

Steel Arches Used in Bridge Reconstruction Over I-5

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This article reviews the design and reconstruction of the Capitol Boulevard Bridge over I-5 in Olympia, Washington. The existing superstructure was retained, and a new arch supporting system was designed to replace the existing concrete piers as part of a \$130,000,000 Interstate widening project. The paper describes the structure types studied, the external peer review comparing the constructibility of the concrete and steel alternatives, and the influence of aesthetics on the selection of the final structure type, two steel arches. The design of the tieback wall system, spread footing, elliptical twin-cell box arches, and load transfer by jacking is also described. The cost of the reconstruction was \$3,900,000, including \$1,600,000 for the tieback retaining wall system.

A decade-long project to widen 7 mi (11.3 km) of I-5 through Olympia, Washington, is scheduled for completion in 1992 and will cost \$130,000,000. Of the 18 bridges spanning I-5, the Capitol Boulevard undercrossing (Figure 1) posed the most interesting engineering challenge. At this location, I-5 is 70 ft (21.4 m) below the Capitol Boulevard Bridge and is confined by embankments on each side. As originally constructed, I-5 had two lanes in each direction separated by a median. Together with shoulders, the overall width was 80 ft (24.4 m); the widening increased the number of lanes to four in each direction and increased the overall width to 146 ft (44.5 m).

The Capitol Boulevard undercrossing was built in 1957 to span the proposed location of I-5, which was to be carved out of a steep natural hillside. This continuous steel plate girder bridge has three spans of 84, 110, and 84 ft (25.6, 33.5, and 25.6 m) with girders spaced at 14.25 ft (4.34 m). The roadway width is 56 ft (17.1 m) curb to curb with two 6-ft (1.83 m) sidewalks. The existing reinforced concrete piers are 75 ft (22.9 m) high, and I-5 below is confined to the 110-ft (33.5 m) middle span between piers. The angle between the centerline of the bridge and that of I-5 is 81 degrees. This angle together with the restrictions caused by the existing piers limited further widening of I-5 to three lanes in each direction with little or no room for inside shoulders.

SITE AND TRAFFIC CONSIDERATIONS

The problem was clearly defined: how to squeeze additional lanes for I-5 under the existing bridge. Several factors made saving the existing bridge superstructure economical. Structurally the deck and girder system were in sound condition.

The steel used in the girders was 50 ksi (344 MPa) high-strength steel, which was not as widely used as ASTM A7 36 ksi (248 MPa) steel in 1957. The concrete in the deck was not cracked, spalled, or deteriorated. There were telephone cables, a gas line, and a recently relocated water line on the bridge. The existing alignment could not be improved without drastically filling and regrading the site. Therefore, building a new bridge adjacent to the existing bridge would be expensive.

I-5 is the only major north-south Interstate highway between the port cities of Portland, Oregon, and Seattle, Washington. It is the primary commercial artery from San Diego, California, to Vancouver, British Columbia. Therefore, a fundamental design consideration, in addition to saving the existing bridge superstructure, was that traffic on the bridge as well as on I-5 below could not be halted or interrupted. However, traffic could be shifted from side to side during construction.

STRUCTURE TYPE

Eight structure types (Figure 2) were considered as replacements for the existing piers. Preliminary sketches were prepared and submitted for architectural review. The concrete sloped-leg portal frame (Figure 3) and steel arch (Figure 4) were selected for further study. Models were built that allowed the aesthetics of the alternatives to be compared, indicated how each blended with the natural environment, and permitted critical evaluation from different perspectives. Specific factors considered in selecting the final structure type were function, form, proportion, color, character, economy, con-

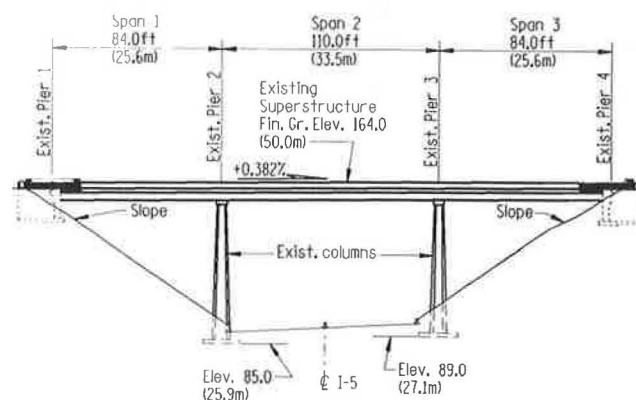


FIGURE 1 Existing structure before rehabilitation.

