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Colorado River Bridge at Glen Canyon Dam, Arizona U.S.A.

Pont sur le Colorado au barrage de Glen Canyon, Arizona

Brücke über den Colorado-Fluß beim Glen Canyon-Damm, Arizona

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The Colorado River Bridge, completed in February 1959, is a major feature of the Glen Canyon Unit of the Bureau Reclamation's Colorado River Storage Project in northern Arizona. The single-span, steel-arch structure has an overall length of 1,271 feet. It is the highest arch bridge in the world and the second longest of its type in the United States. The bridge spans the Colorado River about 860 feet downstream from Glen Canyon Dam, principal structure of the Storage Project, now under construction by the Bureau. The deck of the bridge is 700 feet above river level, crossing Glen Canyon, one of the spectacular canyons cut by the Colorado River.

The bridge, about 12 river miles downstream from the Arizona-Utah state line, serves as a vital link in the new highway to the remote dam site, extending between Flagstaff, Arizona, and Kanab, Utah, a distance of about 200 miles. As there are no rail facilities near the dam, the highway and bridge are essential to the transportation of construction materials and equipment by truck from the established railheads at Flagstaff and Marysvale, Utah, 135 miles and 200 miles, respectively, from the dam site. The 700-foot high dam and its 28-million-acre-foot reservoir are expected to be major tourist attractions, and the highway and bridge will provide access to the many recreational opportunities in the area.

Construction of the bridge began in January 1957, under a contract awarded to the joint-venture firm of Kiewit-Judson Pacific Murphy of Emeryville, California. The contract called for the firm to furnish and install 7,200,000 pounds of structural steel, 340,000 pounds of reinforcing steel, 100,000 pounds of handrailing, and 2,650 cubic yards of concrete for bridge abutments, skewbacks, and deck.

First plans for the dam included the routing of the highway across the crest of the dam. However, the crest is more than 100 feet below the rim of the canyon; terefore, highway approaches designed to meet the curved crest and fulfill specifications provisions for curvatures, sight distances, and grades would have required a deep rock cut on one side and a tunnel on the other side of the canyon. In addition, the dam crest would have required widening to standard highway width; limited parking facilities parallel to and bordering the traffic lane would have created a traffic hazard. Another aspect considered in the early plans was that, until the dam was completed, the east and west segments of the highway could not be joined, and the full value of the highway during construction would not be realized. Also, a temporary vehicular bridge would have been required during construction of the dam.

Evaluation of these factors led to the conclusion that a permanent highway bridge across the river downstream from the dam and constructed in advance of the dam would best serve the needs of the construction program and other requirements as well. Several bridge sites were subsequently studied. As the shape of the canyon does not change for a considerable distance, both upstream and downstream from the dam site, final selection of the bridge site was made on the basis of geological considerations; that is, where the massive Navajo sandstone of the canyon walls and rim at the bridge site is free of visible geological joints.

Economic Studies of Various Types of Arches

Prior to the final design, various types of arches such as fixed arch with solid ribs, fixed arch with trussed ribs, and 2-hinge and 3-hinge arches were studied. The studies of the fixed arch, which were made first, showed that a solid rib structure was somewhat more economical than a trussed rib. However, the solid rib arch was much more flexible and amplification of dead load, live load, and wind stresses due to deflections increased the stresses materially. Further, in view of the narrow roadway width relative to the length of span, it was felt highly desirable to reduce wind effects, which were much smaller for the truss than for the solid rib. For this reason, studies on other types of arches were limited to truss-type structures.

The 2- and 3-hinge arches studied showed that the weight was practically the same and 14 percent less than the fixed arch. Since the weights were nearly the same for 2- and 3-hinge arches, the final selection of a 2-hinge trussed arch, with hinges in the bottom chord, was made because of greater rigidity than a 3-hinge arch, simplicity of details at the bearings, and fairly uniform chord sections.

