The Spencer Creek Bridge crosses a creek and trail that extend from a state park to the shores of the Pacific Ocean near Newport, Oregon. Carrying a part of U.S. 101 that has been included in the Federal Highway Administration’s National Scenic Byways Program, the deck arch is in harmony with other arch bridges along the Oregon coast, including those designed in the early 20th century by Conde B. McCullough.
COASTAL CONNECTION

Along the Oregon coast, where the beautiful arch bridges designed in the early 20th century by the engineer Conde B. McCullough are preserved and revered, even bridges of modest size are designed with aesthetics in mind. But creating a graceful arched crossing on a site that could experience floods, tidal waves, and earthquakes was no easy task.

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Since 1947 the Spencer Creek Bridge, roughly 6 mi (10 km) north of Newport, Oregon, has carried U.S. Highway 101, which is known locally as the Oregon Coast Highway, over a tidal creek and a walkway from a nearby state park. Recently, however, corrosion in the bridge led its owner, the Oregon Department of Transportation (ODOT), to call for the structure’s replacement. Not only is the new bridge wider, stronger, and built to withstand all manner of natural disasters; it is also in keeping with the arch bridge aesthetic of Oregon’s many coastal bridges.

In this region of the country U.S. 101, the primary north–south transportation route along the Oregon coast, forms part of the National Scenic Byways Program, which is administered by the Federal Highway Administration (FHWA), and it has also met that program’s requirements for designation as an “all-American road.” What is more, the highway serves as the major “lifeline” route to the communities, residents, and businesses throughout central Lincoln County and is located just west of Beverly Beach State Park, one of the most popular destinations in the state. The state park contains forest-sheltered campgrounds, wind-sculpted trees, and “nurse logs”—fallen and decaying trees that nourish forests—along with the pebbly Spencer Creek. A walkway along the creek extends beneath the bridge and leads to a long sandy beach and eventually to the ocean. The creek flows westerly from the Siuslaw National Forest through Beverly Beach State Park. There it crosses the broad beach and debouches into the Pacific.

With a natural channel width of 50 to 70 ft (15 to 21 m) and a depth of approximately 9.5 ft (3 m) at the ordinary high-water elevation, the creek provides access for salmon migrations. The bridge structure and its
approaches are required to withstand a 100-year flood without major damage (B. Miller and J.T. Potter, *Hydraulics Report for US 101: Spencer Creek Bridge Replacement* [Oregon Department of Transportation, Region 2, 2005]). Streamflow and ocean waves influence the hydraulic activities in the creek. Riprap armor, including large rocks installed to protect the creek embankment, has washed away during large storms, and significant quantities of logs have moved in and out.

The original bridge, a three-span, reinforced-concrete deck-girder bridge (a girder bridge with a concrete deck) with steel hinge details in the center span, was 182 ft (55.5 m) long and 35 ft (10.7 m) wide. It had deteriorated over a long period, but its shortcomings were not obvious until a comprehensive bridge inspection was carried out by the ODOT in early 1999. The inspection revealed that the bridge was in very poor condition because of corroded reinforcing bars in all the main girders. This resulted in the construction of a 960 ft (293 m) long emergency detour bridge, which was designed and constructed to have a service life of five to eight years. The temporary structure was constructed to the west of the original bridge, and traffic from U.S. 101 was rerouted onto it.

In 2001 the ODOT formed a project management team and a steering committee to determine the most suitable alternative for replacing the bridge. This team included representatives of the ODOT, the FHWA, the Oregon State Parks and Recreation Department, and the U.S. Army Corps of Engineers, and its work spanned several years. The project was developed in several phases over the years with significant public input. Numerous alternatives to the roadway alignment and bridge type were considered to accommodate engineering, geological, and environmental aspects. The selection criteria used in assessing alternatives were developed and modified in response to input from regulatory agencies, local residents, regional stakeholders, and special interest groups.

A draft of an environmental impact statement was released to the public in 2004, and a final impact statement was
Some conditions challenged the bridge designers, including significant lateral loads from the arches; the level of corrosion likely to be caused by salt spray from the ocean; the high seismic design loads; the potential for liquefaction of the soil underlying the bridge, the bridge approaches, and the relatively tall retaining walls during seismic events; and the scour of the structure that was likely to result from the actions of the stream. During the design process, several bridge types were evaluated in detail, including tied-arch bridges, a deck-girder bridge with an architectural arch, a deck-girder bridge with haunched girders, and other configurations of a deck arch.

