The new Mike O'Callaghan Pat Tillman Memorial Bridge at Hoover Dam

David Goodyear
T. Y. Lin International, Olympia, Washington, USA

Robert Turton
Phoenix, Arizona, USA

ABSTRACT:A dramatic new concrete arch joins the setting of the historic Hoover Dam, spanning the Black Canyon between the States of Arizona and Nevada, USA. This 323 meter long arch span is the 4th longest concrete arch in the world, and the longest in the United States. The superstructure is a composite deck system using steel box girders and a conventionally reinforced concrete deck, integral with pier caps, which serves as the lateral bracing system for the bridge. The scale of concrete construction for the bridge was impressive. Four form travelers advanced to the crown of the cast-in-place arch supported by 88 carefully tuned stay cables, while precast segmental construction was used for the tallest precast columns erected to date.

1 INTRODUCTION

A project team of five US government agencies, lead by the Central Federal Lands office of the Federal Highway Administration (CFL-FHWA) collaborated to develop a highway bypass to the existing US93 roadway over Hoover Dam, shown in Fig.1. The existing highway route over the Dam mixed the throng of tourists for whom the Dam is a destination with heavy highway traffic and commercial trucking. The blend of these two created hazard and hardship for both, and served as a bottleneck for commerce along the major north-south route of US93.

The Hoover Dam Bypass Project had a decades long history of planning and process. First discussed in the mid 1960's, plans for a highway crossing of the Colorado River were advanced by the Bureau of Reclamation to address the increasing highway traffic across the top of Hoover Dam. A series of studies ensued, sponsored by several of the project stakeholders throughout the next two decades. In 1997 FHWA Central Federal Lands Division (CFL) became the lead agency for the Project Management Team comprised of the Bureau of Reclamation, Arizona DOT, Nevada DOT, National Park Service and Federal Highway Administration. The project then advanced through the Draft Environmental Impact Statement, Final Environmental Impact Statement and Record of Decision leading to commissioning the project.

Figure 1 : Historic Hoover Dam Site

2 PROJECT DEVELOPMENT

A consortium of firms working under the moniker of HST (HDR, Sverdrup, and TY Lin International) teamed with specialty sub-consultants and CFL to deliver the final design for 1.6 km of approach roadway in Arizona, 3.5 km of roadway in Nevada, and a major 610 m Colorado River crossing about 450 m feet downstream of the historic Hoover Dam. A bridge design group of TY Lin International and HDR was directed by the Olympia office of TY Lin International for development of the bridge type study and final bridge design.

The design project was highly structured by CFL, who was the client for development of design work. Of note in relation to the bridge design work was CFL's formation of both a Design Advisory Panel (DAP) and a Structural Management Group (SMG) as advisory groups for the design.

2.1 Bridge Type Screening Process

With the selection of an alignment so close to Hoover Dam, the new bridge will be a prominent feature within the Hoover Dam Historic District, sharing the view-shed with one of the most famous engineering landmarks in the US. The environmental document set a design goal to minimize the height of the new bridge crossing on the horizon, both from the Dam and from a boater's view on Lake Mead. The State Historic Preservation Officers for both Nevada and Arizona – both members of the DAP – emphasized the need to complement and not compete with the architecture of the Dam.

The typical design approach for a project of this significance would be to conduct a comprehensive type study of all candidate bridge types, carrying design to a level that would permit architectural and economic evaluations of each type. However, since the Hoover Dam Bypass had been studied in one form or another for over 25 years, CFL decided to use previous information developed for prior studies along with new information developed by the design team in an initial Type Screening Process – as a precursor to the type study. This Type Screening process was developed to consider policy-level criteria as a first litmus test on bridge types that should proceed to a more formal type study. The rating matrix in Fig.2 was the result of this process.189.

HOOVER DAM BYPASS - RIVER BRIDGE TYPE CANDIDATE SCREENING FEATURE-IMPACT ANALYSIS

Bridge Options	Structural Redundancy	Height and Mass on Viewscape	Rock Excavation/Canyon Impact	Engineering Cost	Technical Suitability for the Site	Cost of Construction	Construction Wind Risk	Inspection and maintenance requirements	Architectural Potential			RATING
Truss	3	5	2	4	2	3	4	3	1			89
Box Girder	5	2	1	5	1	3	5	5	1			84
Cable-Stayed	2	3	2	3	2	3	1	2	2			70
Suspension	1	4	5	1	5	1	2	1	5			103
Deck Arch	5	5	2	4	5	5	4	4	5			139
Thru Arch	2	2	3	2	1	2	3	3	2			71
Impact Weight	1	4	4	1	5	4	3	5	5			