Loading

The final design of the 2-hinged arch is shown on figs. 1 and 2. The design was made in accordance with the Specifications of the American Association of State Highway Officials. The 30-foot wide roadway was designed for 2 lanes of H 20-S 16 loading. The uniform live load on the 4-foot wide sidewalks was assumed as 60 pounds per square foot when designing the floor system and 35 pounds per square foot when designing the arches. In the transverse direction, a wind load of 75 pounds per square foot was assumed without live load on the bridge. The load was reduced to 25 pounds per square foot when combined with live load. The wind on the live load was assumed as 100 pounds per linear foot. For overturning and stability calculations, twice the uplift loads given in the specifications were used. Longitudinal wind loads were not considered in view of the shielded position of the bridge in that direction.

A temperature variation of 60° F. normal to 120° F. maximum and 0° F. minimum was assumed for the bridge.

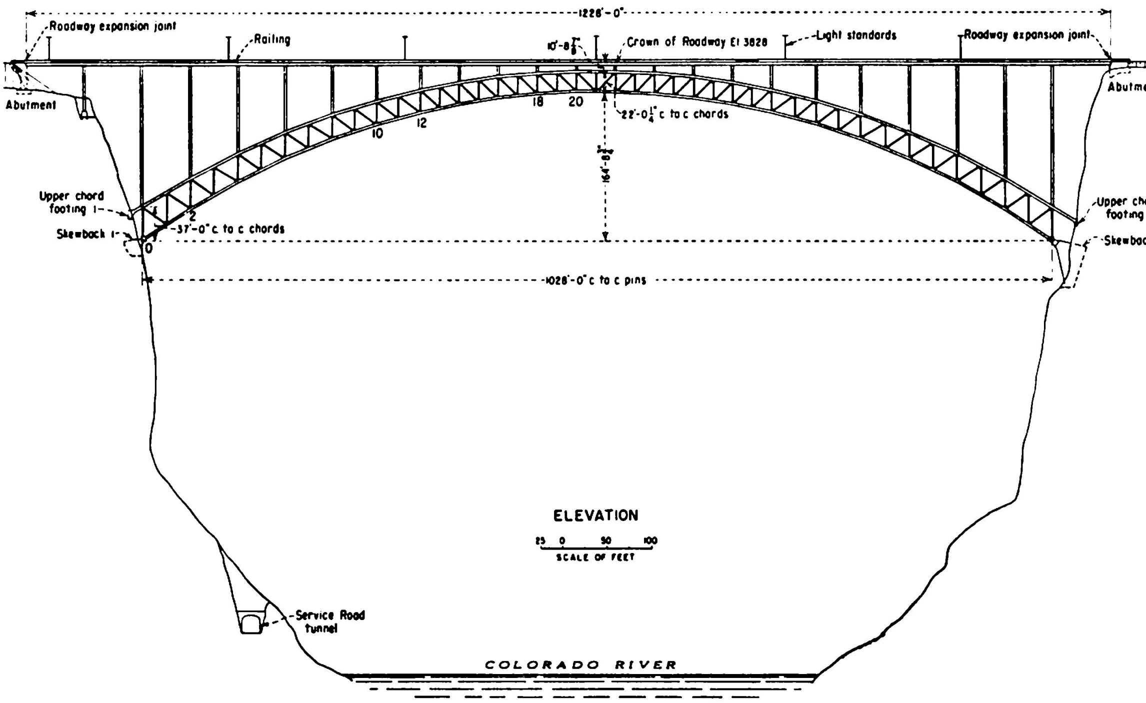

Fig. 1. Bureau of Reclamation drawing.

Floor System

The roadway is a 6-inch-thick reinforced concrete slab. The sidewalk slab is 4 inches thick. The roadway stringers are rolled wide-flange sections and continuous over the top of the floorbeams. Field splices of the stringers are at about the $^1/_5$ point of the spandrel panels. The outside stringers are box sections and have a dual function. They support the sidewalk and also serve as chords the horizontal wind bracing of the floor system. A spandrel length of 44 feet is used at the crown of the arch. At the ends, where the columns are long, the spandrels were increased to 54 feet, which resulted in a saving in weight of 15 percent over a scheme with equal panels. Besides being economical, the increase improved the appearance of the spandrels and gave the paneling for the truss a more pleasing appearance.

