THE DESIGN AND CONSTRUCTION OF THE NEW RIVER GORGE BRIDGE

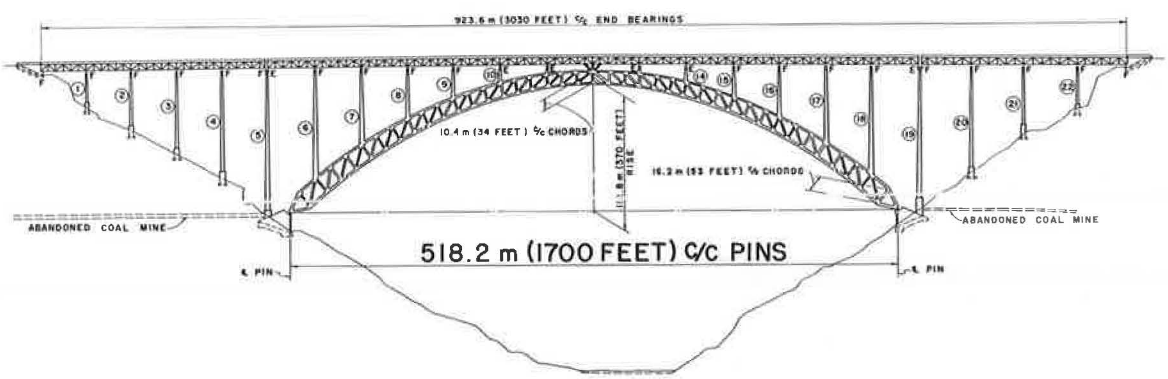
C. V. Knudsen, ASCE-F, Chief Engineer - Structures, Michael Baker, Jr., Inc.
J. F. Cain, Engineer, American Bridge Division, U.S. Steel Corporation

The world's longest steel arch bridge, spanning the New River Gorge in West Virginia, was opened to traffic on October 22, 1977. The overall length of the structure is 923.6 m (3,030 feet), with the main arch spanning a distance of 518.2 m (1,700 feet). During the preliminary design stages, various bridge types were considered. The final decision to build a steel arch was based on a combination of cost and aesthetic considerations. By using a high-strength, corrosion-resistant steel, the weight of the structure was kept to a minimum with the added benefit of maintenance-free steelwork blending with the surrounding rugged terrain. Surface conditions in the coal mining region presented problems during foundation design. Special methods were employed to provide for subsurface stabilization where support bents were located in the proximity of mined-out areas. The computer was a major tool in both the design and erection of the bridge. The computer made it possible to determine the most economical configuration for the main arch, and it was used to study many complicated loading and erecting conditions. Erection of the steelwork presented a tremendous challenge. The height of the structure, 267 m (876 feet) above the river, and the heavy member weights dictated the use of a twin 1,067-m (3,500-foot)-cableway system. Construction of the arch across the gorge proceeded from both sides simultaneously utilizing the unique tieback system to support the cantilevered arch truss halves. The 34-million-dollar structure will be a major link in West Virginia's Appalachian Corridor "L" Expressway System.

The world's longest steel arch bridge spanning the New River Gorge in south central West Virginia was opened to traffic on October 22, 1977. The bridge was designed by Michael Baker, Jr., Inc. for the West Virginia Department of Highways and constructed by the American Bridge Division of United States Steel Corporation. (See Figure 1.)

The total length of the bridge is 923.6 m (3,030 feet). The main 518.2-m (1,700-foot)-arch span is flanked by five 38.6-m (126.5-foot)-continuous deck truss spans on the south and four 43.7-m (143.5-foot)-continuous deck truss spans on the north. Fourteen continuous deck truss spans at 39.5 m (129.75 feet) support the deck over the arch span. The deck of the bridge is 267 m

Figure 1. Elevation View of New River Gorge Bridge.



(876 feet) above the waters of the New River, a height exceeded only in the United States by the Royal Gorge Bridge in Colorado. By way of comparison, the Bayonne, New Jersey arch, built in 1932, has a span of 503.5 m (1,652 feet). The Sidney, Australia harbor arch, also built in 1932, has a span of 502.9 m (1,650 feet).