Rating 1 to 5 - 5 is preferred

Figure 2 : Bridge Type Decision Matrix

Of particular note to both advisory groups was the separation of alternatives in the ranking. The two most favored options were the natural design choices – to span the canyon, or to arch against the canyon walls. But also of note were the extremes of rankings for the various criteria. The clear spanning suspension option (Fig.3) was significantly handicapped in terms of structural vulnerability, first cost and maintenance cost. While being one of the more architecturally alluring options, the suspension span was seen as both the highest life-cycle cost option and the most vulnerable design type, which was a special concern for the Agencies who would soon be maintaining the bridge. As a result of this screening process, the type study proceeded with only deck arch options.

2.2 Type Study

At the time of the type study, detailed geotechnical engineering had just begun. The topography on the Nevada side of the canyon (Fig.4) includes a massive outcropping of rock below the US93 switchback, with a fault line running between this block and the canyon slope behind. Without detailed geotechnical and mapping information, we could not confirm the suitability of the short block as a foundation. Therefore, the type study progressed in parallel with geotechnical exploration assuming either of two different arch spans could be selected; a short span of 323 m or a longer span of 405 m.

Figure 3 : Suspension Alternative

Figure 4 : Nevada Foundations

A family of arch designs was reviewed by both the DAP and the SMG based on architectural and technical criteria. The DAP expressed a need for simplicity, and rejected any notion of ornamentation or art-decco designs that competed with features on the Dam. Six arch designs were developed to the point where general quantities and construction methods could be established for review. The final decision to proceed with the Concrete Composite Arch alternative was made by the Executive Committee, comprised of the operations chiefs from the 5 Agencies on the Project Management Team.

3 MAJOR DESIGN FEATURES

The final design went through an evolution of form dictated by the engineering demands on the structure to arrive at the twin rib framed structure shown in Fig.5. At the outset of design it was assumed that earthquake could control the lateral design of the bridge. A project specific probabilistic seismic hazards analysis was conducted in order to assess the range of ground motion associated with return periods appropriate for design. A 1000 year return period was selected, resulting in a design basis PGA of .2g.

Wind was also a major environmental loading condition from the outset of design. It played a somewhat qualitative role in the type selection, and a key role in design.The unique topography was recognized as a feature requiring special studies. During the preliminary design phase, a site wind study was conducted to correlate the wind speeds at the bridge site with those at the Las Vegas Airport NOAA station in the valley. With this correlation, the long term statistics from the Airport were used to develop site wind speeds for design. As a result of this study, the 3 second wind speed was raised to 56 m/sec, up from the ASCE-7 standard of 40 m/sec. Dynamic studies resulted in a gust loading factor of 2.4, which collectively resulted in wind controlling the lateral forces design. Therefore the ensuing design for seismic forces was based on essentially elastic criteria. Wind recordings continued throughout construction, establishing a more complete on-site record. The forecast produced by West Wind Laboratories based on the NOAA correlation and the local terrain studies (Fig.6) were tested according to the methods of Scanlon based on 4 years of continuous on site record. These records confirmed the accuracy of the initial wind studies conducted with only 6 months of site data.

Figure 5 : Selected Concrete Arch Option

Figure 6 : Wind Tunnel Terrain Model

4 STRUCTURAL FRAMING

4.1 Arch Framing

Once given the arch span, the founding elevations for the springing and the roadway profile, the framing plan for the arch and girders could take a number of forms. The 70 mPa concrete arch is an efficient element for gravity loads in its final form. There were two aspects of design that favored a twin rib layout instead of the typical single box section for this arch. The first is one of practical construction. A single box would be almost 20 m wide, and weigh approximately 30 tones per meter. This section size would rule out a precast segmental option. The second is the matter of performance under extreme lateral forces. At the time the framing plan was devised, the level of seismic ground motion had not been determined. A single arch rib would leave no opportunity for tuning stiffness or for providing for frame ductility, whereas twin ribs could provide an excellent means of creating ductile Vierendeel links that could otherwise fully protect the gravity system of the arch. It is for both of these reasons that a twin rib arch framing system was selected (Fig.7). The arch ribs have a prismatic form instead of the more classical variable depth to the springing. This too was decided based on lateral force demands. Early evaluation for longitudinal seismic loads showed that springing moments increased considerably for a tapered arch rib, so much that the demands on the arch rib and foundation were disadvantaged by the increase in rib section.

