BAY AREA TOLL AUTHORITY
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BATA OVERSIGHT COMMITTEE

Wednesday, May 8, 2013, 9:30 a.m.
Joseph P. Bort MetroCenter
Lawrence D. Dahms Auditorium
101 Eighth Street, 1st Floor
Oakland, CA 94607

The Bay Area Toll Authority Oversight Committee considers matters related to the Toll Bridge Accounts and the Regional Measure 1 (RM 1) Bridge Improvement Program.

Chair: Bill Dodd
Vice Chair: Tom Bates
Members: David Campos, Federal Glover, Joe Pirzynski, Jean Quan, Bijan Sartipi +, Adrienne Tissier, Scott Wiener
Ex-Officio: Amy Rein Worth***, Dave Cortese***
Ad Hoc: All Other Commrs.
Staff Liaison: Andrew B. Fremier

This meeting is scheduled to be audiocast live on MTC's Web site: www.mtc.ca.gov.

AGENDA

	ACTION / STAFF RECOMMENDATION**
1. Roll Call	Confirm Quorum
2. Pledge of Allegiance	
3. Compensation Announcement (Committee Secretary)	Information
4. Consent:	Committee Approval
a) Minutes – Meeting of April 10, 2013.*	
b) BATA Financial Statements – March 2013.* (Eva Sun)	
5. Draft FY 2013-14 Toll Bridge Program Operating and Capital Budgets.* (Brian Mayhew)	Information

The Committee will receive for its information an overview of the draft FY 2013-14 Toll Bridge Operating and Capital Budgets.

ACTION / STAFF RECOMMENDATION**

6. Contract Amendment – Legal Services: (William E. Donovan).* ($200,000/year) (Adrienne D. Weil) — Committee Approval

   The Committee will be requested to approve a two-year sole source contract extension with William E. Donovan, Attorney at Law, in an amount not to exceed $200,000/year to provide legal services in the area of public finance from July 1, 2013 through June 30, 2015.

7. 1st Quarter 2013 Project Progress and Financial Update Report for the Toll Bridge Seismic Retrofit Program.* (Peter Lee) — Information

   Staff will present an update on construction status of the new eastern span of the San Francisco-Oakland Bay Bridge, including a briefing on the high strength steel rods and bolts.

8. Public Comment / Information / Next Meeting — Information

   The next meeting of the BATA Oversight Committee will be June 12, 2013, 9:30 a.m. in the Lawrence D. Dahms Auditorium, 1st Floor, 101 Eighth Street, Oakland, CA.

* Attachment sent to committee members, key staff and others as appropriate. Copies will be available at the meeting.

** All items on the agenda are subject to action and/or change by the Committee. Actions recommended by staff are subject to change by the Committee.

*** BATA Authority Chair and vice-chair are ex-officio voting members of the BATA Oversight Committee.

\+ Non-voting member.

++ Item will be distributed at the meeting.

Quorum: A Quorum of this committee shall be a majority of its regular non-ex-officio voting members (5).

Public Comment: The public is encouraged to comment on agenda items at committee meetings by completing a request-to-speak card (available from staff) and passing it to the committee secretary. Public comment may be limited by any of the procedures set forth in Section 3.09 of MTC's Procedures Manual (Resolution No. 1058, Revised) if, in the chair's judgment, it is necessary to maintain the orderly flow of business.

Meeting Conduct: If this meeting is willfully interrupted or disrupted by one or more persons rendering orderly conduct of the meeting unfeasible, the Chair may order the removal of individuals who are willfully disrupting the meeting. Such individuals may be arrested. If order cannot be restored by such removal, the members of the committee may direct that the meeting room be cleared (except for representatives of the press or other news media not participating in the disturbance), and the session may continue.

Record of Meeting: MTC meetings are recorded. Copies of recordings are available at nominal charge, or recordings may be listened to at MTC offices by appointment. Audiocasts are maintained on MTC's Web site for public review for at least one year.

Accessibility and Title VI: MTC provides services/accommodations upon request to persons with disabilities and individuals who are limited-English proficient who wish to address Commission matters. For accommodations or translations assistance, please call 510.817.5757 or 510.817.5769 for TDD/TTY. We require three working days' notice to accommodate your request.

可及性和法令第六章：MTC 根據要求向希望來委員會討論有關事宜的殘疾人士及英語有限者提供服務/方便。需要便利設施或翻譯協助者，請致電 510.817.5757 或 510.817.5769 TDD / TTY。我們要求您在三個工作日前告知，以滿足您的要求。

Acceso y el Titulo VI: La MTC puede proveer asistencia/facilitar la comunicación a las personas discapacitadas y los individuos con conocimiento limitado del inglés quienes quieran dirigirse a la Comisión. Para solicitar asistencia, por favor llame al número 510.817.5757 o al 510.817.5769 para TDD/TTY. Requerimos que solicite asistencia con tres días hábiles de anticipación para poderle proveer asistencia.

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Memorandum

TO: BATA Oversight Committee

DATE: May 1, 2013

FR: Deputy Executive Director

W. I. 1256

RE: 1st Quarter 2013 Project Progress and Financial Update Report for the Toll Bridge Seismic Retrofit Program

The 1st Quarter 2013 Project Progress and Financial Update Report for the Toll Bridge Seismic Retrofit Program presents program and project updates through the end of March 2013. The report will be delivered directly to the Committee at the meeting. Recent project highlights include:

- On April 24, 2013, the Toll Bridge Program Oversight Committee (TBPOC) updated the Authority on the status of the failed rods at Pier E2 of the new self-anchored suspension span. The TBPOC members confirmed that the 2008 rods securing shear keys S1 and S2 failed due to hydrogen embrittlement. The hydrogen contamination may have come from both internal and external sources. Caltrans is currently working on several retrofit strategies to replace the 2008 rods. The 2010 rods that secure the remaining shear keys and bearings have been tensioned and have not had any failures. A number of 2010 rods will be removed for more extensive testing to identify any differences between the 2010 rods and 2008 rods and to evaluate their capacity to perform as designed. Further, the TBPOC reported that visual inspection and desk audit of other similar rod materials is underway.

- Other work on the SAS is continuing with on-going painting and system installation work and completion of the bridge hinges between the SAS and its adjoining structures. In addition, the contractor is removing temporary support work around the tower and catwalks. All other San Francisco-Oakland Bay Bridge East Span Seismic Safety Replacement Project contracts are on schedule to meet the scheduled Labor Day 2013 Seismic Safety Opening.

- On March 22, 2013, Caltrans reported that all structural repairs to Pier W6 of the West Span of the San Francisco-Oakland Bay Bridge have been substantially completed. The pier was damaged by a ship on January 1, 2013. The cost of the repair was approximately $1.6 million. Caltrans will be seeking reimbursement on behalf of BATA from the responsible parties.

Staff will be available at the meeting to respond to any questions.

Andrew B. Fremier

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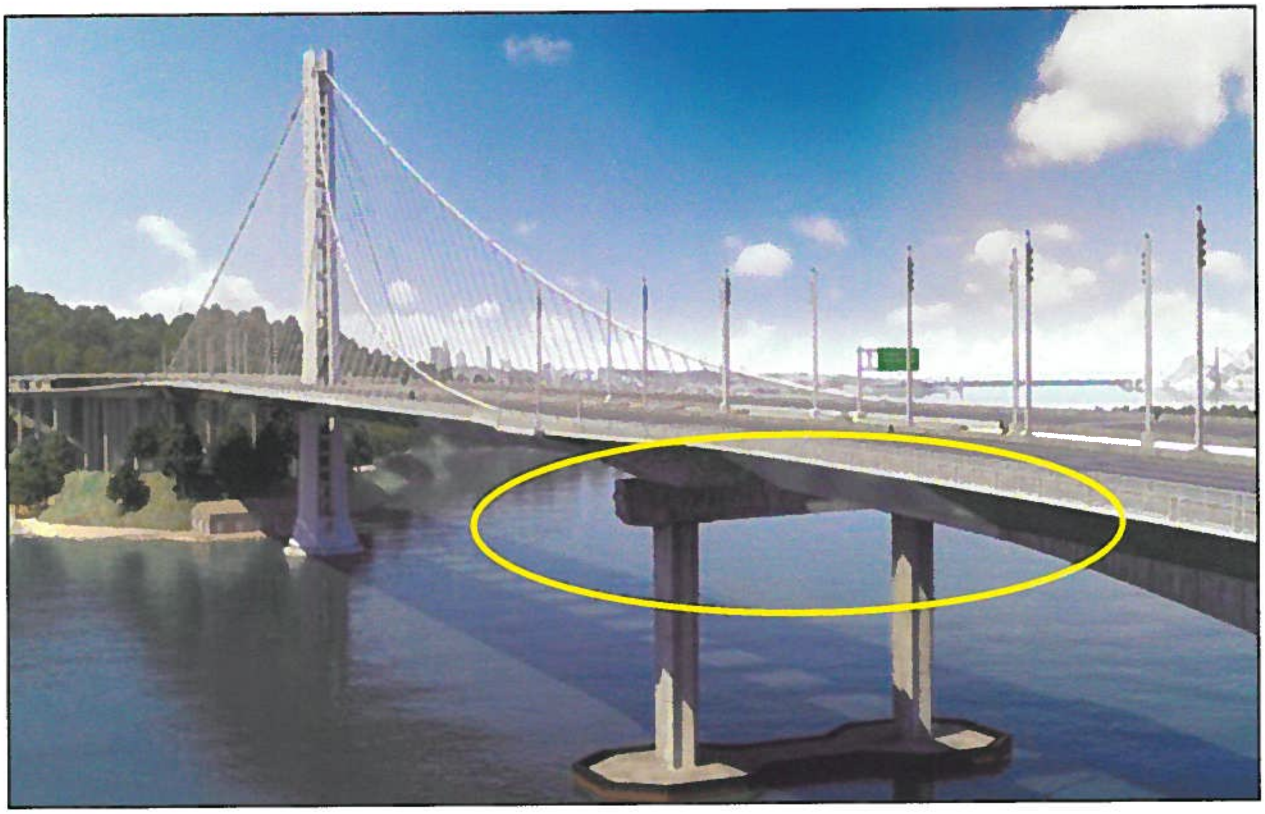
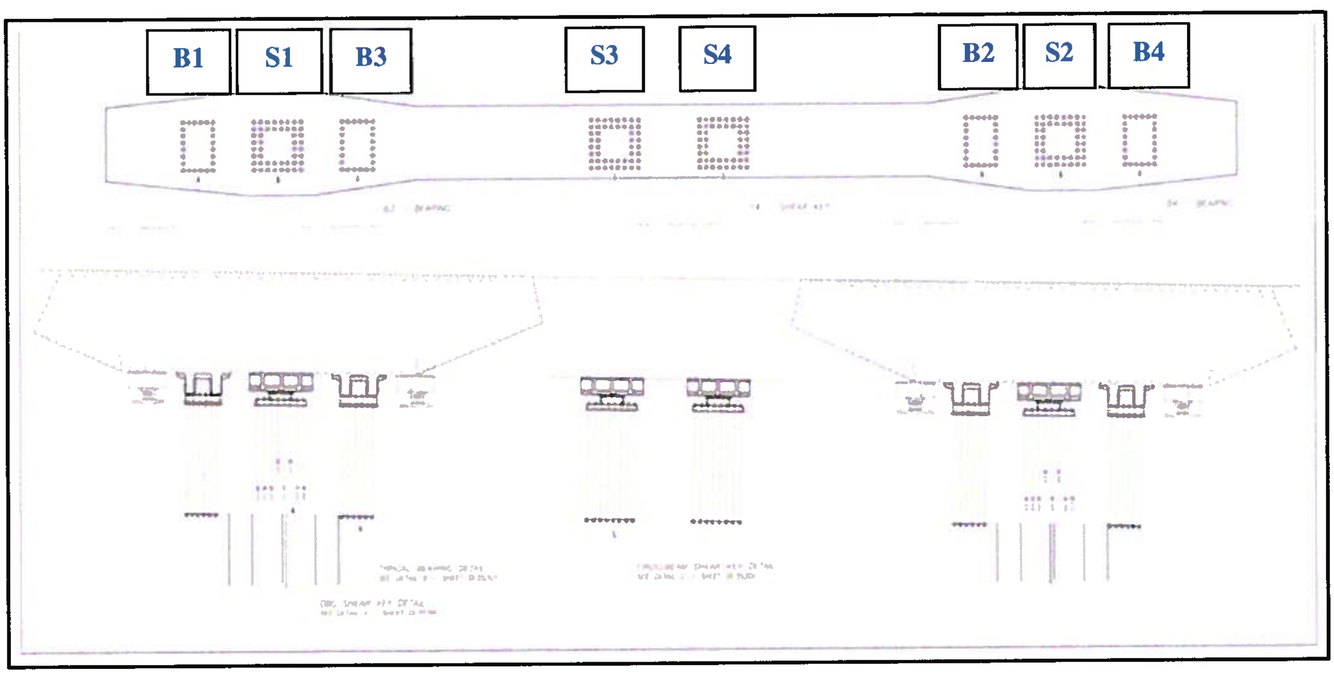


Location of Pier E2



Locations of Shear Keys (S1, S2, S3, and S4) and Bearings (B1, B2, B3, and B4) at Pier E2

SH:pl

Briefing on E2 Anchor Bolts – May 8, 2013

Four Key Questions

1. What caused the E2 anchor bolts manufactured in 2008 to fail?

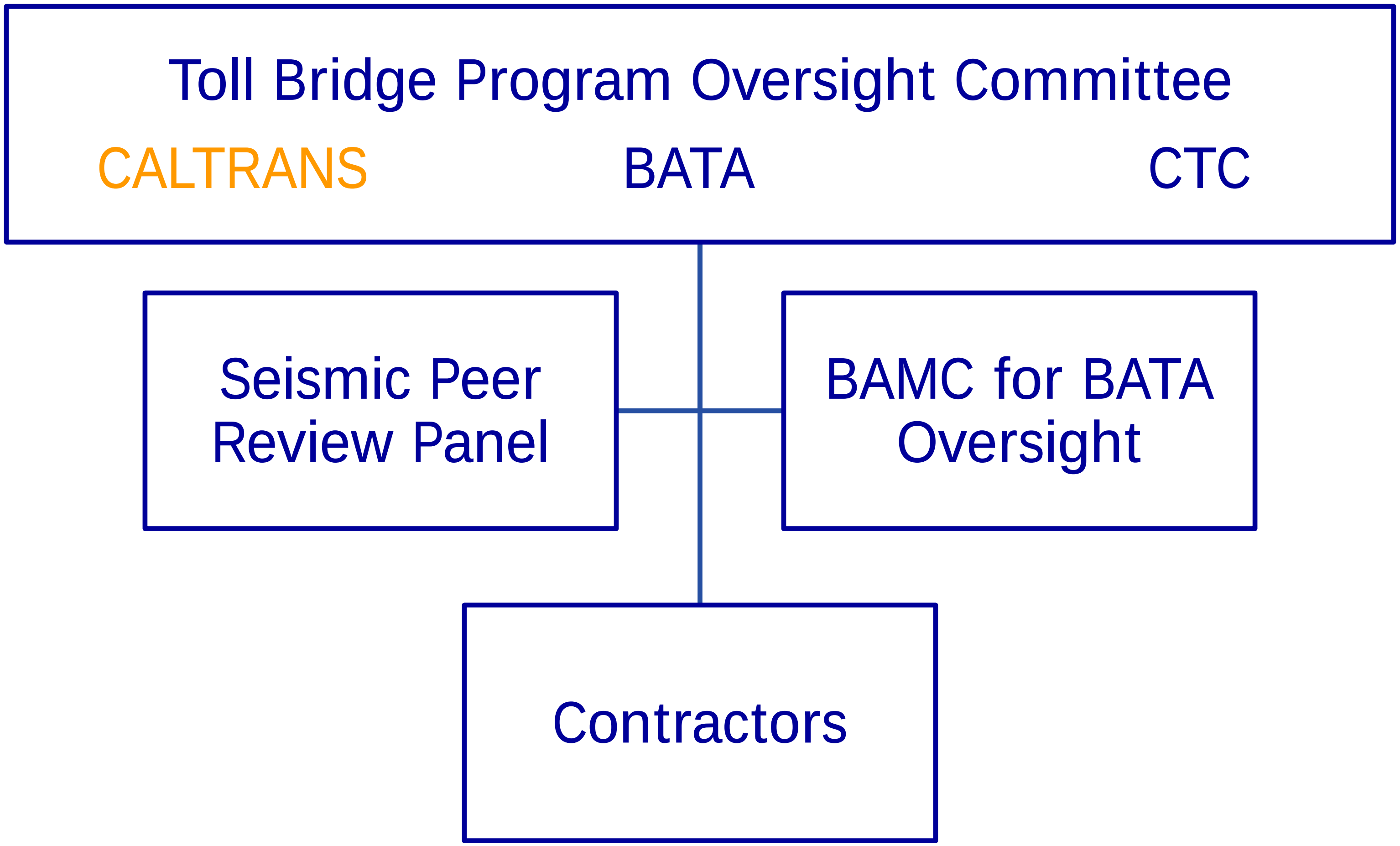
2. What retrofit strategy should be used to replace the 2008 anchor bolts?

3. Should the remaining bolts on the E2 pier manufactured in 2010 be replaced?

4. What should be done about other similar bolts on the SAS?


Pier E2

Oversight Structure



Toll Bridge Program Oversight Committee

- **AB 144 established the *Toll Bridge Program Oversight Committee*, composed of Director of the California Department of Transportation (Caltrans), and the Executive Directors of the California Transportation Commission (CTC) and the Bay Area Toll Authority (BATA), to be accountable for delivering the Seismic Retrofit Program.**

MALCOLM DOUGHERTY
Director
California Department of Transportation

STEVE HEMINGER
Executive Director
Bay Area Toll Authority

ANDRE BOUTROS
Executive Director
California Transportation Commission

Toll Bridge Seismic Peer Review Panel

- Dr. Frieder Seible, Dean Emeritus of the Jacobs School of Engineering at the University of California at San Diego, has consulted on many of the world's long-span bridges and has extensively published related to seismic design and blast resistant design of critical structures.

- Dr. I.M. Idriss, Emeritus Professor of Civil Engineering at the University of California at Davis , is a Geotechnical Engineer who has performed follow-up analysis of every major earthquake since the 1964 Alaska quake and has been part of numerous engineering teams to analyze damage and determine causes of structural collapse.

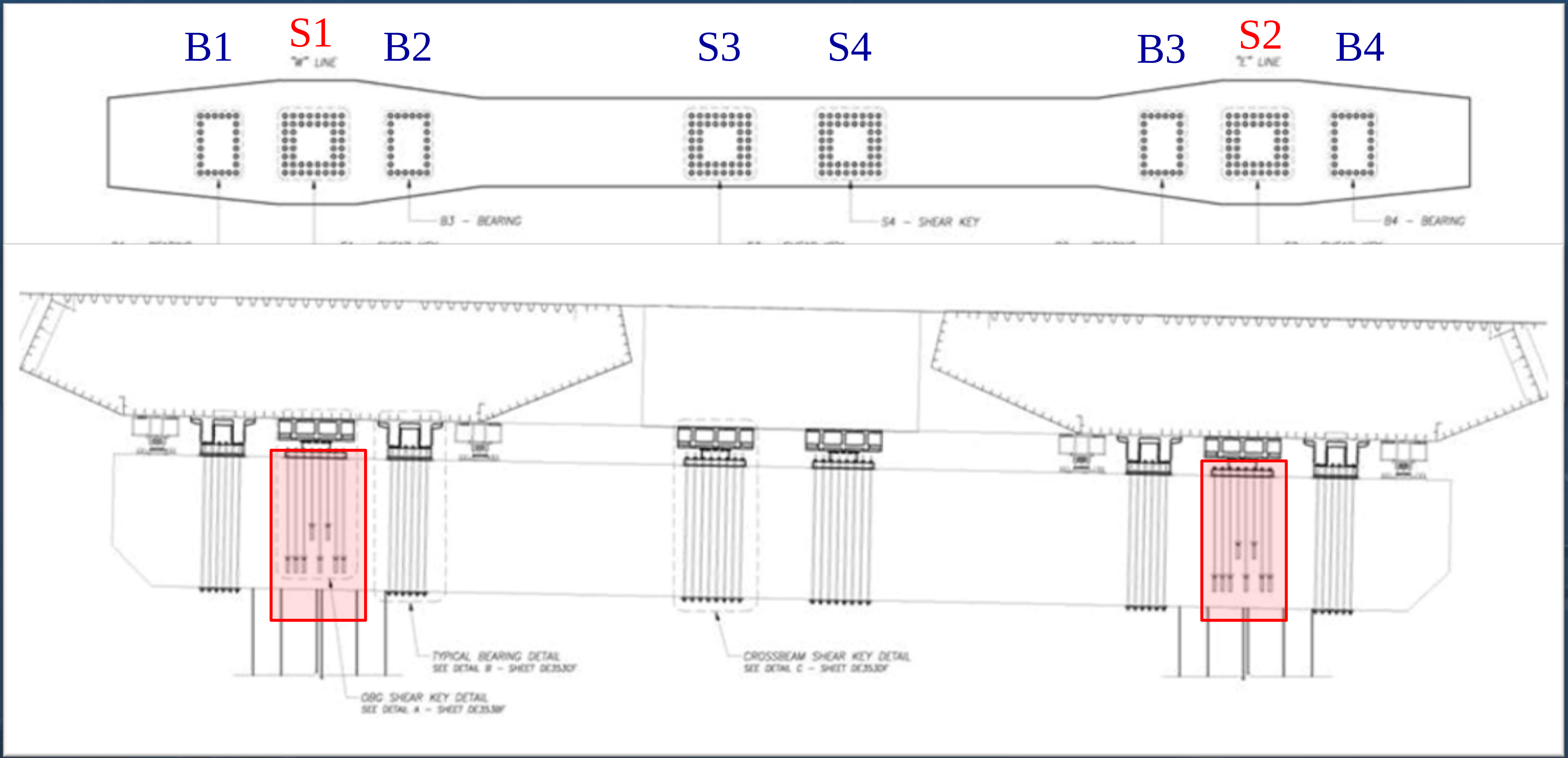
- Dr. John Fisher, Professor Emeritus of Civil Engineering at Lehigh University, has focused his research on the behavior and performance of steel bridges and has examined most of the major failures of steel structures in America throughout the last four decades, including the World Trade Center in 2001.

- All three are members of the National Academy of Engineering.

FHWA Independent Review

- The TBPOC has requested that the Federal Highway Administration (FHWA) conduct an additional independent review of our findings and recommendations concerning the bolts on the SAS.

1. What caused the E2 anchor bolts manufactured in 2008 to fail?

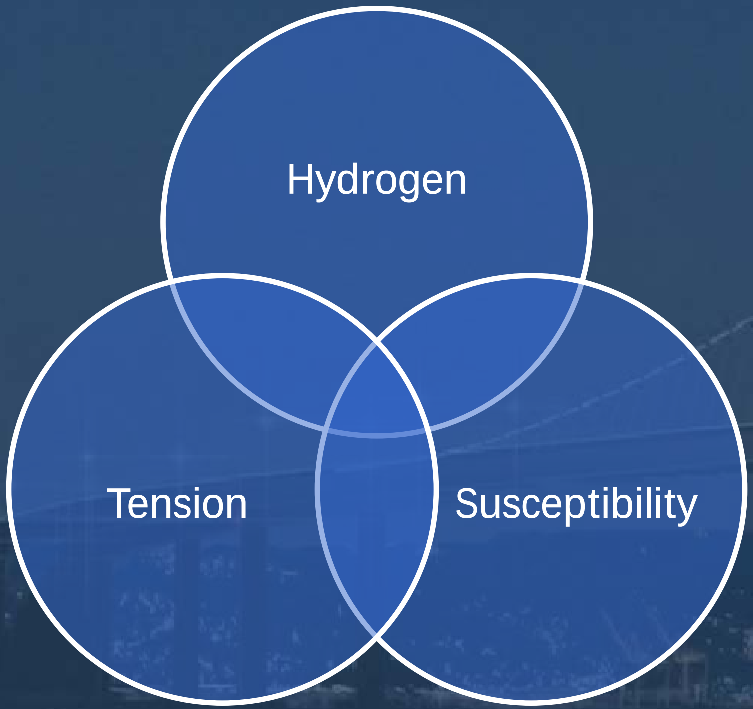


- **Bearings and shear keys are secured to E2 by 3 inch diameter bolts, ranging from 9 feet to 24 feet in length and to the OBG by 2 to 3 inch diameter bolts ranging 2 to 5 feet in length.**
- **96 bolts manufactured in 2008 shown in red are embedded in the pier.**

Failure of 2008 Bolts Due to Hydrogen Embrittlement

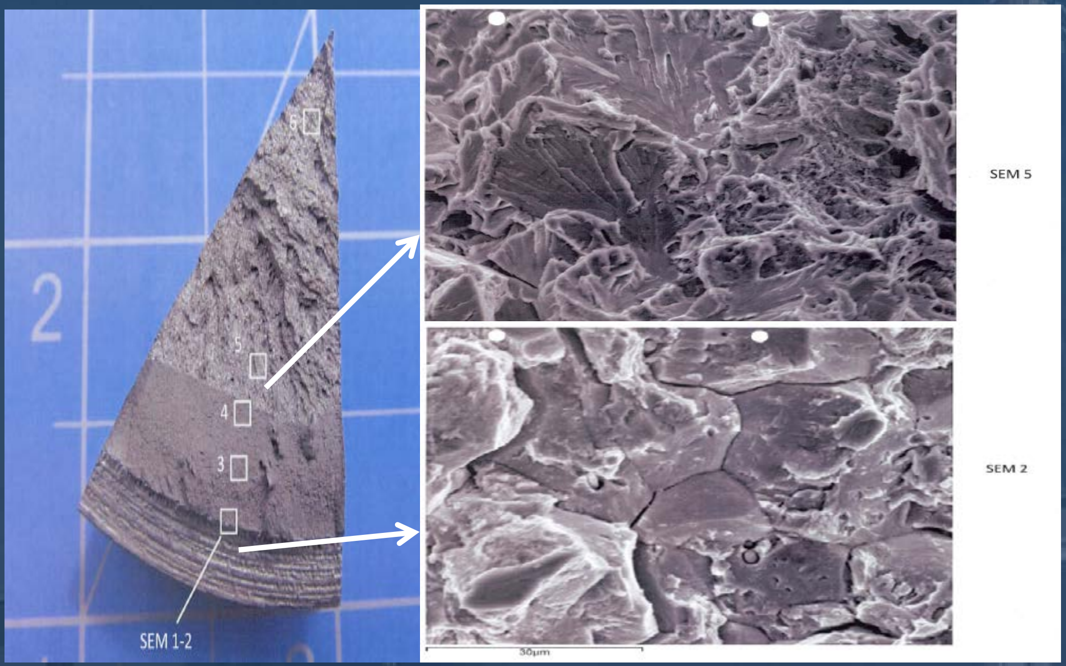
- **As determined by Caltrans and the Contractor, the anchor bolts failed as a result of hydrogen embrittlement.**

Hydrogen Embrittlement



- **Hydrogen embrittlement requires three elements**
  - **Susceptibility**
  - **Hydrogen**
  - **Tension**
- **Root cause of the failure is attributed to higher than normal susceptibility of the steel to hydrogen embrittlement.**

Hydrogen Embrittlement



1000X Magnification

- Metallurgical analysis shows a lack of uniformity in the microstructure of the steel, with large differences in hardness from center to edge and high local hardness near surface. Further, the material exhibited low toughness and marginal ductility.

Report Recommendation

- Procurement of future A354 grade BD anchor rods should include a number of supplemental requirements to assure against hydrogen embrittlement failure.

Specifications for Galvanized A354 BD Bolts

- A technical design specification team evaluated the bridge design and selected A354 BD bolts for this application.

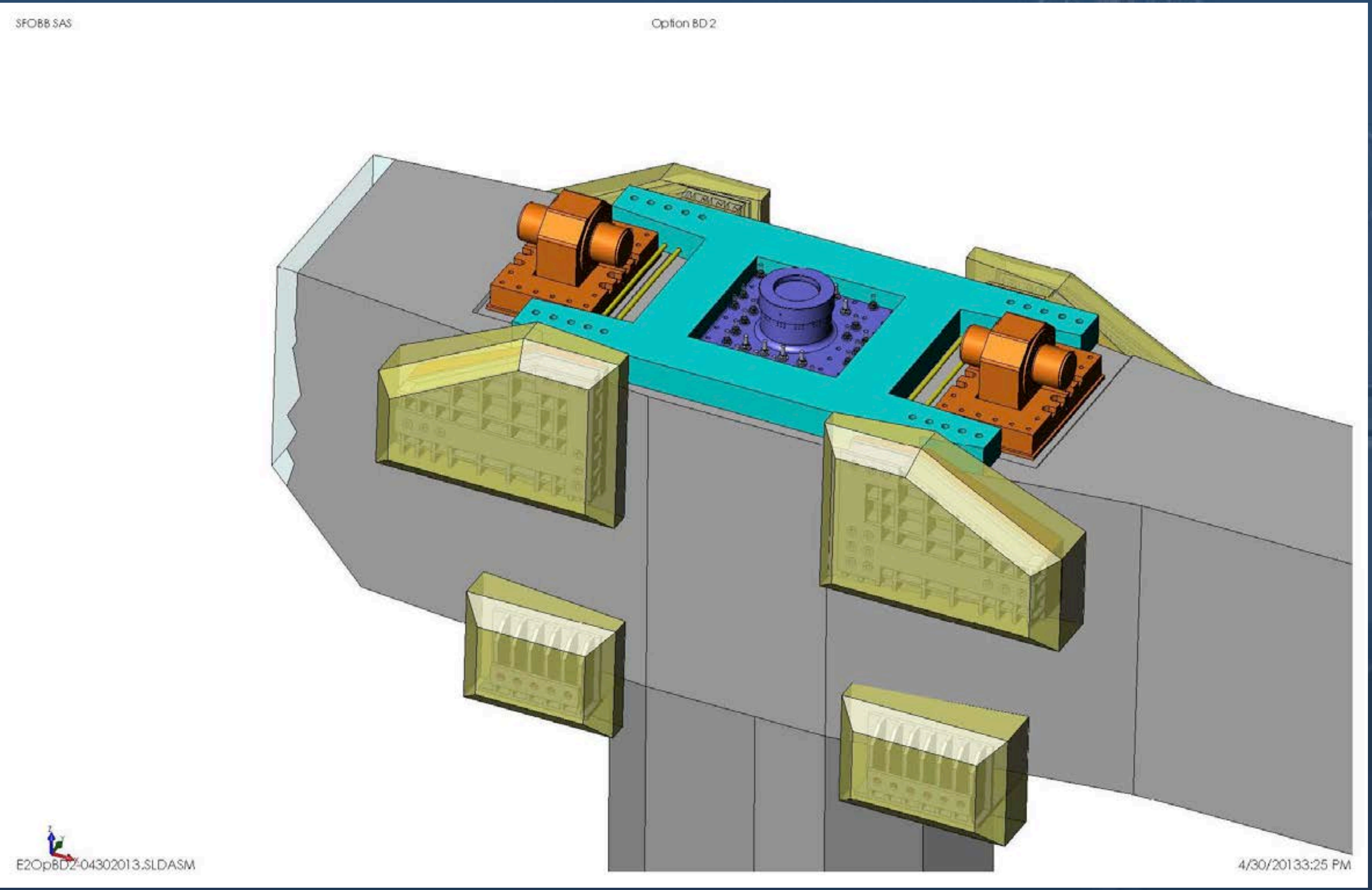
- The design specification team subsequently added a supplemental requirement – blasting instead of pickling – to address the potential for hydrogen embrittlement.

- Current Caltrans Bridge specifications do not allow the use of galvanized A354 bolts for standard bridge applications, but non-standard applications may be considered. ASTM does allow galvanization, but cautions on the potential for hydrogen embrittlement.

