	10.56	5 (2)
Web Wall	1.00	Pier 1	1.45	Pier 1	1.13	4	5.96	Pier 1	5.97	Pier 4

Table 3—Minimum factor-of-safety (FS) at the arch pier footings of the proposed bridge

Load Effect	Bearing Capacity		Sliding-Y		Sliding-Z		Overturning-Y		Overturning-Z	
	FS	Location	FS	Location	FS	Location	FS	Location	FS	Location
Non-Seismic	3.48	Pier 7	1.68	Pier 1	2.65	Pier 6	2.31	Pier 6	2.00	Pier 7
Seismic	9.00	Pier 7	1.74	Pier 1	7.80	Pier 7	2.40	Pier 7	2.00	Pier 1

[1] For Arch Ribs and Arch Rib Struts, the number before the parenthesis denotes the arch span number and the character within the parenthesis denotes the grid number (For Typical Arch Span Legend, see Figure 13).
[2] Roman numerals, I through VI represent design group loadings as per Section 3.22 of AASHTO Standard Specifications of Highway Bridges, 17th Edition, 2002, and loading "VC" represents vessel collision group loading as per Section 3.14 of AASHTO Guide Specification and Commentary for Vessel Collision of Highway Bridges, 1991.
[3] For seismic design, Arch Rib Struts were analyzed with unconstrained rotation degrees of freedom (pinned-pinned) at member ends.