The project steering committee decided to continue with the original deck arch but to make slight revisions that would take into account constructability and operation.

Shannon & Wilson’s geotechnical investigations revealed that the bridge and its approach embankments were located at the transition point between the Spencer Creek floodplain and the infilled estuary that leads to the oceanfront beach. The embankments consist of variable materials overlying terrace deposits, beach sand, alluvium, and estuary deposits with organic materials, and beneath those layers is low-strength siltstone and sandstone bedrock. The top elevation of the siltstone and sandstone is approximately 120 ft (36.6 m) beneath the roadway approaches.

At this site, relatively high ground motions are anticipated from the Cascadia Subduction Zone, which is located nearby. Other nearby faults with the potential to generate seismic activity include the Siletz Bay, Cape Foulweather, and the two

The cap beams atop the arches were constructed before the central arch shoring was released. After this sequence, horizontal jacking was performed to mobilize the passive earth resistance and maintain the position of the arch supports.
Yaquina faults, all of which are within 16 mi (26 km) of the bridge site. Seismic hazard maps released in 2002 by the U.S. Geological Survey provided a probabilistic peak ground acceleration on bedrock of 0.30g and 0.45g for events with return periods of respectively 500 years and 1,000 years. A response spectrum analysis for the site in question was performed and used in the seismic design for the bridge structure. The response spectrum had a slightly lower but longer peak acceleration than did the U.S. Geological Survey predictions because the relatively thick and soft soil profile underlying the site damps the seismic activity there. On the basis of the geotechnical investigation, it was determined that the seismic hazards at the site included strong ground motion and liquefaction of certain zones beneath the bridge and its approaches. Such liquefaction would result in settlement, lateral spreading, and slope instability.

For these reasons, the bridge structure was designed to withstand an event having a 1,000-year return period without collapsing and a 500-year event without being damaged beyond the point of use by emergency vehicles in the immediate aftermath of such an event. To mitigate the seismic hazards, mechanically stabilized earth (MSE) walls were specified for the bridge approaches; these were to be roughly 40 ft (12.2 m) high at the bridge abutments. MSE wall reinforcing strips made of stainless steel were used in the zone below the ordinary high-water elevation.

As explained by V. Elias, B.R. Christopher, and R.R. Berg (Mechanically Stabilized Earth Walls and Reinforced Soil Slope Design and Construction Guidelines, publication FHWA-NHI-00-043 (U.S. Department of Transportation and Federal Highway Administration, 2000), MSE walls have been in use for decades and have performed well during several major earthquakes. However, their history does not adequately predict their performance in the case of excessive settlement and subsurface slope instability. Ground improvements beneath the MSE walls were required, and the engineering team determined that stone columns and wick drains should be used to densify and increase the shear strength of the soil. The treatment area was approximately 80 by 80 ft (24 by 24 m), and the stone columns were spaced at 8 ft (2.4 m) intervals in each direction and extended to depths of 50 ft (15.2 m) beneath the bottom of the MSE walls. Wick drains were used to dissipate the excess pore pressure induced during the stone column installation. (See illustrations on page 82.)

Several alternatives were evaluated for the arch supports, including grouped vertical driven piles, grouped battered driven piles, a single large-diameter drilled shaft, and a group of drilled shafts. The design team and the ODOT decided to use six drilled shafts, each 6 ft (1.8 m) in diameter, beneath each arch support; this decision derived from the shafts' superior resistance both to the lateral loads that could be induced by seismic events and to the deep scour that was anticipated.

Six steel pipe piles 20 in. (510 mm) in diameter and roughly 113 ft (34.4 m) in length were used as foundations for the bridge abutments. Sacrificial section loss from corrosion was considered in the pile design. To allow consolidation settlement to occur before the pile driving, the piles were not driven until a month after the MSE wall construction had been completed. Sleeves with a larger diameter than that of the pipe piles were installed within the MSE walls' fill to isolate the driven piles from the fill material. The construction sequence and details avoided downward drag as a result of negative skin friction along the piles.