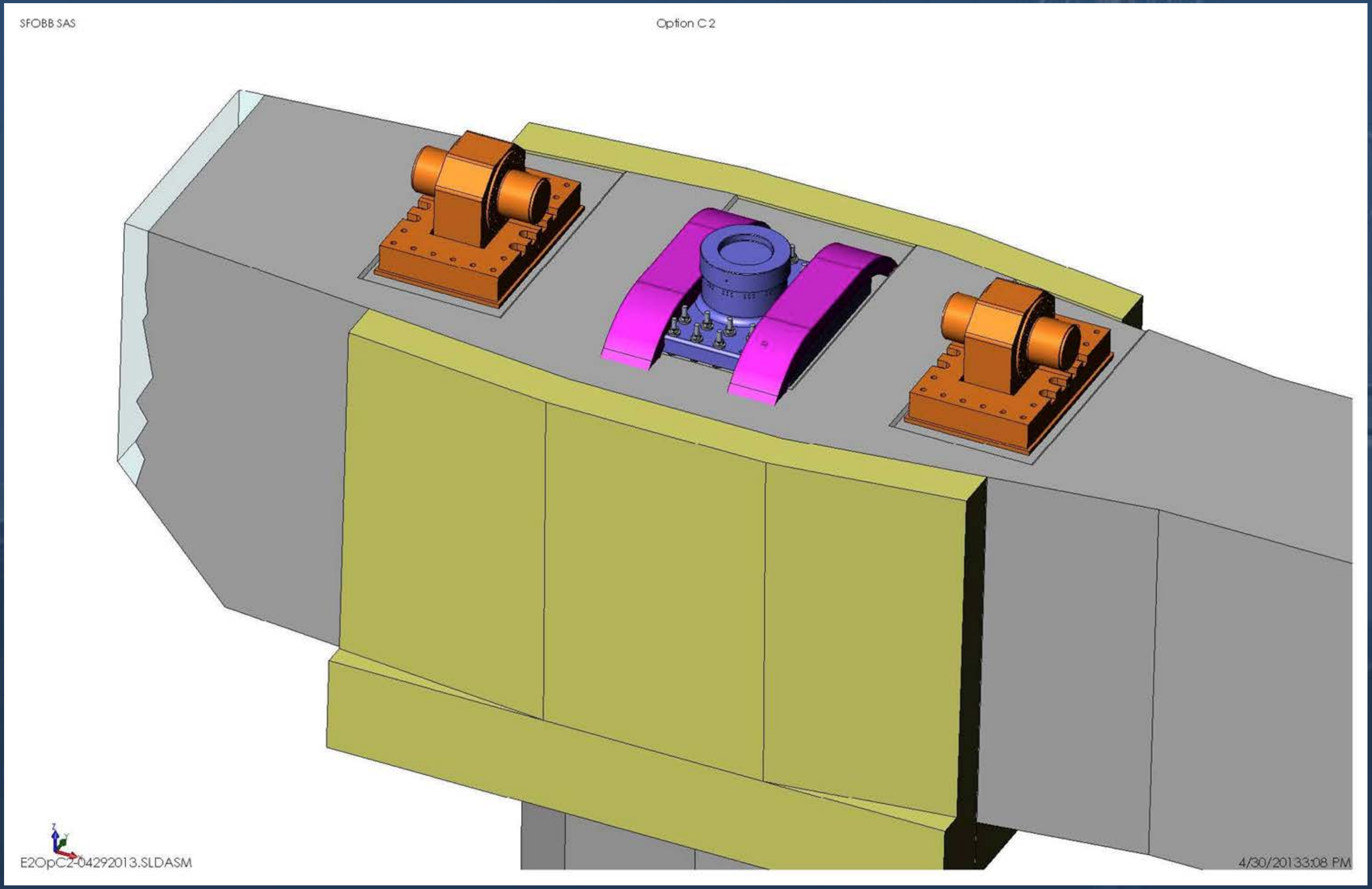
- Caltrans has ordered a limited number of replacement bolts for the 2010 bolts subjected to destructive testing. The special provisions of the specifications for those replacements rods include, but is not limited to, tighter requirements for hardness and additional testing to address hydrogen embrittlement. In hindsight, these supplemental specifications should have been in place for the 2008 bolts.

2. What retrofit strategy should be used to replace the 2008 anchor bolts?

Option 1 – Steel Collar



Option 2 – Steel Saddle



General Comparison of Options

Option 1 – Steel Collar	Option 2 – Steel Saddle
Pros • No need to remove S1 and S2 shear keys • Potentially simpler to fabricate	Pros • No need to remove S1 and S2 shear keys • Less coring of E2 required • Potentially less difficult to install. • Less costly: $5 to 10 m
Cons • Need to find sufficient materials and resources • More coring of E2 required • More costly: $15 to 20 m	Cons • Requires unique saddle system.

Selection of Option 2 - Saddle

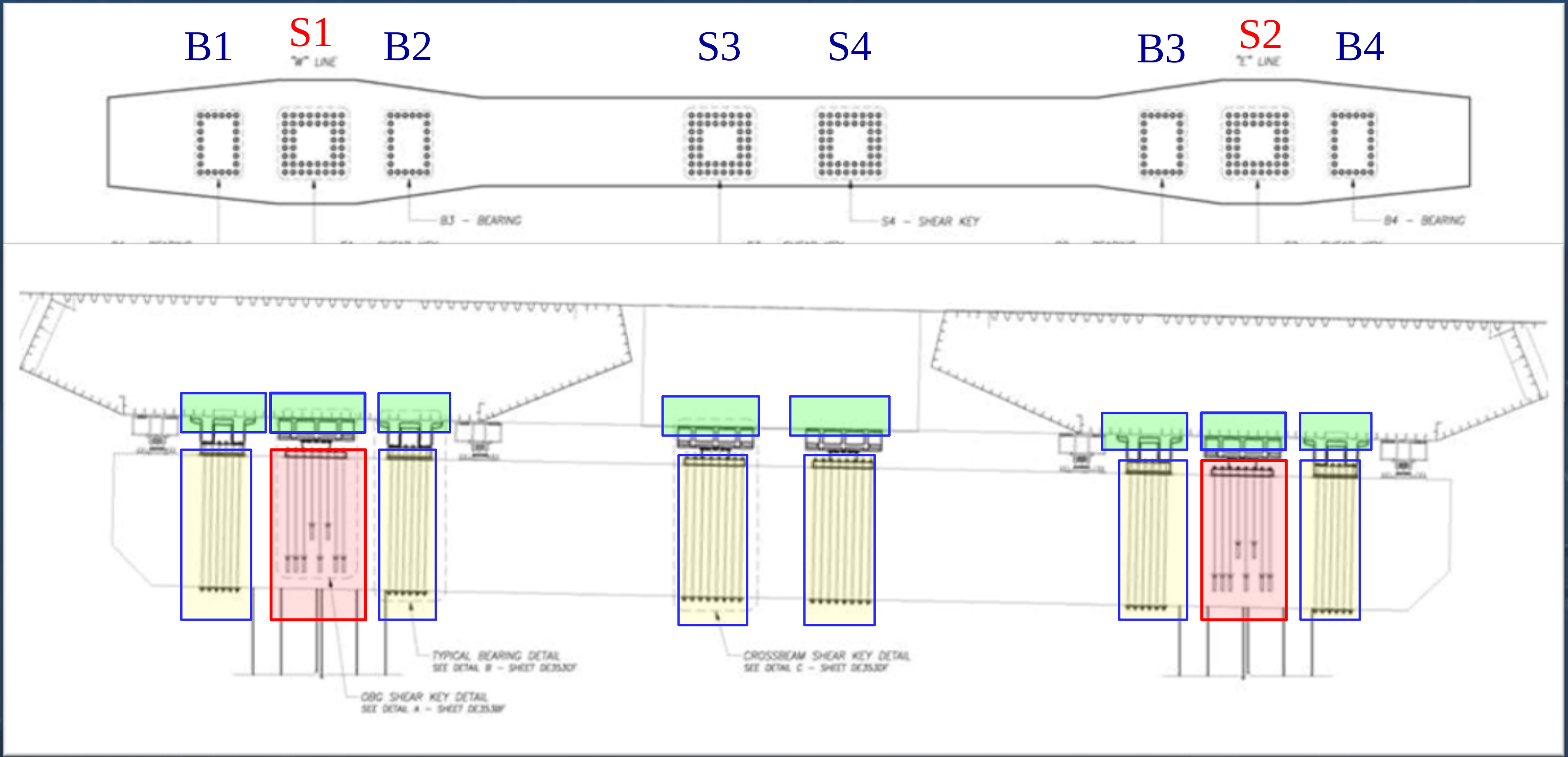
- Both options would provide equivalent clamping force as the original bolt design to hold down the shear keys. The 2008 bolts will be completely abandoned in both options.

- Option 2 has been selected as the retrofit strategy for the 2008 bolts because while requiring more detailed fabrication, installation will be less difficult and require less coring of concrete on Pier E2.

- Estimated cost to construct Option 2 is $5 to 10 m.

- Given the complexities of the retrofit, Caltrans is still working with the Contractor to determine if the retrofit can be completed by Labor Day.

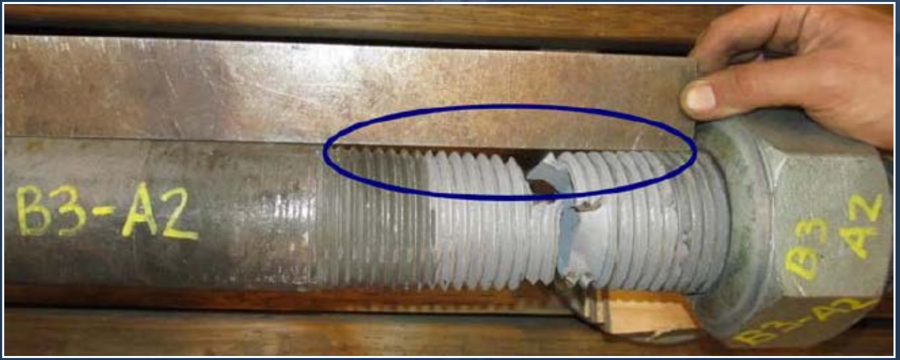
3. Should the remaining bolts on the E2 pier manufactured in 2010 be replaced?



- 96 bolts manufactured in 2008 shown in red are embedded in the pier.
- 192 bolts manufactured in 2010 shown in yellow are <u>not</u> embedded in pier.
- 544 bolts manufactured in 2010 shown in green are connected to the OBG.
- There are an additional 432 bolts of 1 inch diameter, varying from 2 inches to 2 feet in length, that are internal to the E2 bearing assembly.

Preliminary 2010 Bolt Results

- No bolts have broken after more than a month of tensioning.
- Preliminary test results for 2010 bolts, including full sized destructive testing, show more ductile material properties and no hydrogen embrittlement.
- Additional testing results are anticipated, including surface hardness, toughness, microscopic examination, and corrosion testing



	Tensile (KSI)	Yield (KSI)	Elongation (%)	Reduction of Area (ROA)	Hardness (HRC)	Charpy Toughness (ft-lb)
ASTM Requirements D>2 ½”	140 (min)	115 (min)	14 (min)	40 (min)	31-39	N/A
2008 E2 Bottom Average	164	142	14	48	37	@40˚ - 13.5 @70˚ - 16.2
2010 E2 Bottom Average	159	139	16	51	34	@40˚ - 37.2 @70˚ - 37.7
E2 Shear Key Top Average	159	141	16	46	35	N/A
ASTM Requirements D = ¼” to 2 ½”	150 (min)	130 (min)	14 (min)	40 (min)	33-39	N/A
E2 Bearing Top Average	161	135	16	54	35	N/A
E2 Bearing Assembly Average	166	154	18	56	36	N/A
E2 Retaining Ring Average	166	148	16	50	35	N/A

Post Heat Treatment QC/QA Mechanical Tests at E2

Wet Testing of 2010 Bolt Results

- A “wet” test is an accelerated test being prepared to determine the longer term susceptibility of the material to stress corrosion.

- Full sized bolts will be soaked in a controlled concentrated salt solution while tensioned progressively over a number of days until failure.

- Data from this test will be used to determine the susceptibility of the material to stress over time and under various loads.

Stress Corrosion

- Long term stress corrosion susceptibility is a function of the size and hardness of material, and level of tensioning.
- With the "wet" testing data, staff will be able to evaluate all similar high-strength bolts used on the project and help determine if additional remedial action is needed.

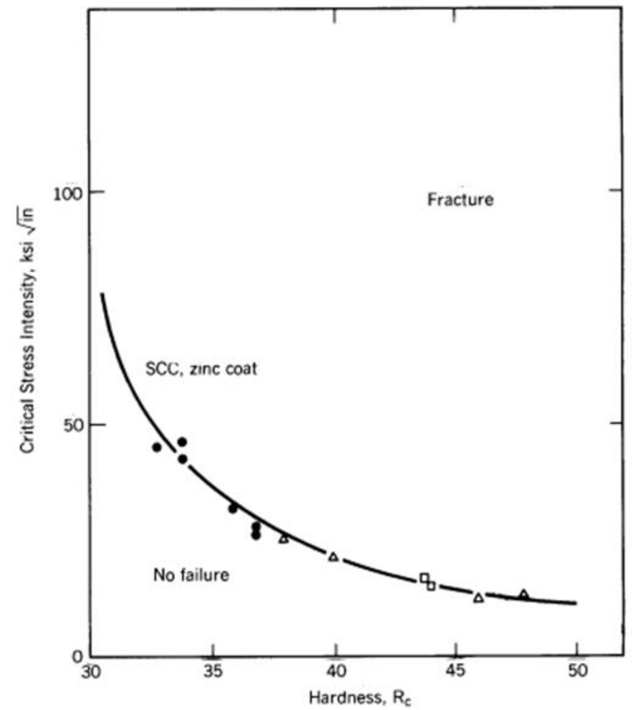
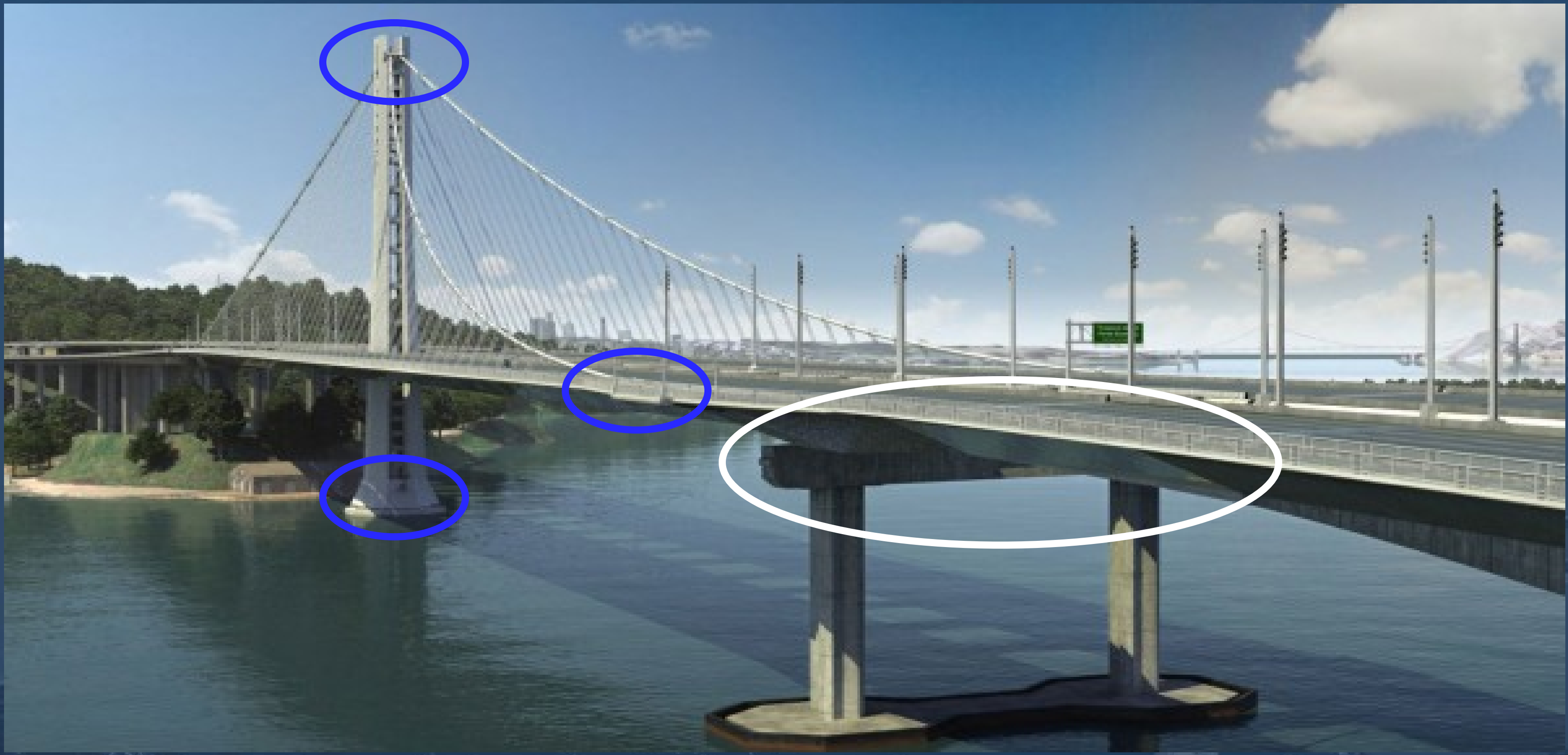


Fig 4.28. Critical Stress intensities in presence of corrosive environment; ● K_{sc}, hot-dip Al-Zn; ▲ K_{sc}, electroplated zinc; ■ K_{sc}, hot-dip zinc.

Sample Critical Stress Curve from ***Guide to Design Criteria for Bolted and Riveted Joints 2nd Edition*** **authored by Geoffrey Kulak, John Fisher, and John Struik and published by American Institute of Steel Construction**

4. What should be done about other similar bolts on the SAS?



- Visual inspections of similar bolts revealed they are performing as required.
- Some E2 Bearing assembly bolts are not accessible to inspection.
- Most bolts at other locations are under lower tension levels.

Other Similar Bolts

Location	Item No.	Description	Diameter (in)	Length (ft)	Quantity Installed	Tension (fraction of Fu)
E2	1	2008 Shear Keys Bolts	3	10 – 17	96	0.7
	2	2010 Shear Keys and Bearing Bolts	3	22 - 23	192	0.7
	3	Upper Shear Key OBG Connections	3	2 - 4.5	320	0.7
	4	Upper Bearing OBG Connections	2	4	224	0.7
	5	Bearing Assembly Bolts for Bushings	1	2.5	96	0.6
	6	Bearing Assembly Bolts for Retaining Rings	1	0.2	336	0.4
Anchorage	7	PWS Anchor Rods	3.5	28 – 32	274	0.4
Top of Tower	8	Saddle Tie Rods	4	6.0 – 18	25	0.4
	9	Saddle Segment Splices	3	1.5 – 2	108	0.1 - 0.5
	10	Saddle to Grillage Anchor Bolts	3	1	90	0.1
	11	Outrigger Boom	3	2	4	0.1
Bottom of Tower	12	Anchor Rods 3"	3	26	388	0.5
	13	Anchor Rods 4"	4	26	36	0.4
East Saddles	14	East Saddle Anchor Rods	2	3	32	0.1
	15	East Saddle Tie Rods	3	5	18	0.1
East Cable	16	Cable Bands	3	10 - 11	24	0.2
W2	17	Bikepath Anchor Rods	1 3/16	1.5	43	tbd
				TOTAL	2306	

Other Toll Bridges

- As part of this investigation, the TBPOC has asked Caltrans to review other toll bridges which may have used similar bolts.

- Caltrans has already identified locations of similar A354 BD galvanized bolts on the Richmond-San Rafael Bridge and completed initial inspections.

- No issues have been found and bolts are performing as required.

Items Expected From Briefing at Special May 29th BATA Meeting

- Pending results from testing of 2010 bolts, decision on whether to replace other Pier E2 bolts and, if so, when.

- Completion of desk review of additional QA/QC results for other high tension anchor bolt locations.

PROJECT INFORMATION

Project# 04-0120F4

SUBJECT

Metallurgical Analysis of Bay Bridge Broken Anchor Rods S1-G1 & S2-A6

METALLURGICAL TEAM

The testing and analysis of the failed anchor rods from shear keys S-1 and S-2 was performed jointly by Salim Brahimi, Rosme Aguilar and Conrad Christensen.

Mr. Brahimi is a consultant to ABF (American Bridge Fluor – joint venture). He is the president of IBECA Technologies. He is a licenced member of the Quebec Order of Professional Engineers and has over 24 years of experience in the fastener industry. Mr. Brahimi holds a masters of materials engineering from McGill University in Montreal. He is the current chairman of the ASTM Committee F16 on Fasteners. He also serves on the ISO TC2 (Technical Committee on Fasteners), ASTM committees B08 (Coatings), E28 (Mechanical Testing), A01 (Steel), F07 Aerospace and Aircraft, Industrial Fasteners Institute (IFI) Standards and Technical Practices Committee, and the Research Council on Structural Connections (RCSC). Mr. Brahimi is recognized and highly respected throughout the fastener industry as a leading expert in fastener manufacturing, fastener metallurgy, application engineering, corrosion prevention, failure analysis and hydrogen embrittlement.

Mr. Aguilar is the Branch Chief of the California Department of Transportation (Caltrans) Structural Materials Testing Branch, responsible for quality assurance testing of structural materials product used in construction projects throughout the state. He has over thirty (30) years of work experience as an Engineer. Twenty three (23) of these years as a Transportation Engineer in Caltrans, two (2) years as a Quality Assurance Auditor for INTEVEP, S.A. (The Technological Research Institute of the Venezuelan Petroleum Industry), and five (5) years as a Researcher in the area of New Products Development at SIDOR (a Venezuelan Steel Mill). Mr. Aguilar holds a Master of Science in Metallurgy (1982) and a B.S. in Metallurgical Engineering (1980) from the University of Utah, Salt Lake City, Utah. He is a Registered Professional Civil Engineer in the State of California. His areas of expertise and responsibility are Quality Assurance and materials testing but in addition he has performed or assisted in the performance of numerous materials characterization and failure analysis for Caltrans and other state agencies.

Mr. Christensen is a consultant to the California Department of Transportation (Caltrans). He is the principal and founder of Christensen Materials Engineering, which provides laboratory testing and materials engineering services. He has over 32 years of experience as a metallurgist specializing in materials testing and failure analysis. His areas of expertise include: microscopic evaluation and characterization of materials, optical microscopy, scanning electron microscopy and fracture analysis. He holds a Bachelor of Science degree in materials science and engineering from the University of California at Berkeley (1981). He is a licenced professional metallurgical engineer in the states of California and Nevada.

EXECUTIVE SUMMARY

Metallurgical testing and fracture analysis was performed on two broken anchor rods that were removed from shear keys S1 and S2. The results indicate that hydrogen embrittlement was the cause of the recent anchor rod failures. Generally, the critical factor to consider when fasteners fail due to hydrogen embrittlement (HE) is the susceptibility of the material to hydrogen assisted cracking. Strength has a first order effect on susceptibility. When the specified tensile strength exceeds 180 ksi (i.e., hardness above 39 HRC), HE susceptibility increases very rapidly. Other variables such as microstructure, fracture toughness and notch sensitivity of the steel have a second order effect that can be significant. Given that (i) critical fasteners are often tensioned to maximize their clamping capacity, and (ii) hydrogen concentrations may be influenced by process conditions and environmental service conditions such as corrosion generated hydrogen, the most effective manner to prevent HE related failures of fasteners is to limit the susceptibility of the material. Conversely, in the rare cases where HE fastener failures do occur, they are often a consequence of the strength/hardness or the metallurgical condition of the material causing the material to become more susceptible than normal or than expected.

This scenario appears to fit the conditions that led to the shear key S1 and S2 anchor rod failures. Although the rods comply with the mechanical and chemical requirements specified in ASTM A354 grade BD, the metallurgical condition of the rods is less than ideal. There is a lack of uniformity in the microstructure which has resulted in regions of high hardness, which has a first order effect on HE susceptibility. Furthermore, the metallurgical structure and substructure of the steel, which are fundamentally a result of alloy selection and heat treatment conditions, has apparently made the rods less tough (i.e., more brittle) and therefore more susceptible to hydrogen embrittlement. These second order metallurgical factors become more critical given the large diameter and length of the rods. Given the material is susceptible, small variations in hydrogen concentrations and/or stress while in service can cause the rod to exceed its HE threshold stress, thus resulting in HE failure.

The metallurgical condition that has led to these failures can be effectively avoided for ASTM A354 BD rods with the addition of a number of supplementary requirements designed to ensure the selection of high quality, high hardenability steel that can be heat treated and galvanized to provide rods with an optimal combination of strength, toughness and uniform microstructure through the entire cross section.

BACKGROUND

A total of 288 ASTM A354 grade BD [1] bearing and shear key anchor rods (3 inch diameter) were installed at Pier E2 per the contract requirements (Figure 1). Ninety six (96) of these anchor rods are installed at shear keys S1 and S2 underneath the E-Line and W-Line OBG's, embedded into the concrete as shown in Figure 2. These rods were fabricated at Dyson between June 4, 2008 and September 6, 2008.

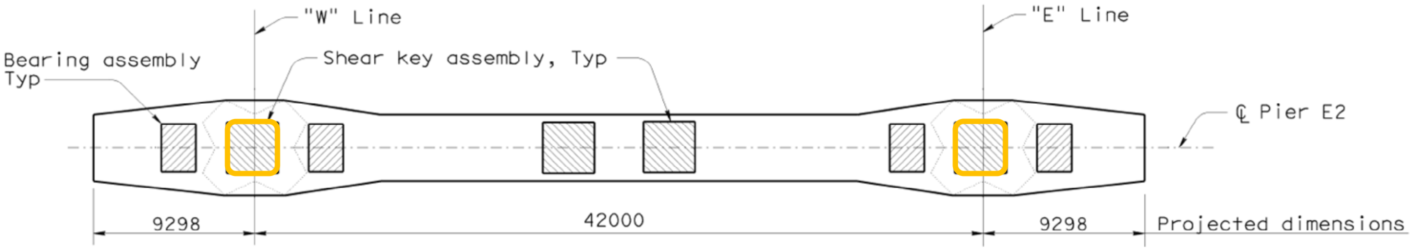


Figure 1: Plan View of Pier E2 Layout

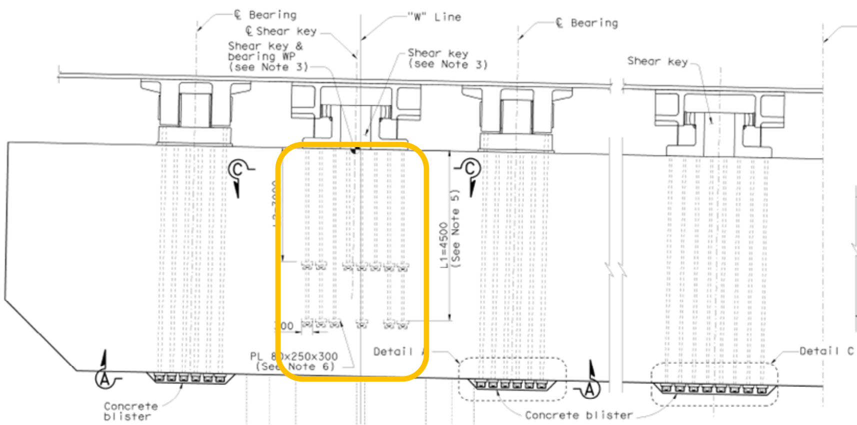


Figure 2: Cross sectional view of the Shear Key Rod Placement

The Contractor started tensioning S1 and S2 anchor rods between March 1, 2013 and March 5, 2013. In accordance with the Contract Documents and approved Submittal 2747, the rods were initially jacked to 0.75 Fu (i.e., 75% of ultimate tensile strength). Due to seating losses as the load is transferred from the jack to the nut, the load is expected to reduce to the final design load of 0.68 Fu. Between March 8, 2013, and March 15, 2013, 32 out of 96 rods fractured. The Contractor extracted three rods (Rod ID's S1-G1, S2-A6, and S2-H6) for further analysis. Figure 3 below shows a schematic of the shear key layout with the rod identification system. Due to small overhead clearance, the rods were extracted in multiple sections. The sections were numbered incrementally from top to the bottom. Therefore the first section pulled out from rod ID S2-A6 is identified as S2-A6 #1 and the bottom section (with the fractured surface) is identified as S2-A6 #11. Pieces S1-G1 #11, S2-A6 #12, and S2-H6 #12 were transported to the Christensen Materials Engineering lab, where they were metallurgically analyzed and destructively tested. This report provides the details of the testing that was performed, and some of the conclusions that were made.

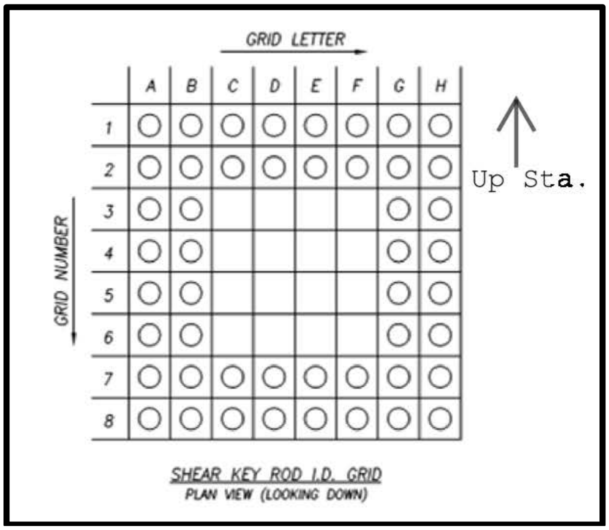


Figure 3: Shear Key Rod Identification Grid

ANCHOR ROD MATERIALS SPECIFICATION

The galvanized steel anchor rods were specified to be ASTM A354 grade BD *"Standard Specification for Quenched and Tempered Alloy Steel Bolts, Studs, and other Externally Threaded Fasteners"*.

The mechanical properties specified for A354 grade BD are as follows:

ASTM A354 Gr BD Mechanical Properties	
Yield Strength	115 psi min.
Tensile Strength	140 psi min.
Elongation in 2 inches	14% min.
Reduction in Area	40% min.
Hardness Rockwell C	31 -39

TEST PROCEDURES & RESULTS

I. Visual Examination/Observations

Approximate 20” sections, which included the retrievable fractured end of three failed anchor rods were provided for testing and identified as S1-G1 #11, S2-A6 #12, and S2-H6 #12. Fracture occurred in the lower threaded ends (i.e., embedded/grouted ends) of all three rods. Anchor rods S1-G1 #11 and S2-A6 #12 were chosen for metallurgical testing to determine the cause of the anchor rod failures.

It was observed that the threads and fracture surface of the as-received rod S1-G1 #11 were covered with Denso paste (part of the Denso Tape system) as required by the Contract Documents. There was Denso paste and grout in the threads and on the fracture surface of rod S2-A6 #12. The fracture surfaces were cleaned and the fractured ends cut from the rods to facilitate further visual and stereo microscopic (up to 80x magnification) examinations.

The overall appearance of both rod fractures was brittle (i.e., there was no thread elongation/stretching to suggest plastic deformation/yielding occurred prior to fracture). Photos 2-8 show the observed fracture features. There was evidence indicating that hydrogen assisted cracks were present in both rods prior to failure. The cracks initiated and extended from the thread root up to a depth of 0.6 inches in Rod S1-G1, and 0.4 inches in Rod S2-A6 (see Photos 4 and 8). The presence and appearance of the cracks, and the delayed nature of the fractures point to time dependence of the failure mechanism. Cracks developed and grew in both rods, which progressively exceeded their capacity with time, and resulted in final failure by fast fracture.

II. Scanning Electron Microscopy

The fracture surfaces were examined at high magnification with a scanning electron microscope (SEM) to further characterize the failure mechanism. Intergranular fracture morphology was observed at, and near, the thread root (i.e., crack origin). See Photos 9-14. Intergranular cracking is a characteristic feature indicative of a number of brittle fracture mechanisms, including hydrogen assisted cracking. Intergranular features were predominant at the thread root (i.e., crack origin). Gradually increasing mixed morphology was observed as the crack progressively grew and extended inward from the thread root (i.e., more ductile tearing, and less intergranular features). See Photos 15-17. Sudden fast fracture occurred when the crack reached a critical size, wherein the reduced capacity of the rod could no longer sustain the applied load. The morphology across the final fast fracture zone was almost exclusively cleavage (brittle fracture mechanism). See Photos 18-20. This observation is considered unusual. Final rupture of bolts (in the strength range of A354 grade BD) caused by tensile overload following crack propagation typically occurs in ductile mode.

Both rods were examined with the SEM and exhibit similar fracture characteristics.

III. Microstructural Examination

Cross-sections were cut from both rods and metallurgically prepared (i.e., mounted/potted, polished and etched). The location of the cross-sections is shown in Photos 21-22. The observed microstructure was generally tempered martensite, which is the normal structure associated with quenched and tempered AISI 4140 steel. However, many areas especially toward the center of the rod, showed evidence of incomplete martensitic transformation. The regions of incomplete transformation, as characterized by the observed presence of ferrite and pearlite, appeared to alternate in banded layers between regions of fully transformed martensitic structure. The banded nature of the microstructure is an indication that the material is not homogeneous. See Photos 23-26. Additionally, there was a relatively high amount of non-metallic stringer inclusions present in the microstructure. See Photos 27 and 28.

IV. Hardness Testing

The Knoop and Rockwell hardness tests are two different hardness testing techniques that correlate to a material's tensile strength, wear resistance and ductility.

The Knoop microhardness test requires a rhombic-based pyramidal diamond indenter pressing into a smooth, polished specimen surface for a specified dwell time. The size of the indentation after removing the indenter, measured in micrometers, is determined using a microscope. The Rockwell test determines hardness using a conical diamond indenter. The specimen is preloaded with the indenter, then increased with an additional force, then unloaded back to the initial preload force. The indentation difference, measured in millimeters, is determined using a Rockwell hardness machine, automating the procedure with little operator influence.
The Contract requirement for the ASTM A354 BD Hardness test is either the Brinell or the Rockwell C hardness tests. The Knoop hardness test is not a Contract requirement.

A. Knoop Microhardness

Knoop microhardness testing was performed on the previously prepared microstructural cross-sections. The locations for microhardness testing include: (i) directly below (along) fracture surfaces, (ii) the contour of the first thread nearest the fracture surface, and (iii) inward from the thread root nearest the fracture surface up to a depth of ¾ in. The results provided in Appendix A generally indicate there are considerable variations in microscale hardness, ranging from 297 KHN to 446 KHN (equivalent to 28.0 to 43.6 HRC). This observation can be attributed to the non-homogeneous microstructure reported in Section III. (Note that KHN and HRC are the conventional abbreviations for Knoop Hardness Number and Hardness Rockwell C, respectively.)

The general trend of the microhardness results indicates repeated microstructural regions with local hardness exceeding the maximum bulk hardness of 39 HRC that is specified in ASTM A354 for grade BD (39 HRC is equivalent to 390 KHN).

