Highlights

- **1988 SUCCESS OF THE MINAMI BISAN-SETO BRIDGE INSPIRES RECORD AKASHI SPAN**
- **RECORD SPAN ADVANCES ENGINEERING & CONSTRUCTION TECHNOLOGY**

SPANS

Hillsborough County Florida

Public Works Department
Bridge Team

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QUAKE INCREASES WORLD RECORD SPAN

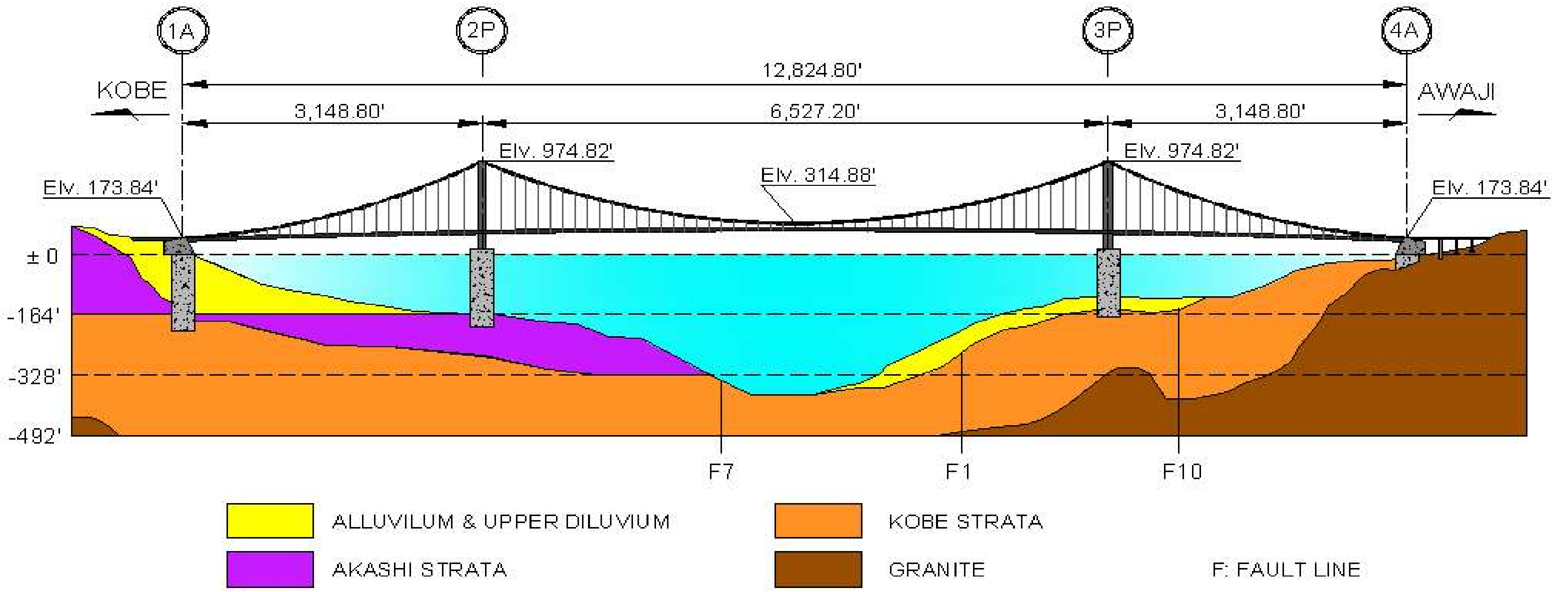


FIGURE 1: Pier 3P positioned to avoid active faults F1 & F10 and it's companion pier 2P location minimizes construction depths

Two ferries collided and sank in 1955 during a violent storm on the turbulent and heavily traveled waters of Japan's Akashi Straits resulting in the death of 168 children. The Nation was outraged and the National Government responded with plans to build a bridge at this location. The original concept incorporated both a highway and rail crossing; however, early in the planning process, it was decided to provide for only highway traffic with six roadway lanes.

The Akashi Strait is about 2.4 miles wide and has a northwesterly to southeasterly orientation at this location. There are as many as 1,400 vessels passing through each day including fisherman who have historically found this area to be a particularly good hunting ground. The Channel has an underwater, 1300' wide, steep sloped valley reaching to a depth of well over 300', midway between the two shorelines.

The geologic and hydraulic profiles at the existing bridge alignment are depicted in Figure 1 along with active fault lines F1, F7 and F10. Pier 3P was specifically positioned to avoid faults F1 and F10 and, due to the exceptionally deep midstream water, Pier 2P was located on the far, opposite side of the underwater valley to minimize water depths and facilitate underwater construction. Coincidentally, these pier locations easily exceeded the specified navigational width of 1490'. The span lengths evolving were obviously in a range never before encountered and it was recognized that the attendant, suspension bridge tower foundations and anchorages would be unprecedented in volume and mass.