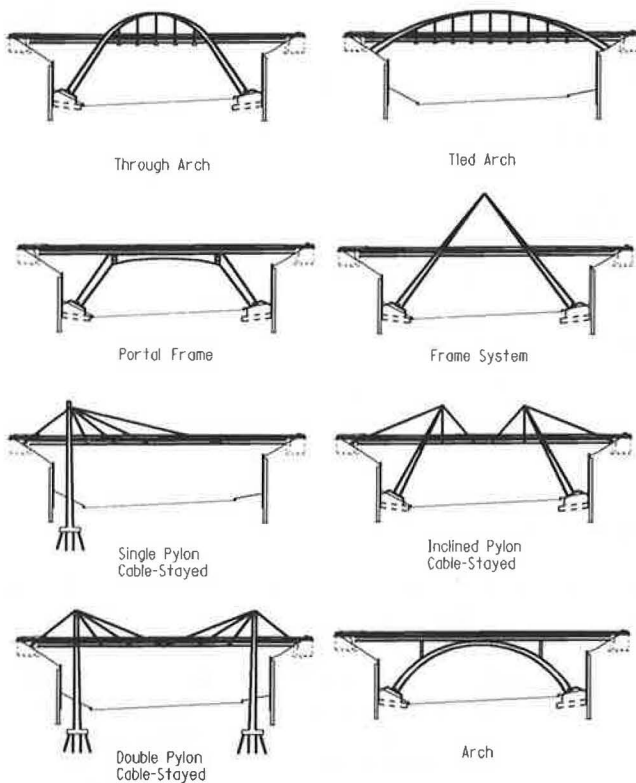


FIGURE 2 Replacement structure types.

structibility, and most important, potential hazard to the highway users.

EXTERNAL PEER REVIEW

The external peer review of the cost estimates and constructibility of the concrete sloped-leg frame and steel arches was prepared by Andersen Bjornstad Kane Jacobs, Inc., Consulting Engineers, Seattle, Washington. It was determined that the concrete sloped-leg frame would be 43 percent less expensive to construct than the steel arch, not including the cost of the tieback walls.

To construct a cast-in-place sloped-leg concrete frame would require falsework on and over I-5 and would necessitate shifting traffic for a period of 5 to 7 weeks. This time frame would allow for constructing falsework, placing forms and concrete, curing of concrete, and form and falsework removal. The outside lanes of I-5 would be closed and a wider median constructed to accommodate a center falsework bent. The construction required for this option would cause traffic interruptions and potential hazard to the highway users.

Precast elements could be used in the sloped-leg concrete frame, which would reduce the traffic hazard period to 3 or 4 weeks. However, erection of two center struts weighing approximately 70 tons (63.5 metric tons) would require closing I-5 if they were cast full-length and erected without a center falsework bent. Halting traffic on I-5 for any period was unacceptable. If the center span struts were cast in two pieces, a

