FINAL REPORT

U.S. ARMY CORPS OF ENGINEERS

EVALUATION & ASSESSMENT OF PROPOSED ALTERNATIVES TO RETROFIT/REPLACE THE EAST SPAN OF THE SAN FRANCISCO-OAKLAND BAY BRIDGE

October 27, 2000

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EXECUTIVE SUMMARY

Introduction

In response to the 1989 Loma Prieta earthquake, the State of California Department of Transportation (Caltrans) began a program to seismically retrofit all bridges in the state, including the damaged East Span of the San Francisco-Oakland Bay Bridge (SFOBB). As the plan to retrofit the East Span progressed Caltrans decided that it would be more cost effective to replace the structure rather than to retrofit it. Caltrans considered several designs for a replacement span and in 1997 selected a "skyway" design as the best alternative. Subsequently, the Metropolitan Transportation Commission (MTC), representing nine Bay Area counties and acting under authority granted by the California legislature, decided to pay the cost of adding a signature span and "amenities" to the bridge. These amenities included a bicycle / pedestrian path and the signature span, which is a self-anchored suspension (SAS) design that the MTC decided was more distinctive than the skyway design.

Scope of Work

The City and County of San Francisco (the City) and Caltrans have asked the U.S. Army Corps of Engineers (COE) to evaluate key technical decisions made by Caltrans in reaching the conclusion to build a replacement bridge. Specifically, the purpose of the COE's assessment is to examine two broad areas of concern as raised by the City. First, the City believes that, from the standpoint of cost and public safety, it is preferable to retrofit the East Span than to replace it with the currently proposed design. Second, the City believes that the self-anchored suspension design that Caltrans is currently proposing for the replacement span is not seismically safe. The scope of work also includes four key questions regarding retrofit / replacement design, cost and seismic safety that require answers from the COE Team.

The COE Team s conclusions and responses are based solely on data submitted and documented in the Data Catalog. The COE Team performed no new analyses.

Conclusions

The COE Team can summarize its review and address the aim of this study by stating the following:

1. Documents provided did not demonstrate that any retrofit alternative met lifeline criteria.

2. Caltrans' proposed retrofit strategy is not reasonable due to concerns regarding the isolation strategy, incompleteness of design, and definition of performance criteria.

3. Based on safety considerations, it is the COE Team s opinion that, at this point in time, a replacement alternative is preferable to a retrofit alternative. A replacement alternative is the path that most quickly resolves the exposure of the public to the seismic vulnerabilities of the existing structure.

4. Costs for the currently proposed replacement alternative are $565 million higher than for the proposed retrofit. Reliability studies, for comparison, have not been found for either the retrofit or the replacement bridge.

5. Seismic safety is being addressed as Caltrans design team works towards meeting the seismic performance criteria established by design authorities including the Seismic Advisory Board (SAB) and the Engineering and Design Advisory Panel (EDAP).

6. The replacement bridge does not meet lifeline criteria as defined in the Scope of Work, but is being designed to conform to a unique Design Criteria, including the Safety Evaluation Earthquake (SEE) performance criteria. The design work is not yet complete and conformance to the SEE criteria cannot be verified. It is the COE Team s opinion that Caltrans design team is highly qualified, using state-of-the-art design methods and is moving along a path to design a bridge that meets the seismic performance criteria.

7. The performance of the replacement bridge during a Maximum Credible Earthquake (MCE) cannot be determined. The bridge has not been evaluated or designed for a MCE event, which is larger than the SEE event.

8. Passenger vehicle access and accommodation has been generally addressed in the Design Criteria, requiring Full service almost immediately following an earthquake. The Design Criteria does not define immediate and the design calculations do not demonstrate how this design requirement is met.

PROJECT OVERVIEW

Background

In response to the 1989 Loma Prieta earthquake, the State of California Department of Transportation (Caltrans) began a program to seismically retrofit all bridges in the state, including the damaged East Span of the San Francisco-Oakland Bay Bridge (SFOBB). As the plan to retrofit the East Span progressed, it became apparent to Caltrans that it would be more cost effective to replace the structure rather than to retrofit it. Caltrans considered several designs for a replacement span and in 1997 selected a "skyway" design as the best alternative. Based on the costs associated with the skyway design, Caltrans formally decided to replace rather than retrofit the east span. Subsequently, the Metropolitan Transportation Commission (MTC), representing nine Bay Area counties and acting under authority granted by the California legislature, decided to pay the cost of adding "amenities" to the replacement span. These amenities included a self-anchored suspension bridge design, which the MTC decided was more distinctive than the skyway design, and a bicycle/pedestrian path.

The City and County of San Francisco and Caltrans have asked the U.S. Army Corps of Engineers, as a body of independent experts, to evaluate key technical decisions made by Caltrans regarding the San Francisco-Oakland Bay Bridge.

The City and County of San Francisco (the City) and Caltrans have asked the U.S. Army Corps of Engineers (COE), as a body of independent experts, to evaluate key technical decisions made by Caltrans. Specifically, the purpose of the COE's assessment is to examine two broad areas of concern as raised by the City and its outside consultants. First, the City believes that, from the standpoint of cost and public safety, it is preferable to retrofit the East Span than to replace it with the currently proposed design. Second, the City believes that the self-anchored suspension design that Caltrans is currently proposing for the replacement span is not seismically safe. To expedite the COE's study of these two concerns, the Federal Highway Administration (FHWA), in cooperation with the U.S. Navy, facilitated the COE's communication with appropriate Federal, State, local agencies and their consultants. The U.S. Coast Guard participated as well.

The City believes that, from the standpoint of cost and public safety, it is preferable to retrofit the East Span than to replace it with the currently proposed design.

The City believes that the self-anchored suspension design that Caltrans is currently proposing for the replacement span is not seismically safe.

Time Line

As background information and to facilitate an understanding of the decisions made, a summary time line is presented in Figure 1 on the following page. These events are further described in the Chronological Table provided in Appendix 1.

INSERT FIGURE 1.
TIMELINE

SCOPE OF WORK

Per the scope of work (Appendix 2), the COE Team conducted its evaluation during the two-phase study.