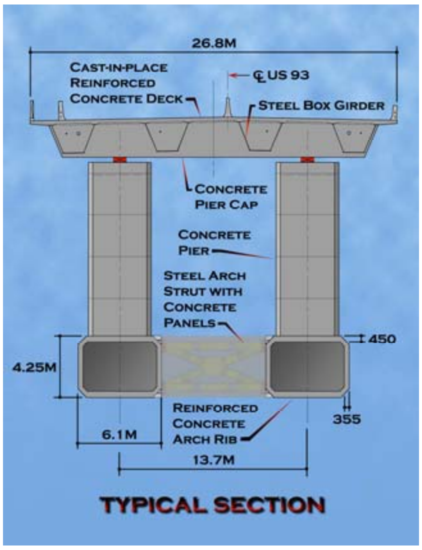

Figure 7 : Arch Framing System

4.2 Spandrel Framing

The composite superstructure was selected for girder erection and to lower weight on the arch. The spacing of spandrels was an extension of the erection concept to erect the bridge using a highline (tramway) crane system. Above 50 tons, there is a jump in highline cost, so the decision was made to target a 50 ton capacity for major superstructure elements. The span was set in the range that a highline crane could deliver the steel box sections, which resulted in a nominal 37 m span. This same span also allows steel girders to be set within the range of most conventional cranes, should an alternative erection system be selected. The statical system includes sliding bearings (Fig.8) for the short, stiff piers over the arch crown and similar piers near the abutments. This was necessary due to the large secondary moments developed in these piers from creep deflections of the arch, and also produced a more even distribution of longitudinal seismic forces among the piers.

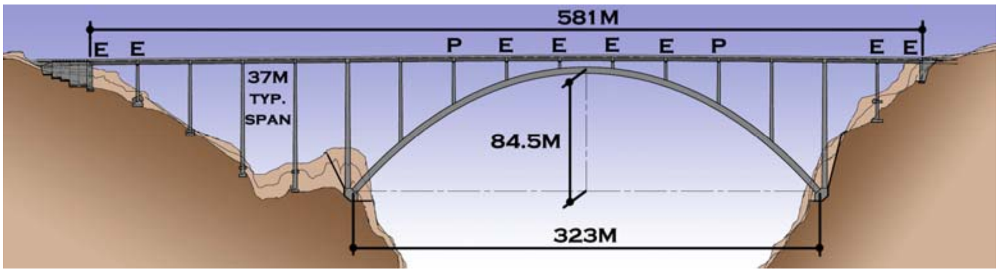

Figure 8 : Layout and Articulation

4.3 Open Spandrel Crown

An open spandrel crown was selected over the option of an integral crown. An integral crown was assumed for the long span concrete alternative for both aesthetic and structural reasons. However, a special consideration for the short span arch was that the composite steel deck would result in a very abrupt, mechanical looking connection at the crown. Equally significant was the high rise of the arch. When studied in either concrete or steel, an integral crown solution for the short span alternatives looked blocky and massive at the crown, and ran counter to the architectural goal of lightness and openness when viewed from Lake Mead.

4.4 Pier Cap Framing

Integral concrete pier caps were selected over steel box cap sections. The integral cap framing (Fig.9) was selected to develop the diaphragm action of the deck used to avoid lateral bracing of the spandrel columns and to provide ultimate stability to the flexible columns along station through direct diaphragm action. Concrete was selected over steel due to the higher maintenance and inspection costs associated with a fracture critical steel diaphragm; even though estimates showed that a steel cap might have a lower first cost.

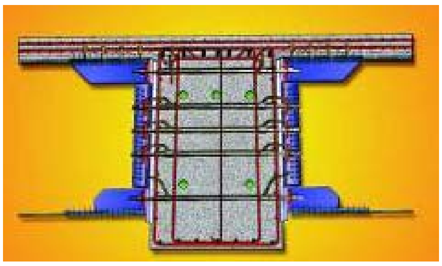

Figure 9 : Integral Pier Cap

4.5 Cross Section Forms

The first natural frequency of the arch system is over 3 seconds – a range normally reserved for flexible cable-supported structures. Since wind forces dominated the lateral load design, shape became a primary design issue.

The tallest of the tapered spandrel columns is almost 92 m tall. Wind studies included considerations of drag and vortex shedding on the main structural sections exposed to the long canyon fetch from over Lake Mead. Studies showed that substantial advantage could be gained both in terms of vibration and drag by chamfering the corners of both the columns and the arch. While this adds somewhat to the complexity of construction, the benefit in terms of reduced demand and material savings was substantial.