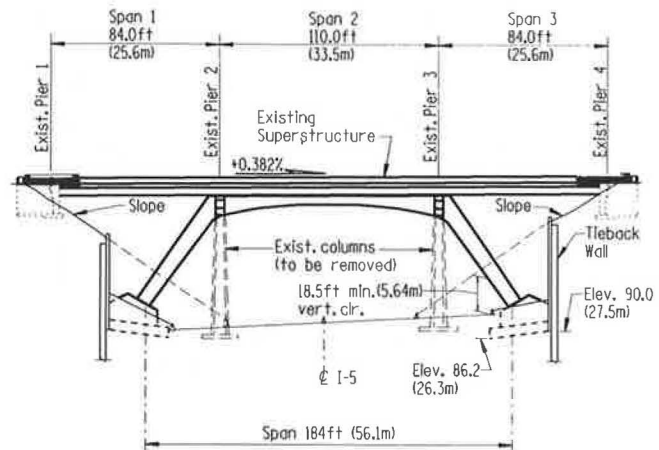


FIGURE 3 Concrete portal frame alternative

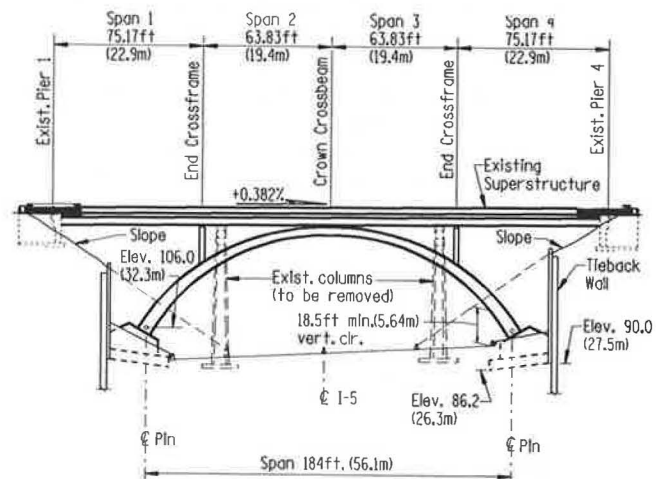


FIGURE 4 Steel arch alternative.

center falsework bent would be required and would pose construction problems similar to those of the cast-in-place frame.

The steel arch alternative would require no falsework. The steel elements could be supported by the existing Capitol Boulevard Bridge girders. It would be necessary to restrict traffic to the two center lanes on the Capitol Boulevard Bridge, without load restriction, until the arch was self-supporting. Traffic on I-5 below would have to be shifted from side to side during delivery and erection of the center crown cross-beam, two end crossframes, and four arch halves. One week would be required to erect and connect all elements of the steel arch assuming that all components of the arch were properly fabricated.

The external peer review concluded that the steel arches would cause the minimum traffic interruption and hazard to the public.

AESTHETICS

An aesthetically correct structure causes the observer to experience a positive emotional experience as a result of his or

her visual perception of the simplicity, strength, and purpose of a structure. According to F. Leonhardt (1):

The arch is the strongest embodiment of a bridge, its shape expresses obviously its ability to carry the loads across a river, valley, or gorge. Therefore, arch bridges are considered beautiful by their evidently suitable shape. This is valid for small and large arch bridges alike.

From the standpoint of aesthetics, the curved lines of the steel arch are more pleasing than the straight lines of the concrete sloped-leg frame.

Three major factors favored the selection of the steel arch: ease of construction, minimal hazard to the highway users, and beauty.

PERMANENT SHORING WALLS

Before constructing the arch foundation, excavation of the steep slopes was necessary. To provide space for constructing the arch foundation and to prevent sliding of the existing end abutments of the Capitol Boulevard Bridge, a permanent retaining wall system with a maximum height of 54 ft (16.5 m) was needed. Site conditions, such as the close proximity of the proposed arch abutments, the limited vertical clearance under the bridge, and the close proximity of the existing end abutment footings, limited the type of wall that could be constructed.

Four wall types were studied: a conventional tieback retaining wall, an element wall, general proprietary wall systems, and a hand-dug reinforced concrete soldier pile wall. The tieback retaining wall with soldier piles placed through holes in the bridge deck was chosen as the most feasible option.

Soil Conditions

All the soils at the site are glacially consolidated with near horizontal bedding planes. In general, the soils consist of loose-to-medium-dense silty fine sand or sandy silt above elevation 90 (27.5 m), and medium-dense-to-dense silty sand below elevation 90 (27.5 m). A 10- to 15-ft-(3.0- to 4.6-m)-thick layer of wet silt was encountered between elevation 95 to 108 (29.0 to 32.9 m). No static groundwater was found in the four test holes drilled at the site.