Phase 1, completed on July 25, 2000, included acquiring and cataloging (see Appendix 3 for the updated and current Data Catalog) all reports, data and analyses provided to the COE Team that address the City s two broad areas of concern. The COE Team assessed the completeness and quality of that information and whether sufficient data was available to answer the four major questions in the scope of work. Also as part of the data assessment in Phase 1, the COE Team visited the East Span of the San Francisco-Oakland Bay Bridge. The visit included the Oakland Mole, Yerba Buena Island, the cantilever section, the failure span at E9, and the pile cap at E3. In addition, the COE Team viewed the bridge by boat and reviewed half scale test specimens for lattice members from a completed test for the Golden Gate Bridge.

The results of Phase 1 are contained in the U.S. Army Corps of Engineers Interim Letter Report, Evaluation & Assessment of Proposed Alternatives to Retrofit/Replace the East Span of the San Francisco-Oakland Bay Bridge, dated July 25, 2000.

Phase 2 answers the four major questions contained in the scope of work and presents the COE Team s findings in two letter reports: 1) *Interim Final Report USACE Evaluation & Assessment of Proposed Alternatives to Retrofit/Replace the East Span of the San Francisco-Oakland Bay Bridge, completed September 22, 2000* and 2) *Final Report USACE Evaluation & Assessment of Proposed Alternatives to Retrofit/Replace the East Span of the San Francisco-Oakland Bay Bridge.*

This report is the Final Letter Report and completes the COE Team s work by addressing Questions 3 and 4 of the scope of work.

Phase
Deliverables

Phase 1
Interim Letter Report — Data Gap Analysis (July 25, 2000)

Phase 2
Interim Final Report (September 22, 2000)

Final Report (October 27, 2000)*

**This document is the second of two deliverables under Phase 2. It addresses Questions 3 and 4 of the scope of work.*

DATA ASSESSMENT

Phase 1 of the project consists of a Data Gap Assessment. Phase 2 consists of answering the questions provided in the scope of work. Through the course of this project, USACE team members have organized, reviewed, and cataloged over 400 documents.

The Data Catalog provided in Appendix 3 is current with over 400 documents, some of which contain multiple volumes, and represent over 75,000 pages of material. Most of the documents are loose-leaf three ring binders and cover such areas as:

- Historical documents of the as-built structure including plans, news articles, and design and construction articles by the designers.
- Test reports covering the performance of steel elements of the existing bridge towers and superstructure.
- As-built analysis and retrofit design calculations.
- Cost estimates for the retrofit contracts.
- Value engineering studies.
- Comparisons of the retrofit alternative to the replacement alternative ranging from Caltrans internal memos to the Governors Action Request (GAR) report.
- Plans and specifications covering the design of the new replacement alternative.
- Project Engineer / Designer notes.
- Seismic Safety Peer Review Panel (SSPRP) meeting minutes.
- University Research.

The data provided by Caltrans is voluminous. To locate information and clarify document content, several meetings have been held with Caltrans. These meetings have aided the team s effort at locating and understanding the data and the data gaps. The data provided by the City consists mainly of letters and reports authored by Professor Astaneh. The City has submitted neither design nor cost information. Follow up meetings have also been held with Professor Astaneh, who represents the City, to allow further clarification of the City s concerns. The COE Team has spent over 600 man-hours in these meetings with Caltrans and City representatives.

The information in the Data Catalog includes all reports, data, and analyses that have been provided by the City and Caltrans over the course of this study. This information represents the basis for the answers given by the COE Team in response to the questions in the scope of work.

Data gaps were initially identified in the report titled *Phase 1 Interim Letter Report, Evaluation & Assessment of Proposed Alternatives to Retrofit/Replace the East Span of the San Francisco-Oakland Bay Bridge,* dated July 25, 2000. These Data Gaps have been modified and/or lined out to account for additional data and information that has been submitted since the completion of Phase 1. The revised Data Gap listing is as follows:

Data gaps initially identified in the *Phase 1 Interim Letter Report*, dated July 25, 2000, that have since been filled due to the receipt of additional

Significant Data Gaps

1. Design criteria summary for the proposed retrofit alternative. Quantify these criteria in terms of allowable stress and strain levels, displacement limits, and other pertinent parameters. Significant to Questions 1, 2, and 3.

   Concise definition of the acceptable level of structural response quantities should be known for seismic performance evaluation of the as-built and retrofitted bridge. Design criteria summary that outlines the pertinent parameters including acceptable stress and strain levels, displacement limits, and other factors that are essential for assessment of the bridge s seismic performance, was not provided in a single document.

2. ~~Documents outlining the decision process for data supporting costs in the Governor s Action Request (GAR). Significant to Questions 1 and 2.~~

   ~~Caltrans recommendation for bridge replacement is summarized in the GAR. The GAR provides cost figures from various sources (i.e., Value Analysis Study, Replacement Study For The East Span of SFOBB Seismic Safety Project, and Retrofit vs. New Bridge Economic Analysis study). There is no outline of the criteria used by Caltrans to support their selection of costs that were used in the GAR.~~

   ~~This information is needed to substantiate Caltrans cost effectiveness position that a replacement bridge is more cost effective than retrofitting the existing bridge.~~

3. Basic Geotechnical and Geology Data. Significant to Questions 1, 2, and 3.

   The information provided for the Oakland Mole contained in Document 319, and in the other documents referred to in Document 319 is comprehensive and appears to be sufficiently complete. However, a similarly comprehensive presentation of geotechnical and geology data for the rest of the bridge alignment is important for foundation analysis of both the retrofit and the new bridge alternatives.

   Several key geotechnical design issues have been identified in the various documents. The information received to date does not include a comprehensive presentation of the resolution to the following issues:

   Soil/pile interaction loads, particularly for battered piles.

   Soil structure interaction model incorporated into the global model of the various sections of the bridge.