Special attention was given to the interaction of floor system and arch, and details were developed so as to reduce secondary stresses. At the crown of the arch, the floor beams rest directly on the arch, and disk bearings were used to permit rotation longitudinally as well as transversely to the center line of the bridge. Special bracing connects this floor panel to the arch so that the floor system is held longitudinally at the crown. Proceeding from the crown to the skewbacks, the distance from floor to arch becomes greater, and the floor beams are supported on columns. As the column length increases, the effects of interaction diminish rapidly and disk bearings at the bottom of

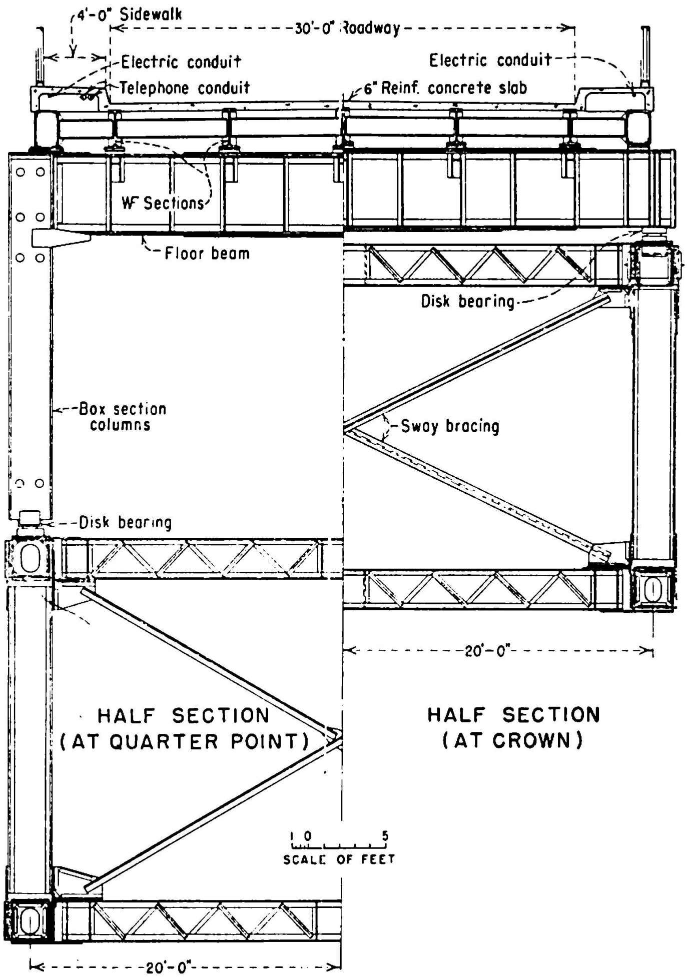

Fig. 2. Bureau of Reclamation drawing.

the columns were required only over the central portion of the arch span. On the long columns near the end, riveted connections were used. At the top of the columns hinge-type bearings were used throughout between floorbeams and stringers.

At the central portion of the arch span, transverse wind loads acting on the floor system can be readily transferred to the arch by the short steel columns which, together with floorbeams, form rigid portal frames. For the long columns at the ends of the span such frames are ineffective as far as transverse loads are concerned. For this reason, a horizontal wind bracing system was provided in the floor from abutment to panel Point 12. This system was designed primarily for wind stresses during erection, as in the final structure the monolithic concrete slab is effective in carrying transverse wind loads.

Expansion joints were provided at the abutments. The joints are of the finger type and allow for a total movement of 9 inches.

Arch Truss

The arch has a span of 1,028 feet from center to center of the pins and a rise of $^1/_6$ of the span. The truss is 37 feet deep at the skewbacks and 22 feet at the crown. Some studies were made as to the most economical proportions, but within the range investigated, no significant weight differentials were observed.