Earth Pressure Coefficients

The tieback wall was designed to resist static and earthquake-induced earth pressure. For a wall with another structure within a lateral distance equal to twice the wall height the average static pressure coefficient, \bar{K} , is determined from the following equation (2, p. 40):

$$\bar{K} = K_o - (x/2H)(K_o - K_a) \quad (1)$$

where

- x = lateral distance from adjacent structure to wall,
- H = wall height,
- K_o = coefficient of at-rest earth pressure, and
- K_a = Coulomb's active earth pressure coefficient.

The pressure diagrams for \bar{K} , the lateral pressure resulting from the soil overburden, and weight of the existing bridge abutment are shown in Figures 5a, 5b, and 5c.

The lateral earthquake earth pressure coefficient was determined from a Mononobe-Okabe pseudo-static analysis. The additional dynamic earth pressure coefficient, ΔK_{ae} , was assumed to be uniformly distributed along the wall and computed as follows:

$$\Delta K_{ae} = K_{ae} - K_a \quad (2)$$

where

K_{ae} = the coefficient of lateral earthquake earth pressure, and

K_a = Coulomb's active earth pressure coefficient.

Soldier Piles

Soldier piles were spaced from 6.0 to 7.5 ft (1.8 to 2.3 m) apart and were fabricated from two HP 12- × 84-pile sections that were butt-welded at the flanges (Figure 6). As many as eight tieback anchors were required for each soldier pile because the maximum depth of excavation was 54 ft (16.5 m). These tieback anchors were drilled at an angle of 15 degrees to the horizontal. Large vertical forces from the vertical component of the tieback anchor forces were resisted by an allowable skin friction value of 1 kip per square foot (47.9 kN/m²) and an allowable end bearing value of 5 ksf (239.4 kN/m²) at the bottom of the soldier pile.

Anchors

The high-strength steel prestressing strands were designed for anchor loads ranging from a low of 60 kips (267 kN) to a high of 125 kips (556 kN). The capacity of each anchor is dependent on the type of anchor installed, drilling equipment used, experience of the contractor, and quality of workmanship. Four anchors on each wall were verified by performance tests. In addition, each anchor was proof-tested before acceptance. To ensure long-term stability of the tieback wall, the free stressing length or no-load zone was extended beyond the critical failure plane, but not less than 17 ft (5.2 m) into the soil in order to avoid unacceptable prestress losses resulting from creep in the anchor or soil. The tieback wall and anchor configuration are shown in Figure 7.

The complete encapsulation of the anchor was accomplished by installing the anchor in a corrugated high density polyethylene tube and pressure grouting inside and outside the tube. In the no-load zone nearest the wall, the strands were individually greased, sheathed, placed in the polyvinyl chloride (PVC) or high-density polyethylene tube and pressure grouted inside the tube. They were not grouted on the outside, but were surrounded by a nonstructural filler such as a weak soil-cement mixture.

Timber Lagging

Treated timber lagging was 4 × 12 in. (10.2 × 30.5 cm) for soldier pile spacing 7.0 ft (2.13 m) or less, and 6 × 12 in.

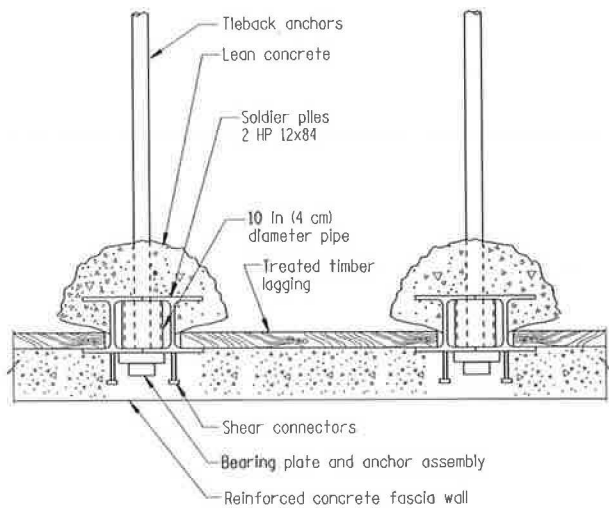


FIGURE 6 Tieback wall cross section.