~~Geotechnical information (boring logs, CPT, field tests, laboratory tests, etc.).~~

~~4. Seismology and Selection of Ground Motions. Significant to Questions 1, 2, and 3.~~

~~For the retrofit alternative, a comprehensive report on seismology and ground motions has not been made available to us. Only limited ground motion information in various design reports has been provided. However, this information does not provide an overall view of the methodology used to develop multi-support ground motions for the soil structure interaction and the structural analysis of the bridge.~~

5. Analysis and design calculation documents for portions of the bridge associated with Contracts 4 and 5 (Foundations E6 — E16) on proposed retrofit. Significant to Questions 1, 2, and 3.

   Without this information it may not be possible to determine if sound analysis and appropriate criteria were used for the subject portions of the bridge.

~~6. Work in progress / most current cost data on currently proposed replacement 65 percent design review. Significant to Question 3.~~

~~Currently Caltrans has submitted for review a 65 percent design along with a 35 percent cost estimate. To ensure accurate, realistic, and complete cost evaluation a 65 percent cost estimate is required. Significant to Question 3.~~

Moderate Data Gaps

7. Stage of design to which work each contract had progressed when the decision was made to go to replacement. Significant to Question 1.

   With many documents at various stages of design it is difficult to identify which documents are pertinent to the most current design. Without identifying the chronology of events, the decision making process is not clear. The level of design stage of each contract (i.e., conceptual, preliminary, final designs) should be known to accurately evaluate the retrofit alternative. This information is only provided by Caltrans for contract 8, in Document 326.

Portions of Data Gap 7 were closed with additional information provided. However there was insufficient detail to close the gap entirely.

8. Meeting minutes, notes and/or letters of meetings for the following groups:

   ~~Caltrans Seismic Advisory Board (SAB) from 1990 to present.~~

Portions of Data Gap 8 were closed with additional information provided. However there was insufficient detail to close the gap entirely.

Caltrans Seismic Safety Peer Review Panel (SSPRP) from 1997 to 1998 and March 2000 to present.

Caltrans retrofit strategy meetings from 1990 to present, including design engineer s preparation for the meetings.
Significant to Questions 1, 2, 3, and 4.

This information is necessary in providing an outline of the review process and identifying changes in project direction recommended by these advisory groups. Significant to Questions 1, 2, 3, and 4.

9. Analysis strategy using various computer models, including the relationship between the various global and local models for the retrofit alternative. Significant to Question 1.

   This information is necessary to show the relationship of local and global models. Although specific sections of the bridge can be analyzed separately, ultimately the bridge must function as a whole. With the information provided it is difficult to determine whether or not the results of the local and global models are compatible. In addition, the information does not show consistency between bridge components.

Minor Data Gaps

10. Material test reports and/or summaries for the condition of the existing foundations, including concrete, steel, and timber. Significant to Questions 1, 2, and 3.

    Lack of this information limits the ability to assess studies of existing foundation information and proposed retrofit designs. Evaluation of soil structure interaction depends on an understanding of existing material properties.

KEY QUESTIONS

In preparing a response to the scope of work s key questions, the COE Team s approach is to state the question and present a summary conclusion. Detailed analysis for each question are referenced and contained in the appendixes.

In responding to the key questions, the COE Team has based its conclusions on the data submitted and documented in the Data Catalog. During the course of the study, discussions were held with Caltrans and City representatives to help the team gain a better understanding of the project and to assist in locating relevant information within the documents provided. Only written, verified documents have been used in development of the conclusions. The COE Team performed no new analyses. Where data can be corroborated and supported by a document in the Data Catalog the document s number is referenced in brackets.

This study combines a short time schedule with the daunting task of reviewing a massive amount of documentation on both the retrofit and replacement projects (studies, university research, plans, specifications, etc.). The documents span nearly a decade since the Loma Prieta earthquake. It has been a challenge for the COE Team to separate and review all the pertinent project data. The goal has been to piece together the relevant data needed to give unqualified answers to the key questions. Both Caltrans and the City have gone to great lengths to provide the needed data to the COE Team. However, for the data provided, the level of completeness is only sufficient for the COE Team to give qualified answers.

The questions are in and of themselves complex and difficult to answer in a straightforward manner. The COE Team provided the most complete answer possible, using the information provided. The answers are based on a holistic (global) perspective, encompassing the total length of the bridge.

The following section makes reference to stages of project planning and design using acronyms; the following provides a key to these design stages as defined by Caltrans:

Advanced Planning (AP)	**0 — 35% Complete**
General Plans (GP)	**35 — 75% Complete**
Plans, Specifications, & Estimates (PS&E)	**75 — 95% Complete**

It should be noted:

Comments contained herein only reflect consideration of technical issues.

Responses are based solely on information made available to the COE Team, absent of independent analyses.

Answers to Questions

Question 1: Was Caltrans' selection of the proposed retrofit alternative reasonable -- i.e., was it based on appropriate criteria and sound analysis, including consideration of realistic, accurate and complete cost figures?

Conclusion: Caltrans had separated the retrofit design into eleven design projects. These designs were in various stages of completion from AP to PS&E. It is the COE Team s conclusion that Caltrans initially used a structured approach to evaluate alternate retrofit strategies, but as explained below, the selected retrofit strategy does not appear to be reasonable due to concerns regarding the isolation strategy, incompleteness of design, and definition of performance criteria.

It is the COE Team s conclusion that Caltrans initially used a structured approach to evaluate alternate retrofit strategies, but as explained, the selected retrofit strategy does not appear to be reasonable due to concerns regarding the isolation strategy, completeness of design, and definition of performance criteria.

Isolation Strategy

Caltrans' proposed retrofit alternative is seismic isolation of truss systems with the exception of the cantilever section. The suspended portion of the cantilever section is cut off and isolated using two new pier supports, while the rest of the cantilever section remains fixed to its piers but is strengthened by edge trusses. Ordinarily, an isolation system is considered for relatively rigid structures to elongate their period of vibration in order to reduce seismic force demands and to provide additional damping through friction or other means. Most spans of SFOBB are long-period structures with fundamental periods of vibration in the range of several seconds.