The arch was designed following the principles of the elastic theory. The deflections were so small that the stresses changed less than 2 percent. The truss chords of the arch are box sections having 30-inch webs and 30-inch cover plates riveted to corner angles. The largest section has a cross sectional area of 302 square inches. Access to the inside of the box chords is provided by manholes located near the field splices at the spandrel columns. A manganese-silicon steel, suitable for riveted work, was used in the chords. The principal chemical properties in percents are: carbon, 0.28 maximum, mangenese, 1.60, silicon, 0.30 maximum, and copper, 0.20 minimum. The basic physical properties are 50,000 pounds per square inch yield point and 72,000 pounds per square inch ultimate tensile strength with small reductions for both values for material thicker than $^3/_4$ inch. The vertical and diagonal members of the arch are structural carbon steel and are I-sections made up of four angles and web plates.

The trusses are braced laterally by a *K*-type bracing system in the planes of the top and bottom chord. Sway frames are provided at all spandrel points. Bracing and sway frames are structural carbon steel. Field connections for trusses and bracing are field riveted; the much lighter field connections in the columns and floor are bolted with high tensile strength bolts.

Skewbacks

The skewbacks are concrete blocks 10 feet wide and are set approximately 16 feet into the canyon walls of the massive Navajo sandstone formation, fig. 3. Since the bridge is in the immediate vicinity of the dam, for which extensive foundation exploration by drilling had been made, only minor investigations for the skewbacks were necessary. The sandstone in the dry has an unconfined compressive strength varying between 4,900 and 6,500 pounds per square inch. The block was so designed that for the final conditions, the resultant reaction for any combination of load, produced no tension at the backface. The maximum bearing pressure did not exceed 200 pounds per square inch. To resist tension forces that might possibly occur during certain stages of erection with high transverse wind forces acting, $1^1/_4$-inch dowels ranging in length from 15 to 30 feet, were provided at the rear face. To facilitate accurate setting of the large skewback bearing, rigid steel frames were designed to securely hold 12 long anchor bolts of $2^3/_4$-inch diameter and the 7-foot square base plate. The base plate is the surface on which the 20-ton skewback bearing is set. The frames were placed on a horizontal construction joint of the concrete in the partially complete skewback where accurate measurements of distance and elevation could be readily made. After adjustments were made and the frame firmly bolted in place, the remaining concrete was placed embedding frame, bolts, and base plate solidly in the completed skewback block.

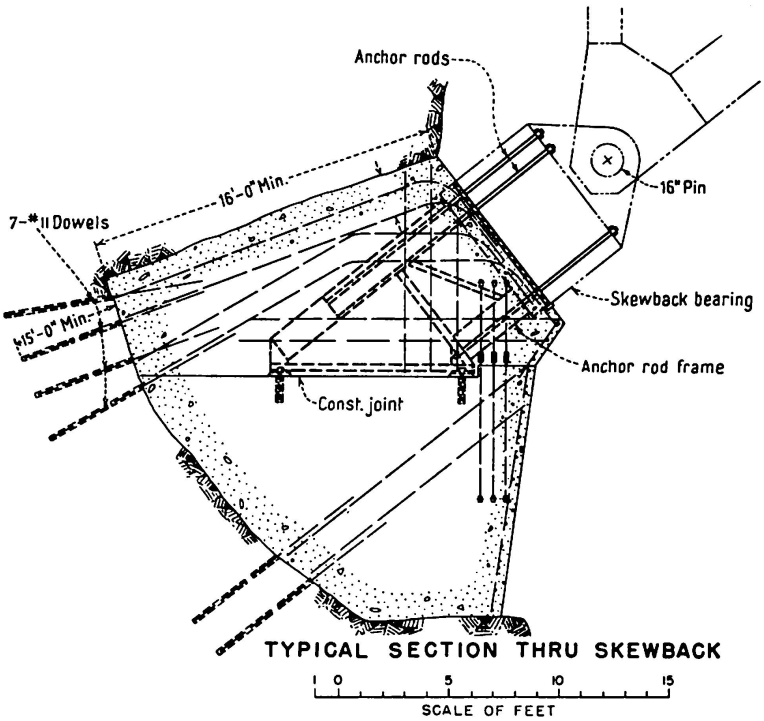

Fig. 3. Bureau of Reclamation drawing.