The overall width of the deck is 22.4 m (73.4 feet) providing space for four traffic lanes, adequate shoulders, and a safety type median barrier. The deck is reinforced concrete with a laytex modified mortar overlay.

The bridge is a part of the Appalachian Development Highway System and will open up a long-needed north-south route across the State of West Virginia. Route U.S. 19 will cross the bridge and will be the connecting link between Interstate 79 and Interstate 77, West Virginia Turnpike.

The cost is shared by the West Virginia Department of Highways and the Federal Department of Transportation. The bid for the construction of the bridge was $34,000,000. The lump sum bid for the superstructure was $25,180,000 for 19,410 M tons (21,400 tons) of steel.

Location Studies

In April, 1967, the West Virginia Department of Highways contracted with Michael Baker, Jr., Inc. for professional services to study a section of Corridor "L" from the north end of the existing Oak Hill Expressway to the vicinity of United States Route 60 near Hico, a distance of approximately 18 km (11 miles). The preliminary report indicated a road to be built within a corridor having a width of from 3 to 5 km (2 to 3 miles). After nearly two years of comprehensive alignment and cost studies, the recommended line was accepted by the State and Federal Departments of Transportation.

New River Gorge was the major obstacle between the two terminals. Due to the steep profile, the cost would have been prohibitive to construct a highway at ground level along the sides of the gorge and crossing the river at low elevation. Thus, studies were directed toward the location of the best alignment for the bridge. Aerial and USGS maps were used to locate the shortest span on an alignment that could be projected to tie in with the approach roads.

Foundation Exploration

A limited number of deep-core borings were drilled on the recommended line to determine subsurface conditions. The area in the vicinity of the proposed crossing had been mined. Preliminary core borings verified open mine shafts and remaining coal support columns. The borings confirmed that the proposed alignment was satisfactory as far as subsurface conditions were concerned. After structure type approval, additional core holes were drilled at the exact locations of each of the substructure units.

The overburden above the coal seam is largely sandstone and of good quality. The abutments for the main arch span are founded on rock below the coal bed seam. Except for the first bent adjacent to both ends of the arch, the distance between the coal bed and the bottom of the footings was well over 30 m (100 feet). The overburden of sandstone was considered adequate to support the superimposed loads from the approach spans.

The proposed bottom of footing for the two bents adjacent to the arch was approximately 18 m (60 feet) above the roof of the coal mine shaft. The mined-out areas were located by drilling a number of 150-mm (6-inch)-diameter holes from the bottom of the footings to the cavities. These holes were used to make a photographic survey of the existing subsurface conditions.

The remaining coal columns appeared stable; however, to assure against possible future subsidence, the mined-out areas below the footings of two bents were filled with a sand-gravel grout mixture. The sand-gravel mixture was dropped through the drilled holes to form a cone with a 1.8-m (6-foot)-diameter against the mine roof. The sand-gravel cones were stabilized by injecting cement flyash into the axis of the cone. The condition of the cones were verified by photographs taken with a camera suspended in unfilled holes.

Span Studies

The recommended alignment required a bridge with an overall length of approximately 915 m (3,000 feet). Various lengths of suspension spans were considered. Due to the height of towers above the gorge and/or difficult tower construction on the sides of the gorge, a suspension-type bridge was not favored.

Continuous deck girder and cantilever truss-type spans were also studied. Numerous piers would have been required that would have been costly and difficult to construct.

By a process of elimination of other types, studies were directed toward a steel arch. Due to the heavy thrust and reaction, it was determined that the abutments for the arch should be founded below the level of the existing coal beds. Existing Route 82 crossed the proposed centerline of the bridge at an elevation near the bottom of the coal bed. Both of these features entered into the selection of the positions for the abutments. Natural features controlled the deck elevation and the elevations and locations of the arch abutments. These features also influenced the geometry of the arch.