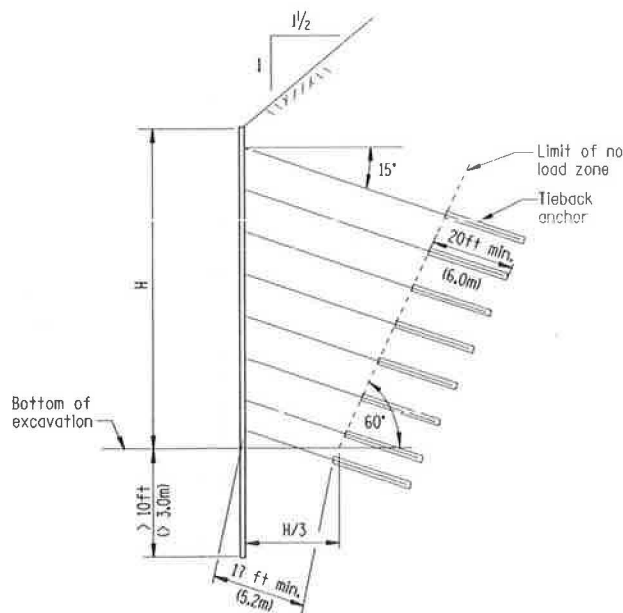


FIGURE 7 Tieback wall anchor configuration.

(15.3 × 30.5 cm) for soldier pile spacing greater than 7.0 ft (2.13 m).

The lagging was designed for a uniformly distributed pressure equal to 50 percent of the lateral earth pressure and was assumed to be simply supported between the soldier piles. The 50 percent reduction was a result of the soil arching effect behind the wall. The uniform pressure distribution approximating the actual parabolic distribution was derived from Terzaghi's experiments (3, p. 267).

Fascia Wall

The reinforced concrete fascia wall has a minimum thickness of 12 in. (30.5 cm). The concrete strength had a compressive

strength of 3,000 psi (20.7 MPa) and was anchored to the soldier piles by two rows of concrete shear connectors spaced 12 in. (30.5 cm) on center. The exterior of the fascia wall had a fractured fin finish.

ARCH FOUNDATION

Arch bridges are normally founded on rock. Any movement of the support, particularly for fixed-end arches, will produce additional stress on the arch and, if not corrected, lead to a sag at the crown. Rock stratum does not exist at this site; therefore, the arch would have to be supported on either a spread footing or a pile-supported footing.

When compared to pile-supported foundations, spread footings are relatively inexpensive. Driving and splicing piles under the existing Capitol Boulevard Bridge would be expensive because of the limited vertical clearance and would conflict with the adjacent tieback retaining wall. The reinforced concrete arch abutment and spread footing (Figure 8) were more economical to construct than a pile-supported foundation. The bottom of the footing was located at the top of the medium-dense-to-dense silty sand layer to avoid a layer of compressible sandy silt. The total weight for each arch abutment, including the 31.5- × 37.5- × 5-ft-thick (9.6- × 11.5- × 1.5-m)-thick footing, was 825 tons (748 metric tons). The settlement for total dead and live load, which produced a maximum foundation pressure of 3 tons per square foot (287.3 kN/m²), was estimated to be 2.7 in. (6.9 cm).

Space under the arch pin bearings was provided for six 100-ton (90.7 metric ton) jacks so that the arch could be restored to its original position (if unanticipated and excessive foundation settlement should occur). All settlement was expected to occur within 2 weeks after load transfer from the existing piers to the arches. Footing settlement was monitored continuously during construction.

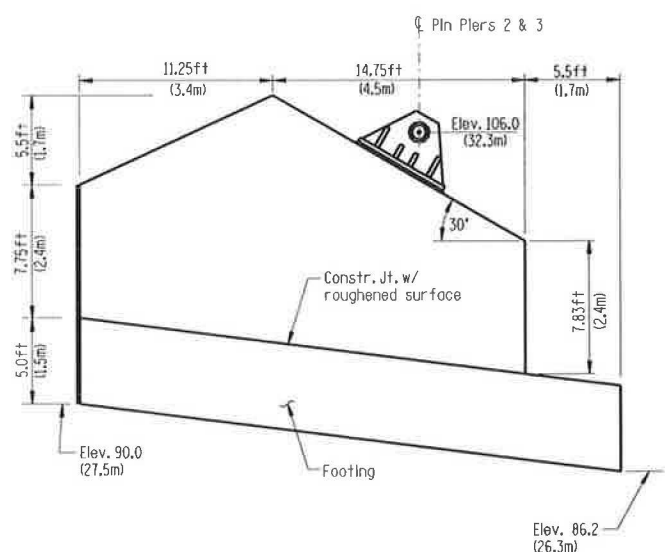


FIGURE 8 Reinforced concrete abutment and footing.