Although these microhardness results are an indication of the metallurgical condition of the steel, microhardness testing is not appropriate or required for determination of conformance to ASTM A354 grade BD specified bulk hardness. This can only be done by using a macro indenter such as Rockwell C (HRC).

B. Rockwell C Hardness

Rockwell C hardness measurements were made across the diameter and at mid-radius locations of both rods. The Rockwell hardness tests were performed by Anamet Inc. and their test reports are provided in Appendix B (see also plotted results in Figure A10 of Appendix A). The results of the Rockwell hardness test show variation in hardness, with the outer diameter approaching HRC 39. The center hardness drops to as low as HRC 25 indicating the material was not uniformly through-hardened. Completely uniform through-hardening is difficult to achieve in large diameter rods such as this case, however, the large disparity in hardness from center to edge indicates that the steel may not have had optimal through thickness hardenability or was improperly heat treated.

It should be noted that ASTM A354 refers to ASTM F606 [2] which specifies that *"…for purposes of arbitration between the purchaser and seller over reported test results, hardness tests shall be conducted at mid-radius (r/2) of a transverse section taken through the threads…"* The mid-radius Rockwell C hardness values were determined by Anamet and ranged between 32.5 and 36.2 HRC. The mid-radius results are in compliance with the A354 grade BD requirements of HRC 31-39.

V. Tensile Test

Tensile testing was performed on machined test specimens taken from near the outer diameter of each anchor rod. Two samples were tested from Rod S2-A6. Piece #12 was from the bottom theaded end of the rod near where the fracture occurred. Piece #2 was from the shank near the top of the rod. The tensile tests were performed by Anamet Inc. Anamet's test reports are provided in Appendix A and the results summarized in Table 1 below.

Table 1
Tensile Test Results

Identification	S2-A6 #12	S2-A6 #2	S1-G1 #11	ASTM A354 Gr BD Requirement
Yield Strength (psi)	149,000	146,000	136,000	115,000 min.
Tensile Strength (psi)	170,000	168,000	159,000	140,000 min.
Elongation in 2" Gage (%)	15.5	14	15	14 min.
Reduction of Area (%)	46.0	48.0	48.4	40 min.

The results indicate the material meets yield strength, tensile strength and elongation requirements for A354 grade BD, although elongation (i.e., ductility) was slightly above the minimum limit.

VI. Charpy V-Notch Impact Test

Notched bar impact tests were performed at room temperature (70° F) and at (40° F) on machined 10x10 mm Charpy test specimens taken from near the outer diameter of each anchor rod. The samples were taken longitudinal to the rod axis with the notched surface facing toward the outer diameter. The tests were performed by Anamet Inc. Anamet's test reports are provided in Appendix A and the results summarized in Table 2 below.

Table 2
Charpy V-Notch Impact Energy Test Results (ft-lb)

Identification	S2-A6 #12	S2-A6 #2	S1-G1 #11
Test Temperature	70ºF	70ºF	40ºF
Sample 1	18	15	13.5
Sample 2	18	14	13
Sample 3	17	15	14
Average	17.7	14.7	13.5

Charpy v-notch impact test results do not pertain to conformance of the rods to the product specification because ASTM A354 does not have any requirements for impact testing. However impact testing characterizes the toughness of the steel, which was called into question especially given the observation of cleavage morphology in the fast fracture region of the fracture surfaces (see Section II). When compared to requirements in other fastener material specifications such as ASTM A320 [3] and ISO 898-1 [4], where the minimum absorbed energy requirements begin at 20 ft-lb (at low test temperatures e.g., -4° F), the results are relatively low. Stated otherwise, this material appears to lack toughness even when tested at room temperature. A more definitive statement on the extent of lack of toughness requires further investigation.

VII. Chemical Analysis

A chemical analysis was performed on samples of material from each anchor rod by Anamet Inc. Anamet's test reports are provided in Appendix A and results summarized in the Table 3 below. The chemistry is consistent with AISI 4140 steel and meets the ASTM A354 grade BD requirements.

Table 3
Spectrochemical Analysis
(Reported as Wt. %)

		S2-A6 #12	S1-G1 #11	Mill Test Report(1)	Mill Test Report(2)	Requirement ASTM A354 Gr BD
Aluminum	Al	<0.005		0.001	0.001	
Carbon	C	0.40	0.43	0.41	0.41	0.33 -0.55
Chromium	Cr	0.97	0.98	0.98	0.98	
Cobalt	Co	0.01	0.01	0.007	0.007	
Copper	Cu	0.22	0.22	0.20	0.20	
Iron	Fe	Balance	Balance			
Manganese	Mn	0.93	0.93	0.92	0.92	0.57 min.
Molybdenum	Mo	0.16	0.15	0.16	0.16	
Nickel	Ni	0.10	0.10	0.10	0.10	
Phosphorus	P	0.012	0.012	0.014	0.014	0.040 max.
Silicon	Si	0.24	0.23	0.23	0.23	
Sulfur	S	0.034	0.039	0.034	0.034	0.045 max.
Titanium	Ti	<0.005	<0.005	0.002	0.002	
Tungsten	W	<0.005	<0.005			
Vanadium	V	0.03	0.03	0.030	0.030	
Zirconium	Zr	<0.005	<0.005			

Note 1) Taken from Gerdau Macsteel certified mill test report for heat no. M058938 reported to Dyson Corp. – Code MIS (Shipped 5/27/08)

Note 2) Taken from Gerdau Macsteel certified mill test report for heat no. M058925 reported to Dyson Corp. – Code MJF (Shipped 5/27/08)

DISCUSSION

The delayed nature of the failures, evidence of progressive intergranular cracking, marginally high surface hardness and apparent lack of toughness are consistent with hydrogen embrittlement (HE) as the cause of the rod failures. The definition of hydrogen embrittlement is as follows:

Hydrogen Embrittlement (HE) — a permanent loss of ductility in a metal or alloy caused by hydrogen in combination with stress, either externally applied or internal residual stress. Source: ASTM F 2078

Generally, hydrogen embrittlement is classified under two broad categories based on the source of hydrogen: internal hydrogen embrittlement (IHE) and environmental hydrogen embrittlement (EHE). IHE is caused by residual hydrogen from steelmaking or from processing steps such as pickling and electroplating. EHE is caused by hydrogen introduced into the metal from external sources while it is under stress, such as is the case with an in-service fastener. The term Stress

Corrosion Cracking (SCC) is a form of EHE that occurs when hydrogen is produced as a by-product of surface corrosion and is absorbed into the lattice. Cathodic hydrogen absorption (CHA) is a subset of SCC and can be explained as follows. Metallic coatings such as zinc are designed to sacrificially corrode to protect say a steel bolt from rusting. If the steel becomes exposed, a reduction process on the exposed steel surface simultaneously results in the evolution of hydrogen.

Three ingredients must be present to cause hydrogen embrittlement failure: (i) steel that is *susceptible* to hydrogen damage, (ii) *stress* (typically as an applied load), and (iii) *hydrogen*. All three of these elements are present in sufficient quantities, and given time, hydrogen damage results in crack initiation and growth until the occurrence of delayed fracture. Time to failure can vary, depending on the severity of the conditions and the source of hydrogen.

(i) ***Susceptibility*** – Material strength has a first order effect on HE susceptibility. As strength increases, steels become less ductile and less tough. By the same token, at equal strength, steel that exhibits lower toughness is inherently more brittle and more susceptible to hydrogen assisted cracking.

The susceptibility of steel fasteners increases significantly when the specified hardness is above 39 HRC. The rod hardness test results indicate the material hardness varies considerably. Bulk hardness readings near the outer diameter surface were high relative to the center of the rods, and Knoop microhardness readings varied up to KHN 446 (HRC 43.6), which significantly increases susceptibility to local hydrogen assisted cracking. However, hardness alone was not high enough to explain the occurrence of HE failure. The high degree of variability in the microhardness measurements and the observed variability in microstructure indicate the material is inhomogeneous. More significantly, the material has relatively low toughness, as measured by Charpy v-notch impact tests. The brittle cleavage features observed during the SEM examination of the "fast fracture" region are further evidence of a material with poor toughness. The tensile tests show that the elongation (i.e. ductility) is within the Contract requirements. However, the reported values show that the measured elongations approach the minimum requirement of ASTM A354BD. Additionally, the microstructure showed evidence of significant amounts of inclusions, which further increases the susceptibility of the steel. These observations together amount to a material that is susceptible to HE.

(ii) ***Stress*** **–** load induced stress is a normal service condition for mechanical fasteners. In this application, the fasteners were initially subjected to 0.75 Fu (627 kips) with a final target load for 0.68 Fu. Fasteners are capable by design to be tightened into yield or to the limit of their elastic range. Therefore, this application amounts to a normal but high loading condition by fastener standards. If all other conditions for HE are met, the greater the load on the fastener, the greater the chance that its HE threshold stress will be exceeded.

(iii) ***Hydrogen*** **–** there are two possible sources of hydrogen: "internal" and "environmental." In this case, although hydrogen may have been available from both sources, the relatively short amount of time between loading and failure (i.e., days) indicates that the hydrogen was already available and mobile in the steel.

a. The principal source of internal hydrogen was likely the freeing of trapped residual hydrogen by the upquenching effect of hot dip galvanizing. A research publication by Brahimi et al. [5] describes this phenomenon which can be summarized as follows. “The source of hydrogen is residual hydrogen trapped in the steel specimens, in reversible trap sites with high bonding energies. In this scenario, hydrogen is released by the up-quench/thermal shock upon immersion in the molten zinc bath. The presence of a thick zinc coating prevents hydrogen escaping, instead causing it to accumulate at grain boundaries. Lower hardness steel specimens, in the range of 25- 38 HRC are not embrittled by the galvanizing process, as evidenced by the fact that most high strength structural fasteners can be safely galvanized.”

b. Although there was no significant visible corrosion on the broken rods (white corrosion or red rust), some of the rods may have been exposed to water and the elements, especially at the bottom, in the period after 2008 when they were installed in the pier until when they were tensioned in March, 2013. More precisely, galvanic corrosion of the sacrificial zinc coating generates hydrogen, which is then absorbed by the cathode (i.e., steel). The quantity of hydrogen absorbed in this manner is exponentially higher than under normal anodic corrosion conditions (i.e., without a coating). If corrosion generated hydrogen contributed to the failures, it was already present and available (i.e., mobile) in the steel.

CONCLUSIONS AND RECOMMEDATIONS

1. The anchor rods failed as a result of hydrogen embrittlement, resulting from the applied tensile load and from hydrogen that was already present and available in the rod material as they were tensioned. The root cause of the failures is attributed to higher than normal susceptibility of the steel to hydrogen embrittlement.

2. The steel rods comply with the basic mechanical and chemical requirements of ASTM A354 grade BD.

3. The metallurgical condition of the steel was found to be less than ideal. More precisely, the microstructure of the steel is inhomogeneous resulting in large difference in hardness from center to edge, and high local hardness near the surface. As an additional consequence of the metallurgical condition, the material exhibits low toughness and marginal ductility. The combination of all of these factors have caused the anchor rods to be susceptible to HE failure.

4. Procurement of future A354 grade BD anchor rods should include a number of standard supplemental requirements to assure against HE failure. The appropriate specification of supplemental requirements is currently under review.

REFERENCES

1.ASTM A354, *Standard Specification for Quenched and Tempered Alloy Steel Bolts, Studs, and other Externally Threaded Fasteners.*

2. ASTM F606 *Standard Test Methods for Determining the Mechanical Properties of Externally and Internally Threaded Fasteners, Washers, and Rivets*.

3. ASTM A320/A320M, *Standard Specification for Alloy-Steel and Stainless Steel Bolting for Low-Temperature Service.*

4. ISO 898-1, *Mechanical properties of fasteners made of carbon steel and alloy steel Part 1:Bolts, screws and studs with specified property classes — Coarse thread and fine pitch thread*.

5. Brahimi, S., et al., *Effect of surface processing variables on hydrogen embrittlement of steel fasteners part 1: Hot dip galvanizing.* Canadian Metallurgical Quarterly, 2009. **48**(3): p. 293-302.

Photos1-28

Photo 1) Anchor rod installation at shear key with broken rod at arrow.

Photo 2) Broken anchor rod S1-G1.

Photo 3) Fracture surface of Rod S1-G1 after cleaning.

Photo 4) The fracture surface showing progressive hydrogen assisted cracking and fast fracture areas.

Photo 5) Broken end of Rod S2-A6. Grease and grout were present on the fracture and threads.

Photo 6) Another view of Rod S2-A6 fracture surface.

Photo 7) Fracture surface of Rod S2-A6 after cleaning.

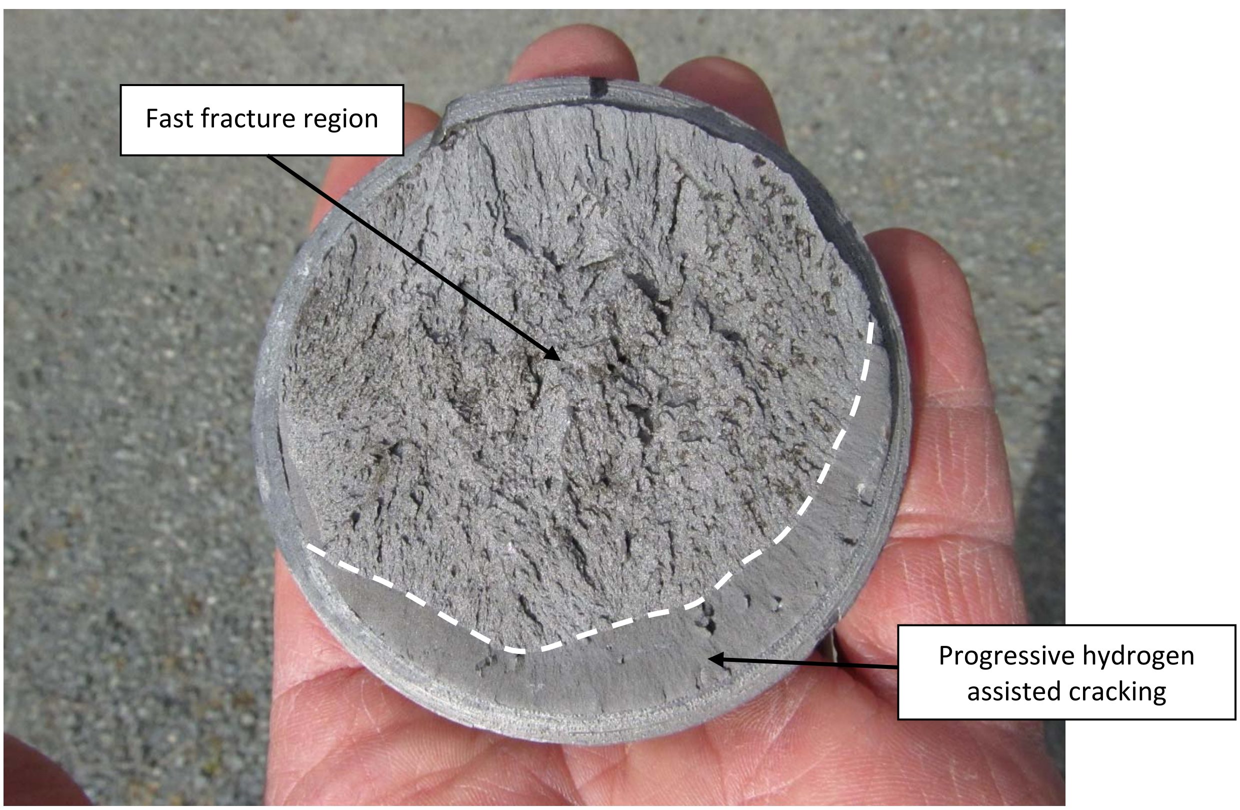


Photo 8) The fracture surface showing progressive hydrogen assisted cracking and fast fracture areas.

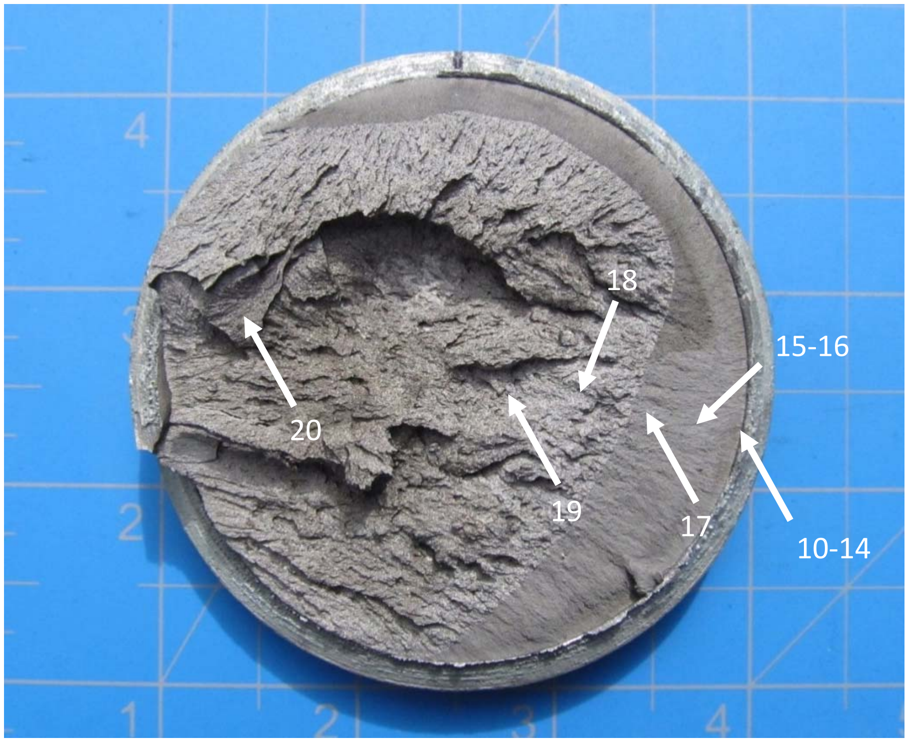


Photo 9) Fractured rod S1-G1 showing the locations of SEM photos 10-20.

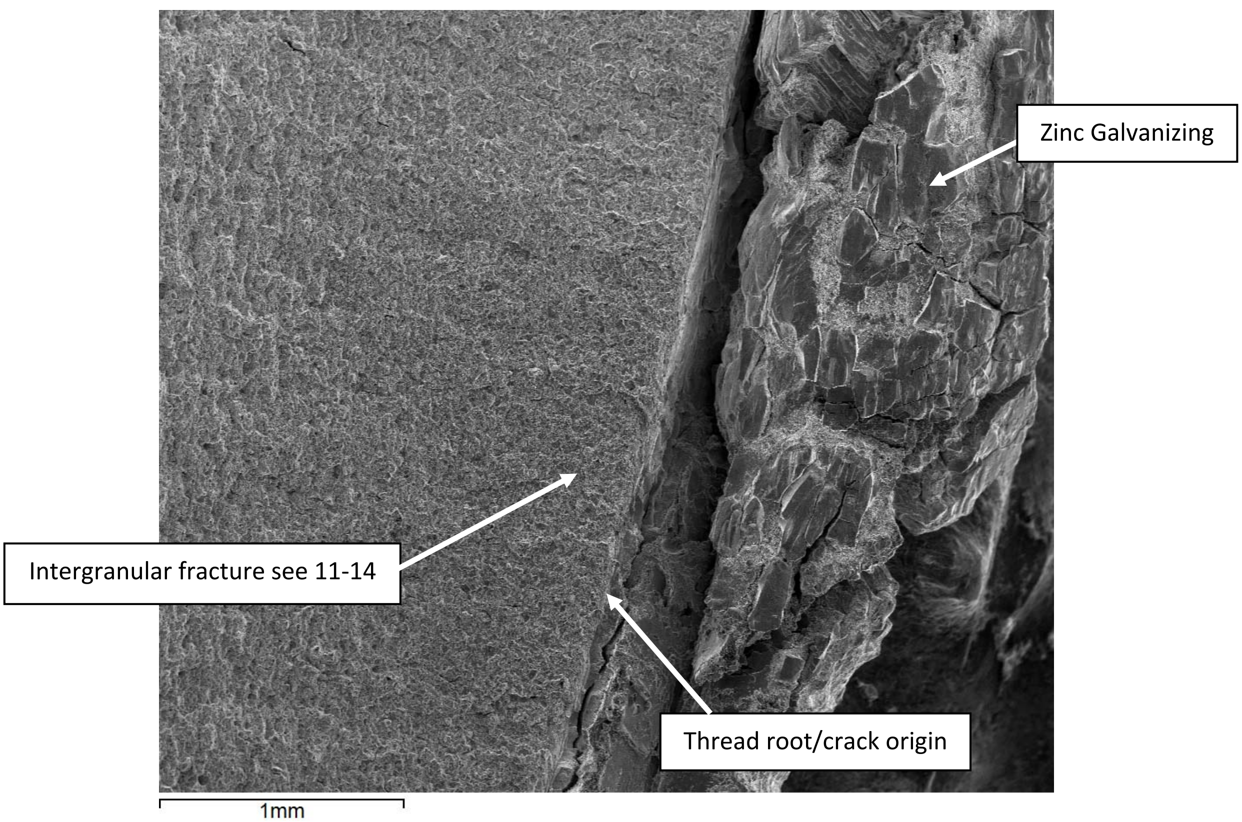


Photo 10) SEM image of S1-G1 at crack origin.

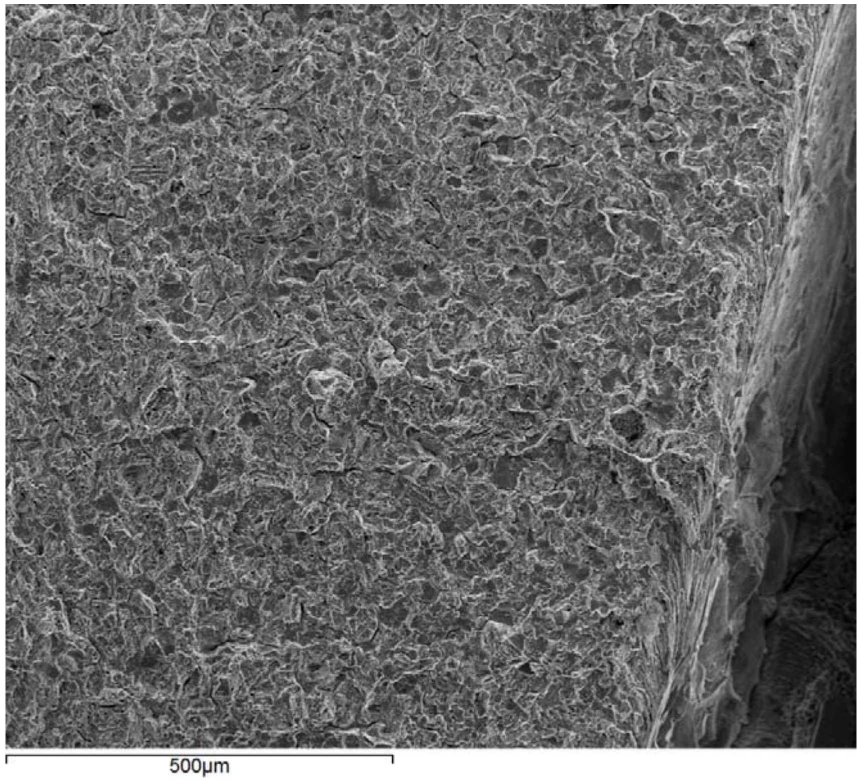


Photo 11) Same as Photo 10 except higher magnification.

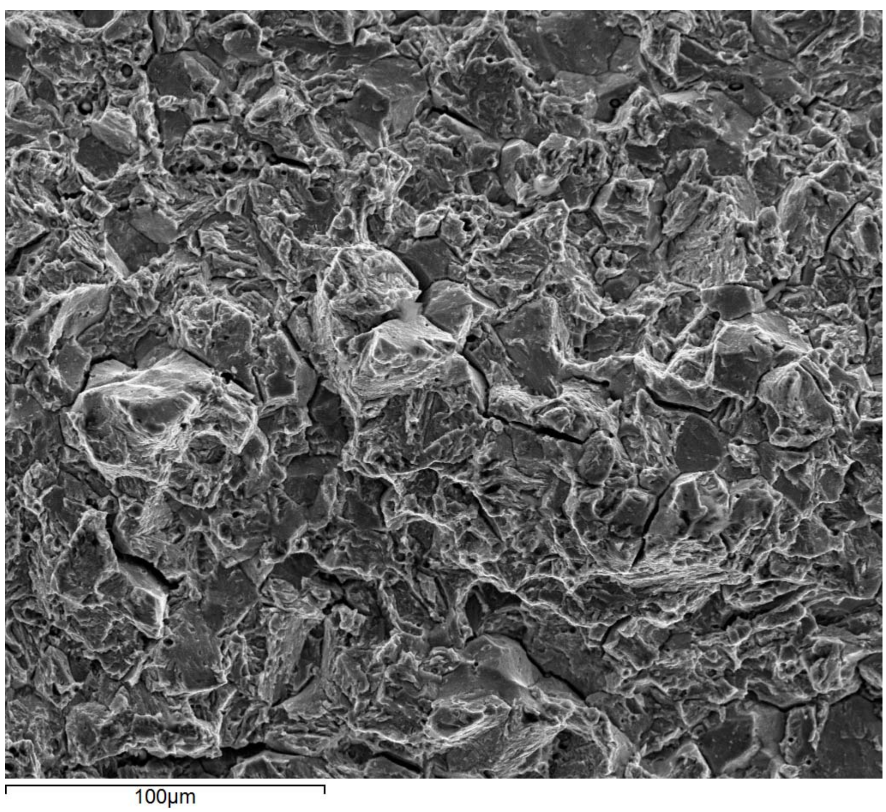


Photo 12) Same as Photo 11 except higher magnification.

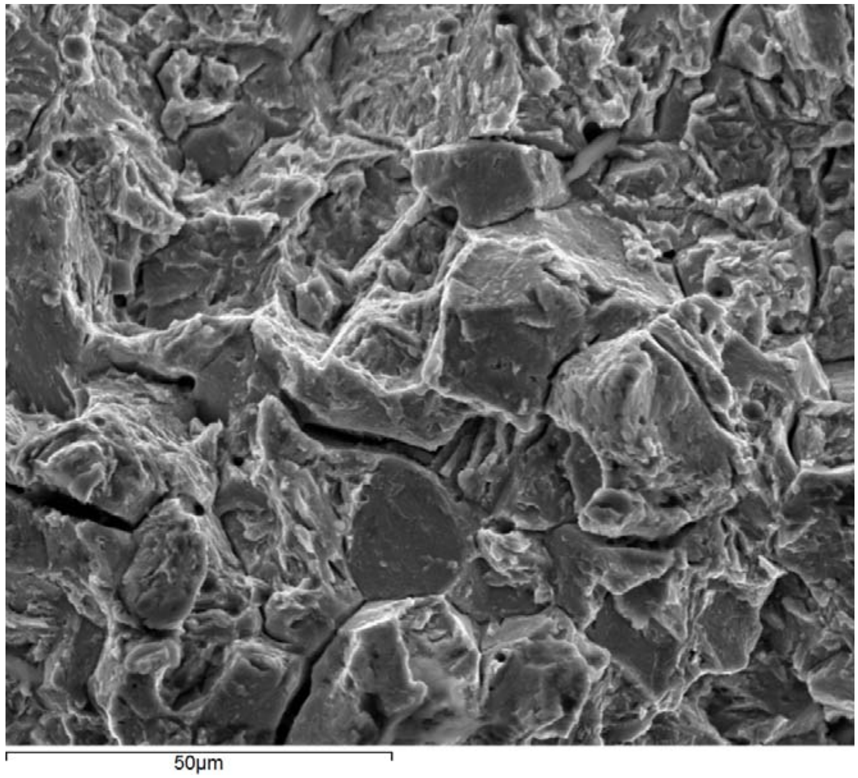


Photo 13) Same as Photo 12 except higher magnification showing intergranular fracture features.

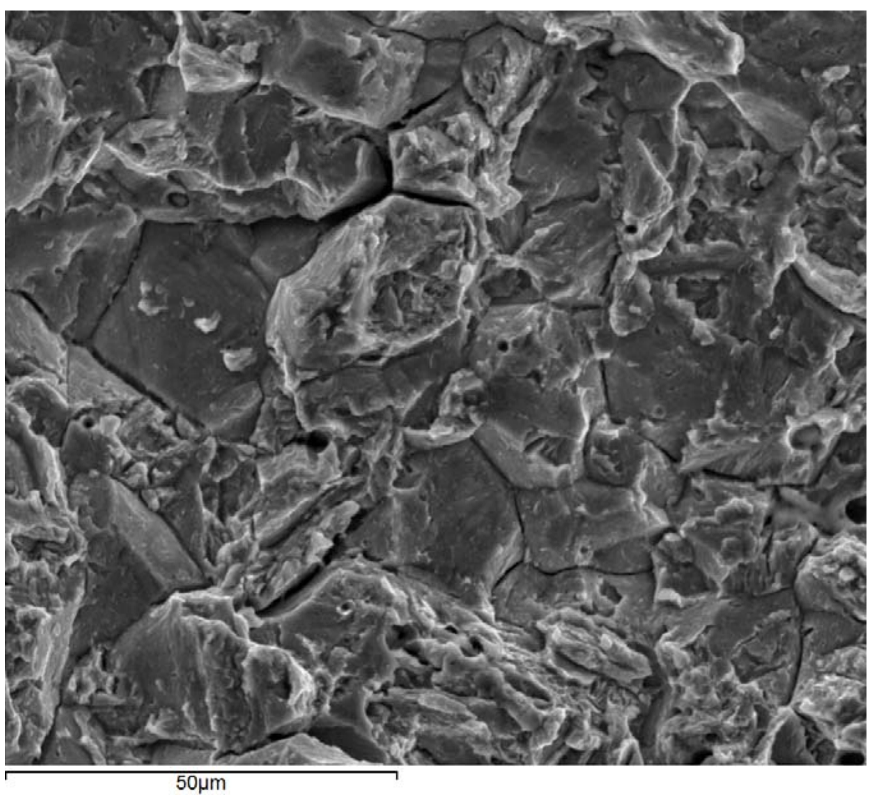


Photo 14) Same as Photo 11 except higher magnification showing more intergranular fracture features.

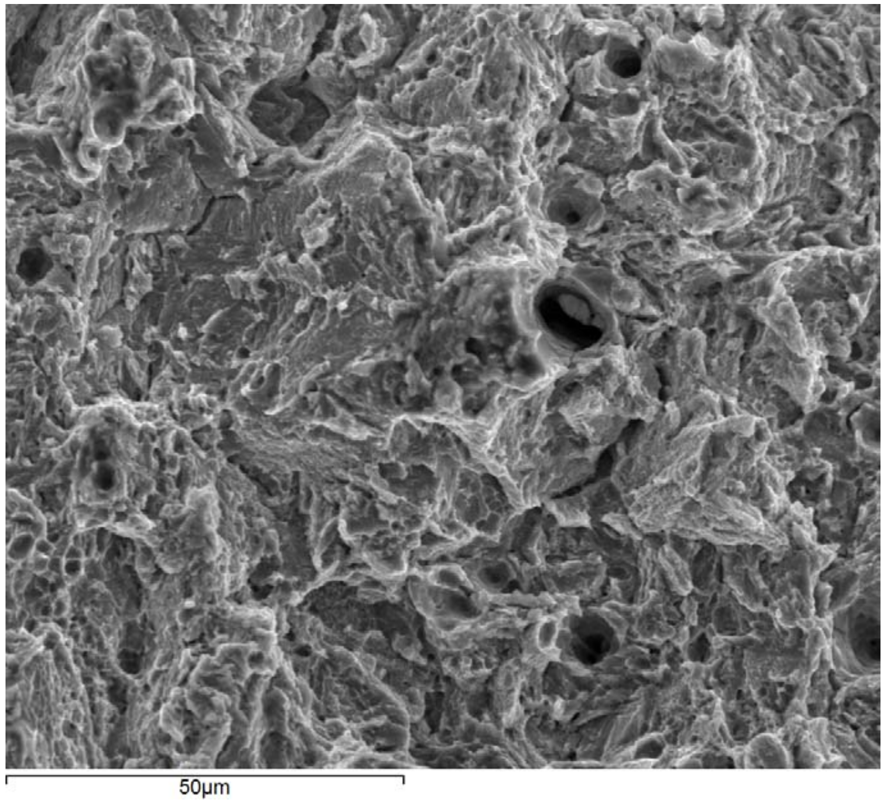


Photo 15) Mixed ductile tearing and intergranular fracture features.