The seismic force demands for such long-period spans in their existing conditions are approximately at the same level of the proposed isolated spans. On this basis, the use of an isolation system appears unreasonable. Documents submitted for review do not demonstrate why a flexible structure with low seismic force demands should be stiffened by concrete encasement and then softened back to its original condition using isolation bearings. Computer analyses of the isolated bridge are based on unrealistic modeling and input assumptions and they provide limited results. The validity and effectiveness of the isolation retrofit strategy has not been demonstrated.

The seismic force demands for such long-period spans in their existing conditions are approximately at the same level of the proposed isolated spans. On this basis, the use of an isolation system appears unreasonable.

The following statement, from the Seismic Advisory Board s meeting minutes (January 3 and 4, 1991), supports the concerns stated above: Because of the sensitivity of base-isolated structures to the longer periods of free-field ground motion, base isolation should be avoided at very soft sites such as those on San Francisco Bay fill [Document 372]. Two letters to Director James van Loben Sels (December 4 and 5, 1995) also document the concern regarding unprecedented use of an isolation system.

Incomplete Design

The proposed retrofit strategy design of the entire bridge is incomplete. None of the 11 design projects that comprise the retrofit have a finalized, verified retrofit solution, particularly the cantilever truss spans and their foundations. Retrofit designs for the cantilever portion of the bridge including the superstructure and foundations, have not been completed and only preliminary concepts have been derived. No analyses have been provided to demonstrate that they are reasonable and workable.

The proposed retrofit strategy design of the entire bridge is incomplete. No analyses have been provided to demonstrate that they are reasonable and workable.

Criteria

A general statement for seismic design criteria has not been defined. Criteria is inconsistently applied and continually modified. Criteria appear to change as the efforts on the cantilever portion of the bridge progressed.

As can be expected for a bridge of this complexity, Caltrans appeared to struggle with design and cost issues to meet lifeline criteria. Further, they did not have any reasonable degree of confidence that a retrofit alternative could be designed to meet lifeline performance criteria. At the time that the decision was made to proceed with the replacement alternative, Caltrans documents indicate that the retrofit design did not meet lifeline criteria and was being designed to meet the lesser criterion of no-collapse.

At the time that the decision was made to proceed with the replacement alternative, Caltrans documents indicate that the retrofit design did not meet lifeline criteria and was being designed to meet the lesser criterion of no-collapse.

COE Team s detailed review of the data supporting this question can be found in Appendix 4, Retrofit Support Document.

Question 1a: Did Caltrans adequately consider/evaluate other retrofit alternatives, including a West Span-type retrofit and other steel retrofits, and did this evaluation include consideration of realistic, accurate and complete cost figures?

For informational Purposes: The West Span retrofit scheme is to directly strengthen the steel tower members with additional steel components (as verbally provided by FHWA). The West Span refers to the suspension bridge west of Yerba Buena Island.

Conclusion: The COE Team has found that Caltrans considered numerous other retrofit alternatives as reflected in Table 1. The alternatives considered apply to all aspects of the retrofit including foundations, towers and superstructure. The alternatives were not usually evaluated to a level of being able to produce realistic, accurate and complete cost figures. However, to make prudent decisions for retrofit, this is not always necessary. Even though the COE Team questions the reasonableness of Caltrans' selected

Even though the COE Team questions the reasonableness of Caltrans selected retrofit alternative, it does not disagree with the decision process that led to that selection.

retrofit alternative, it does not disagree with the decision process that led to that selection.

The pursuit of a valid retrofit scheme should not be compared to the preliminary design stages of a new bridge structure as this question suggests. Choosing several global schemes for the retrofit and taking them to a 30 percent design level in order to find the best solution is not the normal process for retrofit. This is more practical for a new structure because the type is not restricted by existing conditions. Retrofit alternatives, however, are limited by the existing bridge.

	Table 1 **Retrofits Considered**
Towers	• Steel Strengthening • Hollow Concrete Encasement • Solid Concrete Encasement • Solid X-Bracing Encasement • Boxed Section Steel Retrofit
Superstructure	**Cantilever Truss Spans** • Cable System • Edge Arch System • Superstructure Frame • Substructure Frame • Additional Towers with Supplemental Tube • Additional Towers with Base Isolation and Articulated Superstructure **Cantilever and 504 and 288 Truss Spans** • External Edge Truss System • Retrofitted Towers and/or Additional Towers
Foundations	• Ground Improvement (Grouting) • Small Diameter Piles • Large Diameter Vertical and Battered Steel Piles with New Pile Cap/Load Transfer Structure Above the Water Surface • Post Tension Rock Anchors

An accepted process in developing a valid retrofit scheme is to consider (brainstorm) possible options and, based on discussions of technical feasibility, aesthetics, and preliminary costs, bring forward the most promising overall scheme or strategy. Caltrans conducted an initial analysis to identify the seismically vulnerable items for the existing as-built bridge. Given these items, various retrofit schemes for all the components of the bridge were brainstormed and discussed. Schemes considered are shown in Table 1.

Two general retrofit strategies typically evaluated are: 1) to strengthen elements; or 2) to divert (reduce) forces away from elements that lack capacity for the design load. Forces can be diverted by adding members or by using seismic isolation. These two retrofit strategies are generally considered on two levels: 1) global (entire bridge, or at least by superstructure, frame-by-frame, type), or 2) local (individual components or elements). The selected retrofit includes both of these strategies.

The critical step of abandoning a more typical strengthening scheme for the selected scheme using seismic isolation bearings was not adequately documented.

On the local element-by-element level, valid alternatives should have been thoroughly evaluated. The COE review team could not verify in the documents provided that an adequate evaluation took place at this level to support several key decisions regarding the strategy path taken. The critical step of abandoning a more typical strengthening scheme for the selected retrofit scheme (using seismic isolation bearings) was not adequately documented.

Considering the West Span type retrofit, Caltrans had evaluated this retrofit in a comparison with the selected concrete-encasement type. The evaluation was done by the Contract 2 design team for the towers on Yerba Buena Island. The foundations for these towers are supported by rock. Therefore, the effect of the additional concrete dead load is not as detrimental to the tower foundation as it is on the foundations for the caisson and pile supported piers. This comparison, which led to the conclusion that the concrete encasement was the better solution (steel versus concrete), considered only the cost of the tower retrofit and not the impact to the pile and caisson foundations.