ARCHES

Unique Elliptical Arch Shape

A two-hinged arch was chosen because of potential support settlement, which in a fixed arch would produce large moments. The hinges or pins are located at each abutment. For a two-hinged arch with a rise-to-span ratio greater than 0.2, the ideal arch shape for a uniform dead load over the entire span is a parabola (4, p. 121). Modern steel arch bridges have such a shape (5, p. 281–303; 6, p. 626; 7, p. 124).

Three major geometrical constraints dictated the arch shape: minimum span and rise, maximum vertical clearance for traffic on the inside lane and shoulder of I-5 below the arch, and adequate vertical clearance beneath the existing pier cross-arms to permit existing pier demolition. Clearances for parabolic and elliptical shaped arcs were compared. The parabolic arc provided more clearance beneath the pier crossarms, but the elliptical arc provided greater vertical clearance for traffic beneath the arch, which was a major factor favoring the choice of a symmetrical elliptical arc (Figure 9) for the arch shape.

The equation of an ellipse is

$$x^2/a^2 + y^2/b^2 = 1 \quad (3)$$

The parametric coordinates that satisfy this equation are (8, p. 2)

$$x = a \cos \phi \text{ and } y = b \sin \phi \quad (4)$$

Differentiating, the slope at any point on the elliptical arc is (8, p. 2)

$$dy/dx = -(b/a) \cot \phi \quad (5)$$

From the crown of the arch, offsets to any point on the arch, for fabrication purposes, can be determined from

$$Y = b(1 - \sqrt{1 - x^2/a^2}) \quad (6)$$

where a is the major axis and b is the minor axis of the ellipse.

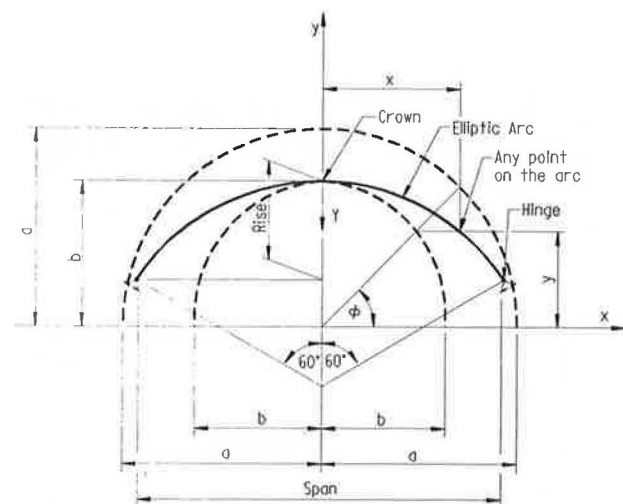


FIGURE 9 Symmetrical two-hinged elliptical arch.

Arch Analysis and Design

There are two arches spaced 54 ft (16.5 m) on center each with a span of 184 ft (56.1 m) and a rise of 49 ft (15.0 m). The depth of the cross section varies from 6.0 ft (1.83 m) at the springing to 4.0 ft (1.22 m) at the crown. The width of the top and bottom flanges are 7.0 ft (2.13 m). There are three webs that divide the box section into two equal cells (Figures 10a and 10b). High-strength steel with a yield strength of 50 ksi (344 MPa) was used throughout. All welds were made in the shop and all field splice connections were made with 7/8-in.-(22-mm)-diameter ASTM A325 high-strength bolts.