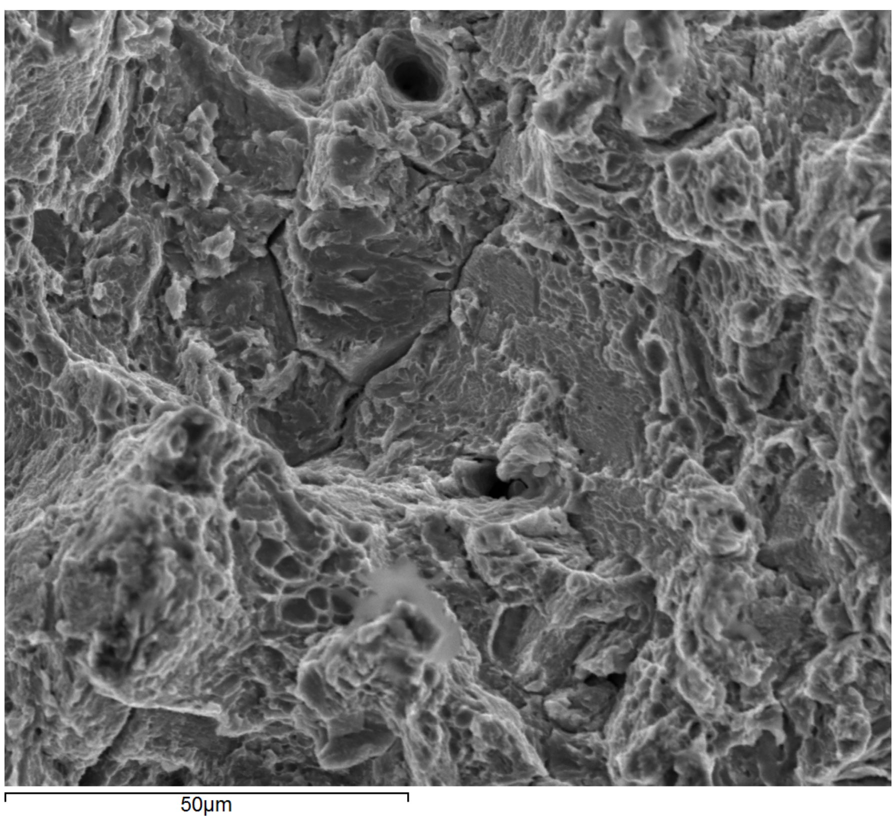


Photo 16) Mixed ductile tearing and intergranular fracture features.

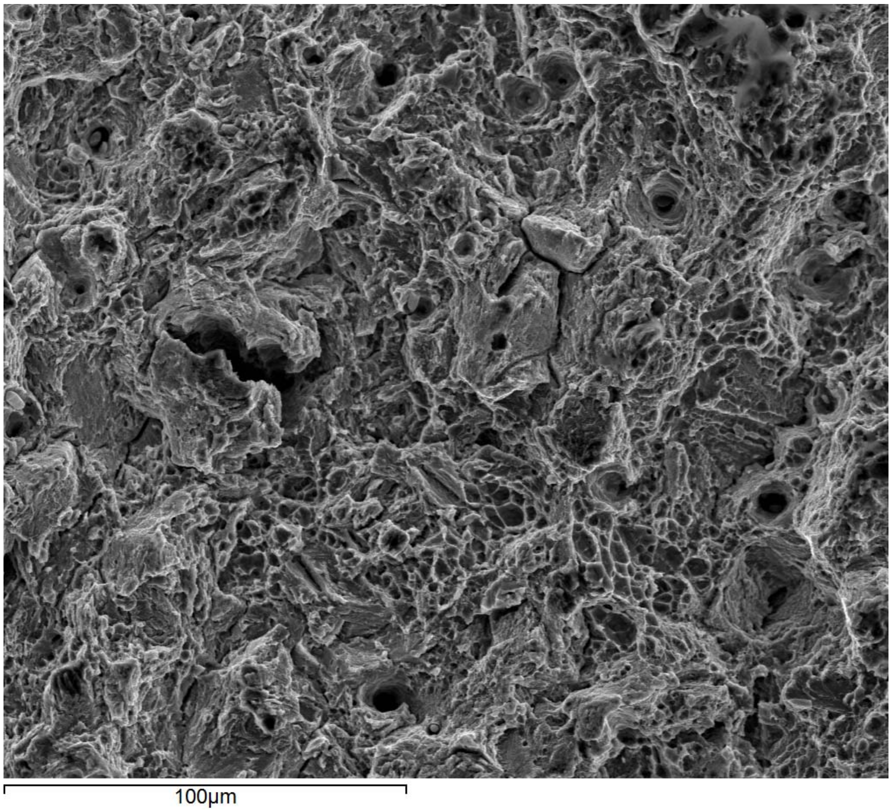


Photo 17) Mixed ductile tearing and intergranular fracture features.

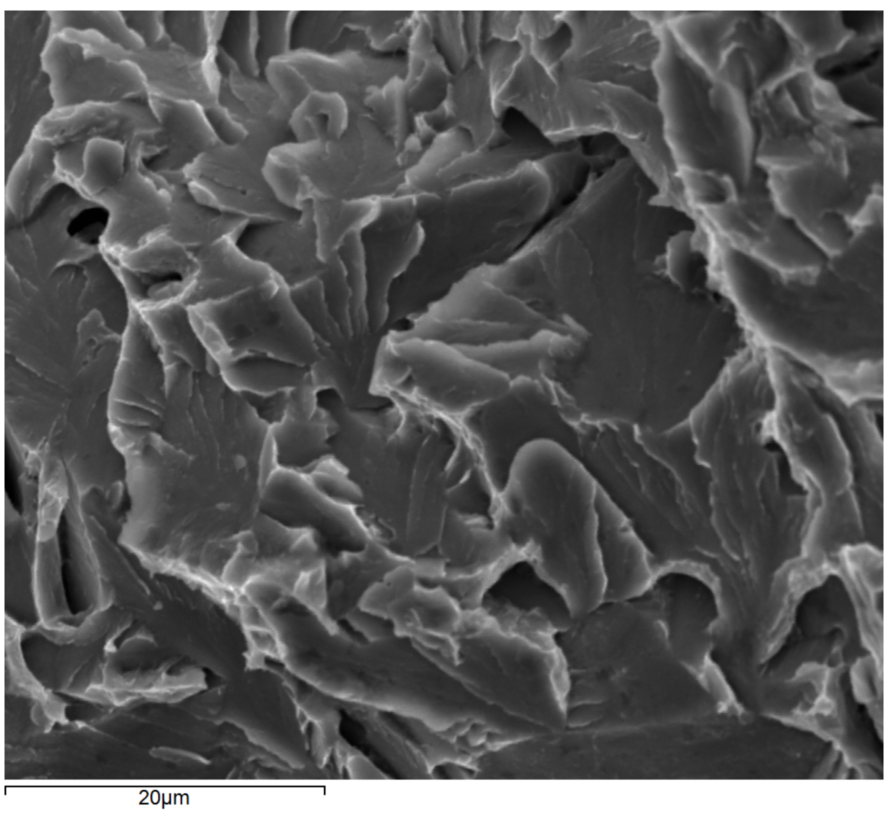


Photo 18) Cleavage fracture features – brittle fast fracture.

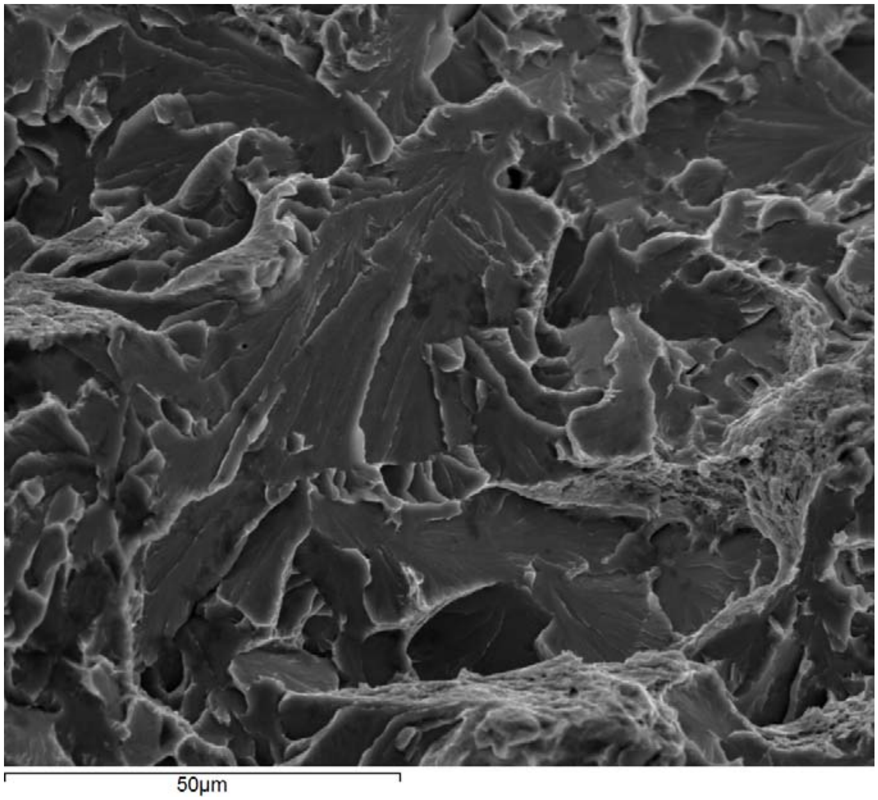


Photo 19) Cleavage fracture features – brittle fast fracture.

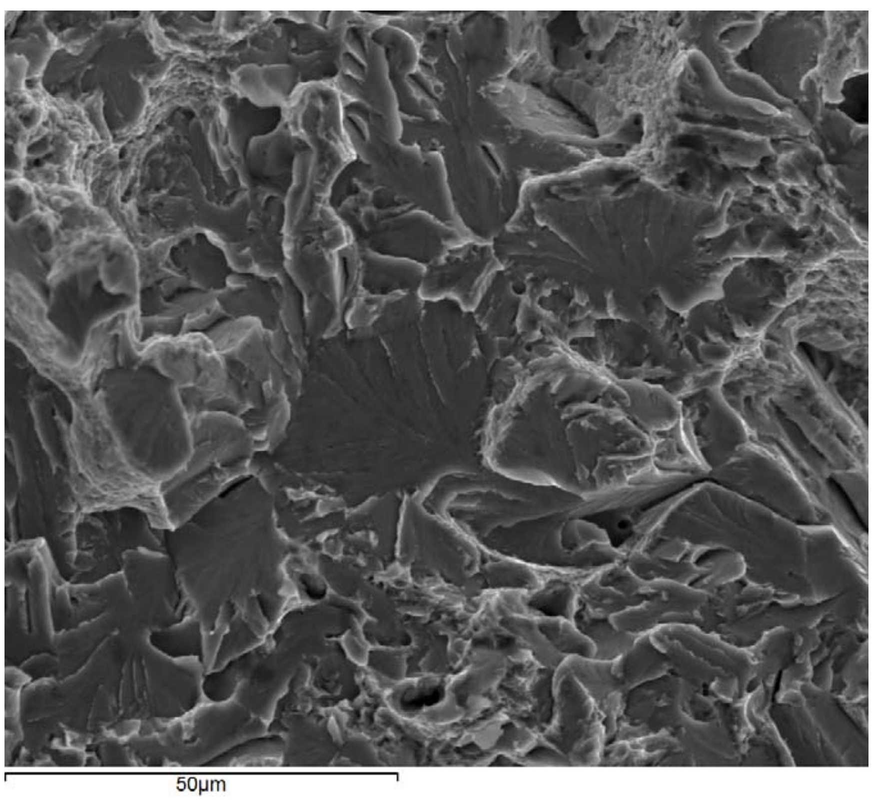


Photo 20) Cleavage fracture features – brittle fast fracture.

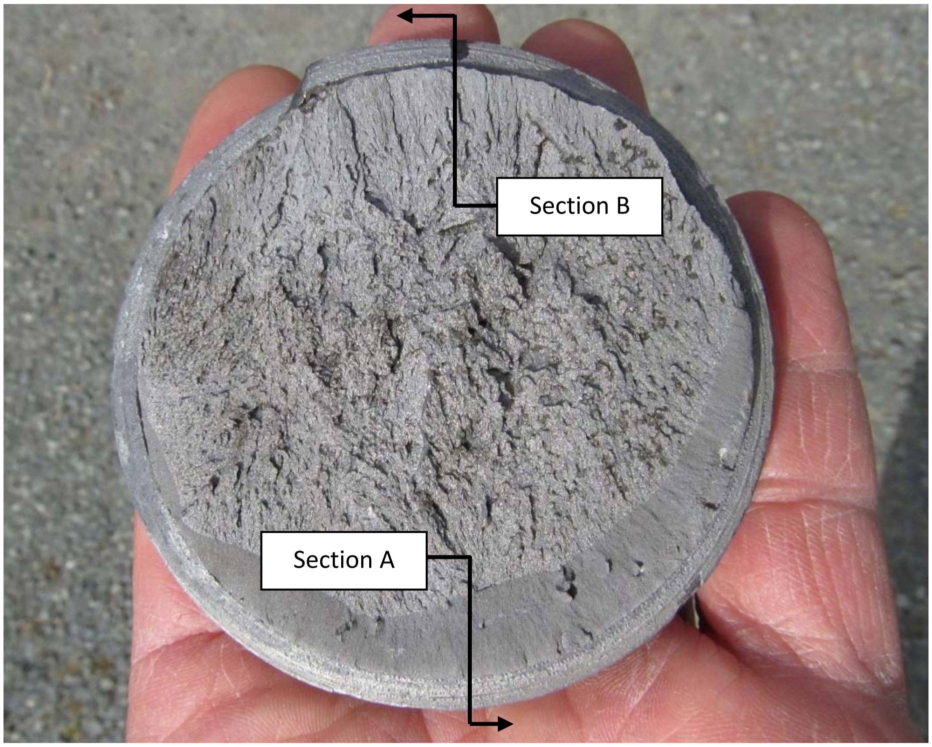


Photo 21) Fractured rod S2-A6 showing the location of cross-sections for microstructural examination and microhardness tests.

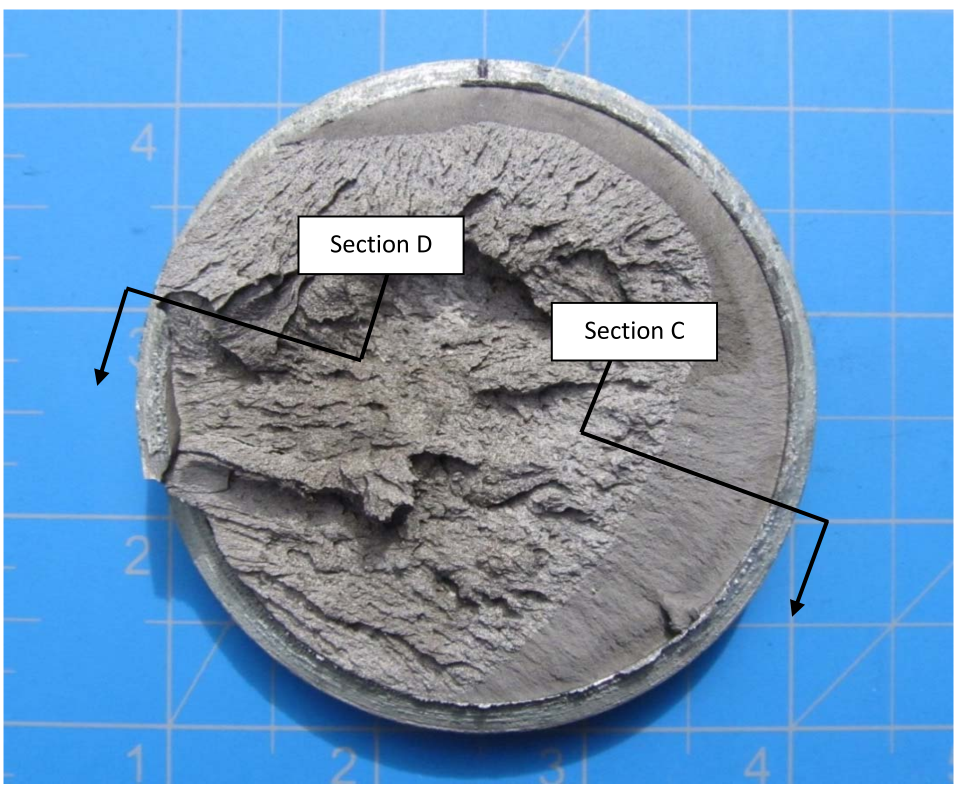


Photo 22) Fractured rod S1-G1 showing the location of cross-sections for microstructural examination and microhardness tests.

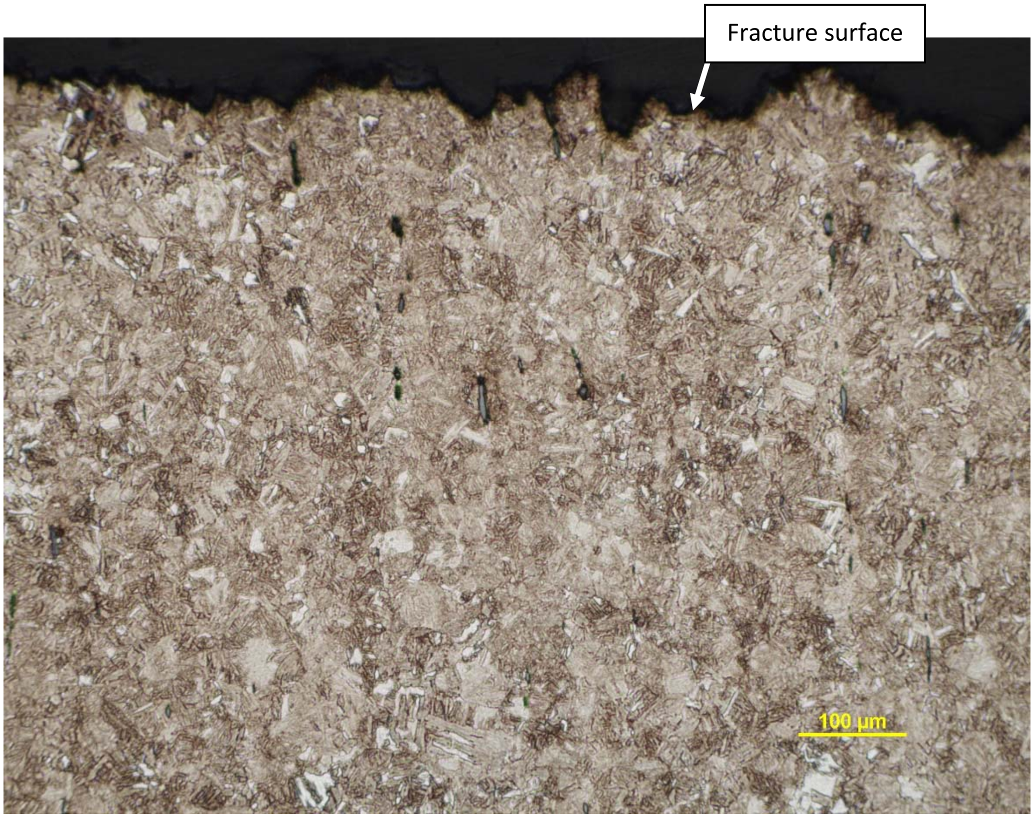


Photo 23) Example of microstructure observed in S2-A6 at Section A. (Etchant 2%Nital)

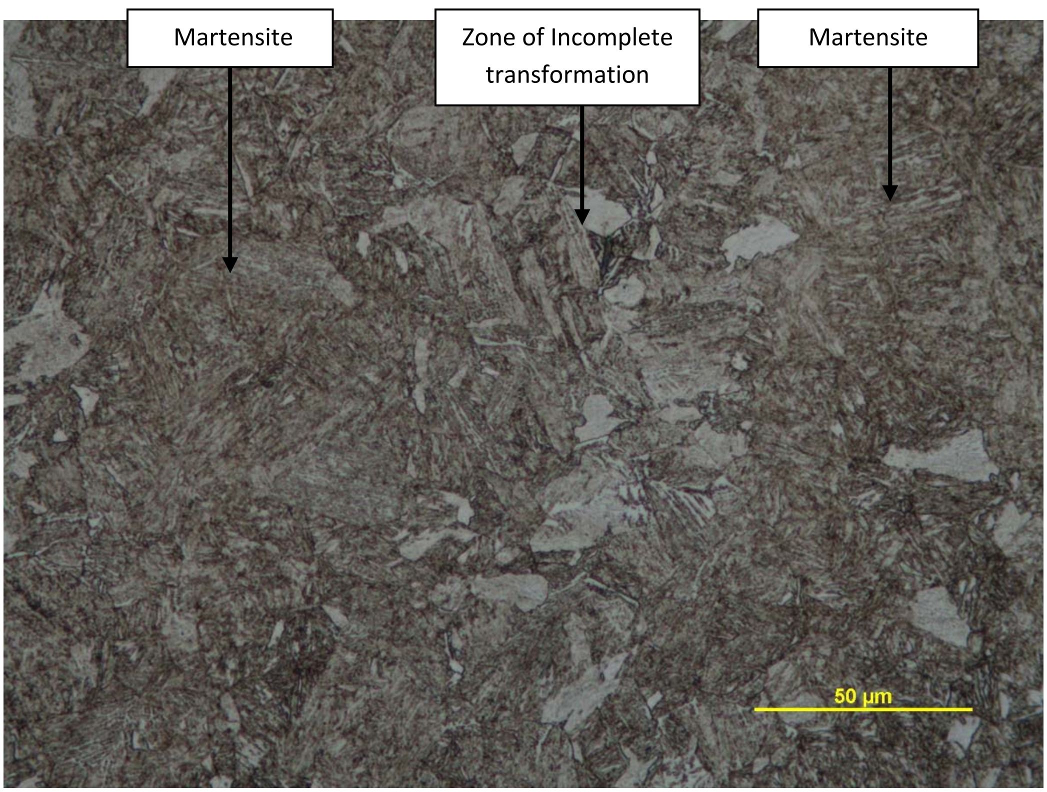


Photo 24) Same as above except higher magnification. Note the structure is not fully tempered martensite. The center region did not fully transformed to martensite.

Photo 25) Another example of microstructure observed in S2-A6. Note vertical banding (alternating light-dark streaks) in grain direction. (Etchant: 2% Nital)

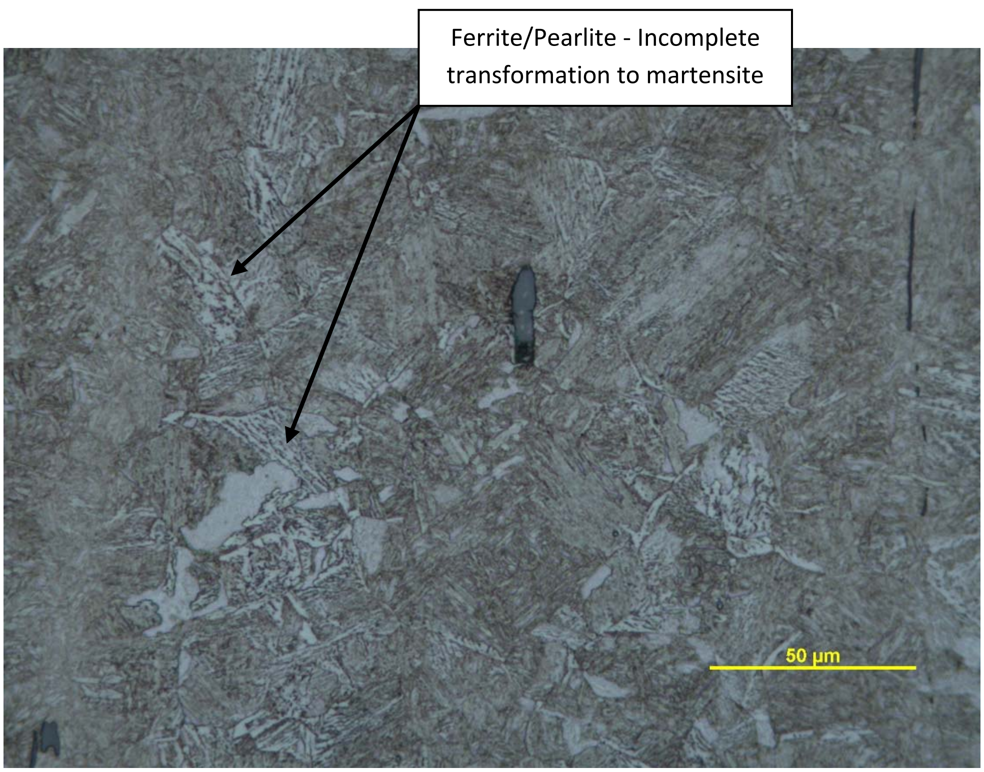


Photo 26) Same as above except higher magnification. Note ferrite/pearlite where the structure is not fully tempered martensite. (Etchant: 2% Nital)

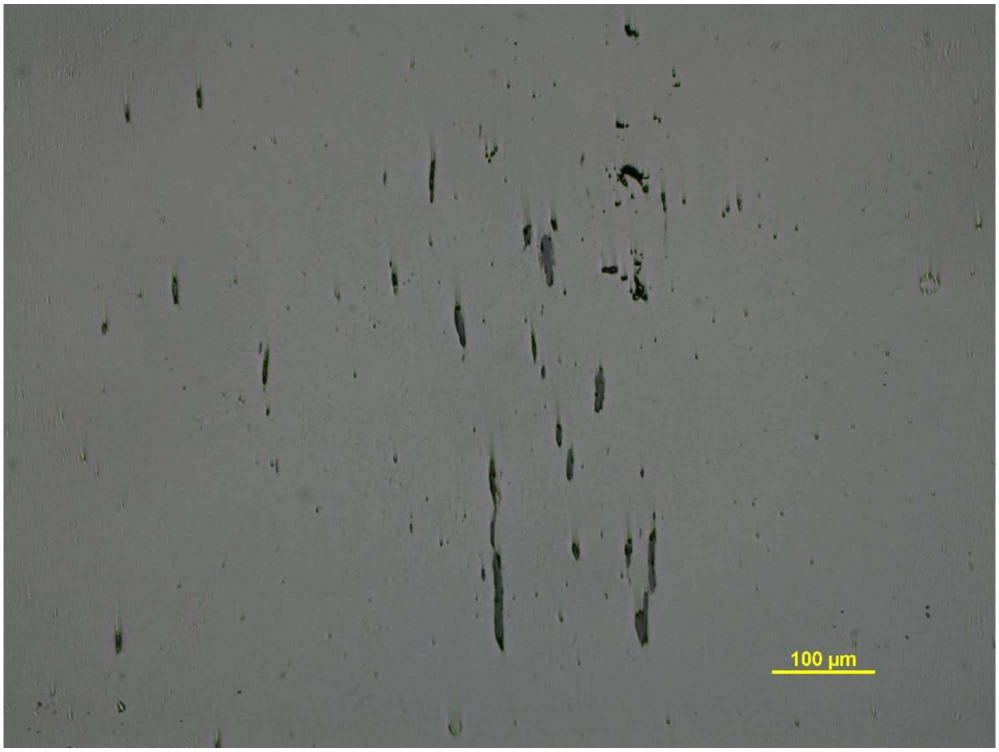


Photo 27) Example of stringer inclusions observed in microstructure. (Unetched)

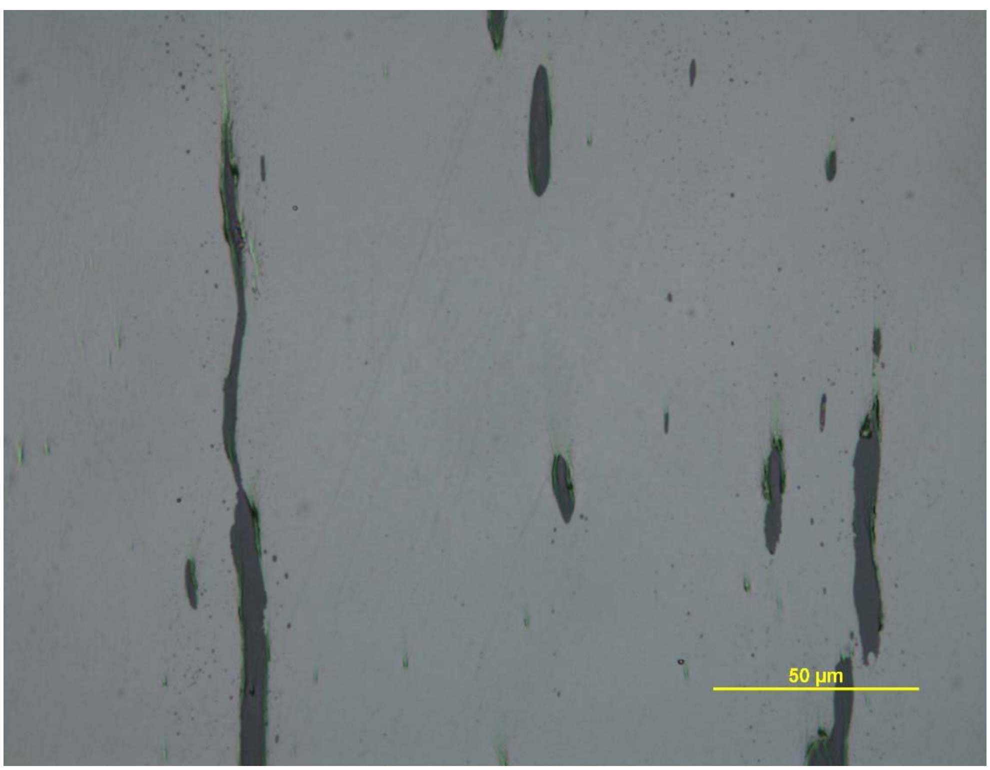
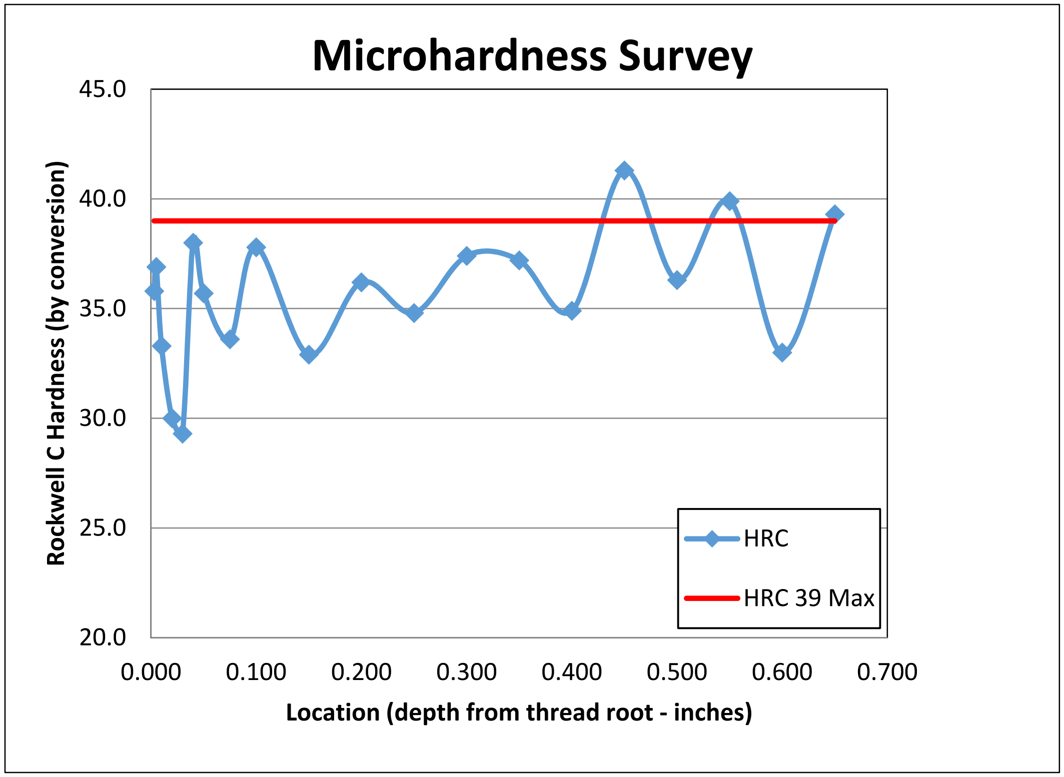
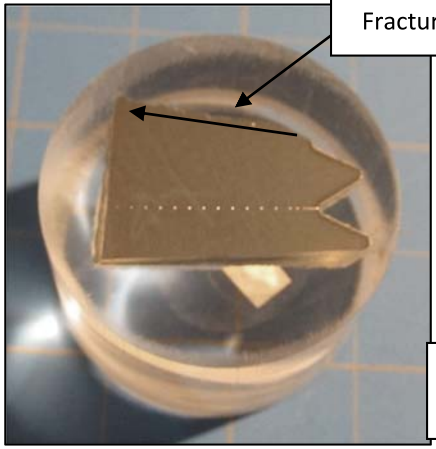


Photo 28) Same as Photo 28 except higher magnification of non-metallic stringer inclusions.