Erection

The bridge was erected by the cantilever method starting at the skewbacks and working progressively towards the crown. A cableway, having a span of 1,540 feet and a capacity of 25 tons was used to place the steelwork. The single column towers of the cableway could be tilted sideways by guy cables, thus permitting placing the individual steel members directly overhead of the final position. In addition, an auxiliary cableway for transporting men was installed 10 feet upstream from the bridge. This cableway had a capacity of 12.5 tons. The temporary tieback cables used for the cantilever method of erection led over steel towers 120 feet high to a concrete anchor block. The long spandrel columns of the bridge were used in this steel tower.

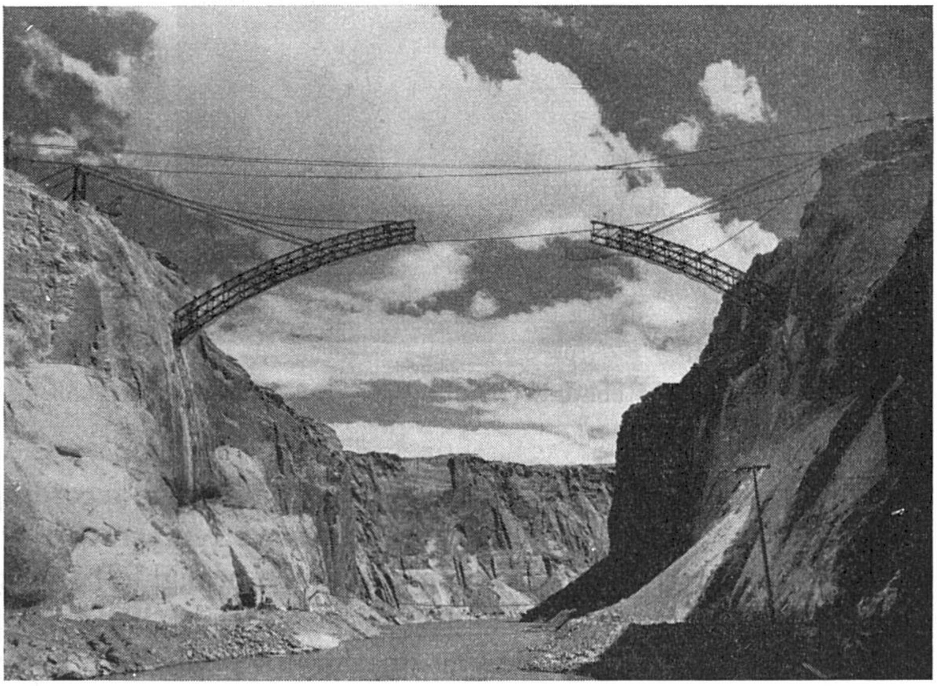

Fig. 4. Bureau of Reclamation photograph No. P-557-420-2438.

The tieback cables were moved forward from panel point to panel point as the erection progressed. Finally, tieback cables connecting to panel points U 9 and U 15 were used and the remaining arch built freely forward for 130 feet to the closure, figs. 4 and 5. Having two sets of tieback cables, the system was statically indeterminate, but by means of hydraulic jacks, proper stresses could be maintained accurately at all times.

The closure of the arch was made at the upper chord by driving a steel pin of 20-inch diameter. Having this pin in place made the structure a self-supporting 3-hinge arch, and the tieback cables were slacked off immediately. The next step was to convert the 3-hinge arch into a 2-hinge arch by means of hydraulic jacks placed for this purpose in the bottom chord at the crown. The jacking load was computed taking into consideration the dead load of the arch and the loads of the erection equipment on the arch and safety net. The reaction on the arch from the slacked-off tieback cables was determined

Fig. 5. Bureau of Reclamation photograph No. P-557-420-2553.

by means of hydraulic jacks and the jacking load adjusted accordingly. The load so computed was 460,000 pounds. In addition, a temperature correction of 3,000 pounds per degree F. had to be made which brought the jacking load to a total of 560,000 pounds. The hydraulic jack, which had a capacity of 1,000,000 pounds, permitted trouble-free operation. Shims were inserted in a specially provided slot which held the member firmly in place while the blind connection was drilled. After drilling, the jacking operation was repeated, the shims removed, and the connection drift pinned and bolted. Finally, the connection was riveted.