The arch was analyzed using the computer program STRUDL. The elliptical arc was modeled by a series of straight members (9, p. 337–338). The following load cases were used:

1. $DL + LL + I$ + differential settlement
2. $DL + LL + I$ + differential settlement + temperature
3. $DL + LL + I$ + differential settlement + wind
4. DL + differential settlement + earthquake.

Load Case 4 governed because the structure is located in a seismically active region.

The arches are 7.0 ft (2.13 m) wide and are braced at the quarter points by the end crossframes and at the crown by a crown crossbeam. The arches can resist transverse forces with-

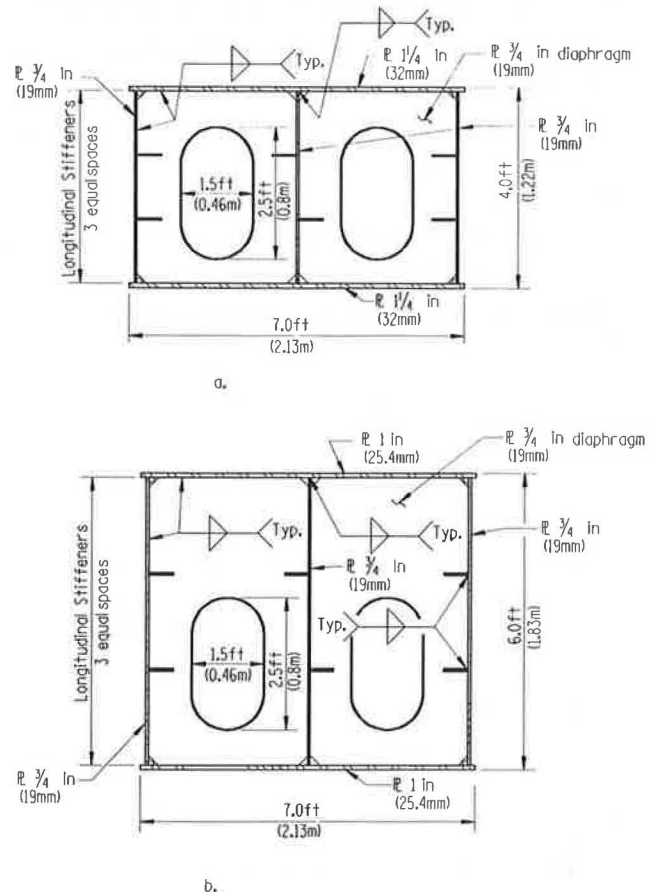


FIGURE 10 (a) Arch cross section at crown, and (b) arch cross section near springing.

out the need for cross-bracing, which gives the structure a streamlined appearance.

The flange plate thicknesses were checked for local buckling according to AASHTO specifications (10, p. 145–146) so that

$$b/t_f = 4250/(f_a + f_b)^{0.5} \leq 47 \quad (7)$$

where

b = width of flange plates between webs and
 t_f = flange thickness.

Two longitudinal stiffeners were used at the third points of each arc web so that (10)

$$D/t = 10,000/f_a^{0.5} \leq 120 \quad (8)$$

$$I_s = 2.2 Dt^3 \quad (9)$$

$$b'/t' = 1,625/(f_a + f_b/3)^{0.5} \leq 12 \quad (10)$$

where

D = web depth,
 t = web thickness,
 I_s = moment of inertia of the web stiffener about its base,
 b' = width of outstanding stiffener element,
 t' = thickness of outstanding stiffener element,
 f_a = axial stress resulting from the applied loadings, and
 f_b = bending stress resulting from the applied loadings.

The arch sections satisfied the following interaction equation (10, p. 145–146)

$$f_a/F_a + f_{bx}/F_{bx} + f_{by}/F_{by} \leq 1.0 \quad (11)$$

where

F_a = allowable axial stress, and
 F_b = allowable bending stress.

The allowable axial stress, F_a , was determined from (10, p. 145–146)

$$F_a = (F_y/2.12)[1 - (KL/r)^2 F_y/4\pi^2 E] \quad (12)$$

where

F_y = yield stress of the steel,
 2.12 = factor of safety,
 K = 1.1 for a two-hinged arch with rise-to-span-ratio less than 0.3.
 L = one-half the length of the arch rib for in-plane buckling,
 r = radius of gyration, and
 E = modulus of elasticity of steel.