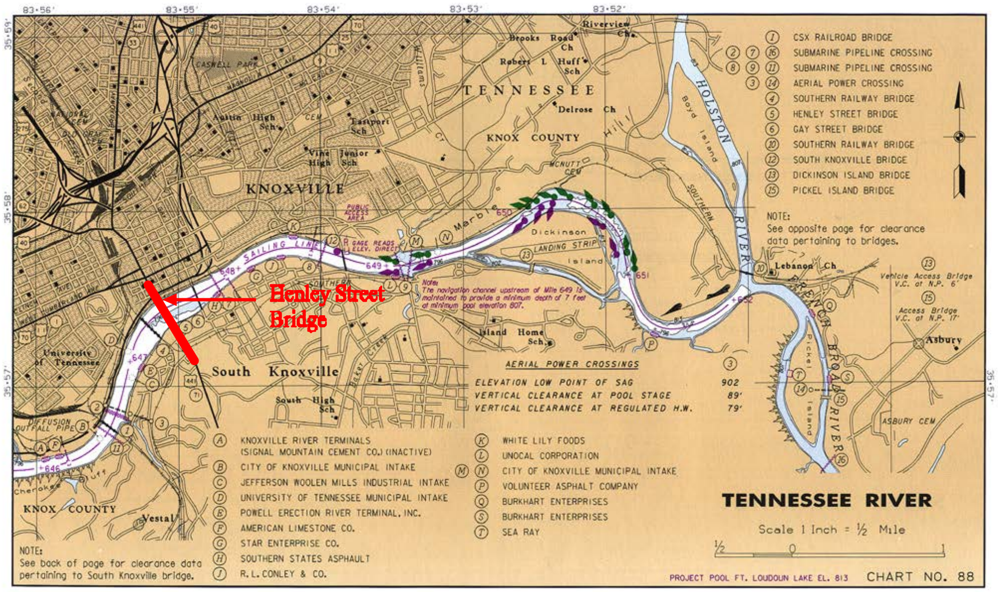

Fig. 1—Project location map

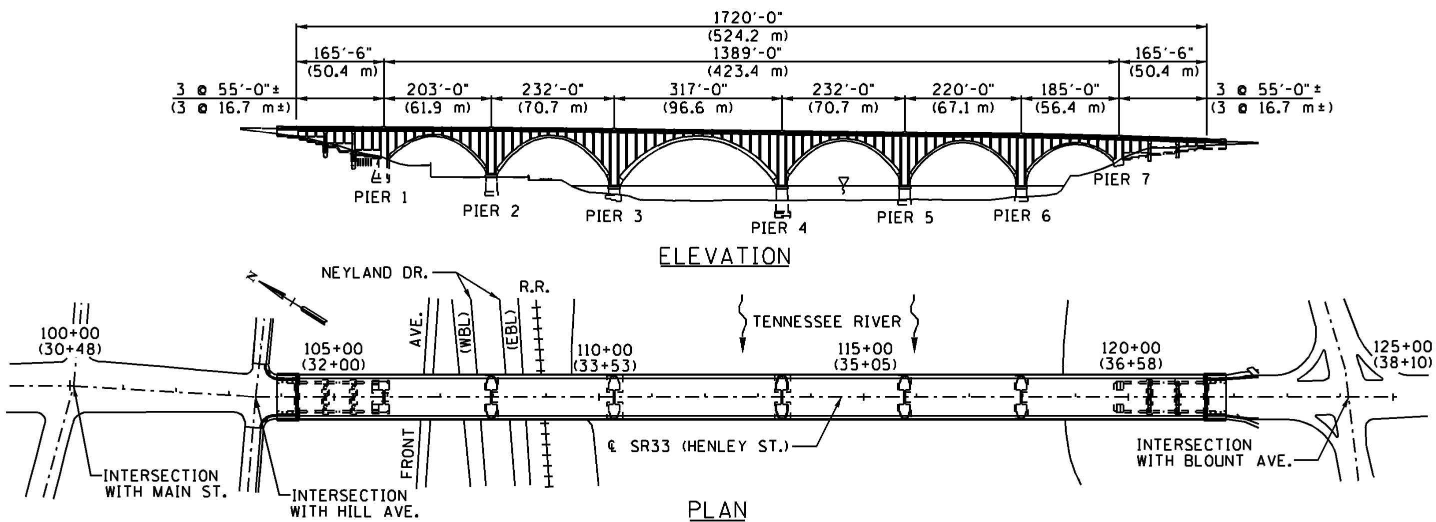

Fig. 2—General plan and elevation

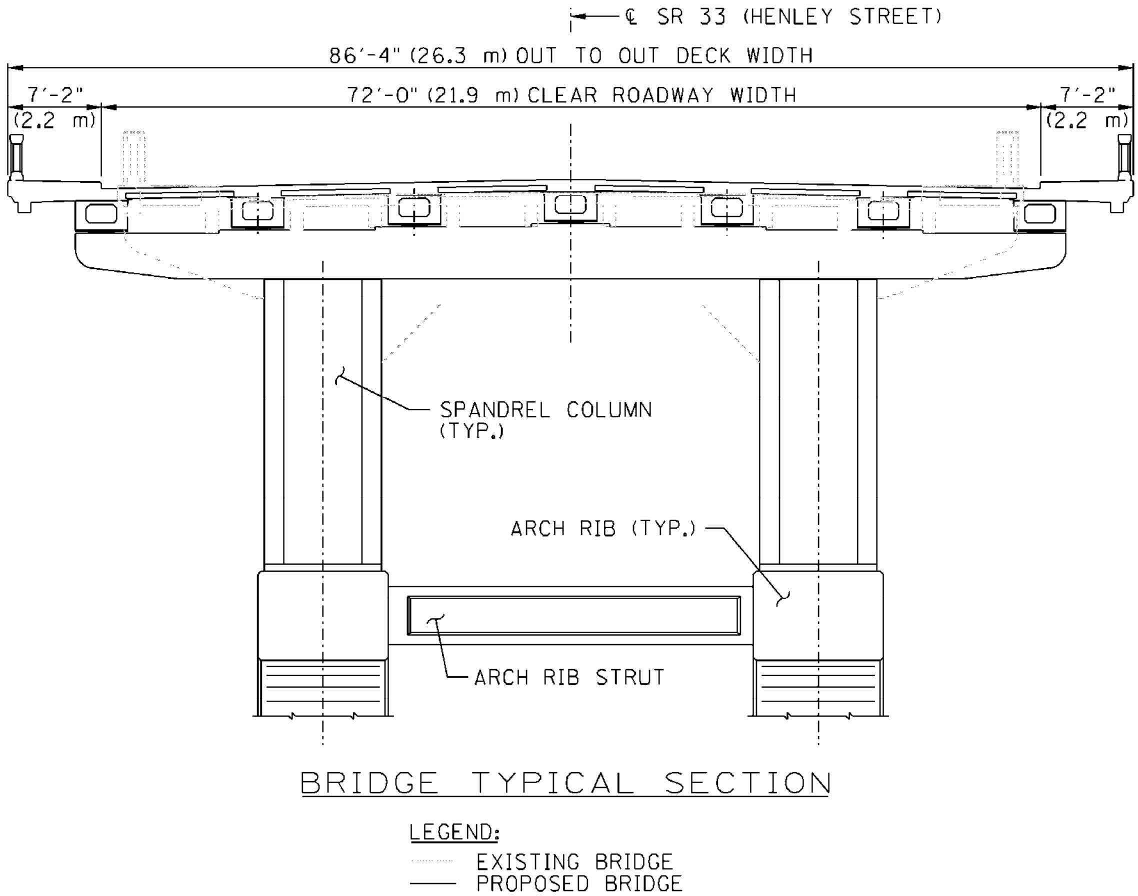