Appendix A
Hardness Test Results

Table 1 Knoop Microhardness Results		
Location Depth from surface (in.)	Knoop Hardness Number	HRC (by conversion table)
0.003	358	35.8
0.005	369	36.9
0.010	336	33.3
0.020	311	30.0
0.030	306	29.3
0.040	380	38.0
0.050	357	35.7
0.075	339	33.6
0.100	378	37.8
0.150	333	32.9
0.200	362	36.2
0.250	349	34.8
0.300	374	37.4
0.350	372	37.2
0.400	350	34.9
0.450	418	41.3
0.500	363	36.3
0.550	401	39.9
0.600	334	33.0
0.650	**394**	**39.3**



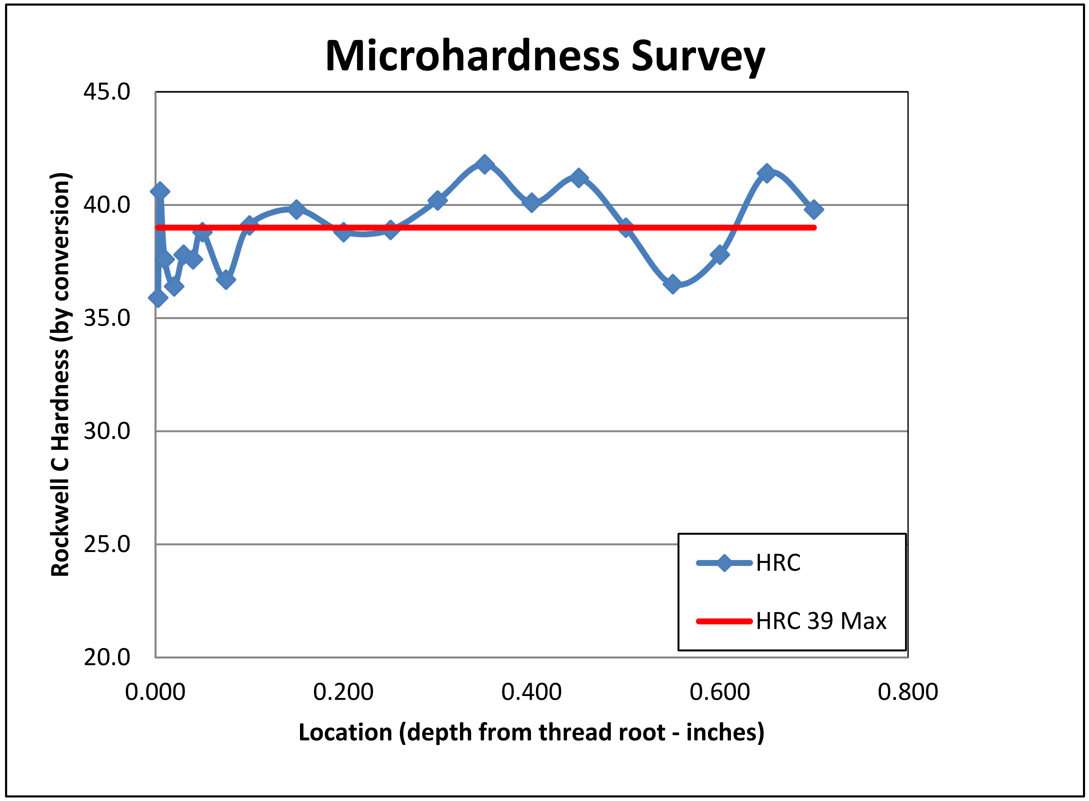


Fracture Surface

Section A - Rod S2-A6 #12
Knoop survey along fracture

Figure A1
(Rod S2-A6 Section A)

Table 2 Knoop Microhardness Results		
Location Depth from surface (in.)	Knoop Hardness Number	HRC (by conversion table)
0.003	359	35.9
0.005	409	40.6
0.010	376	37.6
0.020	364	36.4
0.030	378	37.8
0.040	376	37.6
0.050	387	38.8
0.075	367	36.7
0.100	392	39.1
0.150	399	39.8
0.200	387	38.8
0.250	390	38.9
0.300	405	40.2
0.350	423	41.8
0.400	403	40.1
0.450	417	41.2
0.500	391	39.0
0.550	365	36.5
0.600	378	37.8
0.650	419	41.4
0.700	400	39.8



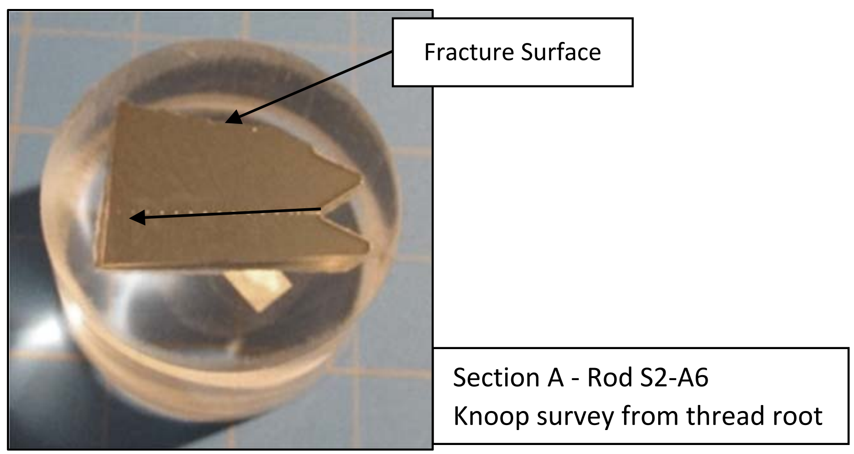
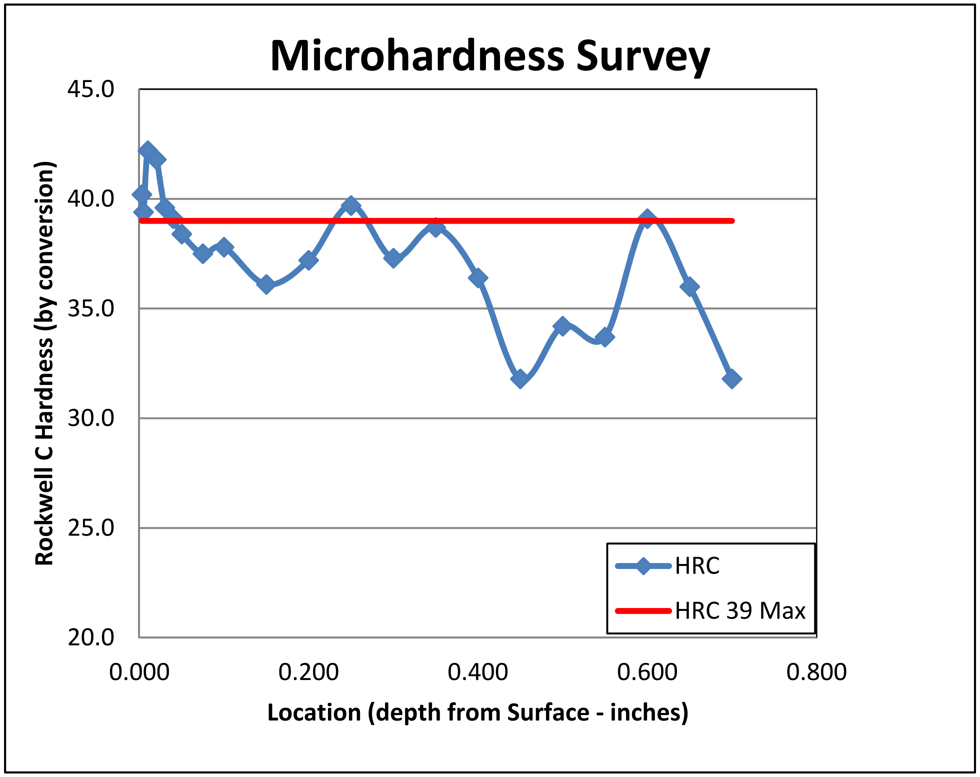
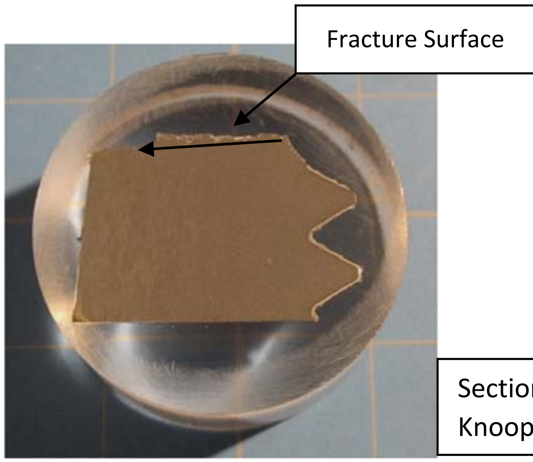


Figure A2
(Rod S2-A6 Section A)

Table 3 Knoop Microhardness Results		
Location Depth from surface (in.)	Knoop Hardness Number	HRC (by conversion table)
0.003	404	40.2
0.005	395	39.4
0.010	429	42.2
0.020	423	41.8
0.030	397	39.6
0.040	392	39.1
0.050	384	38.4
0.075	375	37.5
0.100	378	37.8
0.150	361	36.1
0.200	372	37.2
0.250	399	39.7
0.300	373	37.3
0.350	387	38.7
0.400	364	36.4
0.450	324	31.8
0.500	344	34.2
0.550	339	33.7
0.600	392	39.1
0.650	360	36.0
0.700	324	31.8

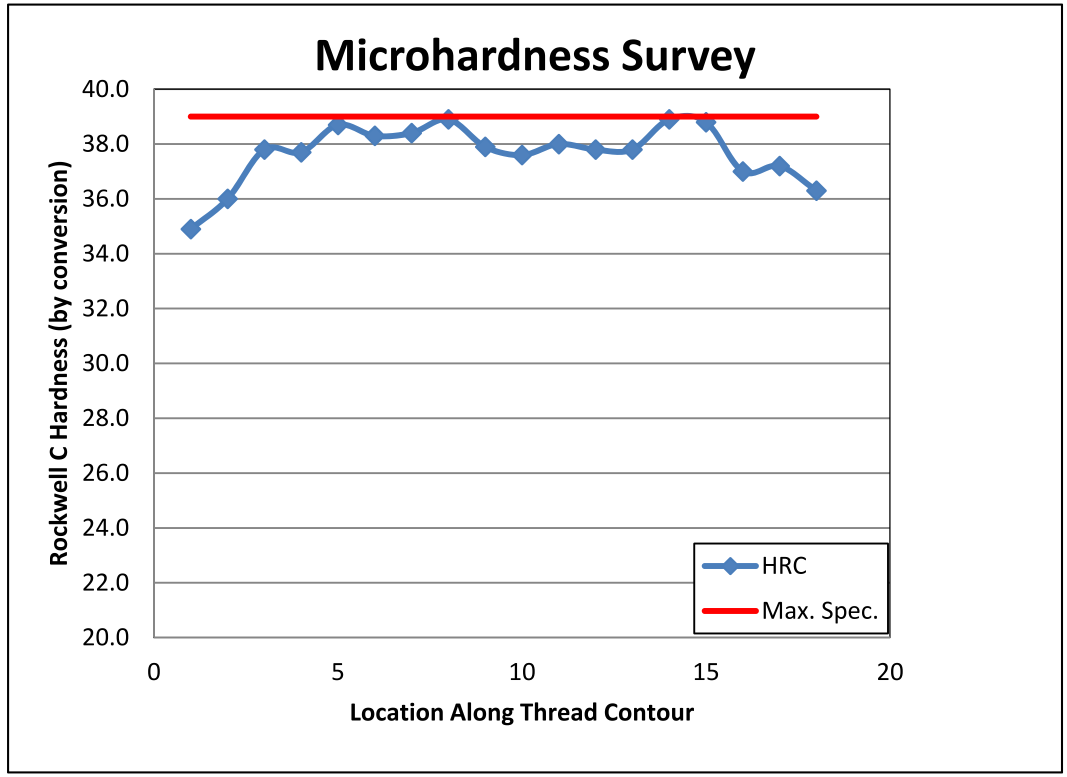




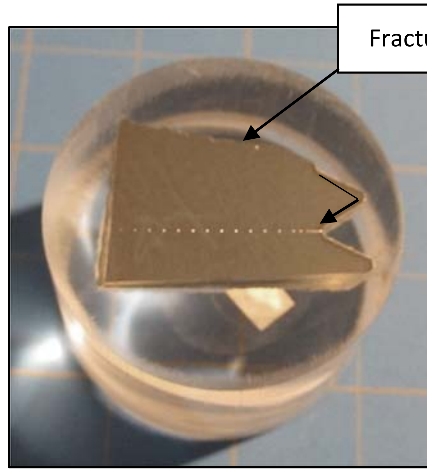
Section B - Rod S2-A6
Knoop survey along fracture

Figure A3
(Rod S2-A6 Section B)

Table 4 Knoop Microhardness Results		
Location Depth from surface (in.)	Knoop Hardness Number	HRC (by conversion table)
1	350	34.9
2	360	36.0
3	378	37.8
4	377	37.7
5	387	38.7
6	383	38.3
7	384	38.4
8	390	38.9
9	379	37.9
10	376	37.6
11	380	38.0
12	378	37.8
13	378	37.8
14	390	38.9
15	389	38.8
16	370	37.0
17	372	37.2
18	363	36.3



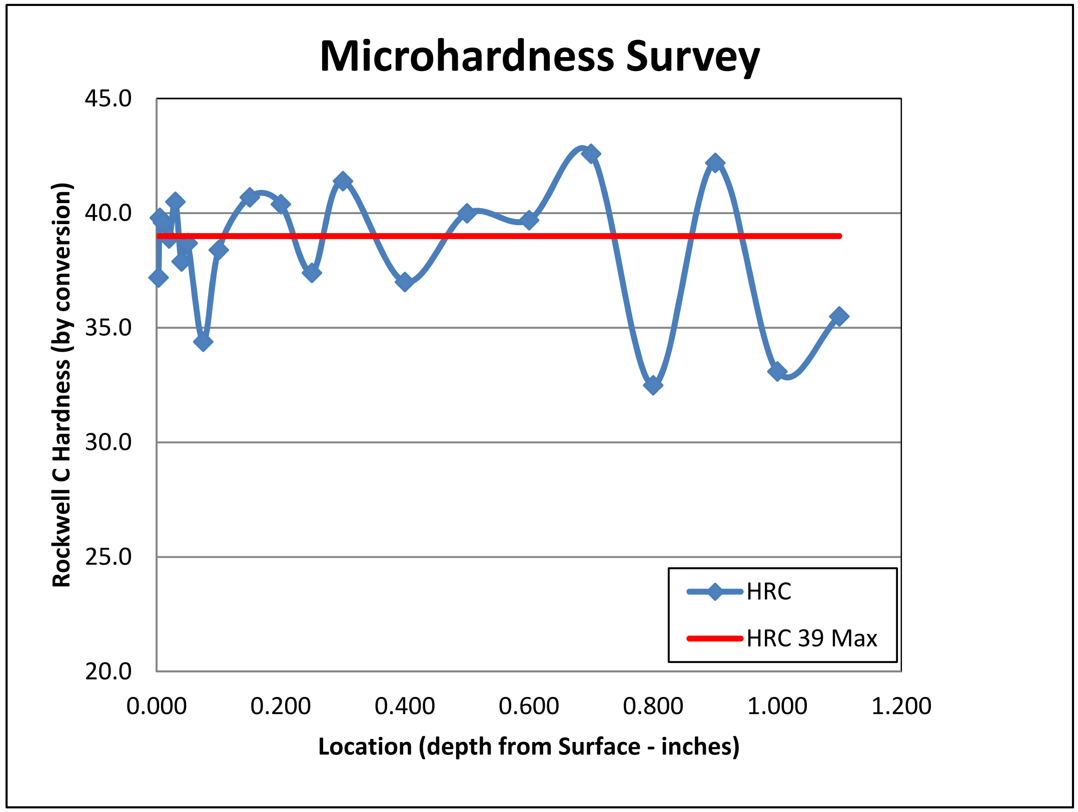
Fracture Surface



Section A - Rod S2-A6
Knoop survey along thread contour

Figure A4
(Rod S2-A6 Section A)

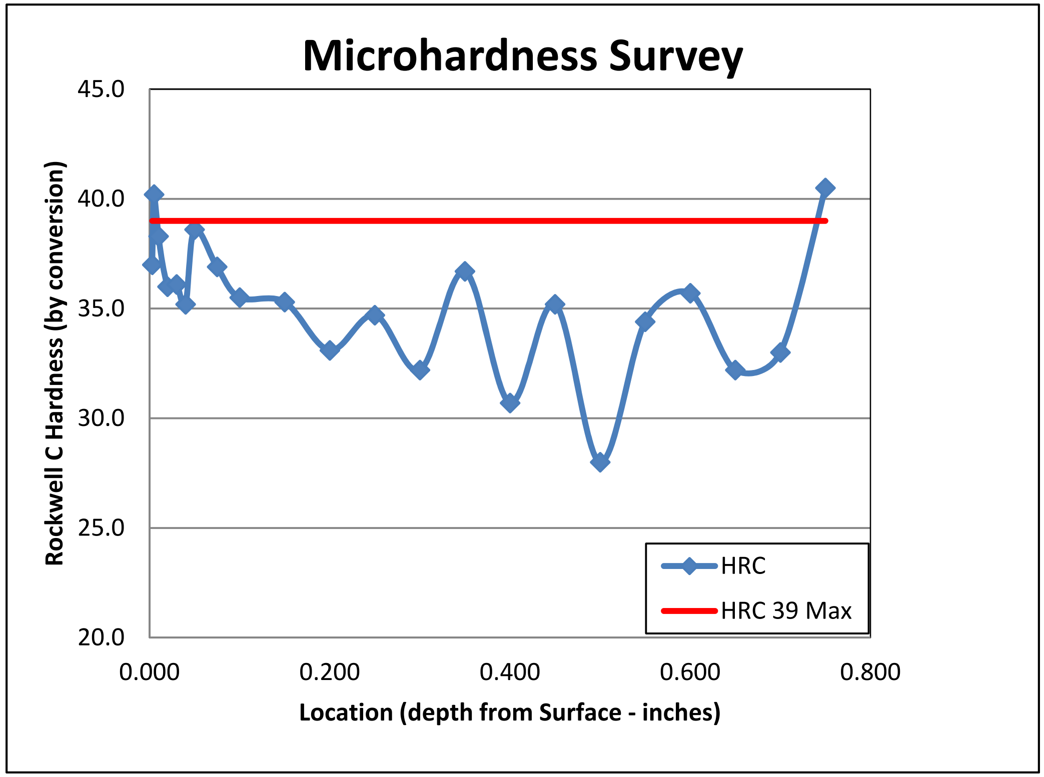
Table 5 Knoop Microhardness Results		
Location Depth from surface (in.)	Knoop Hardness Number	HRC (by conversion table)
0.003	372	37.2
0.005	400	39.8
0.010	397	39.6
0.020	390	38.9
0.030	408	40.5
0.040	379	37.9
0.050	389	38.7
0.075	346	34.4
0.100	384	38.4
0.150	410	40.7
0.200	407	40.4
0.250	374	37.4
0.300	419	41.4
0.400	370	37.0
0.500	402	40.0
0.600	398	39.7
0.700	433	42.6
0.800	330	32.5
0.900	428	42.2
1.000	335	33.1
1.100	355	35.5

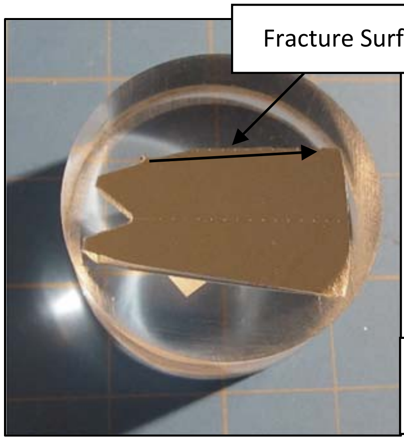


Rod S2-A6 #12
Readings taken radially inward from the thread root at a location approximately ¾ inch from the fracture.

Figure A5
(Rod S2-A6 #12)

Table 6 Knoop Microhardness Results		
Location Depth from surface (in.)	Knoop Hardness Number	HRC (by conversion table)
0.003	370	37.0
0.005	404	40.2
0.010	383	38.3
0.020	360	36.0
0.030	361	36.1
0.040	353	35.2
0.050	386	38.6
0.075	369	36.9
0.100	356	35.5
0.150	354	35.3
0.200	335	33.1
0.250	348	34.7
0.300	328	32.2
0.350	367	36.7
0.400	316	30.7
0.450	353	35.2
0.500	297	28.0
0.550	346	34.4
0.600	357	35.7
0.650	328	32.2
0.700	334	33.0
0.750	407	40.5

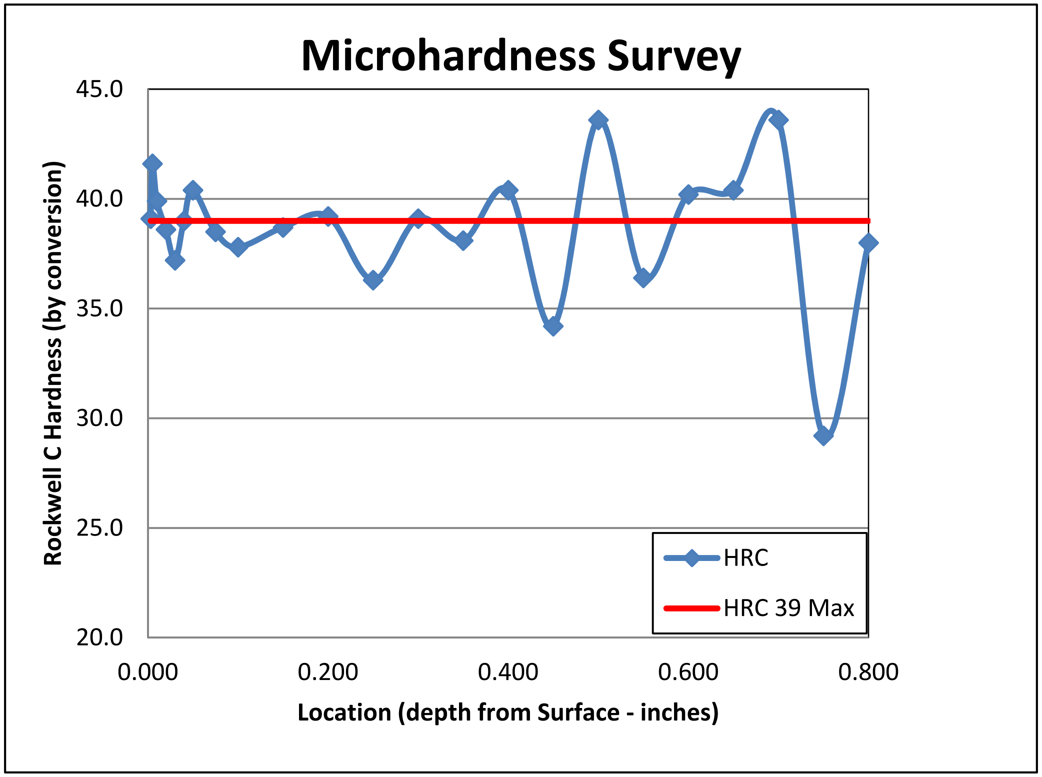


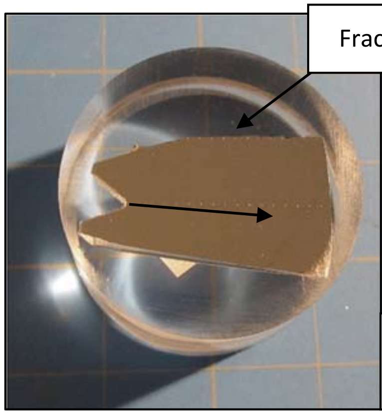


Section C - Rod S1-G1 #11
Knoop survey along fracture

Figure A6
(Rod S1-G1 Section C)

Table 7 Knoop Microhardness Results		
Location Depth from surface (in.)	Knoop Hardness Number	HRC (by conversion table)
0.003	392	39.1
0.005	421	41.6
0.010	369	39.9
0.020	386	38.6
0.030	372	37.2
0.040	391	39.0
0.050	407	40.4
0.075	386	38.5
0.100	378	37.8
0.150	387	38.7
0.200	393	39.2
0.250	363	36.3
0.300	392	39.1
0.350	381	38.1
0.400	407	40.4
0.450	344	34.2
0.500	446	43.6
0.550	364	36.4
0.600	404	40.2
0.650	407	40.4
0.700	446	43.6
0.750	306	29.2
0.800	380	38.0

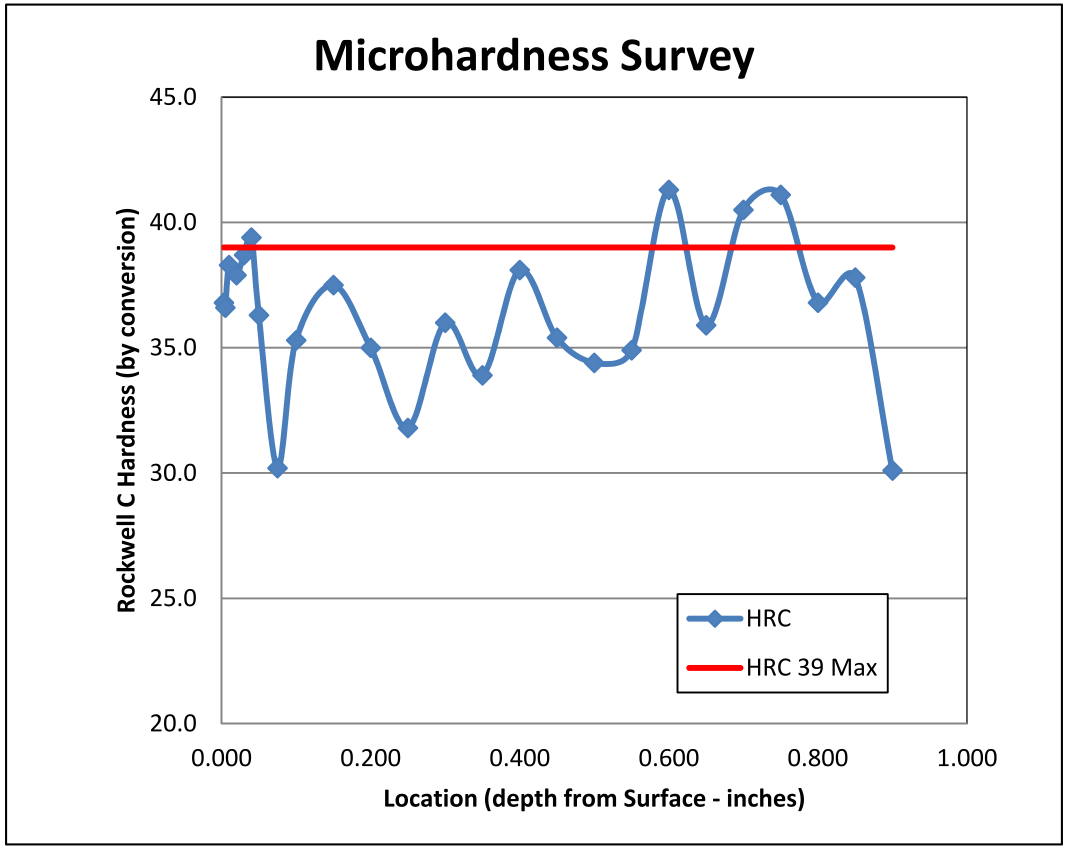


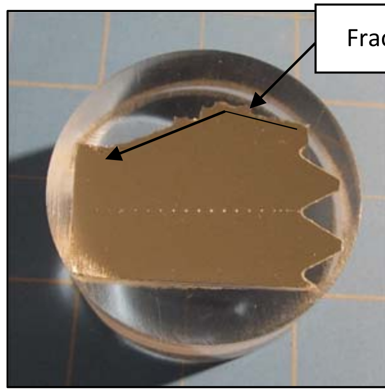


Section C - Rod S1-G1 #11
Knoop survey from thread root

Figure A7
(Rod S1-G1 Section C)

Table 8 Knoop Microhardness Results		
Location Depth from surface (in.)	**Knoop Hardness Number**	**HRC (by conversion table)**
0.003	368	36.8
0.005	366	36.6
0.010	383	38.3
0.020	379	37.9
0.030	387	38.7
0.040	395	39.4
0.050	363	36.3
0.075	313	30.2
0.100	354	35.3
0.150	375	37.5
0.200	351	35.0
0.250	324	31.8
0.300	360	36.0
0.350	341	33.9
0.400	381	38.1
0.450	355	35.4
0.500	346	34.4
0.550	350	34.9
0.600	417	41.3
0.650	357	35.9
0.700	408	40.5
0.750	415	41.1
0.800	368	36.8
0.850	378	37.8
0.900	312	30.1



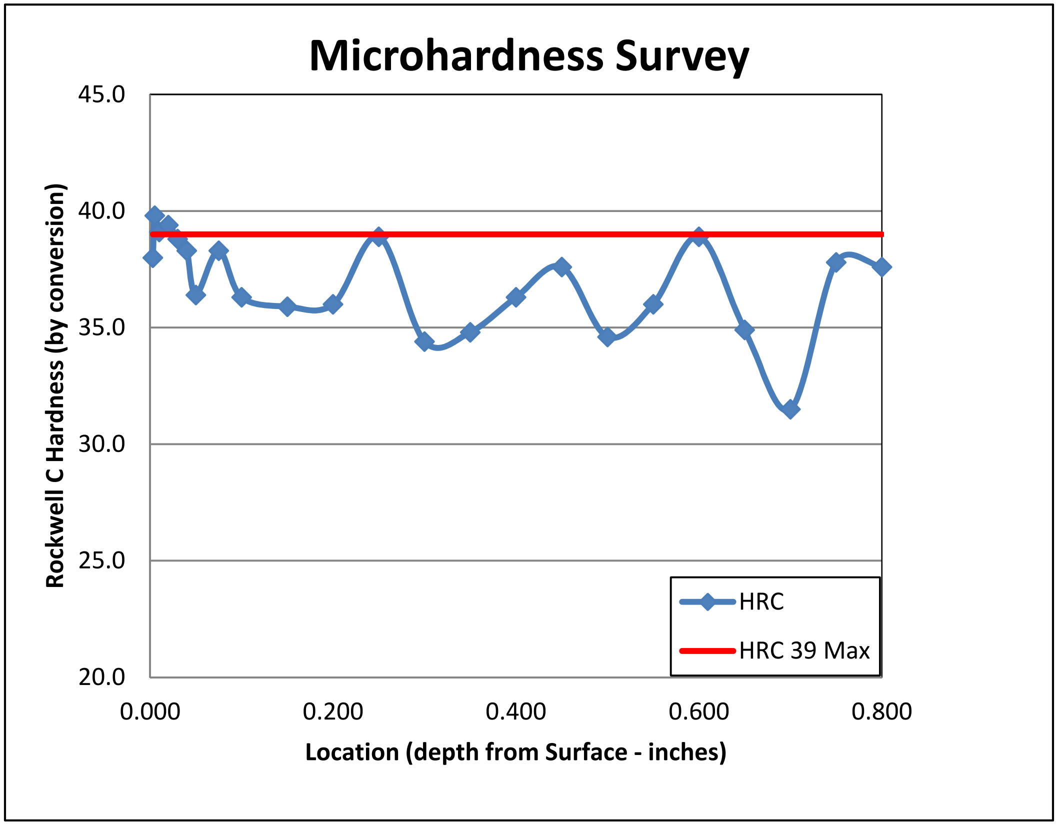


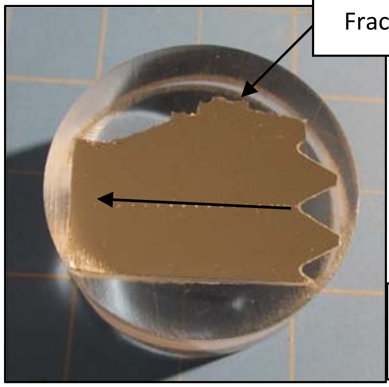
Fracture Surface

Section D - Rod S1-G1 #11
Knoop survey along fracture

Figure A8
(Rod S1-G1 Section D)

Table 9 Knoop Microhardness Results		
Location Depth from surface (in.)	Knoop Hardness Number	HRC (by conversion table)
0.003	380	38.0
0.005	399	39.8
0.010	392	39.1
0.020	395	39.4
0.030	388	38.8
0.040	383	38.3
0.050	364	36.4
0.075	383	38.3
0.100	363	36.3
0.150	359	35.9
0.200	360	36.0
0.250	390	38.9
0.300	346	34.4
0.350	349	34.8
0.400	363	36.3
0.450	376	37.6
0.500	347	34.6
0.550	360	36.0
0.600	389	38.9
0.650	350	34.9
0.700	321	31.5
0.750	378	37.8
0.800	376	37.6

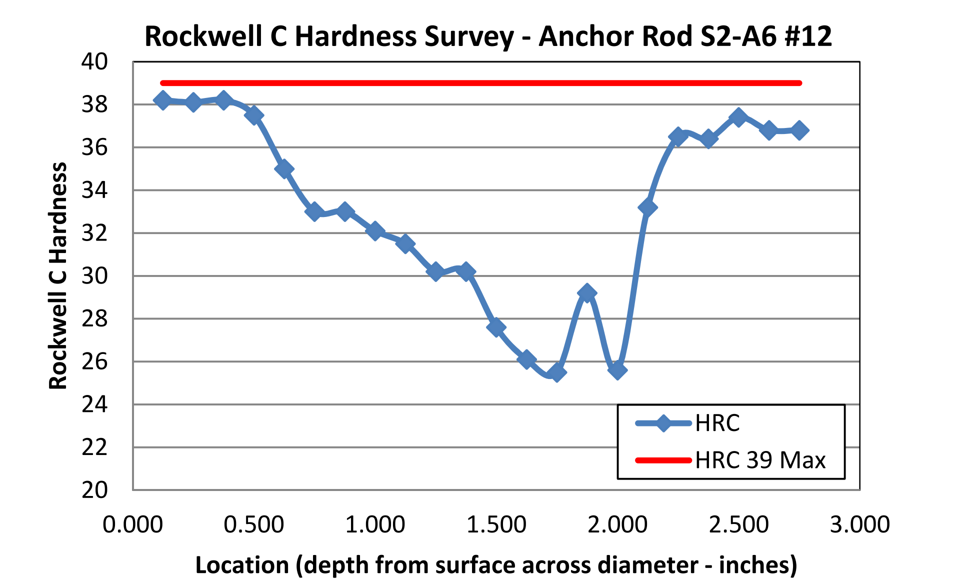




Section D - Rod S1-G1 #11
Knoop survey from thread root

Figure A9
(Rod S1-G1 Section D)

Rockwell C Hardness Results		
Location (in.)	S2-A6 #12	S1-G1 #11
0.125	38.2	36.9
0.250	38.1	36.0
0.375	38.2	36.1
0.500	37.5	34.1
0.625	35	34.9
0.750	33	34.6
0.875	33	32.0
1.000	32.1	28.5
1.125	31.5	31.0
1.250	30.2	33.1
1.375	30.2	29.2
1.500	27.6	30.0
1.625	26.1	29.5
1.750	25.5	30.8
1.875	29.2	29.2
2.000	25.6	30.1
2.125	33.2	35.1
2.250	36.5	35.6
2.375	36.4	35.1
2.500	37.4	36.1
2.625	36.8	36.5
2.750	36.8	36.6



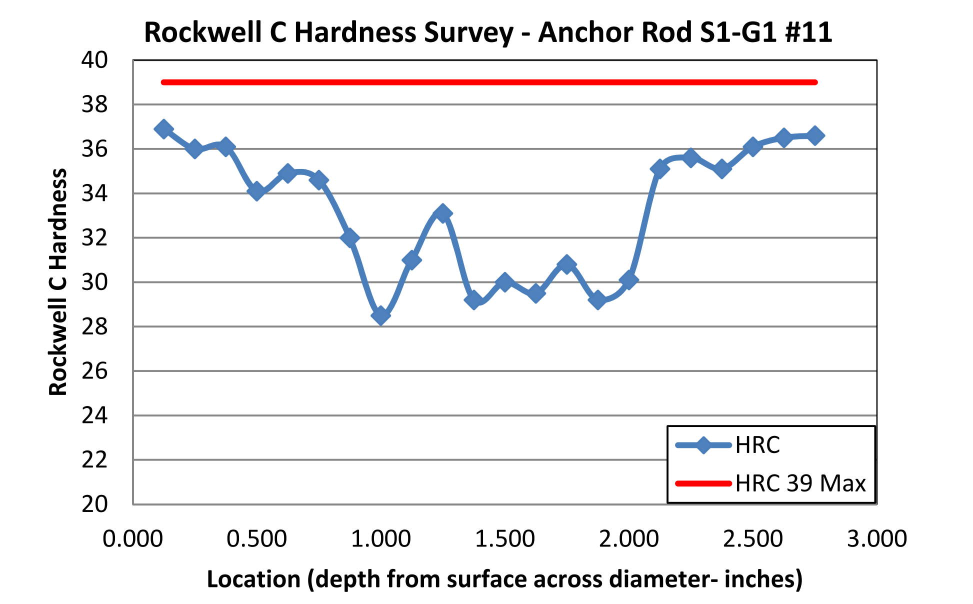


Figure A10

Rockwell C Hardness Across the Rod Diameter

Appendix B
Anamet Labs Test Reports

LABORATORY CERTIFICATE

Note this testing was performed on Rod ID: S2-A6 #12

Anamet, inc

March 21, 2013

LABORATORY NUMBER: 5004.8623

CUSTOMER AUTHORIZATION: Credit Card

DATE SUBMITTED: March 20, 2013

REPORT TO: The Dyson Corporation
Attn: Patrick Linehan
53 Freedom Rd.
Painesville, OH 440177

SUBJECT:
One steel sample was submitted for chemical analysis, tensile and charpy impact testing. The sample was identified as Bay Bridge anchor rod, 3" ASTM A354 Grade BD (approximately 23" long threaded on one end).