The tieback cables and towers were then removed and the floor system erected. The erection proceeded from the center towards the abutments.

To protect the workmen, a safety net was hung below the arch, fig. 6.

All of the steelwork was erected in 7 months. The concrete deck was placed last. The contractor used metal forms, which were left in place. The contractor claimed the metal forms were more economical than wood forms that had to be removed. To avoid overstressing certain verticals and diagonals of the arch, restrictions had to be placed on the method of placing the concrete deck slab. First, a section about 200 feet long was placed at the center, followed by similarly long sections at the ends. The slab sections at the quarter points were placed last. Using the cableway for transporting the concrete, these restrictions on placing the slab in sections caused little inconvenience. The entire concrete deck was placed in 12 working days.

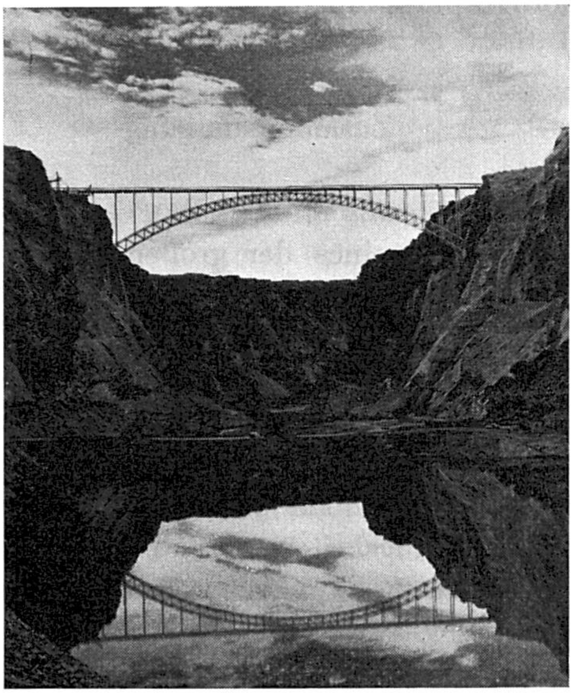

Fig. 6. Bureau of Reclamation photograph No. P-557-420-3325.

Summary

This paper describes the design and construction of the Colorado River Bridge, a major feature of the Glen Canyon Unit of the Bureau of Reclamation's Colorado River Storage Project in northern Arizona. The bridge is the highest arch bridge in the world and the second longest of its type in the United States. Construction of the bridge began in January 1957 and was completed in February 1959. Among the design considerations discussed in this paper are those for the loading criteria, floor system, arch truss, and skewbacks. A selection of drawings and photographs illustrate the text.

Résumé

L'auteur décrit l'étude et la construction de ce pont sur le Colorado, qui constitue l'une des caractéristiques essentielles de l'aménagement de Glen Canyon dans le projet de retenue du Colorado River dû dans le Nord-Arizona, au Bureau of Reclamation.

Il s'agit ici du plus haut pont en arc du monde et par la longueur, il se classe au deuxième rang de sa catégorie aux Etats-Unis. La construction a débuté en janvier 1957 et a été terminée en février 1959.

L'auteur aborde les questions essentielles de l'étude: critères de charge, constitution du tablier, constitution du treillis de l'arc et appuis. Un choix de croquis et de photographies illustre le texte.

Zusammenfassung

Es wird hier die Projektierung und der Bau der Brücke über den Colorado-Fluß beschrieben. — Sie ist eines der großen Merkmale der Glen-Canyon-Anlage im Colorado-River-Stauprojekt des Bureau of Reclamation in Nordarizona.

Es handelt sich hier um die höchste Bogenbrücke der Welt und der zweitlängsten ihrer Art in den Vereinigten Staaten. — Der Bau der Brücke wurde im Januar 1957 begonnen und im Febuar 1959 beendet.

Die vorliegende Arbeit behandelt die für die Projektierung maßgebenden Betrachtungen über Belastungskriterien, Fahrbahnaufbau, Bogenausfachung und Widerlagern. — Eine Auswahl von Zeichnungen und Photographien illustriert den Text.