Fig. 3—Bridge typical section

Fig. 4—View of the existing bridge


Spall in the Bottom Edge of the Arch Rib

Spall in the Corner of Arch Pier


Floor Beam Deterioration near Expansion Joints

Spall in the Corner of Spandrel Column

Fig. 5—Typical bridge condition photos

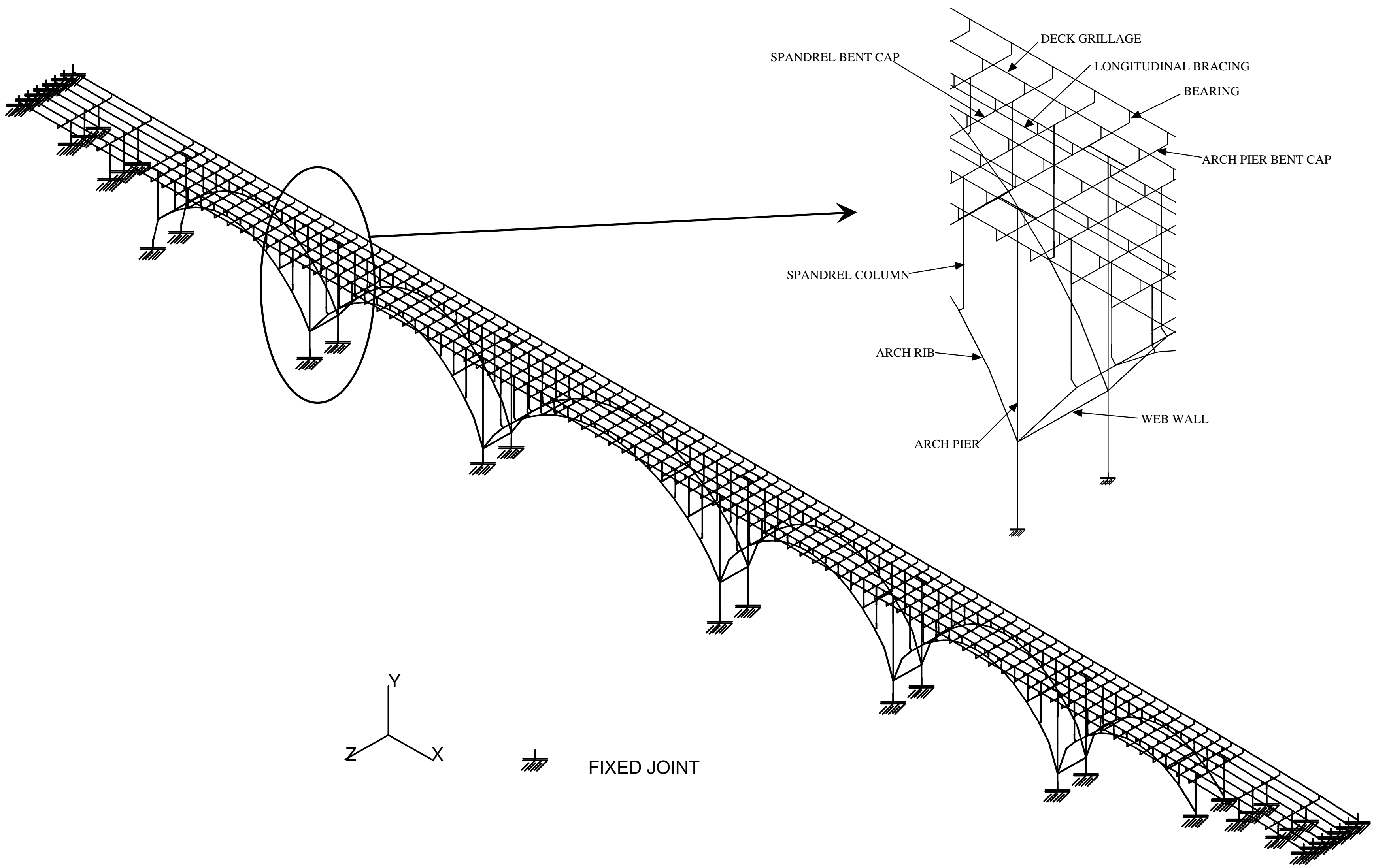