The following results relate only to the item tested

SPECTROCHEMICAL ANALYSIS
(Reported as Wt. %)

			Requirement ASTM A354 Alloy Steel Min.	Max.
Aluminum	(Al)	<0.005	Information	
Carbon*	(C)	0.40	0.33	0.55
Chromium	(Cr)	0.97	Information	
Cobalt	(Co)	0.01	Information	
Copper	(Cu)	0.22	Information	
Iron	(Fe)	Balance	Balance	
Manganese	(Mn)	0.93	0.57	-
Molybdenum	(Mo)	0.16	Information	
Nickel	(Ni)	0.10	Information	
Phosphorus	(P)	0.012	-	0.040
Silicon	(Si)	0.24	Information	
Sulfur*	(S)	0.034	-	0.045
Titanium	(Ti)	<0.005	Information	
Tungsten	(W)	<0.005	Information	
Vanadium	(V)	0.03	Information	
Zirconium	(Zr)	<0.005	Information	

* Determined by LECO combustion

Anamet, inc

Lab. No. 5004.8623

TENSILE TEST (ASTM A370-10)		Requirement ASTM A354 Grade BD
Diameter of Specimen (in.)	0.506	
Area (in^2)	0.201	
Tensile Strength (psi)	170000	140000 min.
Yield Strength 0.2% Offset (psi)	149000	115000 min.
Elongation in 2.0" Gage (%)	15-1/2	14 min.
Reduction of Area (%)	46.0	40 min.

CHARPY IMPACT TEST (ASTM A370-10)
Type: V-Notch
Size: 10mm x 10mm x 55mm
Location: Per Drawing
Temperature: Room Temperature (+70°F actual)

Energy Absorbed (ft·lbs)
18
18
17

Requirements: Energy – Information

This testing was completed on March 20, 2013 and was performed in accordance with the customer's authorization. The testing was under Anamet, Inc. Quality Control Program QCM 66-10, Rev.13 (1/6/2012). The results meet the listed requirements.

Submitted by:

Edward A. Foreman

Edward A. Foreman
Quality Manager

yv

Anamet, inc Materials Engineering & Laboratory Testing
26102 EDEN LANDING ROAD, SUITE 3 • HAYWARD, CALIFORNIA 94545 • (510) 887-8811 • FAX (510) 887-8427

Report No. 5004.8612 **March 18, 2013**

ROCKWELL HARDNESS TESTING OF AN ANCHOR ROD SECTION

Customer Authorization: PO# 615126

Report To: Christensen Materials Engineering
Attn: Conrad Christensen
89 Stephanie Lane
Alamo, CA 94507

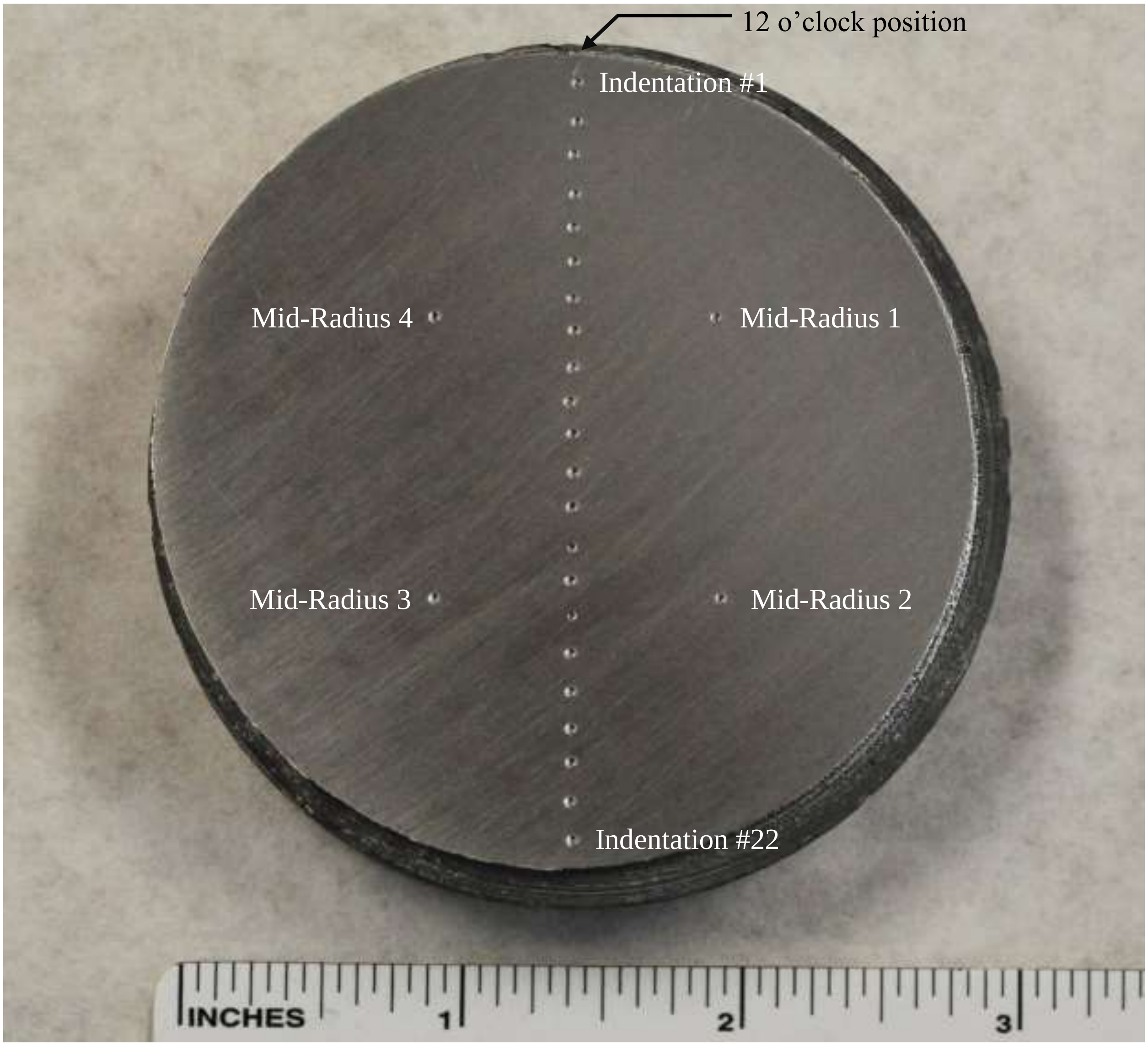
REPORT[1]

One anchor rod section, identified as Bay Bridge 3-inch diameter anchor rod S2-A6 12, was submitted for a Rockwell hardness test. The anchor rod was reportedly an A354 Grade BD alloy steel with a hardness range from 31 to 39 HRC.

The anchor rod section was milled flat with a surface grinder and cleaned with acetone. Rockwell hardness testing was performed at four mid-radii and at twenty-two locations traverse through the cross section at 1/8-inch increments. The photograph in Figure 1 indicates the locations on the anchor rod cross section that were tested. Tables 1 and 2 present the results of the hardness testing and Table 3 presents the hardness readings on two check standards.

Prepared by:	Reviewed by:
Norman Yuen	Audrey Fasching, Ph.D., P.E.
Materials Engineer	Senior Materials Engineer

[1] The magnifications of the optical and scanning electron micrographs in this report are approximate and should not be used as a basis for dimensional analyses unless otherwise indicated.
The conclusions in this report are based upon the available information and evidence provided by the client and gathered by Anamet, within the scope of work authorized by the client, and they are hereby presented by Anamet to a reasonable degree of engineering and scientific certainty. Anamet reserves the right to amend or supplement its conclusions or opinions presented in this report should additional data or information become available, or further work be approved by the client.



(a)

Figure 1 Photograph of the anchor rod with Rockwell hardness indentations at the four mid-radii and traverse through the cross section.

Table 1
Rockwell Hardness Traverse Measurements

Indentation Number	Distance from the 12 o'clock position (inches)	Rockwell Hardness (HRC)
1	0.125	38.2
2	0.250	38.1
3	0.375	38.2
4	0.500	37.5
5	0.625	35.0
6	0.750	33.0
7	0.875	33.0
8	1.000	32.1
9	1.125	31.5
10	1.250	30.2
11	1.375	30.2
12	1.500	27.6
13	1.625	26.1
14	1.750	25.5
15	1.875	29.2
16	2.000	25.6
17	2.125	33.2
18	2.250	36.5
19	2.375	36.4
20	2.500	37.4
21	2.625	36.8
22	2.750	36.8

Table 2
Rockwell Hardness Measurements at Mid-Radii

Mid-Radii Number	Rockwell Hardness (HRC)
1	34.2
2	36.2
3	35.9
4	33.2

Table 3
Rockwell Hardness Standards

Standard – 33.19 HRC	
Indentation Number	Rockwell Hardness (HRC)
1	32.5
2	32.8
3	32.9

Standard – 44.57 HRC	
Indentation Number	Rockwell Hardness (HRC)
1	42.5
2	43.4
3	44.1
4	43.8
5	44.1

LABORATORY CERTIFICATE

Anamet, inc

April 5, 2013

LABORATORY NUMBER:	5004.8677
CUSTOMER AUTHORIZATION:	Verbal
DATE SUBMITTED:	April 1, 2013
REPORT TO:	Alta Vista Solutions Attn: Aaron Prchlik 6475 Christie Ave., Ste 425 Emeryville, CA 94608

SUBJECT:
One anchor rod was submitted for mechanical testing. The sample was identified as Bay Bridge 3" Diameter Anchor Rod I.D: S2-A6 #2, ASTM A354 Grade BD steel.

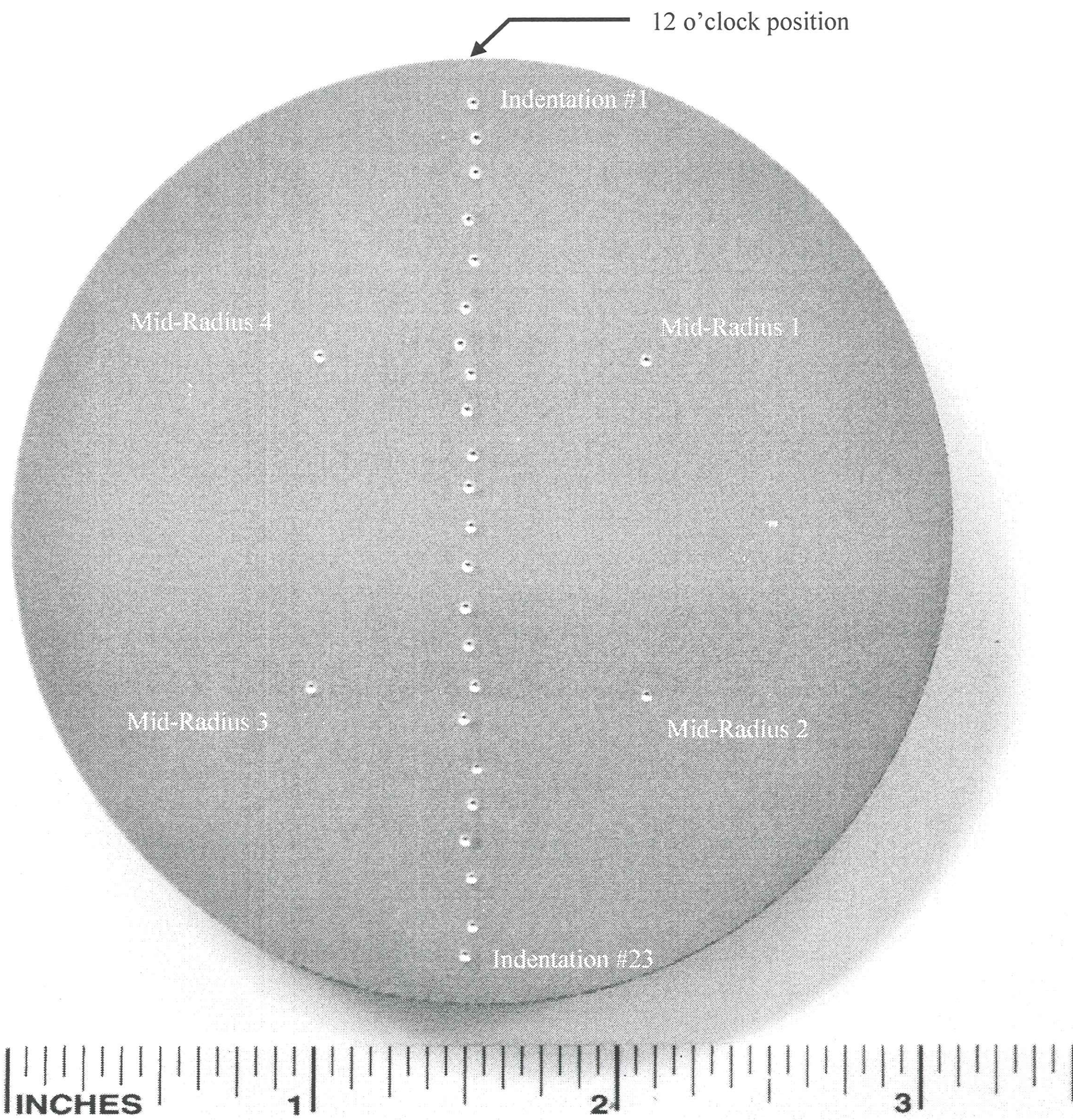
TENSILE TEST (ASTM A370-10)		Requirement ASTM A354-11 Grade BD
Diameter of Specimen (in.)	0.505	
Area (in^2)	0.200	
Tensile Strength (psi)	168000	140000 psi min.
Yield Strength 0.2% Offset (psi)	146000	115000 psi min.
Elongation in 2.0″ Gage (%)	14	14 min.
Reduction of Area (%)	48.0	40 min.

CHARPY IMPACT TEST (ASTM A370-10)
Type: V-Notch
Size: 10mm x 10mm x 55mm
Orientation: Longitudinal
Location: Close to Outside (Surface Notch)
Temperature: Room Temperature

Energy Absorbed (ft·lbs)
15
14
15

26102 Eden Landing Road, Suite 3, Hayward, California 94545-3811 Phone: **510-887-8811** Fax: **510-887-8427** www.anametinc.com

Lab. No. 5004.8677



(a)

Figure 1 Photograph of the anchor rod with Rockwell hardness indentations at the four mid-radii and traverse through the cross section.

Lab. No. 5004.8677

Table 1
Rockwell Hardness Traverse Measurements

Indentation Number	Distance from the 12 o'clock position (inches)	Rockwell Hardness (HRC)
1	0.125	36.1
2	0.250	35.1
3	0.375	36.2
4	0.500	35.3
5	0.625	33.3
6	0.750	33.4
7	0.875	32.0
8	1.000	32.7
9	1.125	32.4
10	1.250	30.4
11	1.375	28.5
12	1.500	31.8
13	1.625	36.9
14	1.750	30.6
15	1.875	31.2
16	2.000	34.5
17	2.125	35.4
18	2.250	35.4
19	2.375	35.2
20	2.500	36.2
21	2.625	37.3
22	2.750	36.1
23	2.875	36.4

Table 2
Rockwell Hardness Measurements at Mid-Radii

Mid-Radii Number	Rockwell Hardness (HRC)
1	32.5
2	34.9
3	35.7
4	34.3

Anamet, inc

Lab. No. 5004.8677

Table 3
Rockwell Hardness Standards

Standard – 33.19 HRC	
Indentation Number	Rockwell Hardness (HRC)
1	32.5
2	32.6
3	33.1

Standard – 44.57 HRC	
Indentation Number	Rockwell Hardness (HRC)
1	43.3
2	43.8
3	43.8

The testing was completed on April 4, 2013 and was performed in accordance with the customer's authorization. The tests were conducted under Anamet, Inc. Quality Program QCM 66-10, Rev. 13 (1/6/2012). The tensile test results meet the listed requirements.

Submitted by:

Edward A. Foreman

Edward A. Foreman
Quality Manager

yv

LABORATORY CERTIFICATE

Anamet, inc

April 12, 2013

LABORATORY NUMBER:	5004.8710
CUSTOMER AUTHORIZATION:	Verbal
DATE SUBMITTED:	April 9, 2013
REPORT TO:	Alta Vista Solutions Attn: Aaron Prchlik 6475 Christie Ave., Ste 425 Emeryville, CA 94608

SUBJECT:
One anchor rod was submitted for chemical analysis and mechanical testing. The sample was identified as Bay Bridge 3" Diameter Anchor Rod I.D: S1-G1 #11, ASTM A354 Grade BD steel.

The following results relate only to the item tested

SPECTROCHEMICAL ANALYSIS (ASTM E415-08)
(Reported as Wt. %)

Requirement: ASTM A354, Gr. BD Alloy Steel, Product Analysis

Element	Symbol	Result	Min.	Max.
Carbon*	(C)	0.43	0.33	0.55
Chromium	(Cr)	0.98	Information	
Cobalt	(Co)	0.01	Information	
Columbium	(Cb)	<0.005	Information	
Copper	(Cu)	0.22	Information	
Iron	(Fe)	Balance	Balance	
Manganese	(Mn)	0.93	0.57	-
Molybdenum	(Mo)	0.15	Information	
Nickel	(Ni)	0.10	Information	
Phosphorus	(P)	0.012	-	0.040
Silicon	(Si)	0.23	Information	
Sulfur*	(S)	0.039	-	0.045
Titanium	(Ti)	<0.005	Information	
Tungsten	(W)	<0.005	Information	
Vanadium	(V)	0.03	Information	
Zirconium	(Zr)	<0.005	Information	

* Determined by LECO combustion (ASTM E1019-11)

26102 Eden Landing Road, Suite 3, Hayward, California 94545-3811 Phone: **510-887-8811** Fax: **510-887-8427** www.anametinc.com

Anamet, inc

Lab. No. 5004.8710

TENSILE TEST (ASTM A370-10)		Requirement ASTM A354 Grade BD
Diameter of Specimen (in.)	0.504	
Area (in^2)	0.200	
Tensile Strength (psi)	159000	140000 psi min.
Yield Strength 0.2% Offset (psi)	136000	115000 psi min.
Elongation in 2.0″ Gage (%)	15	14 min.
Reduction of Area (%)	48.4	40 min.

CHARPY IMPACT TEST (ASTM A370-10)
Type: V-Notch
Size: 10mm x 10mm x 55mm
Orientation: Longitudinal (Surface Notched)
Location: Per drawing
Temperature: +40°F

Energy Absorbed (ft·lbs)
13-1/2
13
14

Requirements: Energy – Information

Lab. No. 5004.8710

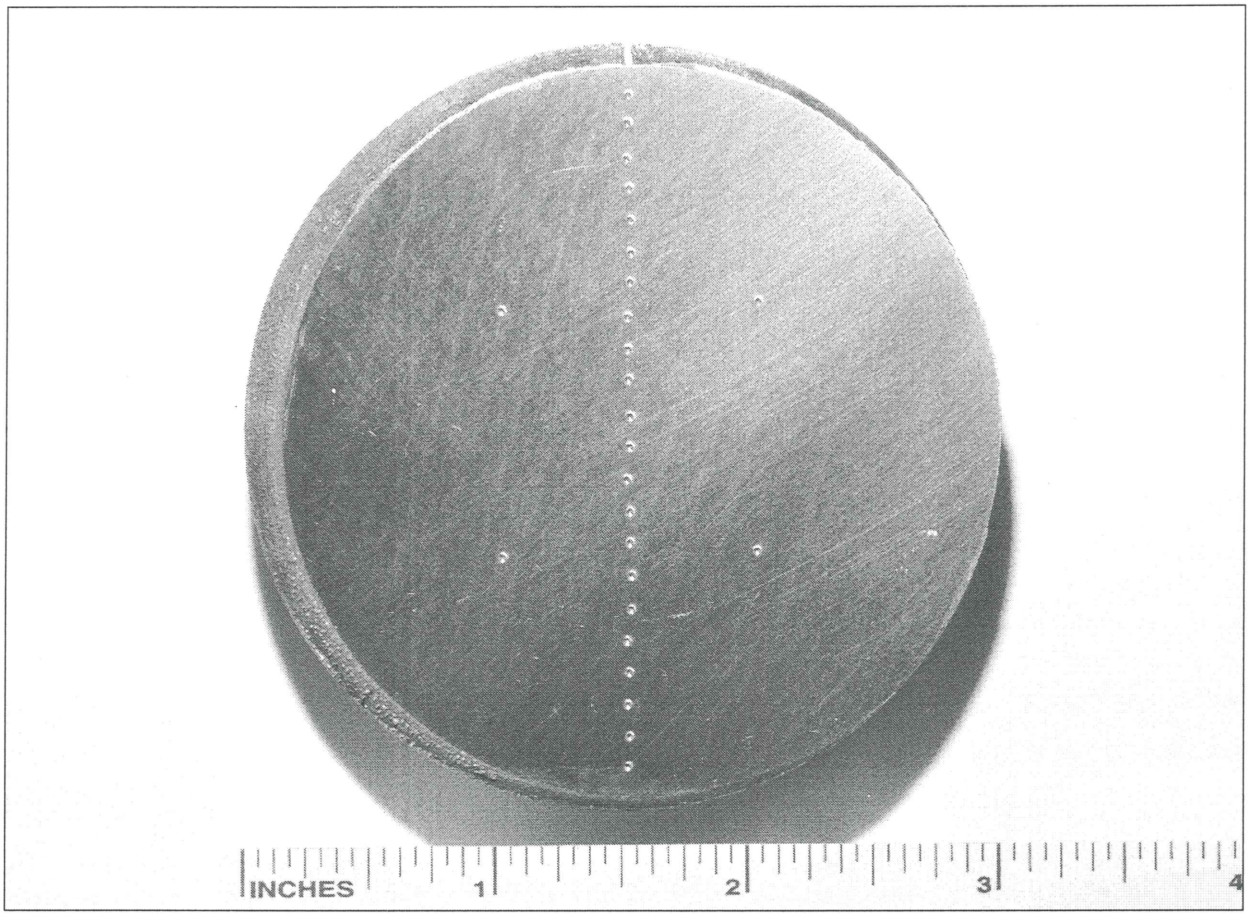


Figure 1 Photograph of the anchor rod with Rockwell hardness indentations at the four mid-radii and traverse through the cross section.

Lab. No. 5004.8710

Table 1
Rockwell Hardness Traverse Measurements

Indentation Number	Distance from the 12 o'clock position (inches)	Rockwell Hardness (HRC)
1	0.125	36.9
2	0.250	36.0
3	0.375	36.1
4	0.500	34.1
5	0.625	34.9
6	0.750	34.6
7	0.875	32.0
8	1.000	28.5
9	1.125	31.0
10	1.250	33.1
11	1.375	29.2
12	1.500	30.0
13	1.625	29.5
14	1.750	30.8
15	1.875	29.2
16	2.000	30.1
17	2.125	35.1
18	2.250	35.6
19	2.375	35.1
20	2.500	36.1
21	2.625	36.5
22	2.750	36.6

Table 2
Rockwell Hardness Measurements at Mid-Radii

Mid-Radii Number	Rockwell Hardness (HRC)
1	34.0
2	34.6
3	31.9
4	31.5

Anamet, inc

Lab. No. 5004.8710

The testing was completed on April 11, 2013 and was performed in accordance with the customer's authorization. The tests were conducted under Anamet, Inc. Quality Program QCM 66-10, Rev. 13 (1/6/2012). The results meet the listed requirements.

Submitted by:

Edward A. Foreman

Edward A. Foreman
Quality Manager

yv

TOLL BRIDGE PROGRAM
OVERSIGHT COMMITTEE
CALTRANS BAY AREA TOLL AUTHORITY CALIFORNIA TRANSPORTATION COMMISSION

Toll Bridge Program Oversight Committee
Department of Transportation
Office of the Director
1120 N Street
P.O. Box 942873
Sacramento, CA 94273-0001

May 8, 2013

Mr. Vincent Mammano
California Division Administrator
Federal Highway Administration
650 Capitol Mall, Suite 4-100
Sacramento, CA 95814

Re: East San Francisco Oakland Bay Bridge Seismic Safety Project

Dear Mr. Mammano:

The Toll Bridge Oversight Program Committee (TBPOC) requests assistance from the Federal Highway Administration (FHWA) to conduct an independent review of our findings and recommendations concerning the galvanized high strength bolts on the new east span of the San Francisco-Oakland Bay Bridge.

The TBPOC is tasked by the California State Legislature to oversee the new east span project and comprises the Director of the California Department of Transportation (Caltrans), the Executive Director of the Bay Area Toll Authority (BATA), and the Executive Director of the California Transportation Commission (CTC). As we work our way through resolution and repair, we ask that FHWA assemble a team to double check our findings and recommendations to ensure that our investigation is comprehensive and our solution is the best and safest possible.

In early March 2013, we discovered 32 galvanized A354 BD bolts that are embedded in Pier E2 had failed days after tensioning (see attached graphic). After a methodical and thorough investigation, we concluded that the bolts failed due to hydrogen embrittlement, with the source of hydrogen being both internal and external. We are currently in the process of designing a retrofit solution to replace all of the bolts, that were manufactured in 2008 and embedded in Pier E2. In addition, there are numerous similar galvanized A354 BD bolts on Pier E2 and at other locations on the new bridge that we are now reviewing, testing, and evaluating to determine if there is any further remedial action required. Please see the attached spreadsheet entitled "Galvanized ATSM A354 BD Material Contract 04-0120F4 SFOBB SAS" dated April 26, 2013, that defines the other locations and uses of these bolts on this bridge that have the same specification.

At the end of this investigation, we intend to classify bolts into three categories for inspection and repair:

1. Bolts that are to be replaced before opening the bridge to traffic,
2. Bolts that are to be replaced after opening the bridge, as a precautionary measure to avoid premature failure due to stress corrosion,
3. And bolts that are to undergo a regular inspection schedule, and replaced as necessary if damage is observed in routine use.

We are asking your team to provide an arms-length review of our analysis and strategy proceeding in that fashion. We believe that resolution of these questions will determine the schedule for opening the new span. We thank you in advance for ensuring the safety of the traveling public and reassuring public confidence by taking on this review. Because time is of the essence we ask that you not hesitate to contact the members of the TBPOC, with any questions your team has regarding this request.

