WJE ENGINEERS ARCHITECTS MATERIALS SCIENTISTS

PROJECT SUMMARY

The Martin Olav Sabo Pedestrian Bridge over Hiawatha Avenue consists of a total of ten spans. Span 3 over Hiawatha Avenue consists of a reinforced concrete deck supported by eighteen stay cables (fore stays) arranged in nine pairs and attached to deck anchorages placed along the 215 ft span. At their upper end, each stay cable is attached via cable diaphragm plates at the east side of the steel pylon set on Pier 2. An additional nine pairs of stay cables (back stays) are attached to the west side of the steel pylon, using similar cable diaphragm plates. These back stays are attached to ground anchorages adjacent to Span 1, and are used to balance the fore stay loads applied to the opposite side of the pylon. The cable diaphragm plate anchorages are identified as Cable Diaphragm Plates 1 through 9 (bottom to top) on the east side of the pylon, and Cable Diaphragm Plates 10 through 18 (bottom to top) on the west side of the pylon. Each stay cable is identified by the cable diaphragm plate to which it attaches, and its position within the structure; the left side (L) of the structure is taken as north and the right side (R) of the structure is taken as south.

Each cable diaphragm plate was cut from 3/4 inch thick steel plate, and then welded to the inside surfaces of the 1 1/2 inch thick pylon flanges using 3/8 inch fillet welds around the entire perimeter of each interface. Two holes were cored through each plate to receive the clevis pins for a pair of stay cables. Each of these holes was reinforced using two ring plates: one on the top surface, and one on the bottom. Each ring plate was fillet welded (1/4 inch fillet) to the diaphragm plate all around its outside edge.

Cable Diaphragm Plate 9 fractured on the evening of Sunday February 19, 2012. The fracture was oriented so as to allow the portion of the plate connected to the two fore stays to fall to the ground, with the cables still attached. During the pylon examination that occurred on February 20, 2012, fractures and cracks were discovered in Diaphragm Plate 8. The Diaphragm Plate 8 damage left each of the associated fore stays connected to the pylon. Winds were reported to be out of the south-southeast at 6 mph to 13 mph for several hours prior to the incident. The temperature was approximately 23 degrees Fahrenheit.

Wiss, Janney, Elstner Associates, Inc. (WJE) was retained by Hennepin County and the City of Minneapolis to perform the following tasks: assist on site during stabilization of the structure; inspect the remaining in-service diaphragm plates using nondestructive test methods; determine the cause of the cable diaphragm plate failures; perform an independent review of the bridge design for Spans 1 through 3; and develop retrofit options to address any noted deficiencies.

During stabilization of the structure, WJE assisted with the de-tensioning of Cables 8L and 8R by providing pre- and post-operation bridge inspections to determine if the structure was responding as expected. Following the de-tensioning operations, WJE performed various nondestructive tests on the remaining sixteen in-service cable diaphragm plates to look for defects. Visual, magnetic particle, and ultrasonic inspection methods were used. This evaluation revealed one small defect at the right lower weld wrap around at the end of the diaphragm plate-to-pylon flange interface for Cable Diaphragm Plate 7. This defect was characterized as a weld lack of fusion defect at the weld toe and was subsequently removed by WJE using light grinding techniques. In addition, cracks were observed at the right (3/4 inch long) and left (1 1/2 inch long) of Cable Diaphragm Plate 5 initiating from the pylon flange-to-diaphragm plate weld toes on the bottom surface of the plate. This discovery resulted in the design, fabrication, and installation of a steel temporary support fixture to provide load-path redundancy at Cable Diaphragm Plate 5.