Design of the Arch

The existing features were instrumental in arriving at a span of 518.2 m (1,700 feet) with a rise of 112.8 m (370 feet) or a span-depth ratio of 4.6. A two-hinged, truss-type arch was selected. The shape of the arch was adjusted to conform to the dead load reaction thrust curve. Live loads and thermal conditions also influence the truss depth and geometrics. Numerous trial runs were made on the computer with variations in shape and depth of truss. The aim was to balance thrusts so that top and bottom chords were practically of equal size, and so that their areas were not significantly affected by secondary stresses due to thermal stresses or wind loads. The depth of the arch truss was varied from 10.3 m (34 feet) at the center span of span to 16.2 m (53 feet) at the first panel from the pin. The top and bottom chords merged to join at the hinge. Geometrically, the arch is a five-centered curve constructed on chords between panel points. At panel points, the ends of the top and bottom chords intersect on radial lines with vertical truss members being on radial lines. This greatly simplified fabrication details for the truss members, lateral bracing, and vertical bracing as compared to similar details if the vertical members had been made truly vertical.

The arch rests on four cast steel bearing shoes. The normal thrust on the abutment is almost at a 45^o angle. The shoes and the embedded grillages were positioned to transmit a maximum thrust of 9,070 M tons (10,000 tons) into the massive abutments. The pins connecting the arch to the shoes are 690 mm (27 inches) in diameter and free to rotate on the bearing shoes. The ends of the bearing pins were fabricated to include a connection for a strut between the pins. The wind bracing from the upper and lower chords was brought down to intersect and be connected to the strut between the pins. This detail turned out to be simple and effective compared to the usual method of either connecting the lateral bracing to the truss or to a shear key in the abutment. A connection of the lateral bracing to the truss was discouraged by the fact that bolts in the area already extended through metal 360 mm (14 inches) thick.

The top and bottom chords are box members. Webs are 1.47 m (58 inches) deep and covers are 1.0 m (39 inches) wide. Members were kept as small as possible, satisfying L/R and thickness of plate requirements. Full depth cross struts were connected to the top and bottom flanges of the chord members but reduced in depth between connections. Thus, wind areas were reduced which in turn reduced lateral bracing requirements. The slenderness of the members actually enhances the beauty of the structure.

The plans and specifications required all truss members to be fabricated to lengths corrected for dead load deformations and to a temperature of 16^oC (60^oF). Contact surfaces at the ends of the chord members were to be milled and shop assembled for check. At least 75 percent of the contact area was to be in full bearing. For the remainder, a separation not to exceed 0.25 mm (.01 inches) would be permitted. This condition was attained during construction.

The close tolerance may have added to the cost of fabrication; however, the added costs was more than offset by a reduction in size and thickness of gusset plates, splice plates, and number of bolts in the connections. Joints were designed on the basis that 50 percent of the load in the member was carried through by end bearing and the other 50 percent by the bolts. (The 50 percent bolting was adequate for the cantilever during construction.)

Except for minor items, all structural steel in the bridge is ASTM-A588. This is a high-strength, weathering-type steel. High-strength steels are economical for long span bridges. This was especially true for the New River Gorge Bridge. Difficult erection conditions made it imperative that the weight of each individual piece be kept to a minimum. The combined weights of all members, being less by using high-strength steel, tended to further reduce the weights of individual members.

The New River Bridge is in an ideal area for the use of weathering-type steel. The atmosphere is clear of fumes or acids that could contribute to corrosion. Due to the extreme height, the structure would have been costly to paint. The steel has not been painted and has weathered to a dark rustic brown that blends well with the mountainous surroundings. Many dollars were saved during construction and will contribute to additional savings in future maintenance costs.