As stated in the conclusion for Question 1, a final valid scheme for the selected retrofit alternative had not been achieved. However, the decision process that Caltrans had followed for this project was adequate. After consideration of various alternatives, what Caltrans considered to be the most promising retrofit scheme, was brought forward into the analysis and design phase. This approach makes use of the vast experience and knowledge available at Caltrans, by quickly considering and eliminating less plausible solutions, and saving the expense of investigating non-viable alternatives.

COE Team s detailed review of the data supporting this question can be found in Appendix 4, Retrofit Support Document.

Question 1b: Did Caltrans adequately consider/evaluate the ability of other retrofit alternatives, including a West span-type retrofit and other steel retrofit, to meet lifeline criteria? Which (if any) retrofit alternatives meet lifeline criteria?

Conclusion: The documents provided did not demonstrate that any retrofit alternative met lifeline criteria. Consequently, Caltrans did not evaluate in detail the ability of other retrofit alternatives to meet lifeline criteria.

Caltrans did not evaluate in detail the ability of other retrofit alternatives to meet lifeline criteria. The documents provided did not demonstrate that any retrofit alternative met lifeline criteria.

COE Team s detailed review of the data supporting this question can be found in Appendix 4, Retrofit Support Document.

Question 1c: Did Caltrans adequately consider/evaluate the costs of retrofitting the span to meet lifeline criteria?

Conclusion: The data reviewed clearly shows that Caltrans did not have a reliable retrofit solution. Therefore, a retrofit solution that could be classified as meeting lifeline performance criteria did not exist. The cost data reviewed by the COE Team were found to be adequate and supportable to the level of design completed. In this case, that level, as stated by Caltrans, was to no-collapse and not lifeline conditions [Document 267]. Analysis to substantiate either performance level is not evident. The COE Team found that Caltrans used sound judgement and estimating procedures, including the use of appropriate cost items, which were consistent and accurate to the level of design under consideration. A cost was not specifically developed for an alternative that would meet lifeline criteria.

Data reviewed clearly shows that Caltrans did not have a reliable retrofit solution. Therefore, a retrofit solution that could be classified as meeting lifeline performance criteria did not exist.

COE Team s detailed review of the data supporting this question can be found in Appendix 4, Retrofit Support Document.

Question 2: Was Caltrans' cost-benefit analysis comparing the originally proposed replacement alternative vs. the proposed retrofit alternative reasonable -- i.e., was it based on appropriate criteria and sound analysis, including consideration of realistic, accurate and complete cost figures?

Conclusion: The COE Team found that the procedures used by Caltrans to form the cost-benefit analyses were reasonable, and Caltrans used sound judgment and estimating procedures, including the use of appropriate cost items. The items are consistent and accurate for the level of design under consideration.

The COE Team found that Caltrans cost-benefit analysis procedures were reasonable and used sound judgment and estimating procedures, including the use of appropriate cost items, which were consistent and accurate to the level of design under consideration.

Caltrans cost figures for the retrofit strategy include the appropriate elements needed to produce a reasonable budgetary tool commensurate with the level of design. The cost presented represent a broad range of numbers and values that were based on engineering and cost assumptions. The lifecycle costs used in the economic analyses could not be substantiated by the data submitted and reviewed, but did represent a reasonable range of costs for this type of analysis.

Key documents conclude that the replacement approach is much more desirable from a lifecycle cost standpoint.

Caltrans cost figures for the originally proposed replacement alternative are based on appropriate criteria and sound analysis. Support documentation is provided in Appendixes 5 and 7.

The lifecycle costs suggest that the decision to select a replacement alternative may have been made even if the retrofit alternative construction costs were substantially less.

Cost items considered in the cost-benefit or lifecycle analysis include traffic delays, hazardous work areas, lane closures, work conducted in traffic, lead base paint abatement, worker and public safety, and maintenance as well as costs associated with working with steel and concrete construction over water.

Document 250 is the primary lifecycle/economic analysis report while Documents 23 and 249 also address the lifecycle costs of the retrofit alternative and the originally proposed replacement alternative. Document 250 is supported by Documents 23 and 249. These documents make the same conclusion, i.e., that the replacement approach is preferable based on lifecycle costs.

Even though backup data is limited, the economic or lifecycle analyses sufficiently addresses the significant issues and costs (limited data includes cost items and probabilistic methods to estimate seismic damage, etc.). The lifecycle analyses are reasonable. The lifecycle costs as presented by Caltrans indicate that the decision to select a replacement alternative would be justified given a retrofit with substantially less construction costs.

Question 3: How does the currently proposed replacement alternative, including as well any work in progress, compare to various retrofit alternatives in terms of a) cost and b) seismic reliability (including ability to meet lifeline criteria)?

Part a) Cost Conclusion: Table 2 summarizes and compares the construction and design cost estimates for the proposed retrofit alternative, the originally proposed replacement alternative (Skyway), and the currently proposed replacement alternative (SAS). For comparison purposes, cost estimates for each alternative have been adjusted with a 3 percent yearly escalation factor to bring costs to a year 2000 level.

Table 2 Notes:

Retrofit costs based on incomplete and unreasonable design

Percentages indicate completeness of design

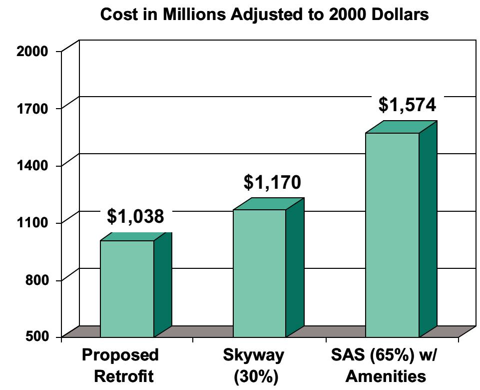
Amenities include bikeway, aesthetic lighting, and light rail loading capacity

3 percent per year escalation taken from Document 263

Ground motion contingency costs, lifecycle costs, operations and maintenance costs, and post earthquake repair costs are <u>not</u> included