Because the bridge is in a flood zone that experiences wave action, a 100-year design flood recurrence interval was used in determining the design of revetments for protecting the bridge. The variables used in the hydraulic analysis took into account wave height during a design-level storm, the spectral wave period, the ordinary water level in the creek, the storm surge levels, and the sea level elevation. The analysis revealed that a significant wave height could occur and that it could result in relatively deep scour at the arch foundations and abutment walls. The revetment system was therefore designed to protect the foundation elements from scour by incorporating gabions, 2 ton (1.8 metric ton) rocks, and sheet piles. Three layers of sheet-pile walls were installed around the arch foundation and abutments. Stainless steel gabions were placed between these sheet-pile walls and were topped with many of the large rocks. The revetment system was designed to withstand a 100-year flood without damage and to sustain only minimal damage in a 500-year flood.

Because the bridge is located within a recognized tsunami hazard zone, tsunami hazards also were discussed during the design phase. If an extreme earthquake originating nearby or at a moderate distance were to be accompanied by the subsidence of a portion of the coast or ocean floor, a tsunami could be generated with significant wave heights. A great deal of seawater would move inland through the creek and impose lateral and uplift forces on the bridge. The water would then rush back out to the ocean, carrying with it a large amount of debris that could damage the bridge structure. An investigation was performed to address this hazard in the design, but at the time provisions for tsunami hazards were not available in the bridge design specifications, the ODOT's bridge design manual, or other specifications. For this reason the relevant loads were difficult to estimate, so the design team could not include the tsunami loading in the design. This absence of specifications relating to tsunamis triggered several research projects that are now being conducted by state agencies and educational institutions to develop standards related to tsunami loading for bridge design.

The arch has a span of 140 ft (42.7 m) and a rise of 31.5 ft (9.6 m). With this arch configuration, the lateral thrust at the foundations was about 45 percent of the vertical loads, and the poor quality of the soils meant that sufficient resistance to the lateral thrust would not be developed. Therefore, a unique approach to an old concept was used so that the arch bridge could be built at this location. A deadman, or anchor log, with compression struts was incorporated into the foundation's lateral-load-resisting system. One deadman was located at each end of the bridge; each took the form of a cast-in-place, reinforced-concrete thrust block 6 by 7 by 52 ft (1.8 by 2.1 by 15.8 m) constructed on the stone columns to provide vertical support. The deadman blocks were buried under the MSE retaining wall fill material at a depth of about 40 ft (12.2 m) at a distance of 50 ft (15.2 m) from the MSE abutment wall face. Each deadman block was then connected to the drilled shaft caps by three 4 by 5 ft (1.2 by 1.5 m) grade beams functioning as compression struts. Horizontal jacking was performed to
mobilize passive resistance from the ground behind the deadman thrust blocks. A small amount of movement—less than 0.5 in. (13 mm)—toward the compacted fill would generate approximately 3,250 psf (155.6 kPa) of passive pressure, or roughly a 600 ton (5338 kN) passive resistance with uncertainties factored in. The jacking location was situated near the drilled shaft cap at the end of the grade beams. The jacking was also used to control the lateral deformation of the drilled shaft caps caused by the arch thrust and to maintain the internal compression within the arch structure. Significant friction around the anchor elements provided resistance to the lateral forces.

To confirm the analysis and assumptions used in designing this foundation system, instrumentation was provided to measure the earth pressures and movements of the anchor system and arch foundations during the construction. Eighteen vibrating-wire sensors, including load cells, pressure cells, displacement sensors, and tiltmeters, were installed before the jacking. The instrumentation program was designed to evaluate the short-term and long-term performance of the new bridge. In the short term, the system evaluated the behavior of the deadman anchor systems against the soil during the jacking. The instrumentation program provided data on the load in the compression struts, the passive pressure of the soil, the displacement of the deadman blocks, and the rotation of the drilled shaft caps. All of the collected data were in the anticipated ranges during the construction.

Over the long term, the instrumentation will continue to provide data that can be used to evaluate the performance of the bridge, the construction of which required a highly complex design to accommodate demanding site conditions. The instrumentation program has been integrated into the ODOT's system for monitoring structural health, a system that is currently tracking several significant bridges in the state. So far the instrumentation is showing no deviations with respect to the anticipated criteria.