Fig. 6—3D finite element model of the bridge

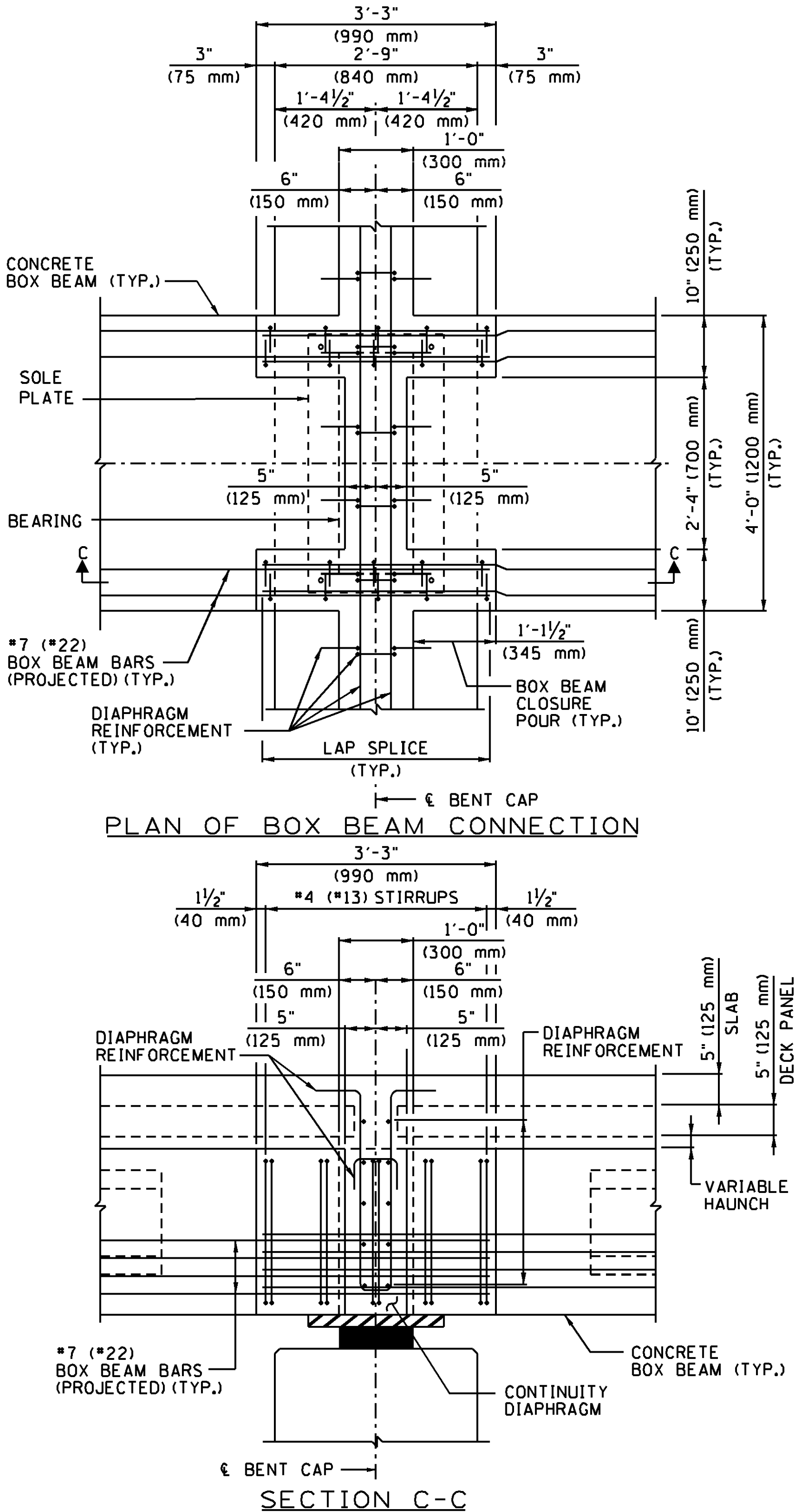

Fig. 7—Box beam continuity in arch spans

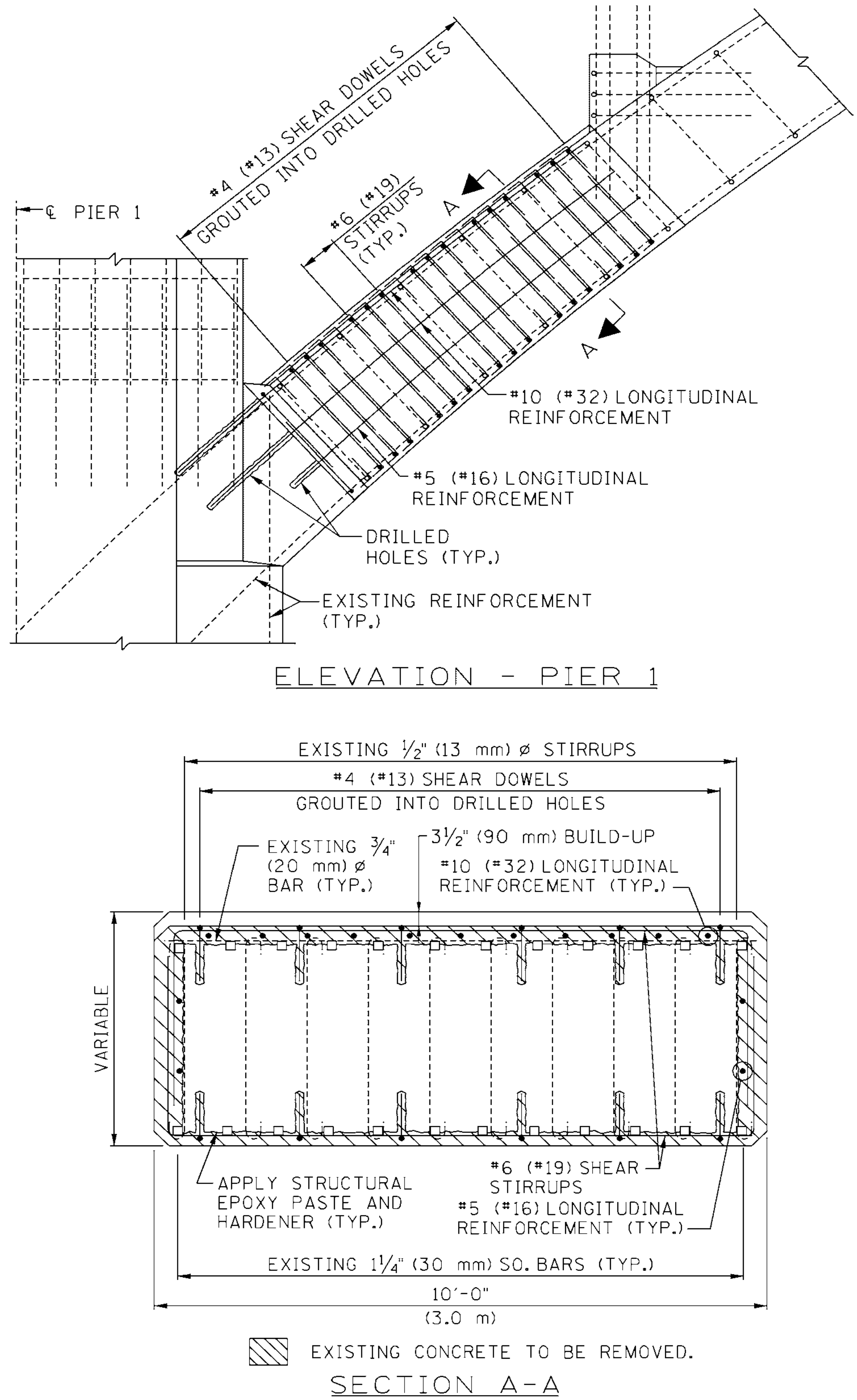

Fig. 8—Arch rib retrofit