Approach Spans

The approach spans at both ends of the arch span and the spans over the arch are a series of continuous deck trusses. These spans rest on rectangular box columns. The columns are 1.2 x 1.2 m (4 x 4 feet) at the top, varying to a maximum of 1.2 x 3.5 m (4 x 11.5 feet) at the bottom. All columns in the approach spans were anchored to the concrete foundation by anchor bolts, tensioned to introduce a positive reaction on the base plate. There are thirty-four 83 mm (3-1/4 inch) anchor bolts in the highest column, and each bolt was pretensioned with a force of 103 M tons (235,000 pounds). The tensioning served to anchor the bents prior to erection of the superstructure and to resist lateral wind loads after the bridge was constructed. Due to the terrain, steel bents were more economical to construct than the more conventional concrete piers. The bents and deck trusses are constructed of A588 steel and are not painted.

Fabrication of Steelwork

The deck system was fabricated using numerically-controlled drilling equipment. Because of the recognized accuracy of these mechanisms, the specification's required only 10% of the main truss chord field splices, which were non-bearing, to be shop assembled for fit verification. The field splices for the remainder of the deck system were not assembled until they were finally placed in the field.

Specification's called for progressive assembly of the bent columns in the shop. Segments were completely welded and milled to length. Following this, successive segments were assembled, with the milled ends in bearing, and the field splice holes were drilled using the pre-drilled splice material as a template.

Numerically-controlled drilling equipment was also used to fabricate the arch members. After the box chord members had been welded, drilled and milled to length, the progressive chord method of assembly was followed to verify fit, bearing at the milled joints, and geometry.

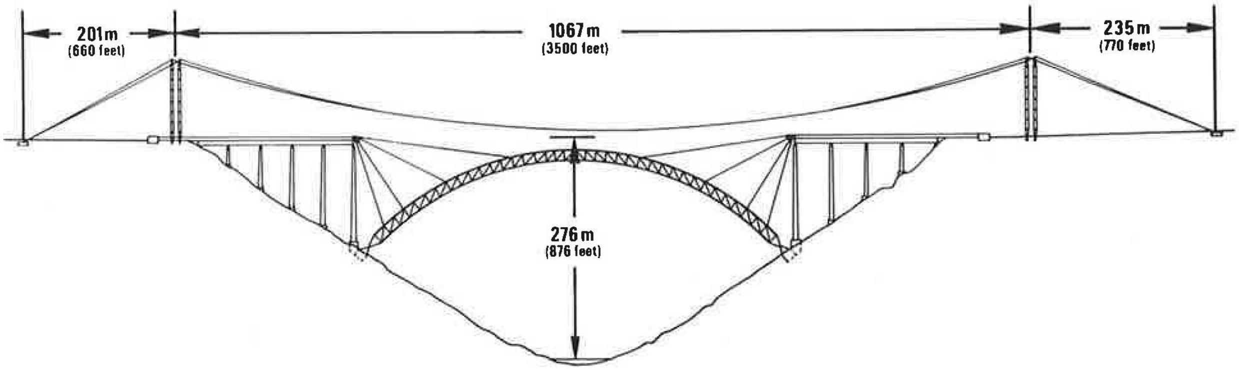
All main bridge members were fabricated using shop welded connections. Exposed welds were made using class E80 electrodes to provide a filler metal compatible with the strength and atmospheric corrosion properties of ASTM A588 material. Field connections were bolted using ASTM A325 Type 3 (weathering type) high strength bolts.

Erection Scheme

The rugged, mountainous terrain presented a tremendous challenge in the erection of the arch. In the early planning stages of construction, engineering studies considered many erection schemes. Because of the gorge's great depth, the use of falsework was ruled out. Finally, the decision was made to cantilever the arch trusses out from both sides of the gorge, using a tieback system for temporary support.

An economical method was needed to move the heavy arch truss members into position over the

Figure 2. Cableway and Arch Tieback System.



gorge. The use of travelling derricks that would move along the top chord of the arch was considered, but this would have imposed prohibitive loads on the steelwork and the tieback system; also, it presented problems in delivering steel to the derricks. The cableway system finally adopted provided the most economical method for both fabrication and erection, minimizing requirements for reinforcement of the main truss members for erection stresses (See Figure 2).

