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September 20, 2005

Mr. Myint Lwin, P.E.
Director, Office of Bridge Technology
Federal Highway Administration
HIBT – 1, Room 3203
400 Seventh St., SW
Washington, D.C. 20590

Re: FHWA Bay Bridge Welding Investigation

Dear Mr. Lwin,

I have reviewed the materials you provided on June 20, 2005 with your email and attachments regarding the allegations of substandard welds in the footings of the San Francisco – Oakland Bay Bridge. Following are my comments and suggestions regarding the possible substandard welds in East Footings E14, E15 and E16, as well as other observations on the weld capacity.

1. You have indicated that approximately 20 substandard welds are alleged to exist in each of these three footings. This is about 10% of the welds in the six pile footings. Each pile has 8 shear plates connecting the pile to the pile-cap sleave and there are two welds at the pile connection and two at the pile sleave connection for each shear plate. It would seem to me that the probability of having both welds at one of these connections being substandard is very low. The existence of one defective weld in each of the 8 shear plates in each of the 6 piles of a footing(i.e. a total of 48 defective welds in a footing) would not have a significant effect on the connections' ability to transfer loads. As I noted in my May 3 report, only one of the welds at each shear plate would be fully capable of transferring the applicable force delivered by the pile into the footing. Twenty substandard welds in one footing would likely be distributed among all six piles. This would mean that only 3 or 4 substandard welds would exist in each piles shear plates and would not be adverse to transferring the pile forces.

2. The CALTRANS report entitled "Foundation Box E9E and E4W QC/QA Document Review" has demonstrated that the QC and QA inspections at Piers E9E and E4W were similar in scope and nature, as well as being in compliance with the contract requirements. The assessment that I provided on the welds in Pier E4W based on our site visit, the field inspections and destructive examination by Mayes Testing Engineers of full depth weld samples which demonstrated that excellent workmanship and no evidence of major and unacceptable discontinuities, verified the adequacy of the alleged substandard welds. These findings appear to be equally applicable to Pier Box E9E based on the QC/QA records.

I suggest that this same type of assessment be carried out by CALTRANS office of Structural Materials and Materials Engineering and Testing Services, on one of the three

East Footings, E14, E15 or E16, with alleged substandard welds. This would permit an extrapolation to all footings if the results are comparable and further structural analysis using a linear elastic finite element module will not be necessary.

3. I note in the Power Point slides that the weld over strength is based on the 76 ksi to 83 ksi yield point test results I cited on page 4 of my report. I cited the yield point as the temperature shift for bridge loading rates is based on a correlation model that uses yield point. Yielding is also applicable to the fracture assessment.

The shear capacity of the weld is based on the tensile strength, Exx. The specified minimum tensile strength of the electrodes is 70 ksi. The weld qualification tests that I cited from the data provided by CALTRANS demonstrated that the corresponding weld tensile strength varied from 89.1 ksi to 93.4 ksi. Hence, the lower bound strength of 89.1 ksi is 27% greater than the assumed design resistance, not 9%.

Conclusions:

It should also be noted that the design shear capacity has an inherent reserve or factor of safety built into it. As the code Equations cited for the weld metal indicate, a phi (φ) factor of 0.8 is used and the shear capacity is taken as $0.6F_{EXX}$ instead of the minimum value $0.7F_{EXX}$ provided by test results.

The capacity of each weld is even greater than the demand from applied loads than indicated. 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 earthquake.

The weld capacity provided by a single weld on each shear plate connection is greater than the limiting pile force at its yield capacity. This suggests that even up to as many as 48 substandard welds in each pile cap would not jeopardize the ability of the pile cap to transfer its load. More importantly, the QC/QA assessment has demonstrated that the probability of seriously compromised weld joints is not very likely.

Sincerely Yours,

John W. Fisher, PhD
Professor Emeritus of Civil Engineering