The ideal bearing material of granite was located at an elevation exceeding the practical reach for three of the four foundations; consequently, Pier 2P was founded in the higher Akashi strata of unconsolidated gravel from the Pliocene epoch. Two bearing points, 1A and 3P, on either side of 2P, were carried to the Akashi Strata, dating from the thick soft rock of the Kobe strata dating from the Miocene epoch and the ideal bearing material of granite rose to the surface only at the 4A anchorage.

With these wide-ranging foundation

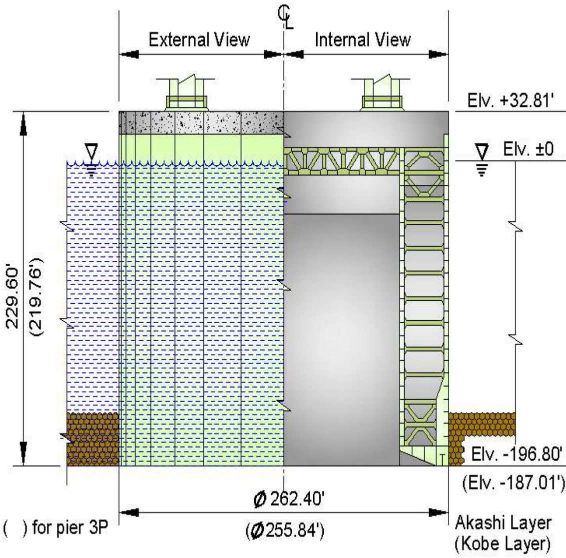


FIGURE 2: Prefabricated caisson for pier 2P

conditions and the unprecedented size of the bearing loads the founding materials had to be very accurately modeled in order to obtain a reliable analysis. Moreover, in order to accommodate these circumstances, new seismic design procedures were developed to account for the non-linear behavior of the ground during an earthquake and its consequent interaction with the contiguous foundation structures.

The Honshu Shikoku Bridge Authority (HSBA) was established in 1970 and has administered the design and construction for the three bridge routes presently connecting Honshu to Shikoku. The Japanese National Railroad first studied the Akashi Strait crossing (eastern route) as early as 1955 but it was not until 1975 that the Government decided to build only the shorter spans of the Seto-Ohashi, middle route. This route is composed of six, over water structures and the most significant bridge in this group is the double suspension span, Minami Bisan-Seto Bridge that was completed in 1988. Before the middle route opened to traffic the decision to build the Akashi Kaikyo Bridge (AKB) was made.

This suspension bridge, carrying both rail and highway, provided the big bridge experience and confidence to again pursue the design and construction of the AKB, the eastern route's north leg in the two bridge, north-south roadway alignment between the Honshu and Shikoku Islands. The actual first shovel of dirt was not turned over until May, 1988 and after a ten year construction period the Akashi Strait highway bridge (also called the Pearl Bridge) opened to traffic on April 5, 1998.

Everything about this bridge has been precedent setting, by a wide margin; the designed, main span length of 6,527.2' was and remains the world's longest span, the 1.85 million cubic yards of concrete utilized for the anchorages and foundations is an unprecedented volume in bridge building, the 220,000 short tons of structural steel used was for the tallest bridge towers, the longest stiffening truss and, finally, the cost of 500 billion Yen (~ 5 billion USD) was the most paid for a bridge of this length. Accordingly, the 2,300 yen ($22 USD) toll charged the 23,000 vehicles per day by the HSBA is intended to defray the construction cost.

Another superlative that subsequently surfaced is that 2 million Japanese had some part in the design and construction of the AKB. Moreover, industrial giants of Japan such as: The Obayashi Corporation, Kawasaki Heavy Industries, Mitsubishi Heavy Industries and the Yokogawa Bridge Corporation all played a part in what was actually a national program used to bring this amazing project to realization.

Consider, each of the main piers (2P & 3P) deliver 132,000 short tons of reaction to their bearing strata and to construct piers in these deep, swift, tidal currents (8.7 knots) special, cylindrical, prefabricated, steel caissons were used. They each weighed 16,500 short tons, measured 262.4' in diameter, and were 229.6 feet in height maximum. An array of tug-boats towed them from the dry-dock to their specified locations where they were flooded with sea water and the encompassed footprint of one and a quarter acres was lowered to the prepared sea bed and leveled to within an elevation tolerance of no more than -3.7" . Once the caissons were firmly positioned three stages of concrete were placed: first, the largest, displaced the seawater; the second acted as a seal and the third provided a pier cap, all of which combined for a total of 346,000 cubic yards per pier (Figure 2).

