SAN FRANCISCO-OAKLAND BAY BRIDGE

SUMMARY OF WELDING STUDIES

April 2005 – July 2005

BACKGROUND

A hotline tip in February 2005 led the FBI and Office of Inspector General (OIG) to investigate welder allegations related to substandard welding in the San Francisco-Oakland Bay Bridge (SFOBB) Skyway contract. The welds in question were in the shear plates (or Pile Head Connection Plates), which connect the pile to the sleeve around the pile in the footing box. Each pile has eight shear plates with four welds each, or 32 welds per pile and a total of 5,248 in the contract. At the time of the allegations, twenty-four of the twenty-eight steel footing boxes had been filled with concrete, making the welds inaccessible in those footings.

Investigation of the Pier E4W Foundation

FHWA hired three consultants to conduct an independent assessment of the Pier E4W foundation, one of the four footings that had not yet been completed. From E4W, three shear plates (and their welds) were removed and tested. The reports generated by these consultants are summarized below and have been previously made available to the State:

Mayes Testing Engineers (MTE)

MTE performed Magnetic Particle testing (MT) of 25% of the welds in Pier E4W, then transferred the removed weld samples to a laboratory in Washington State where they were saw cut into six-inch sections and macroetched. The resulting specimens showed no evidence of unacceptable flaws and were of excellent workmanship. The measured weld cross-sections exceeded the required 40 mm effective throat length, with actual measurements ranging from 45 mm to 57 mm (average 51 mm).

Roy Teal Inc.

Roy Teal was responsible for Quality Assurance inspection of the MT work done by Mayes at the bridge site. Working with MTE, he developed the weld removal procedure, supervised and photographed the sample removal, and provided an independent assessment of the weld quality in Pier E4W. No major or unacceptable discontinuities were found in any of the welds.

Dr. John Fisher

Dr. Fisher evaluated the weld shear capacity and the possibility of fracture and failure based upon design requirements and actual measured strengths from weld qualification data. He concluded that the pile cap design provided a high level of redundancy and that even if a fracture occurred in one of the welds, the others could still transfer the load. He determined that there was no possibility of crack growth from cyclic loading because earthquake loading is infrequent and would not subject the weld to the millions of cycles required for cracks to form and propagate.

STRUCTURAL ASSESSMENT

Because the investigators revealed that there were allegations for other (completed) piers, FHWA conducted further reviews and analyses to verify that the seismic performance of the rest of the bridge had not been compromised. This work was composed of several parts, as described below.

Phase I: Analysis of Design Code Assumptions

The governing code for the design of shear welds offers a conservative design and accounts for material and workmanship defects through the use of capacity reduction factors in the following formula: Shear Strength = $0.8(0.6F_{EXX})tL$. In this formula, the 0.8 and 0.6 are capacity reduction factors, which provide a built in conservatism in the design. The net result is that the contractor supplies a weld that is stronger than what is assumed in the design.

The geometry and physical properties of the welds were also assessed. Contract Change Order number 50 increased the overall weld size by enlarging the bevel in the pile and pile sleeve (see Fig. 1). This was done to streamline the inspection process by eliminating Ultrasonic Testing (UT), originally required by the contract as the method for verifying the depth of preparation. The net result of this change is a larger weld than what was originally designed.

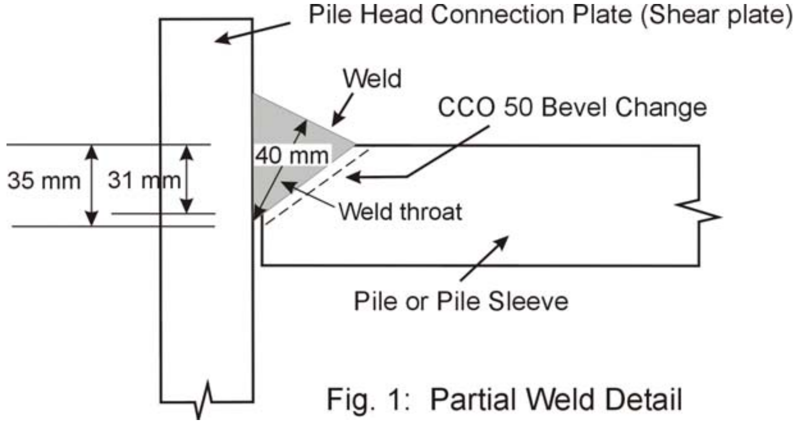

Fig. 1: Partial Weld Detail

The tensile strength of the in-place weld was obtained from the Procedure Qualification Records (PQR), which are developed during qualifying the weld for production. The tensile strengths recorded in the project PQRs ranged from 89 ksi to 93 ksi. The design and physical properties are summarized below:

	Designed	Tested	% Increase
Effective Throat Dimension	40 mm	45 mm – 57 mm (Pier E4W)	12% – 42% (27% avg)
Weld Strength	70 ksi	89 ksi - 93 ksi (PQR)	27% – 33% (30% avg)

Phase II: Quality Assurance/Quality Control (QA/QC)

This Phase consisted of two parts: Phase II (a) was a review of the QA/QC inspection documents for Pier E4W, Pier E9E and Pier E14E (576 welds, total). Phase II (b) was a process review conducted by Roy Teal in June 2005 of the QA/QC procedures used on the job. FHWA provided oversight for both of these Phases.