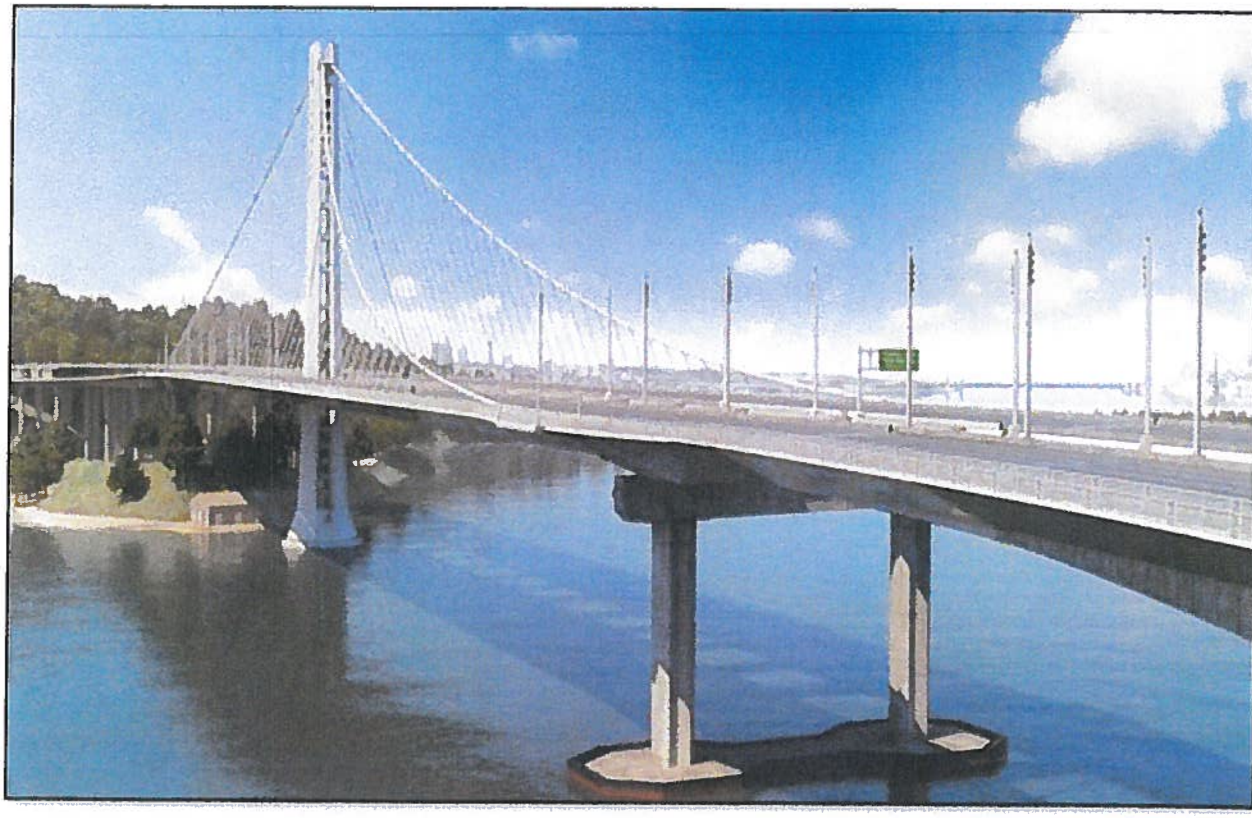
Sincerely,

STEVE HEMINGER
TBPOC Chair
Executive Director
Bay Area Toll Authority

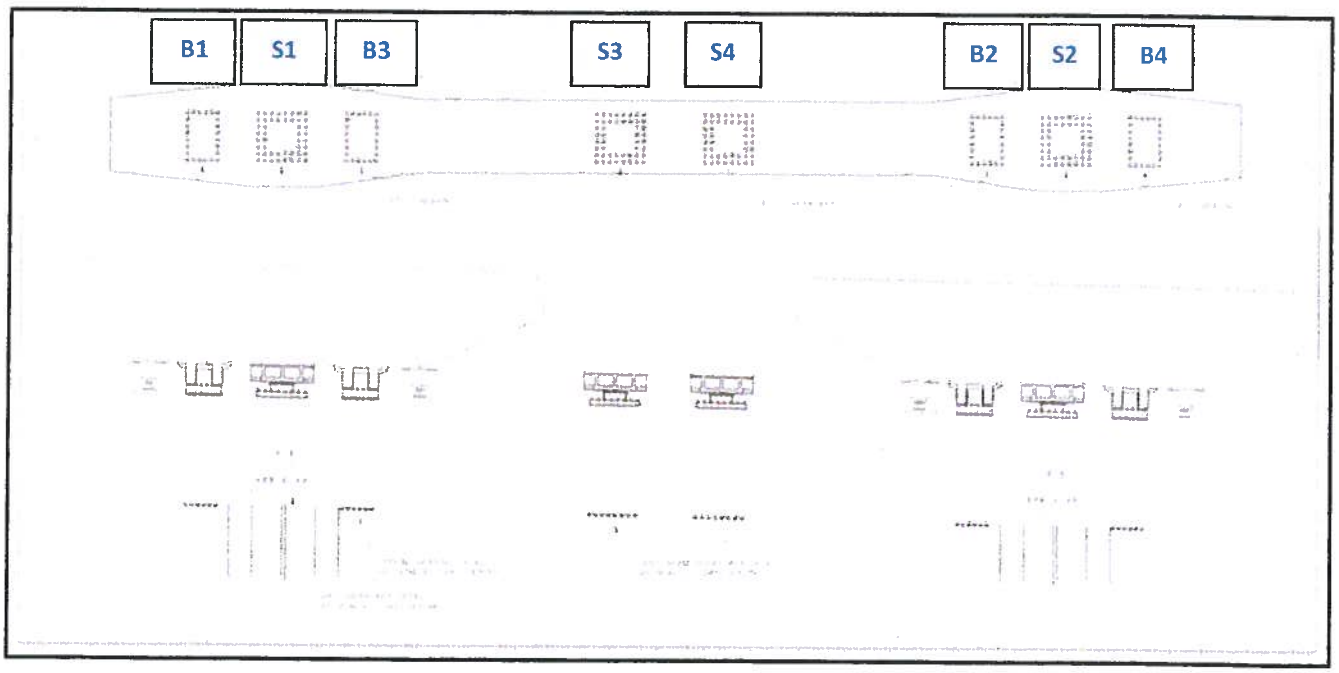
ANDRE BOUTROS
Executive Director
California Transportation Commission

MALCOLM DOUGHERTY
Director
California Department of Transportation

Attachments



Location of Pier E2



Locations of Shear Keys (S1, S2, S3, and S4) and

Bearings (B1, B2, B3, and B4) at Pier E2

Location and Item		Component Description	Rod (no head) or Bolt (with head)	Threads Cut or Rolled	Supplier	Diameter (in)	Overall Length (ft)	Overall Length (mm)	Quantity Installed (not including spares)	De-Humidified Zone?	Tighten Method	Final Tension (fraction of Fu or UTS)	Date Tension or Loading Complete	Date Re-Inspected (by 4/8/13)	Date Re-Inspected (by 4/23/13)	Date Re-Inspected (by 5/5/13)	Notes
E2 Bearings and Shear Keys	1	E2 Shear Key - Connect to Concrete - Above Column, Under OBG [S1, S2]	rod	Cut	Dyson	3	17.2	5235	60	No	Tension	0.7	3/5/2013	daily check	daily check	daily check	32 of 96 rods broke after tensioning, then tension level lowered
							10.0	3035	36 (96)								
	2	E2 Shear Key - Connect to Concrete - Above Bent Cap, Under Crossbeam [S3, S4]	rod	Cut	Dyson	3	21.9	6676	96	No	Tension	0.7	4/1/2013	daily check	daily check	daily check	
		E2 Bearing - Connect to Concrete - Under OBG [B1, B2, B3, B4]	rod	Cut	Dyson	3	22.6	6902	64		Tension	0.7	4/9/2013	daily check	daily check	daily check	
							22.2	6777	32 (192)								
	3	E2 Shear Key - Connect to OBG [S1, S2]	rod	Cut	Dyson	3	4.4	1337	96	No	Tension	0.7	9/12/2012	4/6/2013 4/8/2013	4/17/13 to 4/23/13	5/3/2013	
							1.8	537	64								
		E2 Shear Key - Connect to Crossbeam [S3, S4]	rod	Cut	Dyson	3	4.3	1312	96								
							1.7	512	64 (320)								
	4	E2 Bearing - Connect to OBG [B1, B2, B3, B4]	rod	Cut	Dyson	2	3.6	1105	224	No	Tension	0.7	9/12/2012	4/6/2013	4/17/13 to 4/23/13	5/3/2013	
	5	E2 Bearing Assembly Bolts (Spherical Bushing Halves)	rod	Cut	Dyson for Lubrite for Hochang	1	2.4	733	96	No	Tension	0.61	July 2009	not accessible	not accessible	not accessible	Connect 2 halves of the spherical bushing assembly housing together at Lubrite; rods are internal to bearings and **all rods are not accessible** after bearing assembly at Hochang (December 2009 & January 2010); rods tensioned to 0.7 Fy.
	6	E2 Bearing Assembly Bolts (Retaining Rings)	Socket Head Cap Screw	Cut	Dyson for Hochang	1	0.2	55	336	No	snug + 1/4 turn	~0.4	January 2010	4/6/2013 (for 32 accessible bolts)	4/23/2013 (for 32 accessible bolts)	5/3/2013 (for 32 accessible bolts)	Bolts thread into drill and tap holes to attach retaining rings that secure the Lubrite spherical bushing assembly in the bottom housing; bolts are mechanically galvanized, not hot dip galvanized; bolts are internal to bearings and not accessible after bearing assembly at Hochang, except for a small number of bolts in limited areas -> **32 of 336 bolts are accessible**.
Cable Anchorage	7	PWS Anchor Rods - PWS Socket to Anchorage	rod	55 Cut (20%) 219 Rolled (80%)	Dyson	3-1/2	27.9 to 31.8	8500 to 9700	274	Yes	Load Transfer	0.26	9/26/2012	4/6/2013	4/20&22/2013	5/4/2013	With DL after load transfer (current condition)
												0.29	N/A	N/A	N/A	N/A	With DL + Added DL
												0.32	N/A	N/A	N/A	N/A	Service Load (Group 1)
												0.35	N/A	N/A	N/A	N/A	SEE (Seismic)
Top of Tower	8	Tower Saddle Tie Rods	rod	Rolled	Dyson	4	6.0 to 17.5	1840 to 5325	25	Yes	Tension	0.41	7/14/2012	4/6/2013	4/19/2013	5/3/2013	Tensioned to 0.5 Fy
	9	Turned Rods at Tower Saddle Segment Splices	rod	Cut	Dyson	3 @ Threads [~3-1/16 @ Shank]	1.5	463	100	Yes	Tension	0.45	4/6/2011	4/6/2013	4/19/2013	5/3/2013	Located at the 2 field splices connecting the 3 tower saddle segments; 100 rods tensioned prior to saddle erection; 8 rods only snug tight after tie rod tensioning due to conflict with tie rods.
							1.4	415	8 (108)		snug	~0.1	7/14/2012				
	10	Tower Saddle to Grillage Anchor Bolts	Hex Bolt	Cut	Dyson	3	1.2	360	90	Head Yes, Nut No	snug	~0.1	3/25/2013	4/6/2013	4/19/2013	5/3/2013	Snug tightened before and after load transfer
	11	Tower Outrigger Boom (for Maintenance) at Top of Tower	Hex Bolt	Cut	Dyson	3	2.1	630	4	No	snug	~0.1	July 2012	4/6/2013	4/19/2013	5/4/2013	Act as pins for swinging out and then securing the maintenance outrigger boom at the top of 2 of 4 tower head chimneys. At each boom, one bolt is loaded and other bolt is unloaded in the current boom position. The currently unloaded bolt will be installed snug tight when the boom is swung out for use (future position).
Bottom of Tower	12	Tower Anchor Rods - Tower at Footing (3" Dia)	rod	Cut	Vulcan Threaded Products for KOS for KFM (04-0120E4)	3	25.6	7789	388	Yes	Tension	0.48	4/17/2013	N/A	4/20/2013 4/22/2013	5/5/2013	Tensioned to 1800 kN = 404.7 kips Tension before and after load transfer
	13	Tower Anchor Rods - Tower at Footing (4" Dia)	rod	Cut		4	25.7	7839	36	Yes	Tension	0.37	4/17/2013	N/A	4/20/2013 4/22/2013	5/5/2013	Tensioned to 2530 kN = 568.8 kips Tension before and after load transfer
East Saddles	14	East Saddle Anchor Rods	rod	Cut	Dyson for JSW	2	2.6	800	32	Yes	snug	~0.1	May 2010	4/7/2013	4/21/2013	5/3/2013	specified gap under nut/washer at one end of rod and 2 nuts snug against each other at other end of rod -> snug tight for portion of rod
	15	East Saddle Tie Rods	Hex Bolt	Cut	Dyson	3	4.7	1420	18	Yes	snug	~0.1	4/13/2012	4/7/2013	4/21/2013	5/3/2013	Snug tightened before load transfer
East Cable	16	B14 Cable Bands - Cable Brackets - at East End of Bridge - Strongback Anchor Rods	rod	Rolled	Dyson	3	10.3 to 11.1	3129 to 3372	24	No	Tension	0.16	2/8/2013	4/7/2013	4/21/2013	5/4/2013	neoprene between strongback and cable band is in the grip
W2 Bent Cap	17	W2 Bikepath Anchor Rods	rod	Cut	Dyson	~1-3/16 [Metric M30]	1.5	460	43	No	Not Determined Yet		N/A	N/A	N/A	N/A	Details for bikepath connections are being redesigned and are not final. The 18 anchor rods at the bottom connections will be abandoned. The 25 anchor rods at the top connections will be used and supplemented with additional anchor rods. These rods will be tensioned on the separate YBITS-2 Contract.
									Total = 2306								

Transcription of
May 8, 2013 BATA Oversight Committee Meeting

COMMISSIONER DODD: Move to Item No. 7. We have an update…financial update and progress report for the Toll Bridge Seismic Retrofit Program. Peter?

PETER LEE: Good afternoon Commissioners. I'm going to pass it over to Steve, who's going to present on the anchor bolt issue.

STEVE HEMINGER: Poor Peter has been preempted for meetings on end here. And I'm afraid we're going to preempt him again today, and give you another briefing on the bolt situation. Again, I'm Steve Heminger, Executive Director of the Bay Area Toll Authority. To my right Malcolm Dougherty, the director of Caltrans. To my left Andre Boutros, the Executive Director of the California Transportation Commission. The three of us are the Toll Bridge Program Oversight Committee, which has been conducting the investigation into the bolts on the new bridge.

We wanted to start today with this picture, just to remind everybody why we are building this project. This, of course, is from 1989 during the Loma Prieta earthquake, which was 60 miles away and, nonetheless, collapsed a large deck section on the old bridge. And I must say that on the new bridge we have faced many challenges. We are going to have to fix some bolts. We have fixed welds. We have fixed concrete. We have fixed all kinds of things in construction – all kinds of problems that we encounter. That's something you do in construction; you deal with unexpected problems and fix them. But I must say also this picture – this old bridge is the biggest problem we've got. And there is no fix to it other than moving traffic onto the new span. And that is why we feel such a sense of urgency to get this bolt matter resolved and to get the fixes put in place and to move traffic over to the new bridge as soon as possible. Next slide.

You'll recall that at the last couple of briefings, I think we tried to organize the information we were presenting to you around three questions. We have added a fourth, which is sort of an offshoot of the third. The first question again: what caused the bolts on the east pier manufactured in 2008 to fail? Malcolm again is going to address that question and provide some new information, including a report that we are releasing today. Secondly: what retrofit strategies should be used to replace those bolts? Andre has an answer to that question, actually, today. Thirdly, I'll be dealing with the question of the remaining bolts on the east pier that were manufactured in 2010, as well as the fourth question about the other similar bolts that are on the remainder of the self-anchored suspension bridge that are also – and, again, the word we're going to use, which is just a category of material, is called A354BD galvanized bolts – that's the issue at hand. Again, that's where the east pier is; I think you know that quite well by now.

I did want to – at the outset – talk a little bit about our oversight structure, because I think there have been questions raised in the public discussion about this issue about whether we have an independent review under way. And my strong belief is that we do. And this slide I think tries to depict that. At the top you see the Toll Bridge Program Oversight Committee, composed of the three parties that I've just mentioned. One box down to the left you see a peer review panel, which is independent of us. It's composed of three individuals that I'll mention in a minute, all three of whom are in the National Academy of Engineering, which is the most prestigious engineering body in the United States. To the right – to their right you see BAMC, which is an acronym for a consulting enterprise that we have hired – we, the Bay Area Toll Authority – to deal with issues that arise in this project, and they…their expertise spans all kinds of fields. And we have brought in additional expertise when we need to do so. And then finally, we have a whole series of contractors who are actually implementing the work. The reason we put Caltrans in orange – that's their color, after all – but to make the point that that is where Caltrans is, and everything else in this operation is not them. And so there is, as I indicate, a good deal of independence here, of multiple agencies and multiple checks who are doing this work with us and making sure that we do the work right. Again, this is us. I won't dwell on this, other than to say that we are a statutory mechanism that was created by the Legislature in 2005, and have been dealing with issues like this – maybe not as many cameras as that – but issues like this for many years while we've been building the bridge. I mentioned the peer review panel, and I do want to mention these three individuals by name because their careers are really stellar, and they have been invaluable throughout the history of not only this project, but the Toll Bridge Seismic Retrofit Program. Frieder Seible, who is the Dean Emeritus at U.C. San Diego of their School of Engineering; very well known world-renowned expert on long-span bridges; he's now a professor at a university in Australia. Ed Idriss who is an emeritus professor of civil engineering at U.C. Davis, and he's one of those guys that whenever they have an earthquake somewhere around the world, he parachutes in and figures out what happens, and helps learn that information as the science of earthquake engineering advances. And the third – and especially relevant, I think, for this inquiry – is John Fisher, who is a professor emeritus of civil engineering at Lehigh University of Pennsylvania; very, very capable man, who has helped us enormously on this project because of his background in examining steel failures. And that is exactly what we had here. And he has looked at those kinds of failures in all kinds of places, including on many bridges around the world, and he was also called upon, if you'll remember, after the collapse of the World Trade Center in 2001. He was one of the experts called upon to opine about why that happened. As I mentioned, all three members of the National Academy of Engineering. And I know there have been questions raised in some media circles about whether these gentlemen are really independent because we pay for their airfare and whatever else we might do to get them out here. I can tell you from personal experience that they are not ever bashful about telling us things we don't want to hear, and that is continuing in this process as we go.

I also wanted to announce today a step that we are taking, in addition to all of that review, to ensure that we have an independent – I'll call it a double check on what our investigation yields in terms of findings and recommendations, and that is we have asked the Federal Highway Administration to conduct an arms-length review of our review, and they have agreed to do so. You have at your places a letter that we have sent today to Vince Mammano, who is the Division Administrator – California Division Administrator for the Federal Highway Administration, and we look forward to working with them. They will make whatever findings and judgments they make independent of ours, and I think that will provide a level of reassurance to you and to members of the public and the Legislature and the Administration that we are getting to the right answers and we are getting to the right solutions. So, I will turn the presentation over to Malcolm to deal with the first question on the 2008 bolts.

MALCOLM DOUGHERTY: Thank you, Steve, Mr. Chair, Commissioners. I, again, appreciate the opportunity to be here and, collectively, as part of the Toll Bridge Program Oversight Committee answer questions and provide information and an update. Again, just for clarification, this is a slide that we've talked about before, but again, specifically I will talk about the 2008 bolts that are located at Sheer Key 1 and Sheer Key 2, that are highlighted on that PowerPoint presentation in red. And I think we were very descriptive about those specific bolts in the past. Again, we know what caused the failure of those bolts from 2008, and it was determined that it was hydrogen embrittlement. And we have completed…let me go to the next slide and talk about hydrogen embrittlement, just as a refresher. We talked about this as well last time. Hydrogen embrittlement requires three key components, or three key elements: the susceptibility of the metal and the rods and/or bolts, the presence of hydrogen in a certain quantity, and then also the tension that is imposed on those rods or bolts, and we will actually elaborate on that later in the presentation as well. The root cause of the failure is attributed to higher than normal susceptibility of the steel to hydrogen embrittlement.

This again is a detailed picture under an electron microscope of a metallurgic analysis that shows the lack of uniformity. I won't necessarily elaborate on the detailed picture, but what I do want to draw your attention to is the report, the metallurgic report that has been made available to you as of today, and the analysis that was completed. And really what I want to draw your attention to is Page 12 where it talks about the conclusions and the recommendations. Um, in summary it talks about the metallurgical structure and substructure of the steel in these bolts, which are fundamentally a result of alloy selection and heat treatment conditions has apparently made the rods less tough and, therefore, more susceptible to hydrogen embrittlement. On that page, the first conclusion is that the anchor rods failed as a result of hydrogen embrittlement resulting from the applied tensile load and from hydrogen that was already present and available in the rod material as they were tensioned. The root cause of the failures is attributed to higher than normal susceptibility of the steel to hydrogen embrittlement. The third bullet is the metallurgical condition of the steel was found to be less than ideal. More precisely, the microstructure of the steel is inhomogeneous, resulting in large difference in hardness from center to edge – and that's what this picture represents – and high local hardness near the surface. As an additional consequence, the metallurgical condition, the material exhibits low toughness and marginal ductility – and again, we're talking about specifically the batch of bolts in S…Sheer Key 1 and 2 that came in 2008. The combination of all these factors have caused the anchor rods to be susceptible to hydrogen embrittlement, which ultimately was the cause of their failure. The recommendation is that future procurement of A354 Grade BD anchor rods should include a number of standard supplemental requirements to assure against hydrogen embrittlement failure. The appropriate specification of supplemental requirements is currently under review, and I think we'll talk about that a little bit later on as to what we're doing with the 2010 bolts such that it will lead us potentially modify the amount of testing we do on this type of bolt going forward. This recaps the recommendation that I just iterated to you. And again the conclusions of the metallurgical study are on Page 12, and I also will draw your attention to the information that is on the front page that highlights the individuals that authored this report. One of them is the chair of ASTM Committee on Fasteners, and then the other two, one is a consultant and one of them is on staff in our materials testing laboratory

The specifications…we've talked about the specifications that went into calling for the A354BD bolts…uh, the way that works is a technical design specification team evaluates the bridge design and selects the type of bolts for the application that was called for in the plans. The design specification team subsequently added a supplemental requirement for these bolts – blasting instead of pickling – to address the potential for hydrogen embrittlement. That is consistent with the recommendation in the cautionary note in ASTM 354 standards if you're going to galvanize those bolts. Current Caltrans bridge specifications do not allow the use of galvanized A354 bolts for standard bridge applications. But non-standard applications may be considered, and that's where we are on this project, where we have project-specific specifications and, again, ASTM does allow for galvanization, but it cautions on the potential for hydrogen embrittlement, and that refers us back to the previous bullet as to what steps you should take to avoid or lower the risk of hydrogen embrittlement.

Caltrans has ordered replacement bolts for the 2010 bolts that we were subjecting to destructive testing. So…not on Sheer Key 1 Sheer Key 2, there's another batch of bolts that came in 2010, and they are being tested out in the field in situ right now. We are also doing destructive testing to those bolts. The replacement bolts for those, we have asked for special provisions of those replacements, and it includes, but is not limited to, tighter requirements for hardness and additional testing to address the potential for hydrogen embrittlement such that we do not have that experience again. And in hindsight, these are the types of requirements that we would have been well-served to have on the 2008 bolts. And this is what we would look at going forward for testing in situations when we were calling for these types of bolts. With that, I will turn it over to Andre Boutros from the CTC to talk about the retrofit to the 2008 bolts that I just described that were problematic.

ANDRE BOUTROS: Thanks, Malcolm. Mr. Chair, Commissioners, Andre Boutros, Executive Director of California Transportation Commission. If you recall from the last discussion, we talked about two options that we were investigating to put the retrofit in place. The first option was called a steel collar. And just before I describe that, let me give you a little bit, maybe, of an orientation here, too – what the colors mean. The orange colors here are what we're calling the bearing assemblies. The dark blue is the sheer key element that we're talking about. So that's really the element that's connected with the bolts that failed -- those are the 2008 bolts. The light blue is what's really termed here as the steel collar. That element is essentially clamping the existing sheer key in place, into the concrete on top of the columns. The bolts of the existing sheer key will all be abandoned; they'll be essentially taken out of the way so we can perform this retrofit. The yellow are elements that would be required to assist in that clamping force of that steel collar plates. So this was Option 1. And we have another option, Option 2, which was called steel saddle. And again, what you're seeing here is the bearing assemblies, the sheer key. And then the purple is the new element, which is now referred to as a saddle, that essentially sits on top of that sheer plate – sheer key plate. And that will be connected to steel cables that will be post-tensioned, and they will be essentially covered in that yellow section that you're seeing there, which will be additional concrete elements that will cover all these cables so no one can see what's there. And that will be performing the same function as, again, the original design in terms of providing the sheer capacity for that sheer key to resist the lateral forces.

So after looking at these two options quite a bit, for several weeks now, we've analyzed the options for both constructability, as well as cost, as well as pros and cons, and some of the pros and the cons are described on this slide. Option 1, in terms of pros – you know, both options actually – the pros are that we do not mess with the sheer key itself. We don't take it out to replace it or anything of that nature. For Option 1 the pro is, potentially it is simpler to fabricate because it is stacking steel plates on top of each other and anchoring them together and then into the concrete. As far as cons, there is a lot of steel here that would be required for that particular operation, and there will be a lot of milling and a lot of shop machining to make sure those plates are smooth and they can fit together. There's definitely more coring related to Option 1 into the concrete, that we're kind of concerned about in terms of potential damage to other elements of that cap beam. And then one of the cons is, of course, the cost, which is right now estimated at $15-20 million. As far as Option 2, which is the steel saddle, again there's no need to remove the key, there's less scoring required; it is potentially less difficult to install because most of the installation would be really external to the concrete, so we're wrapping those elements around the concrete cap. It is definitely less costly – it is $5-10 million. And then as far as cons are concerned, there is a special or unique saddle system that would have to be fabricated to make this all happen.

So, after debating both options, like we said, we're recommending Option 2, which is the saddle. It essentially, as would Option 1, provide the clamping force that would be required to hold down the sheer keys as originally designed. Again, the 2008 bolts altogether would be abandoned; they'll be taken out of the way where they conflict with the installation of the new assembly. Option 2 would require more detail fabrication; however, installation will be less difficult, as I mentioned, and would require less coring of the concrete on top of Pier 2. It is definitely less cost.

As far as the schedule is concerned, we're working with the contractor right now to – Caltrans is working with the contractor to make sure that this retrofit can be completed by Labor Day. We do not have an estimated time yet; this is in the negotiation process, and it will have to be worked out in terms of a Contract Change Order with the contractor. But we're hoping that this can be in place before the planned opening on Labor Day. And with that, I'm going to turn it over back to Mr. Heminger to describe the 2010 bolt issue.

STEVE HEMINGER: Thank you, Andre. So the remainder of this presentation will focus on the bolts that have not broken. And that's obviously an important distinction. Here again is Malcolm's slide, but with a little more detail and a little more color. And, as you know, we have been learning as we're going in this investigation, and I think one place where we provided you an incomplete picture in the last couple of briefings is on the east pier. So we want to give you a more complete picture here.

We have spent most of our time briefing you on what I will call the downstairs bolts – the bolts that have been connecting the bearings and sheer keys to the pier cap. And they are shown in the longer boxes that you see on this slide. The red ones, again, those are the 2008 bolts that failed, that Malcolm has described to you. There are similar bolts made in 2010 in yellow that are also downstairs bolts, and they are connecting the remaining bearings and sheer keys to the pier. One difference that we have emphasized to you in the past is that the 2008 bolts in red are embedded in the pier because of their location over the column leading down to the water. Whereas the yellow ones – the 2010 bolts – are not embedded; they're through-bolts – they go all the way through the pier cap and have nuts at either end.

What we have focused less attention on in our briefings to you, and we want to include today, are another couple of sets of bolts. The second major set of bolts are what I would call the upstairs bolts. They're the bolts that take the sheer keys and bearings and anchor them to the roadbed – what we call the OBG, the Orthotropic Box Girder. So the bearings and sheer keys are in between two very large things: a concrete pier cap beneath them, and a steel OBG that's above them. And as you can see here, in terms of the green areas, there are another 500 bolts, also manufactured in 2010, that connect those sheer keys and bearings – the upstairs bolts – to the deck that's above it, the steel deck. And above the steel deck is the riding surface that you drive on. In addition to hose, there are also an additional 400 or so bolts that are smaller. All the bolts that I've described so far are roughly three inches in diameter. The ones that go through the pier cap are very long: they're 15, 20 feet long. The upstairs bolts are much shorter, but they're as wide in terms of diameter, because they have a very small distance to cover just to connect the bearings and sheer keys to the OBG. The bolts that I'm now going to describe – the additional 432 bolts – are one inches in diameter. They're much smaller and, in fact, they vary from two inches to two feet in length. And they are internal to the bearing assemblies, the Bs that you see on this sheet: B1, B2, B3 and B4. And so that comprises the totality of the bolts that are on the east pier. It's about 1200 of them and, again, manufactured in different batches: 2008, 2010; some of them of different sizes; most of them – both the downstairs and the upstairs bolts – at a fairly high tension level and the remainder of the ones internal to the bearings at a slightly lower tension level. And as we've discussed with you, that's a relevant fact in terms of this issue of hydrogen embrittlement or the long-term problem, the long-term potential that we're concerned about with stress corrosion.

So, having that as background, what have we been doing? We have been testing these bolts. And as Malcolm indicated, we've been testing them not only on the site with some field equipment – and I think you've seen some of that testing going on in the press, which is good for the public to see it – but we've also been testing them in labs. And when you test this stuff in the lab, you break it. And so that destructive testing has been underway, and that breakage involves pulling them apart and then examining how they fail and then cutting it up and putting it under the microscope and doing all sorts of tests to it so you know what is going on. The most important finding, of course, is out there in the field on the pier, and that is that none of these bolts have broken. And so the longer they are there not doing that, the more daylight we are seeing between the 2008 bolts, which had a hydrogen embrittlement problem – and generally speaking this problem develops in the short term – and the 2010 bolts that do not seem to be developing this problem. The preliminary test results as well as showing them to be a more ductile or flexible material, not as hard, especially on the surface, as Malcolm indicated – which was a major culprit in the failure of the 2008 bolts – and we are not seeing any hydrogen embrittlement. That little break at the bottom, if you can see that circle there, if you remember the break that occurred on the 2008 bolts, from a prior presentation, it was a brittle break, and it was fairly clean. You see here that this break is much…I won't say dirty, because that's the opposite of clean, but it…what it reveals is something that pulls apart and is more ductile, more flexible, which is what you would like to see in this test so that it's not as brittle, not as hard. We are also anticipating additional test results after today that will be going on for the next few weeks, and then there's another test that I'll describe that will take a little bit longer than that.

Uh…lots of numbers here, and I beg your pardon about that. But I do want to focus on two sets of numbers here, which are at the far right of this chart. I think we showed you a version of this at your last briefing, primarily concentrated on the first two rows, which are the 2008, again, downstairs bolts, and the 2010 downstairs bolts. And if you move over to the right and you look at the hardness characteristic of those bolts, you'll see a difference. In other words, the 2008s are quite a bit harder than the 2010s, on average. What you will see above that is the ATS ASTM standard, which has a range there of 31 to 39, so the higher up you are in the range, the harder you are. And then on the far right is a new test that we have been conducting. It's a toughness test, and it's called the Sharpie test. And it essentially involves taking a device that's like a pendulum and swinging it into the material and seeing what kind of toughness that material has. You basically put a notch in the material so that when you swing it you can break it. And how far you swing it, how much you need to swing it into the material is an indication of the toughness. And here again you can see a significant difference between the 2008 and the 2010 bolts. This particular test you do it at two different temperatures because obviously steel gets more brittle when it's cold, as opposed to being hot. And what you see are the values, in other words the "swing width" I'll call it. And you can see for the 2008 bolts much lower values; for the 2010 bolts much higher values.

Now what we've arrayed on this chart that is new information also for you – the toughest information is new, and you can see we haven't finished the toughness tests on everything – but we've also arrayed for you the hardness information, as well as these other metrics that I won't dwell on in particular, for the other bolts that are on the east pier that I just described to you. And you can see – Peter if you'll help me with the cursor again – right below the 2010 bolts you'll see the E2 sheer key bolts, and again you will see a hardness value that is more consistent with 2010 than with 2008. And if you go down the list, you will again see values most of them closer to 2010 than 2008. The reason we have two ASTM standard rows on this chart is because they have a different standard the bigger the bolt. So the big bolts are on the top, greater than 2-1/2 inches in diameter, and the smaller bolts are on the bottom. So again, the data we're seeing is showing a divergence between the 2008 bolts that failed and the 2010 bolts that have not on various elements of the pier cap.

The other test that I mentioned earlier is something we're calling the wet test. And what it is, is it's a test that is trying to…I see some snickering going on. You'll have to stop that. The uh…we're trying to simulate a long-term phenomenon, which is stress corrosion. And I think we mentioned to you earlier that hydrogen embrittlement and stress corrosion, both of them are time dependent. In other words, when you do a test on a material, you don't see it in the material. You have to subject them to conditions that simulate a time-dependent failure. And in the case of the hydrogen embrittlement, as Malcolm indicated, what we're specifying for any replacement bolts is some kind of time-dependent test before you put them on the pier. For example, you take them into a lab and you put them under tension and you see if they break. We didn't do that with the 2008 bolts, and we should have in hindsight, because that would have probably shown us that if we had a problem, we had a problem in the lab instead of on the bridge after they were already included. And so that's the kind of test you need for the embrittlement question. For long-term stress corrosion, you're trying to simulate something that occurs over years or decades, and that's considerably more difficult. So we have come up with a…a…really a custom test that is going to try to simulate that longer term susceptibility to stress corrosion. We're going to take full-sized bolts off the pier from the 2010 batch, and they're going to be basically placed into a bath of a salt solution and tensioned progressively over a number of days until they fail. And from that test, we will be able to learn the particular susceptibility of the 2010 material to stress over time. And that will give us a lot of good information about how to deal with those bolts in the future, whether they ought to be replaced in the near term, whether they ought to be replaced in the longer term, whether we ought to reduce the tension on them if we can, given the design of the bridge. We really do think this test will help guide what we do with these bolts in the future and over the life of this new structure.

Stress corrosion…it's worth just a little bit of a digression to emphasize the importance of the metrics that I've been describing to you. And this curve is probably the best picture we've got of what you need to worry about with stress corrosion. And on the bottom axis is hardness, and again, the harder the material is, the closer that line is to, uh…again, the line shows you two basic parts of this…fields in this graph. Below the line is good. Below the line you have less likelihood of failure. Above the line you have more likelihood of failure, or fracture as it's stated there. And so the axis at the bottom, the horizontal axis, is a function of hardness. The axis over there on the left, the vertical axis, is a function of two things: the tension level that the material is under and the size of the material. And size here does matter, and as you go up that scale, you have more of a problem. As you go right on the horizontal scale, you have more of a problem. So if you have a very hard material, you want to have it at fairly low tension. If you have a material at very high tension, you want to have it be not so hard – you want it not to be subject to the brittle condition that can lead to the failures. This, by the way, is from the textbook of John Fisher, who is one of our experts.

So, those are the bolts on the east pier, again, about 1200 of them. There are another, roughly, thousand bolts that are 354 BD galvanized on the rest of the SAS, and what have we done about those? As I think we informed you at your last meeting, we have been conducting visual inspections of those bolts for some time now on a regular basis, and they are performing as they are required. Now, it's worth noting and emphasizing as we did last meeting that some of the E2 bearing assembly bolts – the ones that are inside those bearings – we cannot inspect because they're in there. Now they were observed before they were put in there, and the bearings were closed up. And we are tracking down, as we've shown you here, their quality control data. So we know quite a bit about those bearings. The one thing we can't do with the internal bearing bolts is pick apart the bearings and look at them. But we have the observations we made before. We also have the quality control information that we are assembling. And as I indicated before and want to emphasize again, most of the bolts at the other locations off the eastern pier are at much lower tension levels, and that is a very significant difference that they exhibit from the others. And, in fact, here is a roster of them all, and working your way from top to bottom, you'll see the first several sets are the E2 bolts on the east pier, and then you have some inside the anchorage, which is toward the eastern end of the bridge, some at the top and the bottom of the tower, the suspension tower, some in the east saddle, some in the east cable, some in the west pier where there is also a saddle. Even the bikers get their own anchor rods. Everybody gets anchor rods on this bridge. And the relevant characteristics for you to look at on this chart we've depicted, first of all the diameter, and the bigger they are the more difficult they are to manufacture and also the higher up they go on that chart that I showed you in terms of stress corrosion potential. Length is not as important, although, you know, the longer they are, that's a bit of a manufacturing challenge, but not as operative as the diameter. You can see the quantities depicted here, and it adds up to about 2300 bolts on the SAS – again, this is contained to the self-anchored suspension bridge, the steel element of the bridge. The vast majority of the length of the new bridge is a concrete skyway – no bolts. And finally – and we've highlighted this is purple – that's the tension. And, as you can see, it's a fraction of FU. FU is not what you think it means. It's its ultimate capacity – in other words, when it would fail. And you can see that on the east pier, we've arrayed these from roughly more tension to less tension. On the east pier is where we have the highest tension levels, which is why this issue we have focused so much of our attention on the east pier. As you head down the chart, you see much, much lower tension levels, in some cases what we would call snug tight, just as you would snug up a bolt anywhere in your garage. Next one, Peter.