Load Transfer

New girder bearing stiffeners were field-bolted to the existing girders at the end abutments, end crossframes, and crown crossbeam. Permanent steel jacking beams were placed between the existing girders, which were spaced at 14.25 ft (4.34 m)

centers and field-bolted to the bearing stiffeners. These beams were designed so that the existing superstructure could be lifted off the existing piers and the load transferred to the new arches.

Jacks were located adjacent to the girders (Figure 11) with space allowed for new bearings under the girders. Jacking was completed at the end crossframes before jacking at the crown crossbeam. This applied most of the superstructure dead load as two symmetrical concentrated loads 28 ft (8.5 m) from the arch pin bearings, which reduced the bending moment on the arch.

The contractor was permitted to jack the superstructure up 2.5 in. (6.4 cm) maximum at each end crossframe and 2 in. (5.1 cm) at the crown crossbeam so the girders would not be overstressed. During jacking, strict tolerances for differential deflection of the existing deck/girder system were established to prevent damage to the existing 7.25-in.- (18.4-cm)-thick reinforced concrete deck. A maximum differential settlement of 0.125 in. (3 mm) between adjacent girders was specified. Survey readings were taken 12 hr, 24 hr, and 15 days after load transfer to monitor foundation settlement. Minor settlement could be corrected by jacking and shimming the girder bearings, whereas excessive settlement could be corrected by jacking and shimming the four arch bearings. Fortunately, foundation settlement was less than 0.25 in. (6 mm) after the arch was erected; there was no additional settlement after load transfer.

COST

The contractor, David A. Mowat, Bellevue, Washington, bid \$3,900,000 for the bridge reconstruction, which was 14.7 percent over the engineer's estimate of \$3,400,000. Included in this amount was \$1,600,000 for the tieback retaining walls, which were constructed by Malcolm Drilling Co., Inc., San Francisco, California. The steel was fabricated by Universal Structural, Inc., Vancouver, Washington. A total of 446 tons (404.5 metric tons) of steel was bid at \$1.46/lb (\$3.22/kilogram).

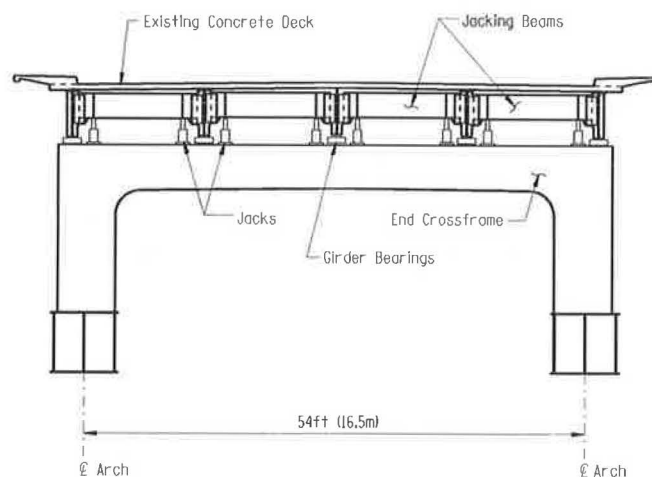


FIGURE 11 End crossframe, jacking beams, and jacks.

SUMMARY

Two steel arches replaced existing reinforced concrete piers of the Capitol Boulevard undercrossing so that I-5 below could be widened from four to eight lanes. Variable depth elliptical arches with no transverse wind bracing were designed, fabricated, and erected over I-5 with minimal interruption and hazard to highway users.

Several alternative structure types were considered, and models were used to study the aesthetics of the bridge types. An external peer review was an important aid to compare cost estimates and constructibility of the alternative bridge types.

Tieback walls were used for shoring during construction and as a final earth retaining structure. Spread footings, not normally associated with arch structures, were constructed and foundation settlement monitored continuously during construction. The transfer of load to the arches from the existing piers was accomplished by simple jacking.

Cost of the project was \$3,900,000, including \$1,600,000 for tieback wall construction.

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