5 CONSTRUCTION METHODS

As with any large bridge structure, the dead load design is dominated by the assumptions of a construction scheme. The typical approach in the US is to nominate an erection scheme, but to show it only schematically, and defer responsibility for both the scheme and the details to the contractor. The management team believed that more informed bids could be developed if there was a more complete erection scheme shown with the plans, even if the contractors elected to use alternative methods. Therefore, the decision was made to show a complete erection scheme for dead load on the plans and allow the contractor to use that scheme or his own.

There are at least two practical erection methods that can be used to erect a cantilevered arch. One is a simple cable-stayed cantilever erection (Fig.10). The second is the use of temporary stay truss diagonals, erecting the arch, deck and spandrels as a cantilever truss. In selecting the simple cast-in-place stayed method, the design team opted for the most conservative method in that arch geometry can be controlled and corrected at each step of construction with stays and traveler settings. In addition, this method allows the most flexibility for closing the arch without affecting the geometry of columns and deck (since they are not in place until after closure).

Figure 10 : Stayed Erection Scheme

Both precast (Fig.11) and cast-in-place methods were permitted for the arch and spandrel columns. The contract was written to allow alternative methods of erection, however both the arch and the columns each were to be of a single type (precast or cast in place) in order to conform to the time dependent assumptions inherent in design. All equipment and ancillary temporary works were also to be designed by the contractor.

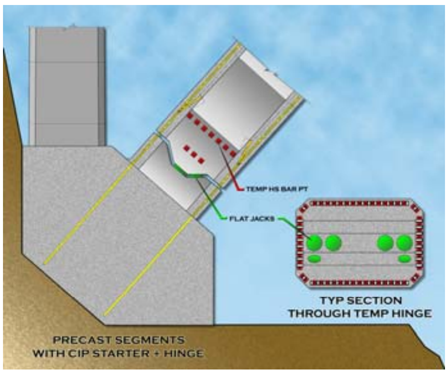

Figure 11 : Precast Segmental Springing Option

6 CONSTRUCTION

The construction contract was awarded in September of 2004 to Obayashi-PSM, JV, after about a year delay in the funding process. A limited notice to proceed was issued for November, 2004, with full field work beginning in 2005.

The first challenge for the construction team was creating a foothold for foundation construction. Climbing on the side of the cliff 250 meters over the river below was difficult enough, but excavating (and doing so within the loss limits in the specification) was an incredible challenge. The subcontractor who met this challenge was Ladd construction from Redding, CA. They not only met the tight schedule for this work, but completed the excavation allowing about half of the rockfall into the river that was permitted.

Figure 12 : Foundation Excavation

Initial bridge construction began on site with footing and abutment work, and in the precast yard outside of Boulder City where the contractor set up their own precasting facility and self-performed the precasting. Column sections were trucked to the site as needed for erection, and set into place using both the high-line crane and conventional cranes located at the hairpin in Nevada.

Figure 13 : Precast Column Erection

In September, 2006, the high line crane collapsed in a strong wind, removing the high line crane from service. The contractor mobilized additional land cranes to continue construction in Nevada, and an S-70 derrick in Arizona, to both complete approach columns and set approach steel box girders until a new high line could be designed, fabricated and erected.

Arch Erection was being initiated at the time of the high line crane collapse. The temporary cranes were able to service the first few arch segments from the springing, allowing the piece work for the starters and the initial setup of the form travelers to proceed.

Figure 14 : Arizona Girder Erection

Four form traveler headings were operated in concert for erection of the arch. After erection of the new high line and restarting the main arch erection, the contractor reached a reliable cycle of 2 weeks, and often bested that cycle on segments that did not have a stay.

The arch was closed in August of 2009 within an impressive 20mm tolerance at closure. Spandrel columns were erected using the high line crane, and superstructure girders are being set as of this writing. The bridge is scheduled for opening in the November of 2010.

Figure 15 : Four Form Traveler Headi

Figure 16: Arch Closure

CREDITS

Program Manager: Central Federal Lands Highway Division; US Federal Highway Administration. David Zanetell, Program Manager; Bonnie Klamerus, Structures Manager.

Project Management Team: Nevada Department of Transportation, Arizona Department of Transportation, US Bureau of Reclamation, US National Parks Service, US Federal Highway Administration.

Bridge Design: T.Y. Lin International in collaboration with HDR Engineering.

Bridge Construction: Obayashi-PSM, Joint Venture.