As part of our investigation, we have also asked Caltrans to review the other toll bridges in the Bay Area and whether or not they have used any of these 354 BD galvanized bolts, either in new construction or in the retrofit program that we've had underway. And Caltrans has identified one location where they were used on the Richmond-San Rafael Bridge retrofit. Caltrans has been inspecting that area on that bridge for some time and so has records, and they also went out and did some visual inspections over the weekend. We have observed no performance issues. There are a couple of very important points to make about the Richmond-Bridge bolts. The first is they were installed well before the bolts that we're talking about here on the SAS. We are going to track down their quality control records and the like. But we're not certain where they were manufactured, but they were put in well before these bolts at issue. I think the most important difference is they are also snug tight. They are not under any real tension load at all, and that tension is really what matters in terms of near-term embrittlement, which they do not exhibit, as well as long-term stress corrosion potential, which they should have a very low susceptibility to. But we're going to do our due diligence on these, just as we're doing on the SAS, just to track down the quality control and continue inspections to make sure we're okay.

So, what's left? We do appreciate your indulgence in us sort of barging into your meetings every couple of weeks. And we're going to have to do it at least one more time. And I appreciate that Chair Worth has agreed to call a special meeting of the Bay Area Toll Authority at the end of the month, so we won't be barging into a regular agenda; we'll be on our own, by ourselves. And at the May 29th meeting, which is when it will be scheduled, we believe we will be able to give you a decision on the 2010 bolts because we will have all the relevant test information we need, with the exception of the wet test. And the wet test is something that we can use to gauge our behavior going forward. It's not something we think we need immediately to make a decision necessarily about what to do about the 2010 bolt locations, especially pier E2. And we will have also completed all the document review that we need, not only for the bids, the two bids that were conducted to build this bridge, but all the quality control information that was provided.

If I could, in closing, just draw your attention again to that letter to the Federal Highway Administration because I think it gives you three helpful categories into which we believe we will be classifying the bolts on the SAS in terms of our future activities on inspecting and repairing them. The first category we intend to classify bolts into is bolts that are to be replaced before opening the bridge to traffic. And we already know that the 2008 bolts fall into that category. We do not know yet whether any other sets of bolts fall into that category, but we will be able to tell you that, I believe, on May 29th. Secondly, is a category of bolts that are to be replaced after opening the new bridge to traffic, as a precautionary measure to avoid premature failure due to stress corrosion. So what that means is we may decide to replace some bolts that haven't broken. But we may decide to do it as a precaution to avoid a concern about stress corrosion in the future. And finally, there will be bolts, we expect – and we expect the vast majority of them will probably be in this category – that are to undergo a regular inspection schedule and only be replaced as necessary if damage is observed in routine use. And one helpful way, I think, of starting to think about how we might differentiate is between the bolts that are on the east pier under high tension, and the remainder of the bolts on the bridge. That is the drift of the data that we have received to date, but we do not have all of it yet, and so do not want to make that judgment until we do.

And so again, Mr. Chairman, that concludes our presentation. We appreciate your patience, and we're available to answer your questions.

BILL DODD: Thank you, Steve, Malcolm, and Andre. Appreciate that report. We're going to start off with Commissioner Campos.

DAVID CAMPOS: I know that the Chair wants to start….

AMY WORTH: Well, I just had a couple of quick questions.

DAVID CAMPOS: Yeah, I do, too.

BILL DODD: You're the Chair.

DAVID CAMPOS: I'll defer to the Chair.

AMY WORTH: Thank you, sir. I have several, but if I could just start with a couple of them. I just wanted to clarify this Option 2, Steel Saddle, and try to kind of reconcile it with another photograph that we got initially. It shows the 2008 bolts that have failed, and then it shows the 2010 bolts on either side. So is it clear to say that this Option 2, Steel Saddle – the 2010 bolts would continue to be in that system? And basically what this does merely is replace those 2008? So we have to visualize the other ones also in it?

ANDRE BOUTROS: Right. So, um, the 2008 bolts will be abandoned. So essentially we will not be counting any capacity of any of the bolts, so ignoring them altogether. Some of them will be cut on top, at their connection on top of the plate to allow for the saddle to ride the plate, essentially. So it can sit on top of the plate and hold the plate down. We're not taking them out of the concrete, because there's really no point of doing that. We may be causing more damage to the concrete, you know, trying to get them out because they are embedded; we can't get to them from underneath like the other bolts on the bearings, as an example.

AMY WORTH: Okay. And then the 2010 bolts will go in as designed on either side of those, where the…

ANDRE BOUTROS: The 2010 bolts, you know, they're essentially anchored on top of the bearing assembly, and they're anchored at the bottom of the concrete. So we can get to them from either end.

AMY WORTH: Okay. And I guess the other thing, I wonder if you…you know… appreciate the information on the retrofit. Can you talk a little bit more about the process that you went through and tie this within, for example, we've got these, you know, we've got additional experts on our, you know, that Steve identified in terms of giving us perspective on this. Can you talk in a little bit more detail about how the design process went and how the, you know, the independent experts weighed in on, in helping come to the conclusion that this was the best way to go?

ANDRE BOUTROS: Sure. Let me start, and maybe Malcolm, you can? Essentially the design engineers – which is TY Lin International – as well as assistance and feedback from the contractor and Caltrans, of course – a lot of engineers in Caltrans are performing oversight on top of this – have determined that any assembly that will be put in place will have to resist the same loads that this structure will be subjected to regardless. So that was really the starting point, and that's what we need to keep in mind, that we're not really diminishing any capacity or any intended behavior of this bridge by this retrofit. This retrofit will provide the same design capacity that was intended. So that was really the first thing. And designing sheer keys is essentially really taking a deflection, a movement of the bridge into account. So you're limiting the movement of the bridge by putting the sheer key in place. So there are allowances that the design had put into place when the original design was put in, so those same allowances came into play. And I may refer to Malcolm here, because this is getting a little bit more technical for me at this point.

MALCOLM DOUGHERTY: If I could just add, both alternatives were designed to a 65% design stage, such that we had enough information to make a decision on the pros and cons and which one to move forward. As Andre pointed out, our consultant of record, TY Lin, as well as Caltrans engineers were involved. And then both alternatives were also provided to our peer review panel to comment on both of them and offer up information to the TBPOC before a decision was made on which alternative was the best one to move forward with. But both of them provide the functionality that we need.

AMY WORTH: So…and I guess a final question…so it sounds like there was kind of complete concurrence by everybody we talked to, including the independent review panel, that this option was the best way to go in terms of protecting the integrity of the existing structure, but providing the work that we needed.

MALCOLM DOUGHERTY: That is correct, yes.

ANDRE BOUTROS: Absolutely. And really, just to give you more assurances from our standpoint as the POC here, we would not be presenting this solution to you here if it weren't providing the solution that would provide for that safety for that intended design for that behavior of that bridge.

AMY WORTH: Absolutely, and I understand that. What I think we just want to understand is, kind of, what the process is when you go back to design an element. So, who is weighing in on it, who is reviewing it, and that kind of thing, so, um…and I guess I have one other question, and we can take the answer now or later after we focus on this. But I wanted to see…one of the other things that has come out over the last week are questions about the welds in the tower. So I don't know if you want to answer that now or prefer to handle the remaining parts of the report, you know, the presentation, and then go back. Whatever is best.

STEVE HEMINGER: If it's your pleasure to answer it now, I'd like to call Brian Maroney up to the podium. Brian, I think…at least Commission Spering will remember Brian because he's been with us from the get go on this project, and he is our go to guy on this bridge and, in fact, was one of the folks who were instrumental in persuading the Governor at the time to build a new bridge. And I'm sure there are days when he regrets that, like I do, but it was the right decision. And Brian has been involved in all the major innovations of the bridge, and this welding, in fact, is one of those innovations.

BRIAN MARONEY: Commissioners, thank you for…good morning. Thank you for letting me have an opportunity to talk about that. With respect to your question, I guess I'll just start off with the self-anchored suspension bridge literally has miles and miles and miles of weld, in the tower, in the deck…just all different types of welds. Contractors will select a type of weld based on what's going to be best for that particular type of weld, and it might be because it's flat, it might be because it's going up and down vertical, it might be because it's overhead, it might be because of its size, or the weld material that a contractor will have to melt as they connect pieces of steel together. The base of the towers – you're probably referring to the electroslide welding, is that right? That is one of many different types of welds on this job. The tower – if I can paint a little picture here – the tower is basically four different tower legs. And that's where all the self weight is supported, and that's where all the live load – and we call live load things like trucks and cranes and cars and bicycle/pedestrian weights. So the tower, four tower legs, they carry those kinds of loads. We call them service loads. Now in event of a large earthquake down at the very bottom – and again, these towers are, these four tower legs are about 500 feet tall, round numbers – well, the bottom, about 30 feet or so, the tower legs are connected with sheer plates. These sheer plates are on the order of about, about four inches thick; they vary slightly. And on every weld on every bridge I've ever worked on, when you lay down miles and miles and miles of weld, we, by contract, force the contractor to do their own inspection. We call that quality control. Now we have experience. We know that that's not even enough. And we have our own people, our own inspectors – qualified – and we have our own equipment. We bring in our own equipment, and we re-inspect those to various degrees, depending on the geometry, the type of weld, its function of weld, following American Society of, um…AWS Bridge D15 Code, the Welding Code. And then we do that. Well, when you inspect…when you perform miles and miles of weld and then you necessarily inspect it, we inspect it because we know there is no mile of weld that's perfect. There are always imperfections. Now those imperfections, some of them need to be cut out and repaired. Others are imperfections that have no consequence whatsoever to the performance of the bridge. So like, for example, when you fly an airplane, okay, there might be a small imperfection in the wings, and it doesn't stop the airplane from very safely, successfully flying from San Francisco to New York and back. So with respect to those welds, we have found some imperfections due to that inspection – the contractor's inspectors and our own independent – we currently have mapped three dimensionally out the imperfections, and some of them we've already communicated with the contractor, these are going out. Please remove them, and then we'll follow up and inspect them again. Now, even if we have to tear them out and fix them and inspect them again, if we have to do that three times, we'll do it three times. We'll do it, and when we leave they will be fine. Now there will be some that we evaluate we agree, consensus, that yes, this is an imperfection but it has no consequence whatsoever to the performance, and we're in the process of doing that now. And it's just, quite frankly, it's a very large weld; it's a very exciting type of weld, electroslide welding. Federal…FHWA has been investing for about 15 years, up at Portland State, research, and many states do allow it, and we're excited to see this kind of technology coming to here in the state of California. And when we leave, this weld will be just fine. But we're in the process of doing what we do. We've probably been working on it for about, I'd say nine months. Are there any follow-up questions?

BILL DODD: Chairman?

AMY WORTH: So, basically we have the four towers that you mentioned, and having been out there, I saw the way it comes down. When they were originally put in, there were just two of those sheer plates because then you had to install the rest of the towers, and then…are the sheer plates installed around and then those are basically welded together? Is that, kind of….?

BRIAN MARONEY: The legs…the five-sided legs, each one of them, they were fabricated and lifted and brought in. And then additional plates were brought in, and they are not service levels. They are only there for large seismic events. And then we basically connect all the four tower legs with these sheer plates down at the bottom 30 feet.

AMY WORTH: Okay. Thank you. Thank you very much.

BRIAN MARONEY: I should also add that I actually brought out the independent peer review panel, and they've actually been out there more than once. They have…they review our weekly notes and progress on this. I've taken them out there. I've actually given them demonstrations on how the inspection technologies that we use. So it's fully vetted.

AMY WORTH: Thank you.

BILL DODD: Commissioner Campos.

DAVID CAMPOS: Thank you, Mr. Chairman. I want to begin by thanking the Oversight Committee and the staff of all the different agencies, including the MTC staff, for all the time that has been spent on this item. I also want to thank the Chair of the MTC. It's very clear to me that the MTC is playing a very important role in making sure that there is transparency and accountability with respect to where we are here. So I want to thank our Chair, I want to thank our Executive Director. I'm very proud of that role. You know, to paraphrase the Governor, you know, to use the exact words, stuff happens. And I think that when stuff happens, it is really important for us to be very transparent and forthcoming in the information. I have to say – and I can only speak for myself – one of the things that's frustrating about this process is that I feel that we have actually gotten more information from the press in terms of some of the things that have happened than from this process. And I actually want to thank the press, whether it's Matier and Ross or Mr. Vanderbeek, and they have done a great deal of service to the public in terms of airing out where we are…in the East Bay as well. I'm sorry, the Times as well – my apologies; didn't want to leave anyone out. But the first time that this was presented to us, the impression that I got – and maybe it was just my misunderstanding of how Caltrans was talking about It – it was described as sort of maybe a manufacturing issue, and the way in which these bolts were manufactured. And maybe that's, in the end, part of the issue. But there was no discussion about how these bolts were selected to begin with. And I know you were talking about the wet test. You know, I have a very simply mind; I'm not an engineer. I follow the smell test, and what doesn't, to me, pass the smell test is how it is that Caltrans chose these bolts. And so I want to ask Caltrans a little bit more about that. Uh, what is the difference between an A354BD bolt and an A490 bolt? Let's begin with that.

MALCOLM DOUGHERTY: I appreciate the question and the opportunity. I don't know that we varied substantially from the initial assessment that it's a materials issue. Whether or not the materials issue and the susceptibility in the hydrogen embrittlement was born out in the processing or why it was in the field, I think there are…that is kind of captured in the metallurgic report as best it can. The difference between a 354BD and a 490 has to do with the size of the bolt. When they go to ASTM standards – and I can actually talk about a 354BC; it may also help. A 490 bolt is a bolt that is limited in its length. It is also limited in its diameter, and it also has a hex nut at one end of it. An A354BD bolt is a…is for bolts or anchor rods that are larger than 2-1/2 inches in diameter. They are similar – a 490 and an A354 – in the strength requirements and some of the tensile requirements, and those types of things. But they are different in size and magnitude. So that's one difference. Difference between an A354BD and an A354BC is the tensile strength that the two different steel anchor rods have. So you get yourself into a point where you're trying to decide how much tensile strength you need, what size it is, what function it's going to serve, and then it fits into a different category of a bolt type. But these are not 490 bolts because 490 bolts are limited in size, length and diameter.

DAVID CAMPOS I understand that. So with the 490 bolts, are 490 bolts allowed to be used in these kinds of bridges?

MALCOLM DOUGHERTY: Sure. In the right application, the right size, the right specifications, if those specifications met the application, sure.

DAVID CAMPOS: And is there a limitation in terms of that use? I mean, are there things that they're not allowed to be used for?

MALCOLM DOUGHERTY: Uh, there are differences in the specs. I think you may…490 bolts aren't allowed to be galvanized.

DAVID CAMPOS: Okay. And they're not allowed to be galvanized because….

MALCOLM DOUGHERTY: Those specifications are lined out in ASTM. I think there is a concern about hydrogen embrittlement, but I don't what other factors led to the direction in ASTM not to galvanize 490 bolts.

DAVID CAMPOS: Given that there are some similarities between the two types of bolts, and you're talking about the 490 bolts not being allowed to be galvanized, did that raise any red flags for Caltrans in terms of moving forward with the use of the A354BD bolts?

MALCOLM DOUGHERTY: I think it certainly…and our specifications called out special procedures to be followed with the use of the A354BD bolts. But I don't know that was brought to our attention by the 490 specifications. It was brought to our attention by the 354BD specifications that have a cautionary note about galvanization and actually have a process outlined if you are going to use these and galvanize them what steps to take. Another difference, going back to 354BC bolts which are a lower tensile strength, there are no restrictions on galvanizing those bolts.

DAVID CAMPOS: Now in your presentation you say that current Caltrans bridge specifications do not allow the use of galvanized A354 bolts for standard bridge applications. Uh, but then it says that it is allowed for non-standard applications. Can you explain the difference?

MALCOLM DOUGHERTY: Our standard direction to our bridge designers is not to galvanize 354 bolts because if you are, you have to be very thoughtful about it. It is within ASTM standards, but our standard direction to bridge designers, as a basic principle, is to not galvanize 354 bolts unless you do due care and add special provisions, which we did in this case. It's not uncommon to have…to vary from standard specifications, and that's what was done in here. And it's very common to have specific specifications for an individual project.

DAVID CAMPOS: I guess, not being an engineer, I don't understand how it is that you would deviate from the standard specifications that you have. If, generally, you're saying you should not be galvanizing these bolts, why would you do that with respect to this project?

MALCOLM DOUGHERTY: Our standards also tell us not to have less than eight-foot shoulders on a two-lane highway, or not to have less than ten-foot shoulders on a freeway, but we do in certain circumstances, and we are very thoughtful about when we do that. It's not that much different in a technical application. You have to stay consistent with ASTM, I think, would be a good rule of thumb. But we vary from our specifications, whether or not it's regarding bolts, whether or not it's regarding geometry; but you have to be thoughtful about it when you vary from the specifications. When we did that in this case, we were within the ASTM requirements, and we added what they called out to do if you were going to galvanize these type of bolts.

DAVID CAMPOS: And what does it mean to be thoughtful? I'm still trying to…so, let's say that you're going to deviate from your standards. You're going to galvanize these bolts. What is it that Caltrans requires in terms of being thoughtful when you're deviating from their own standard?

MALCOLM DOUGHERTY: I think that…back on Slide 14 it described a couple of bullets that talked about the technical design specification team evaluating the bridge design and the specification. So there would have been thorough conversations about what specifications to use. And there should be documentation going into why the…what the logic was to vary from those specifications.

DAVID CAMPOS: Is the wet testing that was talked about, is that part of the process of being thoughtful – something that you want to do?

MALCOLM DOUGHERTY: Well, let me add to this, and then Steve may also…let me answer and Steve may want to add to this as well. One of the things that I think that is very apparent is when we were ordering the replacement bolts for the 2010, we have put a lot tighter specifications and a lot more testing requirements on those replacements. And, in retrospect, that would have served us very well to have those same tighter specifications as far as additional testing when we called for the A354BD bolts to be galvanized.

STEVE HEMINGER: Commissioner, if I could, the wet test is something that we are essentially inventing. And it's trying to deal, and simulate this issue of long-term stress corrosion. I think the more relevant concern here is the short-term hydrogen embrittlement problem. And, at least in my mind, there's no question that if we had specified with the 2008 bolts that we do a time-dependent test in the lab before we put them in the bridge, we would have been a lot better off because I think it's likely they would have failed in that test. And we wouldn't have installed them on the bridge, and the complicating factor, as you know, is we installed them in a location where we basically couldn't get them out again. So, that, in addition, I think should have cautioned the folks at the time to say "Geez, if we can't get these things out again, shouldn't we test them off the bridge before we put them in?" And so that is, I think, one of the clear lessons learned, and it's on the slide that looking back, you know, I think as Malcolm indicated the process – and we have yet to get all the documents in front of us so we can actually look at the minutes of the meetings, what was discussed, what happened – but once we have that, I think the question really is…they clearly were aware of the issue because the prohibition on picking was included in the specs. But we think there should have been more included in those specifications, which is why we're doing so in the replacement bolts that we're ordering.

DAVID CAMPOS: Just following up through Caltrans. So, you deviated from your specifications in terms of the galvanizing of these bolts…of the A354 bolts. Is there any other area in the construction of this bridge where Caltrans deviated from its own specifications?

MALCOLM DOUGHERTY: I'd have to actually defer to Brian Maroney and the design team, which included our consultants. I…from a technical standpoint, from a geometric standpoint, I don't know that I can answer that question definitively without going back to the design team.

DAVID CAMPOS: And I think that's part of the concern here – that we see, based on what happened, what does occur when you, in fact, go down the path of deviating from your own specifications, and some of the challenges that come with that. And so I think it's fair for us to ask if it happened with respect to these bolts where else with respect to the construction of the bridge did it happen?

STEVE HEMINGER: Commissioner, if you'd like Brian Maroney to try to address that….

DAVID CAMPOS: Thank you.

STEVE HEMINGER: …I'm sure he could.

BRIAN MARONEY: In 25 years, as a bridge engineer, I've never worked on a project – in design or construction -- where we didn't have something we call special provisions. Think of a package for a plan for a bridge. There are a bunch of plans, and those are the drawings. And then there are the specifications, and those are kind of like the directions – how you do it, what you can do, what you cannot do. And there's two types of those directions. One are the standard specifications, and it's a book that we use and it's, you know, hundreds of pages long. But every single project has something that we call special provisions. And it's because those standards don't really fit, and you have to come up with an engineering technical solution to how you still get the performance of the bridge structure that you need. The Bay Bridge – like every other bridge – has many, many, many, many special provisions, or many, many, many special deviations away from the standard. Like, for example, um, let me give you a simple one. As you come out of the tunnel, the roadway on the west spans of the Bay Bridge, the roadway in the tunnel it does not meet our standards. It doesn't. And the brand new bridge that is on Yerba Buena Island as it transitions to the brand new self-anchored suspension bridge and the skyway structure, that brand new bridge, it also doesn't meet those full lanes because it's connected to substandard lanes, and then it transitions out to larger ones. The superelevation is the same way.

I personally was involved in one such deviation. Caltrans standards at the time of the design of this bridge was to use something called maximum credible events. And that sounds like a very interesting term and a confidence building term – maximum credible event. But as a bridge engineer, and somebody who specializes in earthquake engineering, I can tell you the maximum credible earthquake motions can be exceeded. They are a statistical characterized set of ground motions. I, quite frankly, led or drove a variation to use "probablelistic" motions – 1500-year return "probablelistic" motions. A typical building in San Francisco is designed for about a 500-year return period motion. Outside of California, AASHTO has fully adopted probablelistics, 1000-year return period motions. Here we're using 1500. Now, how did I make sure that this was an improvement? First of all, I augmented the team with specialists in this area – people like Bruce Bolt, people like Norm Abrahamson, people like Joe Canzeen…these are the people who design the ground motions for nuclear power plants here in California. These are the people who design the ground motions for things like dams in Southern California. These are the people who design the ground motions for Yucca Mountain, where the world keeps the most nasty things. When I develop it with the proper skilled professionals on every single one of them, then I have to vet it out through my external independent group. So, typically – at least when I'm involved in it – I'll develop it with the folks that have the proper background, and then I'll elevate that to my external independent peer review panel on ground motions. So there are many, many of them, and every job is like that.

DAVID CAMPOS: Well, thank you. I just have a couple of final questions. And I guess one would be for Steve. Steve, I actually think it's a great idea that this letter was written to the Federal Highway Administration. And I guess the question that I have is, in terms of ensuring the safety of the bridge, you know, who has the final say, and what role will the federal government have in, sort of, the technical guidance on that issue?

STEVE HEMINGER: Well, under the statute, this committee – you're looking at the three of us – have been overseeing the project. And we have selected the retrofit solution you've seen, and we're going to proceed with it, as we've proceeded with many things. Now, at some point, as I indicated in this letter, you need to make a call about which bolts fall into which categories. And I think the most important distinction is between the bolts that you might want to replace before you open the bridge and the bolts that you might want to replace, or monitor, after you open the bridge. And we've already made the call on the 2008 bolts, and I don't think there's any debate about that. We can't open the bridge without them because we need those sheer keys to be operational. I think there may be a debate about whether other bolts should be replaced before or after. And I think the operative question there goes back to the picture at the beginning of this presentation. Do you want to have people traveling on a bridge you *know* is unsafe in the event of a major earthquake while you replace bolts on the new bridge that haven't broken? I'm not sure you want to say that is your answer. But it's a debatable question, and I think we're dealing here with not only engineering concerns, but public confidence. And I think that public confidence has taken a beating over the last few weeks. So we are mindful of that. To answer your question directly, we are asking for the Federal Highway Administration to bring their expertise to bear, that once we make a judgment on one, two and three in this letter, we want to see if they concur.

If they do, then I think ultimately this bridge is owned by the State of California, and the head of the State of California is the Governor. I think his office will eventually make whatever call needs to be made about opening the structure. We will advise the Administration when we believe it's ready. But I think that will ultimately be a judgment of elected officials, as it should be. If the Federal Highway Administration doesn't agree, then we've got a new problem, and we'll have to go through that. But the purpose of an independent check is not to necessarily get an answer that says you're right. It's to be willing be told that you're wrong, or that there's another way of looking at the issue. And we're willing to do that.

DAVID CAMPOS: I think that makes sense. I think having that independent judgment is important. Last question: given where we are right now, given that we're talking about this federal agency also coming in, what does it look like in terms of the opening of the East Span?

STEVE HEMINGER: Well, you'll notice, Commissioner, in the presentation we didn't give you any guarantees, so I'm not going to give it to you orally either. We do believe when you look at the volume of work that needs to be done for the retrofit of the 2008 bolts, that that work can get done by Labor Day. Now, it will probably involve extra shifts, and perhaps even 24-hour-a-day operation, and that will tend to cost you more money. But…so, on that question, we do need more work with the contractor, as Andre indicated, and we're not going to be comfortable giving you a forecast on that, I don't think, until the end of the month. I think the wild card, again, is whether or not there needs to be any other work done before we open the bridge. And, again, we hope to give you our best sense of that at the end of the month, because – especially on the east pier – we need to make a call about the remaining bolts on that pier, as Chair Worth was driving at. Do we need to replace those 2010s right next to the 2008s now, or can we replace them later, or do we watch them for awhile and decide at some future point to replace them? This bridge, as soon as it opens, it goes into a maintenance program, and we've got some early candidates for a lot of attention during that maintenance program, and it's all these bolts.

DAVID CAMPOS: Thank you very much.

BILL DODD: Commissioner Cortese.

DAVE CORTESE: Just a couple questions. One is just a general question that's been on my mind since this started…this process started. And it admittedly comes from my upbringing as a farmer and being around people who were involved in building heavy equipment and doing an awful lot of welding. Besides the enemies of welds and bolts that we've heard about here, the one that I learned about as a youth was vibration. And I'm just wondering…we haven't heard much of anything about that…if we're satisfied that vibration itself isn't part of the causation of what's happening out there. I'm not dissatisfied with those things that have been pointed to by the investigation thus far, as part of the problem. And ultimately, I suppose, if you replace bad bolts with good bolts and flawed welds with good welds, you know, somehow you're taking that into account. But I just want to be reassured that whatever engineering has been done, it's taken in the tremendous amount of vibration on that bridge that's going on.

MALCOLM DOUGHERTY: Um, I'm certain that if I don't get into as much detail as you'd like, we can invite Brian Maroney back up and he can talk about it. But, as far as the welds go, as far as the bolts go, if we do not have a hydrogen embrittlement problem with the bolts, for example, if we don't have a long-term corrosion problem with the bolts – which is exactly what we're evaluating now – we have taken into consideration the vibrations of live load and the vibrations of wind and weather into the design of the bridge. That is absolutely an important component of the design of the bridge. We have to make sure that we have the components that are going to perform as we designed them. And I think that's what we're trying to get to the bottom of. The 2008s are already out; the evaluation continues on the others. But once we determine that they're going to function as we requested them to, or require them to, they will absolutely take into consideration vibrations of live loads, winds, and everything else.

STEVE HEMINGER: Yeah, and Commissioner, if I could mention, I asked a very similar question of John Fisher at one of our workshops. And that was his answer, that the stress loading is far more important than any of the other live load conditions that you might encounter.

DAVE CORTESE: Again, I appreciate the response, and I just wanted that reassurance that that discussion, that analysis has been going on. I'm sure a lot of other things have been looked at that you haven't had time to present here, but that was the one I was interested in. The other question I have, if I may, is…I'm leaning toward a comfort level – personally, as one Commissioner here – with regard to those bolts that have, that are performing, that haven't failed or that show no signs of failure, or expectations of failure, you know, to deal with them through this rigorous, ongoing maintenance type of a process. And it sounds like we're going to hear more about that option later, not today. The one question I'd have is how do you deal with the ones that you described as being embedded in the bearings that you can't pick apart. I mean, it seems to me if you can't pick them apart, you can't see them, ongoing maintenance and ongoing testing, it still…it's still sort of the fly in the ointment.

STEVE HEMINGER: Commissioner, you're right. And one thing is we don't have the full pedigree on those bearings yet, in terms of their manufacturing history. And that's, as you recall, what we've been doing on all the other bolts. So we want to advance that practice and that base of information so we have it. You'll recall that Malcolm brought a couple of phone books to one of your briefings that we had assembled on the 2008 bolts, and so we want to review that information on these bearing assemblies. Secondly, again, they are considerably smaller in there. They are at slightly lower tension. And they were observed before they were put together. So we, again, have that sense of assurance that we did not have such an immediate problem. I think what you probably do over the long run – and I think this bear some further thought on our part – but what you probably do over the long run is just again observe the performance of those bearings. The bearings do have an everyday use, you know, as the steel expands and contracts, they're there to make sure that that's occurring. And at some point, clearly, if you had sufficient failure inside the bearing, you would be observing that they're not performing as they ought to. And at that point you might need to tear one apart and inspect and do repair.

MALCOLM DOUGHERTY: I think the only thing I would add to that is, not only do we go back and look at the material and the testing and the observation we had in the installation, we would go back and look at the risk that we were facing. Many of those bolts are snug plus a quarter turn, so the tension or the stress that we're going to put those under is just not going to be in the same category where we're concerned about susceptibility. That will be another thing we look at when we talk about all those bolts.

DAVE CORTESE: Sounds like the plan may be to test the bearing, rather than the bolt, and see if there's problems there. It would be interesting to hear a little more detail later about what that test will actually look like, so we can explain it to constituents and others that are asking us.

STEVE HEMINGER: We'll get back to you on that one.

DAVE CORTESE: Let's see…if we can't make it by Labor Day, we should pick Thanksgiving because we're all going to be a lot to be grateful for once this is over. That's an unofficial recommendation, by the way.

BILL DODD: Commissioner Spering. Let's see if you can top that.

JIM SPERING: Thank you, Mr. Chairman. I just want to commend the staff for the process that's been put in place. There was a little criticism about we haven't been forthright with the information. You know, we really don't have the luxury of speculating and making premature announcements, and I think the staff has been very responsible, going through a process of testing and analyzing what is the problem. And I think we have to appreciate that, and that does take time. And the development of this solution, it's clear it has to be vetted; it has to have peer review. These things take time. And I know there's a frustration on the part of the press. And much of the stuff we've read *has* been speculation, and we just don't have that luxury. And I just wanted to commend the staff, that I appreciate what you're doing is the right process. I think it's been vetted, I think it's under complete Sunshine, and, from my perspective, I think it's been done in a very responsible way. And I want to thank the staff. And if it takes additional time, so be it. I think it's important we take the time to find out what caused the problem, and this is the right solution. So I commend the staff.

BILL DODD: I'd like to associate my comments with Commissioner Spering. Well done. Anybody else? Commissioner Quan.

JEAN QUAN: I'm just going to…I understand that basically, just this one sentence…that it sounds like on May 29th we'll decide whether we can make the Labor Day holiday or not?

STEVE HEMINGER: I'm going to say I hope so.

JEAN QUAN: I think that if we can't decide then, that we really then have to start discussing alternative dates.

STEVE HEMINGER: And, look, maybe I can just take the opportunity, because I know there's some concern publicly that Labor Day's important because there's a party associated with it. Labor Day's important because it's a long weekend. And I think you're aware we have used several long weekends to do very important construction work on this project. And this one, we probably need not three days, we may need four, we may need five, because we are tying this new bridge into where the old bridge is. And so there's…it's not just restriping and moving some cones around. There's heavy construction work that needs to occur, and so that's why we have targeted Labor Day. Personally, I wish it could have been two Labor Days ago, because the urgency, again, with that picture, is to get traffic onto this new bridge. So, as, I think Commissioner Spering and the Chairman indicated, we are working as quickly as we can, but we need to be deliberate. And that's why I want to tell you I hope so, and not promise it.

BILL DODD: Commissioner Glover.

FEDERAL GLOVER: Let me be very clear as one member of this body, is that what I'm looking for is a safe bridge. So if the timing is not Labor Day, that date is not important to me or to the constituents in which we serve, but that we have a safe bridge to go across. And so I just want to say, as one member of this body, that when you're doing your work, be deliberate, but not to cut corners in order to meet a date.

BILL DODD: Commissioner Worth.

AMY WORTH: Yeah, I just wanted to echo the thanks to our staff, and also to the Commissioners for your willingness to have a special meeting at the end of the month to go over this, and really the thoughtful questions that we're asking. And much of it is we're lay people, like the public, so that we can all collectively understand both the source of the problem and the recommended solution. And I appreciate the fact that each meeting we come to, we get more material, and I think that reflects the seriousness with which the Commission is taking this, as well as staff, in terms of conveying the information. So I know the…I think the other thing it's important to remember, from the standpoint of some of the questions that came during the day, the next steps for the May 29th meeting focus no some of the issues about schedule and documentation. And, again, I appreciate your deliberate, kind of sense of really doing the homework before…in order to really provide thorough and thoughtful answers to the questions. So thank you very, very much. And thank you to everybody for their ongoing work on this.

BILL DODD: All right. Last speaker on this is Commissioner Bates.

TOM BATES: Thank you. Thank you very much. I just want to say that I really appreciate the fact that we're seeking independent review. And I think that's reassuring to the public, and I want to thank the staff and people for doing that, because I think that's the real issue. People need to know that we're not doing this just to get it done. We're going to do it right, and we're going to do it on time, maybe. And if we don't do it on time it's not a big problem – we can do it later. But we, you know, as Commissioner Glover said, it's not a question of time, it's a question of getting it right. So thank you.

BILL DODD: Very good. Thank you all very much for being here today. We have one, uh…I don't have any speaker cards for public comment. I don't see any members of the public coming forward. So with that, that concludes the business before the BATA Oversight Committee, and the next meeting will be July…excuse me, June 12th, 2013, at 9:30.