Plate samples, including the fracture surfaces from Cable Diaphragm Plates 8 and 9, were removed from the structure and sent to Dr. John W. Fisher at Lehigh University for assessment. Dr. Fisher visually examined each of the fracture surfaces, performed a fractographic study using a Scanning Electron Microscope (SEM), and performed metallographic examinations of selected welded connections. Tests were also performed to determine the chemical, physical, and toughness properties of the steel diaphragm plate material. Dr. Fisher's findings included the following:

- The welded details used on the cable diaphragm plates provide low fatigue resistance.
- Prior to its fracture, Cable Diaphragm Plate 8 sustained fatigue cracking with approximately 7 1/2 inches of stable growth occurring at the right side of the plate between the lower end of the pylon flange-to-diaphragm plate weld and the right ring plate-to-diaphragm plate weld, initiating in both locations. These two fatigue cracks coalesced and then extended in an unstable manner to intersect the top edge of the plate, dividing the plate into two pieces. A similar but shorter fatigue crack existed at the left side of the plate, initiating at the lower end of the pylon flange-to-diaphragm plate weld, but there were no signs of significant unstable crack extension.
- Cable Diaphragm Plate 9 sustained extensive fatigue cracking before it fractured. A stable crack extension of approximately 11 1/4 inches occurred at the right side of the plate between the pylon flange-to-diaphragm plate weld and the right ring plate-to-diaphragm plate weld, initiating in both locations. A separate stable crack extension of about 9 inches was noted at the left ring plate weld. These fatigue cracks extended in an unstable manner, separating the front section of the diaphragm plate, including the ring plates, from the remainder of the plate.
- The noted fatigue cracks initiated and extended due to cyclic stresses caused by wind-induced cable vibrations.
- The steel plate material conformed to the design specifications.

WJE devised and implemented an instrumentation program to characterize the response of selected cable diaphragm plates to wind-induced cable vibrations. Various instruments, including strain gages, displacement transducers, accelerometers, wind monitors, and thermocouples, were installed on the structure at selected locations, and monitored over a continuous period of 18 days. Structural responses were correlated to wind speed and direction throughout the monitoring period. Calculations using this data were performed to determine the effective stress range for the fatigue sensitive weld terminations. "Pluck testing" of selected cables was also performed, where a statically induced 250 lb load applied perpendicular to the cable alignment was instantaneously released, and the corresponding responses to the induced cable vibrations were recorded. The results of the instrumentation program included the following:

- Low wind speeds (approximately 5 mph to 10 mph) with a direction approximately transverse to the structure alignment result in cable vibrations that induce damaging stress range cycles at fatigue sensitive details in the cable diaphragm plates.
- These cable vibrations occur at high frequencies, resulting in rapid accumulation of stress cycles during the causative events.
- The effective stress range (S_{re}) calculated for instrumented regions of the cable diaphragm plates adjacent to the terminations of the pylon flange-to-diaphragm plate welds, and ring plate-to-diaphragm plate welds is in the range of 2.5 ksi to 3.9 ksi for the events monitored.
- The number of effective stress range cycles accumulated over the 18 day monitoring period was as high as 2,000,000 at some locations.

An independent review of the original bridge design for Spans 1 through 3 was performed by WJE using a finite element model of the structure. The model was used to verify forces reported in the original design calculations package. Structural checks for strength and serviceability limit states were performed for selected primary structural elements, including the steel pylon, the cables, the bridge deck, and the piers. A review of the original design calculations package and the design drawings was also performed. The results of this design review included the following:

- Forces in the primary structural elements determined from WJE's independent finite element analysis model are in general agreement with those reported in the original design calculations package.
- The primary structural bridge elements are adequate for the reported design loads.
- Stay cable vibrations (wind induced) were not included in the original design calculations package.

WJE developed options for replacing the stay cable connections that had been provided by Cable Diaphragm Plates 8 and 9, and for retrofitting the sixteen remaining in-service cable diaphragm plates. All retrofit options were designed to provide increased strength and fatigue resistance for the cable connections.

Failure of the Sabo Pedestrian Bridge Cable Diaphragm Plates 8 and 9 occurred due to fatigue cracking that initiated at weld toes with low fatigue resistance. These welds connected the 3/4 inch thick diaphragm plates to the pylon flanges, and to the reinforcing ring plates used at the cable connection points. The fatigue cracks extended to critical size in both plates due to wind induced cable vibrations that resulted in unstable fractures.