Mr. Masaormi Higashimoto, a principal engineer with the Yokogawa Bridge Corporation, traveled to the US in 1994 and visited with the author. Mr. Higashimoto was taken on a tour of the Dames Point Bridge, in Jacksonville, and the Sunshine Skyway Bridge, across the mouth of Tamps Bay, Florida. Our visitor returned to Japan to continue work on the AKB and mailed a descriptive report on its construction to the author on December 19, 1994 with an invitation to visit his bridge that was 6 years into its 10 year construction period. On January 17, 1995 the Kobe earthquake struck with devastating results and the responding trip was never made.

Even though the AKB was designed for an 8.5 Richter event and the 7.2 Kobe quake had its epicenter within 2.5 miles of the bridge, very little damage was incurred by the bridge because its construction state presented a very flexible target (Figure 3). The foundations and anchorages were in place and the 98 story towers were at their full height with the 50,000 short

FIGURE 3: Limber configuration at time of '95 Kobe quake minimized impact

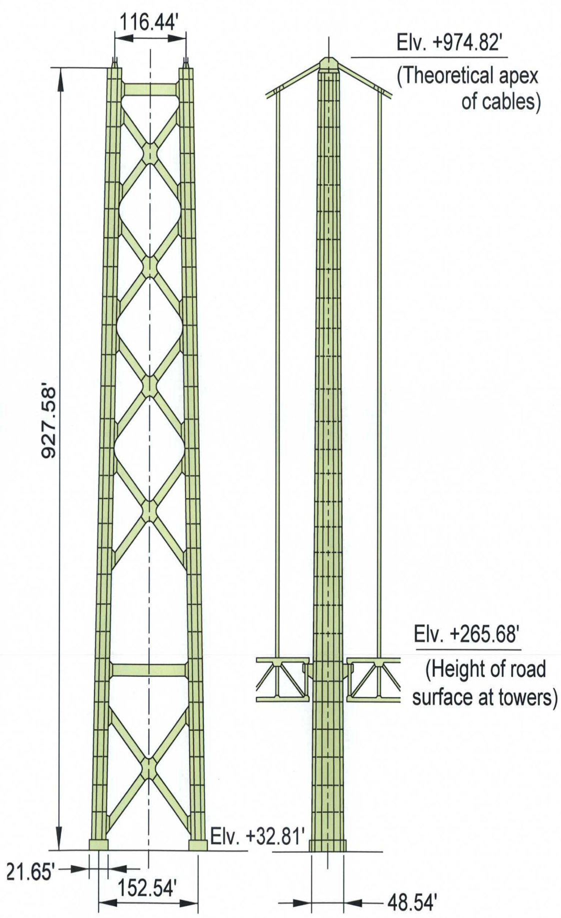


FIGURE 4: Each tower stitched together with 700,000 bolts

ton suspension cables resting in their saddles as they were being compressed into their compact, 3'- 8" designed diameters.

It was further explained that the all steel towers were erected using a free-standing, self climbing derrick that had a maximum lifting capacity of 176 short tons. The derrick was located midway between the two tower shafts and off-set in the longitudinal direction to allow the placing of the two shaft sections and the tower diagonals by the one derrick. Base-plates 27.22' wide and 54.12' long spread the cellular, cruciform tower shaft reactions through this 7" thick, steel slab. The shaft segments were stacked one on top of the other, in 31.8' lifts, thirty times. These lifts were divided into three sections according to their cellular cross-section to accommodate the 176 short ton, derrick, lifting capacity. These stacked steel lifts had to be leveled to within the tolerance of a 1/100", thick gap-gauge (Figure 4).

The research and design that went into the cables provided a major breakthrough with the development of a new, high strength steel cable (was 227,000 psi to 255,000 psi) that made it possible to direct all of the loads into only two cables instead of dividing the suspended loads into four cables, two on each side of the tower tops. The pilot-line that was strung between the anchorages and over the towers was installed by using a helicopter to carry the 3 15/16" diameter, high strength, light weight, poly-aramid fiber rope between anchorages. The cat-walks used to install all of the cables were suspended from these ropes. The twin cables were 2.53 miles in length and were composed of 290, full length, steel strands, each of which were prefabricated and delivered for installation.