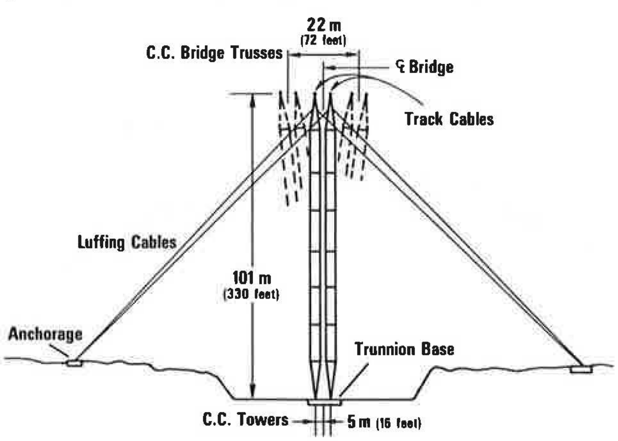
All structural steel was erected using the cableway system. The land bents and approach deck trusses were completed first. Then, the arch trusses were cantilevered out from both sides of the gorge supported by the temporary tieback system. The arch trusses were joined at mid-span, and the tiebacks were removed. Finally, the arch bents and arch deck trusses were erected, completing the steelwork.

Cableway System

Cableways have been used in the past to erect structures over deep canyons, but the twin system developed for the New River Gorge Bridge is in a class by itself. The 1067 m (3,500 foot) span and 90 M ton (100 ton) lifting capacity made it the largest yet constructed.

Each cableway carried a 45 M ton (50-ton) trolley and hoist on two 76 mm (3 inch) diameter track cables that were supported by a 101 m (330 foot) tower on each side of the gorge. The tower bases were set 2.4 m (8 feet) on either side of the bridge center line, and were pinned to permit lateral tilting, or luffing, of each tower. By luffing the towers, the track cables could be positioned over any portion of the 22 m (72 foot) bridge width (See Figure 3).

Figure 3. Luffing Diagram for Cableway Towers.

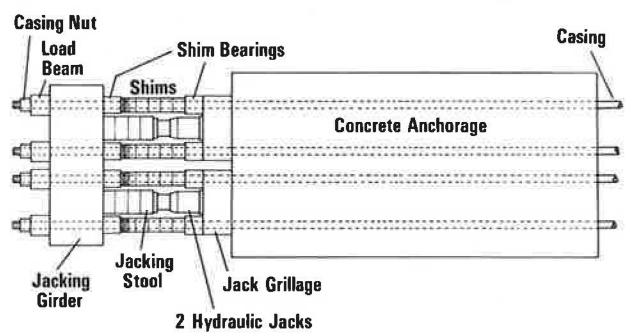


The cableways were operated independently for lifts up to 45 M tons (50 tons); for heavier lifts, the two cableways were operated in unison, providing a lifting capacity of 90 M tons (100 tons).

Arch Tieback System

The tieback system was primarily designed to support the massive arch structure; however, it also included a jacking system that played an important role in the arch closing procedure. Each of the cantilevered arch trusses was independently supported by a concrete anchorage located approximately 70 m (200 feet) behind the abutment at each corner of the bridge (See Figure 4).

Figure 4. Plan of Arch Tieback Anchorage.



Four strings of pipe casing passed through the anchorage and connected to a moveable steel jacking girder. Behind the anchorage were four 1134 M ton (1250-ton) hydraulic jacks that were used to adjust the position of the moveable jacking girder. Shim blocks placed over the casing between the girder and anchorage transferred the tieback load from the girder to the anchorage.

The four strings of tieback casing extended out on the approach steelwork, where they connected to a rocker and linkage device located at the end approach bent (See Figure 5).

Figure 5. Plan and Elevation of Rocker and Linkage Device.