After the jacking operations, the concrete closure connections were placed. The connection details between the grade beams and the drilled shaft cap were carefully designed to accommodate significant movements in response to seismic events or settlement.

Three-dimensional finite-element modeling using the STAAD.Pro program, developed by Bentley Systems, Inc., of Exton, Pennsylvania, was used to analyze the behavior of the bridge structure as well as to predict the structural response to static and seismic loadings. (See the figure at the top of page 83.) The model used 1,834 nodes, 1,649 two-node beam elements, and 1,056 four-node plate elements. All applicable load combinations were considered in accordance with the third edition of the AASHTO LRFD Bridge Design Specifications (Washington, D.C.: American Association of State Highway and Transportation Officials, 2004). For this site, a wind speed of 100 mph (161 km/h) with open surface conditions was considered; however, the wind load combinations did not govern the bridge design. Vertical load combinations controlled the design of the superstructure, cap beams, abutment piles, and arch, while various load combinations, including earthquake loads, controlled the design of the columns and drilled shafts.
Construction activities in the water were limited to a roughly 10-week time frame to ensure that there would be no adverse effects on the most vulnerable life stages of native fishes, including migration, spawning, and rearing. This restriction, combined with the general environmental fragility of the site, led the team to select precast concrete for the principal structural members of the bridge, including the arches and the slab superstructure. This decision not only reduced the amount of fieldwork and formwork required but also diminished the effect on the environment and reduced the required construction time. As an added benefit, the precast-concrete construction provided products of a higher quality than is typically possible with cast-in-place construction. The three precast arches were constructed in six segments by Knife River Corporation at its manufacturing plant in Harrisburg, Oregon, which is 85 mi (137 km) from the project site. Each arch rib was 70 ft (21.3 m) long along the arch axis, weighed 80 tons (73 metric tons), and contained 28 uncoated reinforcing bars 1.7 in. (4.3 mm) in diameter as flexural steel and 0.75 in. (19 mm) diameter double stirrups as shear reinforcement. The radii of curvature of the arch ribs were 90 ft (27.4 m), 93.6 ft (28.5 m), and 97.4 ft (29.7 m) for respectively the intrados, center, and extrados arches. The arch sections ranged from 3.5 by 4.5 ft (1.1 by 1.4 m) at the supports to 3.5 by 3.5 ft (1.1 by 1 m) at the crowns. Moreover, the arch ribs were posttensioned to prevent cracking during shipment as well as to control cracking throughout the bridge’s life.

The precast arches were delivered to the construction site on their sides in piecewise fashion. It was a challenging task to flip the 80 ton (73 metric ton) arch ribs without damaging them into a position that would allow them to be lifted by crane. Gravity combined with concrete blocks and sand bags accomplished the task. The contractor used one of the largest cranes in the state. Supported by a temporary span built on one of the bridge’s approach spans, the crane lifted each arch from its delivery truck, swung it into a prepared arch support socket, and set the top end on a shoring platform spanning the creek. The segments were subsequently connected together—two to an arch—at the crown with a cap beam. At the ends of the arches, the arch ribs were surrounded by a 3.5 ft (1.1 m) deep reinforced-concrete socket located on top of the drilled shaft caps. The three completed arches were spaced 22 ft (6.7 m) apart. At an intermediate point between the arches, cast-in-place reinforced-concrete beams were cast to provide additional lateral stability.

To complete a full arch, the reinforcing bars extending from the inside of each arch rib were connected at the crown using mechanical splices filled with rebar metal (CADWELD rebar splices, provided by ERCO International, of Solon, Ohio). The type of rebar splicing used was selected because of its high strength and generous construction tolerances. At this location, the 28 bars with, as mentioned above, a diameter of 1.7 in. (43 mm) were spliced in a 6 ft (1.8 m) opening. Each splice required a work space large enough to accommodate the splicing equipment that pours the filler material into the provided splice sleeve. Two splices per rebar were used, except on the bottom layer, for which one was used. The splices were staggered from the bottom to the top layers. After the splices were completed, steel reinforcing cages for the cap beam at the crown were installed. To ensure a long service life, the cages were made of stainless steel rebars that were isolated from the uncoated rebars to prevent the corrosion that can occur when metals with different electrochemical potentials come into contact.