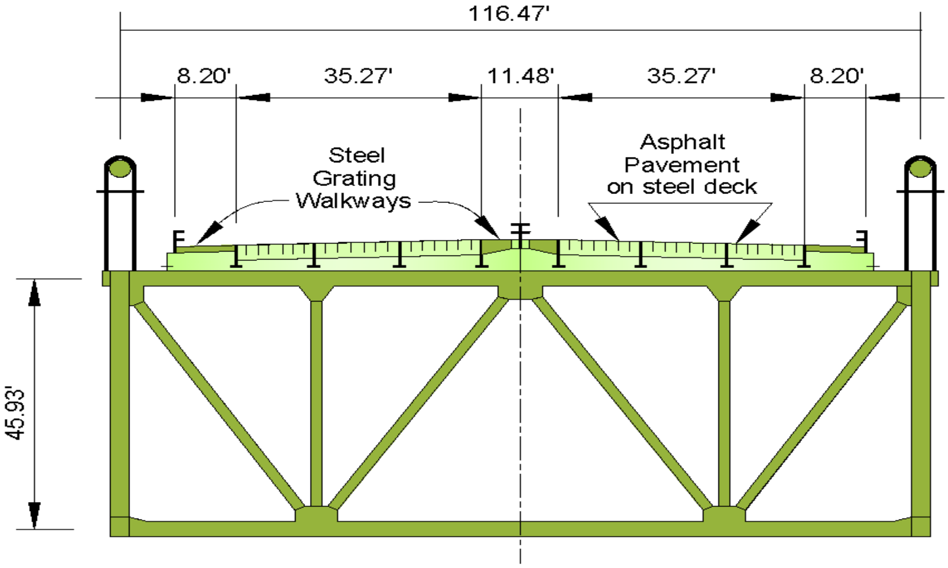


FIGURE 5: Typical section of the 100,000 short ton stiffening truss

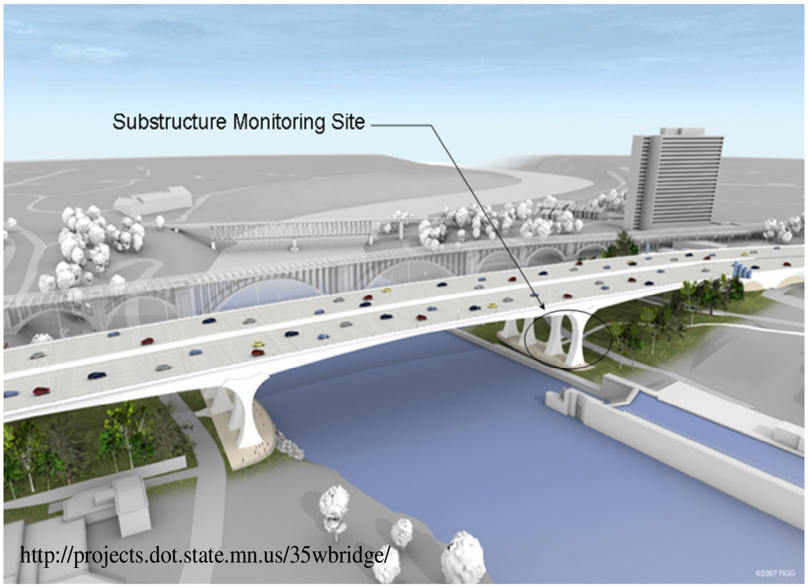
A through inspection of the bridge above and below the waterline immediately following the quake, on January 17 thru the 19, revealed that no serious structural damage was sustained. However, it was determined that pier P3 had moved 2'-7 1/2" along the alignment to increase the main span by that amount and the 4A anchorage had moved 10 3/8" along the alignment to also increase the side span by this amount. The stiffening truss (Figure 5) only required slight modification before the entire 100,000 short tons of steel was hung from the two main cables, after the quake, to complete the construction of the Pearl Bridge (Figure 6).

FIGURE 6: View of the resplendent Pearl Bridge in its distinctive green-gray coat

Guest Commentary

By: Gray Mullins

I-35W Bridge Replacement, With an Ounce of Prevention



Interstate Highway I-35, through Minnesota, has a north-south orientation but I-35 W serves to tie together the easterly north leg and the westerly south leg as a connector, through the city of Minneapolis. This highway connector meets I-35, from the north, just north of the Mississippi River at St. Anthony Falls and turns west after crossing the river with the new, eight lane, highway bridge. Its predecessor collapsed on August 1, 2007 at the peak of the afternoon rush hour traffic. This collapse killed 13 people and opened the eyes of policy makers and engineers alike to the serious nature of America's failing infrastructure.