Phase II (a): Piers E9E and E14E were selected because there were allegations that they contained defective welds; Pier E4W was the control pier because physical samples had been extracted from it. This approach was formulated in order to utilize the information that was known about Pier E4W and extrapolate it to the other piers. In other words, if the level of inspection for Piers E9E and E14E was similar to Pier E4W, then it was assumed that the quality of the welds was similar.

The QC documentation reviewed for this Phase was contained on three forms developed by the contractor: the Daily Visual Inspection Report, the Report of Magnetic Particle Examination of Welds and the Daily Production Log. The QA information was taken from the Caltrans Welding Inspection Report, the TL-6031.

The inspection procedures utilized on the shear plate welds were a combination of Magnetic Particle Testing (MT) and Visual Inspection (VT). At the beginning of the welding, audible sounds and cracking in the base metal were occurring, which led to the use of Acoustic Emission Testing (AE) on Pier E14E, only, to trace the source of the sounds and resolve the problem.

The main findings of the review are summarized below:

- QC inspected 100% of the welds for fit-up and performed MT of 100% of the root passes and 100% of the cap passes (as per contract requirements).
- QC performed VT throughout the production of the welds and documented checks of welding parameters for a significant number of welds.
- QA documented independent confirmation of welding parameters and independent verification testing.

- Rejected items were noted and their follow-up corrections were documented in the reports.
- QC and QA frequently documented the same findings independently. This occurred 273 times in Pier E14E, 166 times in Pier E9E and 152 times in Pier E4W.

Overall, the records show that the welds were thoroughly examined and problems were addressed and solved, such as the early cracking issue. Numerous refinements to the welding processes and inspection procedures were implemented, some of which were not required by the contract. These refinements led to the successful process that was in place for all the piers welded after Pier E14E and demonstrate that proper care was exercised to ensure a high quality product that met the contract requirements.

Phase II (b): Roy Teal performed an independent process review of the QA/QC procedures for welding inspection on Pier E5W. He reviewed inspection documents and interviewed various state, contractor and consultant personnel. No major flaws were found in the process. The report stated, “Based on this QA/QC process review, workmanship, quality standards and the owner/contractor relationship appear to be excellent, and conform to or exceed the requirements of the contract documents.”

Phase III: Dr. John Fisher’s Second Report

The investigators determined that there were approximately ninety locations with compromised welds, located mainly in Piers E14E, Pier E15E and Pier E16E. Since it had previously been relayed to FHWA that the deficient weld locations were in other piers, FHWA elected to apply two-thirds of the ninety to Piers E14E thru E16E for the analysis. This resulted in 20 locations assumed to be in each of those three piers. Dr. Fisher concluded the following:

- The probability of two welds on the same side (i.e., pile or sleeve side) being substandard is very low.
- It is likely that the alleged twenty substandard welds per footing would be distributed among the piles, resulting in a probable upper limit of three to four substandard welds per pile in a six-pile footing and five substandard welds per pile in a four-pile footing.
- If one weld on a plate were substandard, the other weld on that side could still transfer loads to the shear plate because the capacity provided by a single weld is greater than the limiting pile force at its yield capacity. Thus, loads could still be transferred with eight substandard welds per pile, or forty-eight welds per six-pile footing, and thirty-two welds per four-pile footing.

Dr. Fisher concluded by saying “I am more than ever convinced that the results to date have demonstrated that the footing will be able to fully resist the cyclic loading from the 1500-year design.”

CONCLUSIONS

In summary:

- It has been demonstrated that the welds are larger and have an average of 30% more strength than the design requirements because of design conservatism, higher weld material strengths and CCO 50.
- The inspection documentation supports the fact that the State and the Contractor worked together to develop a successful solution to the cracking problem at the beginning of the work.
- The documentation demonstrates that the inspectors provided a continuous, active presence throughout the work.
- Dr. John Fisher’s analysis concludes that even if every pile in the footing had one defective weld the structure could still function because of design conservatism, redundant load paths and weld over strength.

The ninety locations that have alleged defects represent less than 2% of the total number of shear plate welds in the structure. The probability is low that there are other significant locations because of the continuous crosschecks and verifications built into the QA/QC process. FHWA therefore concludes that the welds meet or exceed contract requirements.