All the other cap beams were constructed before the arch shoring was released in the center. After this sequence, horizontal jacking was performed to mobilize the passive earth resistance and maintain the position of the arch supports. The precast slabs were erected, and an additional jacking operation was carried out to ensure the tightness of the deadman anchor system before the remaining connections were constructed.

The bridge superstructure included 18 in. (457 mm) precast, prestressed voided slabs placed side by side and transversely posttensioned on the cap beams. The slabs were also grouted at key points along the slab length and made continuous for live loads by adding rebar at the bent locations that would resist negative bending moments. The slabs were then topped...
with a minimum of 5 in. (127 mm) of reinforced-concrete decking. To create continuity, prestressing strands extending out from the precast slab ends were hooked at the bent locations. The hook lengths were calculated on the basis of the design recommendations made by J.R. Salmons in End Connections of Pretensioned I-Beam Bridges (report FHWA-RD-77-14 [Federal Highway Administration, 1974], 41–46.)

The shear reinforcement rebar in the precast slabs was stainless steel and extended into the reinforced-concrete deck to develop composite sections. Stainless steel bars were also used in the concrete deck and bridge rails. High-performance concrete was used in the deck, cap beams, and columns to facilitate placement and consolidation and to increase the service life of the bridge in the harsh marine environment.

Architectural treatments providing special surface textures, along with a concrete surface colorant, were applied to the MSE wall panels and the exteriors of the bridge rails. The architectural treatments for the MSE wall panels included an exposed aggregate pattern on the top part of the walls that matched the pattern in the bridge rails. An ashlar stone pattern was used for the bottom half of the walls. The MSE wall panels and bridge rails were stained in dark brown, while the bridge's structural components were left in the natural concrete color.

Beverly Beach State Park is open to visitors year-round, and beach access had to be maintained during the construction. To keep the beach open and protect visitors from construction debris, the contractor used shipping containers to protect pedestrians along the walkway beneath the bridge. Pedestrian traffic was halted only when a major portion of the construction was occurring directly above the temporary access, for example, the arch erections and precast slab placements.

After the completion of the bridge structure, in October 2008, the traffic on U.S. 101 was directed back to the original roadway and the new Spencer Creek Bridge. The detour bridge was demolished during a period when no work was allowed in the water inside the ordinary high-water delineation zone. The delineation zone was quite wide at the new bridge, 50 to 70 ft (15 to 21 m), and much wider moving toward the beach, which prevented the contractor from constructing a long temporary structure spanning the no-work zone for the removal of the detour bridge. Without a structure crossing the creek, it would have been difficult to remove the detour bridge from the creek embankment and beach. As a result, the new bridge was used for crane support and as an access point for the trucks that hauled away the precast slabs used in the portion of the detour bridge that crossed the creek. The rest of the detour bridge's slabs were removed across the remaining abutments of the detour bridge itself. Despite this complexity, the demolition of the detour bridge was accomplished without any major problems.

The entire project was completed in May 2009, and the final construction cost was $19.5 million.

Throughout the construction project, the primary contractor took full responsibility for constructing the bridge as designed and in accordance with the specifications. Highly skilled, the contractor's crews used high-quality formwork and paid close attention to the details specified for the project.

Feedback from the owners, park visitors, and members of the community of Beverly Beach indicate that the design and the methods of construction used for this project have met everyone's expectations. The design team worked alongside ODOT engineers, environmentalists, and construction support staff to design an aesthetically pleasing structure that not only meets the current and future transportation needs of the area but also honors the state's coastal bridge tradition and makes the transportation corridor even more scenic.

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**PROJECT CREDITS**

**Owner:** Oregon Department of Transportation

**Structural engineer:** H.W. Lochner, Inc., Chicago

**Geotechnical engineer:** Shannon & Wilson, Inc., Seattle

**Roadside development:** Jones & Jones Architects and Landscape Architects, Ltd., Seattle

**Roadway, environmental, and hydraulic engineers:** Region 2, Oregon Department of Transportation

**General contractor:** Sloyen Construction Group, Stayton, Oregon

**Precast-concrete manufacturer:** Knife River Corporation, Bismarck, North Dakota