A tremendous cooperative effort ensued to quickly and safely replace the bridge. The urgency of this task opened minds and softened attitudes resulting in the harnessing of the latest technologies to build and monitor the long term performance of the structure. To that end, the goal of the Federal Highway Administration, through their consultants (the University of South Florida, Tampa and Foundation & Geotechnical Engineering, Plant City) determined how internal instrumentation could be utilized to increase quality assurance, monitor construction loading, and subsequently show traffic and wind loading effects for the long-term pier performance.

This was the first part of a larger program involving the entire bridge structure and included only the South Bound (SB) Pier 2 columns and foundations. Therein, two types of strain gages and thermocouples were installed to monitor three conditions of the bridge's foundation system: (1) the curing temperatures of the concrete foundation elements, (2) construction loading effects on these foundation elements, and (3) long-term monitoring of the structural elements as they continue to perform.

For the sake of brevity I will only focus on the construction load monitoring, item (2) above. Many of the design assumptions used by the engineer are based on probability and historical, codified data (Design Codes). However, with the new sensor technology and remote accessing it will be possible to view in real-time the structural responses to a wide range of loading conditions. With this knowledge one can determine the efficiency of the design and may be able to foresee areas that may be working close to their ultimate capacity so remedial measures can be put in place before an ultimate condition is encountered.

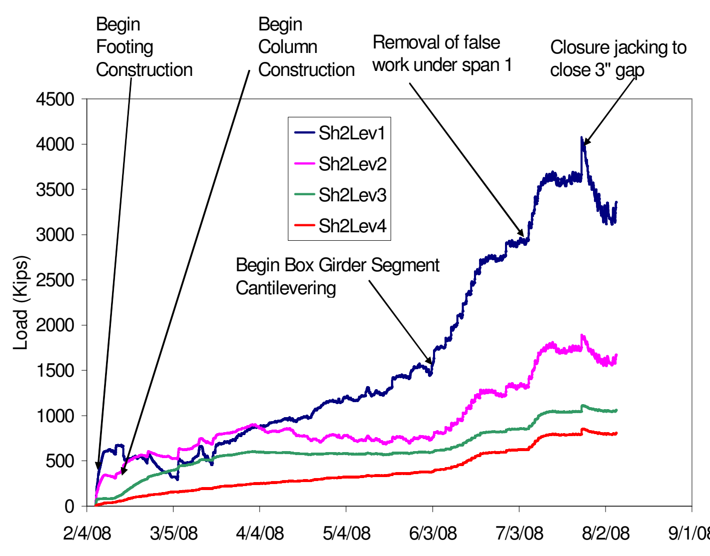
The traditional engineering design uses margins of error (safety factors) which will inflate the predicted loads, reduce the estimated capacity of the structure or include a combination of the two. The result is that the worst case scenarios, that control the design, may never occur and; hence, the structure's design is never fully verified and optimized. One way for the design engineer to confirm and safely optimize his design is to accurately determine the loading as it is applied and actually view the structures response to these loads.

The FHWA goal was to establish a prototypical, monitoring system (at Pier 2 South) for the expedited, I-35W bridge replacement program in order to develop a data-base that will more accurately describe the loading / structural response conditions during construction and for both short and long-term operational aspects for this type structure. The monitoring for the entire Bridge, including the prototypical work by USF/FGE at the SB, Pier 2, will be carried on by the Minnesota Department of Transportation in conjunction with the University of Minnesota.

The structure is founded on steel H sections at the south abutment and on drilled shafts at Piers 2, 3, and 4. The superstructure is a combination of cast-in-place (cip), concrete box-girders built on false-work at the two side spans. The main spans are pre-cast, segmental, concrete box-girders built in free cantilever. The main span piers are positioned on footings that are supported on drilled shafts. Instrumentation for two of the eight drilled shafts at SB, Pier 2 included four levels of 6 strain gauges in these 100' deep shafts. Gauge levels were designated so as to identify load carrying contributions of the various levels of rock formations along the shaft lengths. Gauge levels were at the top of shaft (ground level), top of soft rock, top of competent rock and at the bottom of the shaft.

Each gauge level consisted of 4 vibrating wire (VW) strain gauges positioned at quarter points around the circumference of the shaft cross section. Two of the four VW gauges at each level were coupled with resistance type strain gauges which were situated at opposite sides of the shaft. This scheme capitalized upon the long term stability associated with VW strain gauges and their capability to take instantaneous readings from dynamic loading events.

The attached graph (below) clearly charts the evolution of the construction loads and their accumulation onto the drilled shafts. Correspondingly, we can see how resistance to these loads is distributed: Sh2 Lev 1(top of Drilled shaft) = 3200 kips; Sh2 Lev 2 = 1500 kips; Sh2 Lev 3 = 900 kips and Sh2 Lev 4 = 800 kips.



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