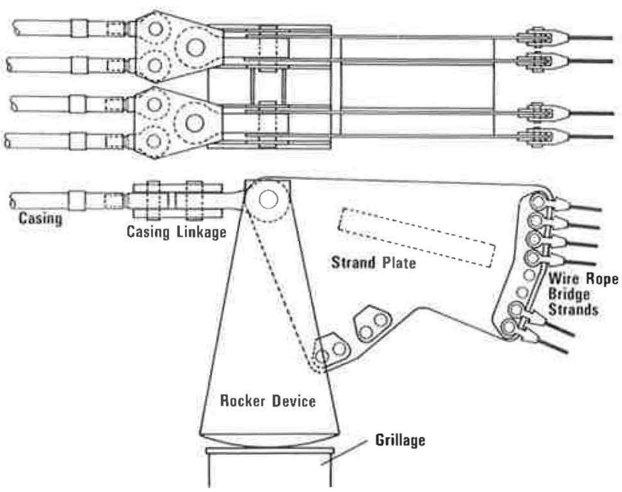
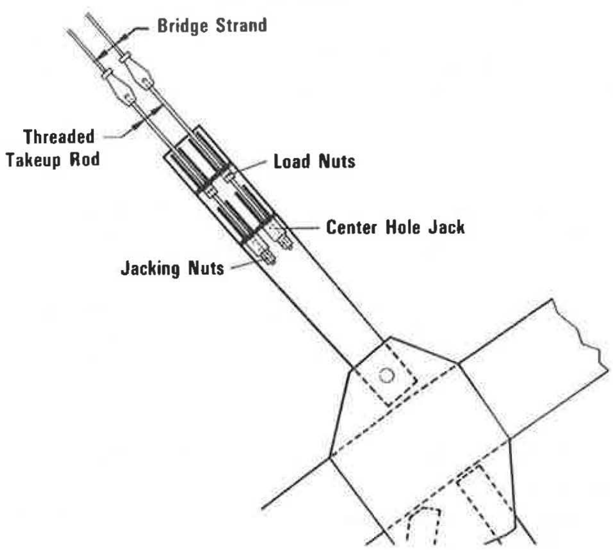


Figure 6. Elevation of Adjusting Device for Arch Tieback Strands.



The rocker device had four strand plates with pin holes for connecting wire rope bridge strands that extended down to various support points on the arch truss. The strands were connected to the arch steelwork using a special tieback adjusting device (See Figure 6).

By means of center hole jacks mounted in the device, the strands were adjusted to equal lengths and equal loads. The tieback strands progressively supported the truss at panel points 2, 4 and 8. While supported at panel point 8, the steelwork was cantilevered out to panel point 14, where another set of tieback strands were connected. Because of the weight of the structure at this stage of erection, tiebacks at both panel points 8 and 14 were used to support the steelwork as it was cantilevered out to mid-span at panel point 20 (See Figure 7).

Arch Closing Procedure

Previously, major arch bridges had been closed by inserting jacks in the gap between the two arch halves to attain the design stress, then filling the gap with a custom-made steel insert piece. The New River Gorge Bridge was the first major arch bridge to be closed without an adjustable closing member.

Deflection calculations showed that the top chord splice points would come into contact first during the closing operation. Therefore, during fabrication, the design details for the top chord splice points were modified, adding pins in each web plate to control alignment (See Figure 8).

Figure 8. Top Chord Web Alignment Pin.