Table 2 Cost Comparisons in Millions ($) *See Notes in Margin*

ITEM	Proposed Retrofit 1996 (Document 253)	Originally Proposed Skyway 30% 1998 (Document 263)	Currently Proposed SAS 65% w/ Amenities 2000 (Document 370)
Mainspan	*NA*	*149.10*	*390.79*
Skyway	*NA*	*526.60*	*565.59*
YB Trans	*NA*	*50.50*	*88.58*
OTD	*NA*	*29.00*	*101.43*
YB Detours	*NA*	*49.00*	*46.45*
Demo	*NA*	*54.10*	*54.10*
Struc. Total	***733.50***	***858.30***	***1,246.94***
Roadway Cost	*62.00*	*89.50*	*83.56*
Support Cost	*126.70*	*155.40*	*244.25*
TOTAL	***922.20***	***1,103.20***	***1,574.75***
Escalated to 2000 $s	***1,038***	***1,170***	***1,574.75***



The cost of the SAS alternative is approximately $565,000,000 higher than the cost of the proposed retrofit. The cost of the SAS alternative is approximately $405,000,000 higher than the cost of the Skyway alternative. This increased cost is due primarily to the addition of the signature span and amenities — bikeway/pedestrian path and lighting. Support costs have also increased significantly.

Part b) Seismic Reliability Conclusion: In reviewing the available documents, reliability studies have not been found for the retrofit alternative(s) or the currently proposed replacement alternative. For the currently proposed replacement, there are discussions that relate to reliability. The discussions can be found in the Ventry Value Analysis Reports [Documents 169 and 170]. However, these discussions do not specifically address the currently proposed replacement bridge, nor do they present quantitative reliability reports.

Reliability studies have not been found for the retrofit alternative(s) or the currently proposed replacement alternative.

In keeping with the Scope of Work, the COE Team is not producing any new data or analyses, and cannot answer this question directly.

In keeping with the Scope of Work, the COE Team is not producing any new data or analyses, and cannot answer this question directly without performing a reliability analysis.

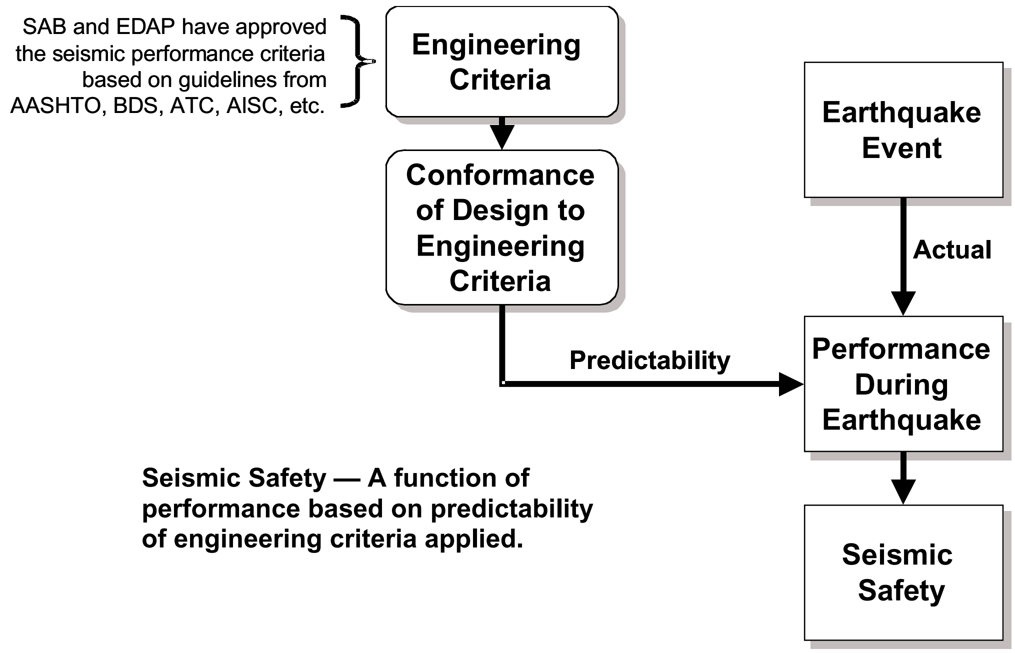
Question 4: Is the currently proposed replacement alternative seismically safe? How will the currently proposed replacement alternative perform in a maximum credible earthquake? Specifically, does the currently proposed replacement alternative meet lifeline criteria? To what extent and how quickly could it accommodate passenger vehicles?

This question is answered in 4 parts:

Part 1. Is the currently proposed replacement alternative seismically safe?

Conclusion: It is the COE Team s opinion that Caltrans design team is moving along a path to design a bridge that meets the seismic performance criteria established by the SAB and EDAP. The COE Team s response is based on the following.

Seismic safety depends on the actual performance of the proposed replacement bridge during a seismic event. The expected performance is determined by the predictability of the engineering criteria that is used for design and construction. The criteria are agreed upon by authorities in various fields and are updated as new events provide additional information and experience. Such criteria become the applicable standard of practice.



Seismic Safety — A function of performance based on predictability of engineering criteria applied.

The degree to which a design conforms to these criteria determines if the bridge will perform in an expected manner based on past experience.

The proposed replacement bridge is a unique structure and a unique set of seismic performance criteria have been developed to guide design and construction. The criteria have been developed by Caltrans based on industry guidelines (AASHTO, BDS, ATC, AISC, etc.) and input from industry experts (i.e., Seismic Peer Review Panel). The SAB and EDAP have approved the seismic performance criteria needed to produce a seismically safe bridge.

The COE Team cannot verify conformance to the design criteria. However, a review of the design work completed to date shows intent to meet the seismic design and performance criteria.