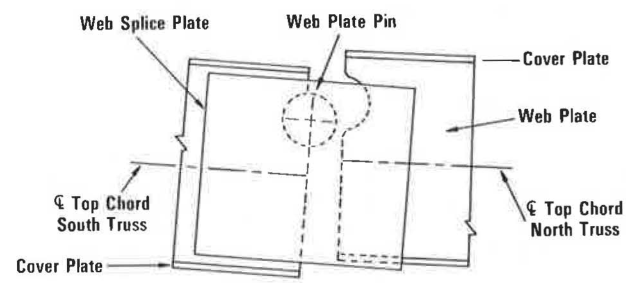
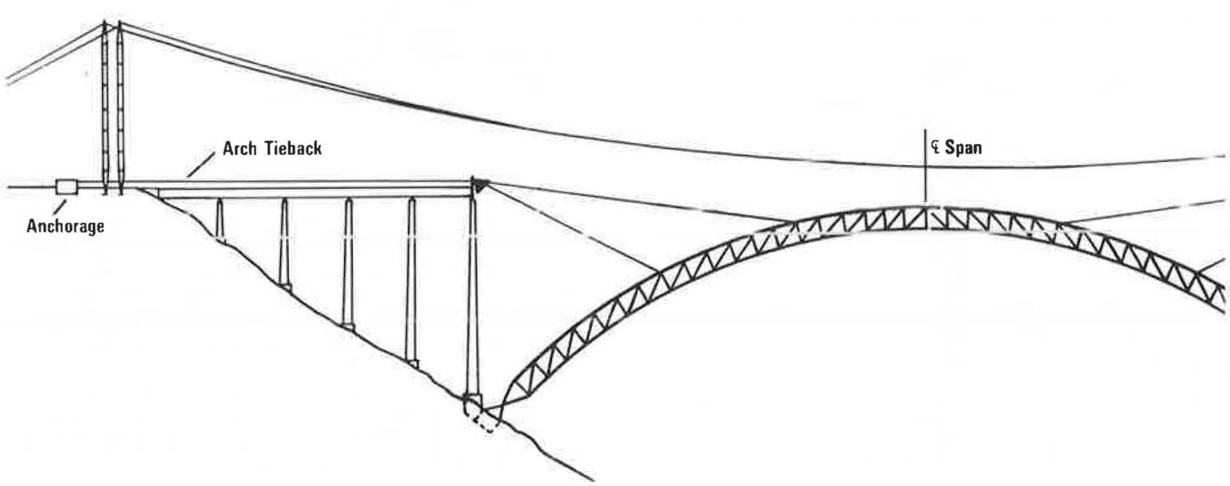
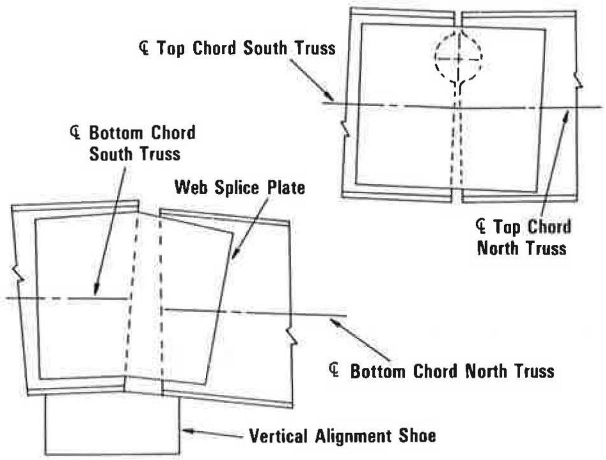


Figure 7. Elevation of Steelwork Cantilevered to Mid-Span While Supported at Panel Points 8 and 14.



The erection procedure called for the truss halves to be erected high. Then, to close the arch, the tieback system was gradually released using the anchorage jacking system. This caused the truss halves to rotate about their hinges and seated the web pins at the top chord splice points (See Figure 9).

Figure 9. Top Chord Web Alignment Pin in Contact, With Bottom Chord on Alignment Shoe.



With the web pins in contact, further relaxation of the tie-backs induced compression forces in the top chords, causing the crown of the arch to rise and the top chords to rotate about the web pins, thereby closing the gap at the bottom chord points. When the bottom chords came into bearing, the top and bottom chord splices were pinned and bolted. The tieback system was then released completely, thereby "swinging" the arch span.

Construction Time

Shortly after the contract was awarded in June 1973, construction work began on the bridge abutments and approach bent foundations. Also, foundations were installed for the cableway system; then the cableway was erected and made operational.

Erection of the approach steelwork began in May 1974 and, following the installation of the arch tieback system, the arch erection was started in July 1975. In May 1976 the arch was closed, and the final deck truss steelwork over the arch was placed in November 1976. The deck slab over the arch, along with the parapets and median were poured in the Spring of 1977; in October 1977 the structure was opened to traffic.