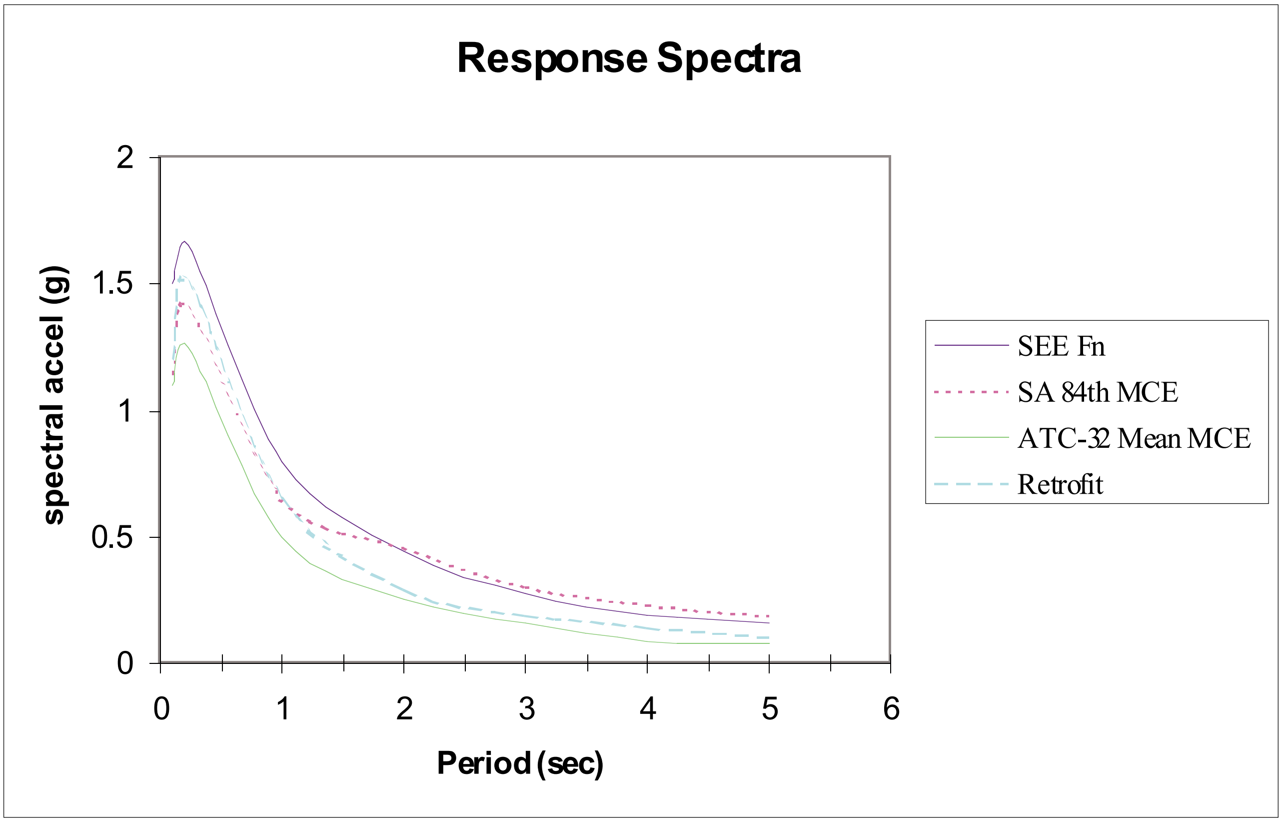
The design for the SAS is in various stages of progress and is not complete. As such the COE Team cannot verify conformance to the design criteria. However, a review of the design work completed to date shows intent to meet the seismic design and performance criteria.

Part 2. How will the currently proposed replacement alternative perform in a Maximum Credible Earthquake (MCE)?

Conclusion: The SAS replacement alternative is not being designed for MCE ground motions, rather it is being designed based on the 1500-year SEE ground motions (further explained in Appendix 6).

The SAS has not been evaluated or designed for the larger MCE event.

The SAS has not been evaluated or designed for the larger MCE event. Caltrans and other authorities have decided to use the SEE and not the MCE.



The Response Spectra graph above compares the various seismic events used to design the SAS. A comparison is made between the SEE and the MCE events for the replacement bridge. The dark, solid line depicts the SEE event, while the dotted line depicts the MCE. For this replacement bridge with its inherent period, the MCE is a greater force than the SEE.

Additionally, the graph shows the seismic event that has been used for the retrofit. The graph shows that the SAS is to be designed with a higher force than the retrofit and is therefore assumed to be more reliable.

Part 3. Specifically, does the currently proposed replacement alternative meet lifeline criteria?

Conclusion: According to the definition of Lifeline criteria contained in the Scope of Work, the current replacement design does not meet lifeline criteria. The Scope of Work defines Lifeline criteria in terms of a MCE event. Caltrans is not designing the SAS to a MCE event; rather Caltrans is designing the SAS to a SEE event. Given this conflict, the COE Team has evaluated the bridge against its given design performance criteria for the SEE event. The SEE event criteria include parameters, in general terms, for

The current replacement design does not meet lifeline criteria as defined in the Scope of Work in terms of a MCE event.

Caltrans is designing the SAS to a SEE event.

design, performance, damage, and repair for both daily and emergency operations.

Design work on the SAS is progressing and is not yet complete. The documents reviewed by the COE Team lack the content necessary to verify conformance with the SEE performance criteria, but do show ability to conform. For instance, it has been noted that the design engineer is using additional analysis to develop his judgment and understanding as a means to predict the performance of the bridge.

However, from the documents presented, it is difficult to verify that correct loads are being used in both analysis and design for such bridge components as the suspension-span tower and deck, as well as the skyway pile caps. For example, member sizes appear assumed or are unidentified in models. Consistent with the assumed intent of the SEE criteria, the ability to replace deck joints in a timely manner remains to be shown. Appendix 8 provides assessment results concerning lifeline criteria, and Minimum and Important Bridges. Appendix 6 provides an assessment of MCE vs. SEE ground motions and additional details and examples of unverified analysis.

Given the ongoing work and the qualifications of the engineers, it is reasonable to believe that conformance issues will be resolved as design progresses. Present review notes a lack of verification and reference in the work presented by the engineer of record. This noted lack of verification can either portend a source of error or it can become an impetus to demonstrate conformance of the bridge to the Design Criteria and its SEE criteria.

Part 4. To what extent and how quickly could it accommodate passenger vehicles?

Conclusion: The COE Team has found no information to indicate how quickly passenger vehicles can be accommodated. According to Document 367 (Volume 1 of 41), the design goal for the bridge is to return to full service almost immediately after an earthquake. The term full service almost immediately after an earthquake is not defined in Document 367, Design Criteria.

The COE Team has found no information to indicate how quickly passenger vehicles can be accommodated. The term full service almost immediately has been provided but not defined. [Document 367]

Information in Documents 344 shows an expected time-scenario for a post-seismic event. This is not a design document, but is the basis used to develop Design Criteria found in Document 367, Volume 1. The post-earthquake scenario calls for steel plates to be placed at failed deck joints within hours to allow for traffic at reduced speeds. Construction activities to replace deck joints would begin within 3 months. [Document 344]

The development of the design is based on the elements of the structure remaining essentially elastic during the SEE. Displacement damage is assumed to be limited to replacement of the deck joints. It is assumed by Caltrans that any other damage can be addressed without impacting traffic. [Document 367]

In summary, no design information demonstrates restoration of traffic for any time frame other than Document 367, Volume 1, which requires almost immediate service.

Recommendations

As indicated in the Scope of Work, actions needed to answer the Questions should be identified. In response, the following actions should be considered to further answer, or refine answers for Questions 3 and 4:

1. Design Calculations should be completed for a comprehensive document. This document should be complete with references, narratives, discussions, and conclusions. The intent is to provide a ready reference for the bridge owner. Future engineers will be able to rapidly determine the designer s intent to facilitate the work for repairs, modifications, etc.

2. An independent check of the design should be completed.

3. The bridge should be evaluated for a design that addresses the San Andreas MCE ground motions. These ground motions appears to be more forceful than the SEE ground motions in the period range significant to the bridge.

4. The possible effects of permanent ground movements on the bridge response should be addressed. These movements are associated with accumulation of seismically induced strains in the soils surrounding and/or beneath the pile foundations.

5. The stability of the rock slope at Pier 1 should be reviewed to confirm that it is seismically stable and consistent with the Fugro-Earth Mechanics, Inc., recommendations.

6. A feasibility evaluation should be performed comparing the performance of vertical and battered piles in order to justify the installation costs and complexities of battered piles.

7. The currently estimated permanent pile settlements during an earthquake should be checked during the iterative design process.

8. Consideration should be given to performing a cyclic pile load test to check the assumed soil degradation rates.

9. Movement at joints should be evaluated and prototype joints should be laboratory tested with loadings that would simulate the MCE displacement demands.

10. A constructibility review should be performed for the bridge. In particular, the COE Team has identified the pile cap/ pile connection as a prime focus. The bridge design should be reviewed for constructibility to ensure reliable conformance to the SEE performance criteria.

SUPPLEMENTAL QUESTIONS

During the course of this study many questions, some of which were not specifically contained in the agreed to scope of work, were asked of the COE Team. In an effort to help all parties reach agreement and make informed decisions, the COE Team, within the context of its mandated scope of work, has addressed these questions in Appendix 9.

List Acronyms and Abbreviations

A/E	Architecture/Engineering
AASHTO	American Association of State Highway and Transportation Officials
ADT	Average Daily Traffic
AISC	American Institute of Steel Construction
AISC	American Institute of Steel Construction
AP	Advance Planning
API	American Petroleum Institute
ARS	Accelerated Response Spectra
ASA	Assistance Secretary of the Army, Civil Works
ATC	Applied Technology Council
BDS	Bridge Design Specifications (Caltrans)
Caltrans	State of California Department of Transportation
CISS	Cast-In-Steel-Shell
City (the City)	City and County of San Francisco
COE	US Army Corps of Engineers
CPT	Cone Penetration Test
CPT	Cone Penetrometer Test
D/C	Demand Capacity Ratio
DEIS	Draft Environmental Impact Statement
EDAP	Engineering and Design Advisory Panel — MTC Task Force Established Early 1997
EIS	Environmental Impact Statement
EQ	Earthquake
FEE	Functional Evaluation Earthquake
FEM	Finite Element Model
F-EMI	Fugro-Earth Mechanics Inc.
FHWA	Federal Highway Administration
GAR	Governor s Action Request
GP	General Plans
LRFD	Load Resistance and Factored Design
M&O	Maintenance and Operations
MCE	Maximum Credible Earthquake
MTC	Metropolitan Transportation Commission — Regional transportation planning agency for the Bay Area
PS&E	Plans, Specifications, and Estimates
RSA	Response Spectra Analysis
SA	San Andreas
SAB	Caltrans Seismic Advisory Board — Established Summer 1990 by Governor s Executive Order D-86-90
SAS	Self-Anchored Suspension
SEE	Safety Evaluation Earthquake
SFOBB	San Francisco Oakland Bay Bridge

SSI	Soil-Structure Interaction
SSPRP	Seismic Safety Peer Review Panel — Established Spring 1997
Task Force	MTC Bay Bridge Design Task Force — Established early 1997
TBPRP	Caltrans Toll Bridge Peer Review Panel — Established 1994 to review and guide retrofit strategies for State owned toll bridges
THA	Time-History Analysis
UCB	University of California, Berkeley
YBI	Yerba Buena Island

Proprietary Computer Software Referenced

SHAKE	WFRAME	XSECTION
GTSTRUDL	ADINA	COM624
GROUP	QUAD4M	DRIVE
ANSYS	SAP2000	ADRIANNA-M

Appendix 1
Chronological Table

CHRONOLOGICAL TABLE

DATE	R/S	Doc#	Major Decisions	Cost	Seismic	Geotech	Performance Criteria / Lifeline
	Retro / Signa						
1998							
17Oct89		321	Loma Prieta Earthquake				
1990							
31May90		247	**Competing Against Time** report from Governor's Board of Inquiry.				
02Jun90		248 & 321	Order to create **Caltrans Seismic Advisory Board (SAB)** per Governor Deukmejian, by Exec. Order D-86-90.				
00Sep90		248	Caltrans appoints 8 members to **SAB** to review seismic design, retrofit, and hazard mitigation activities as these relate to policy and technical procedures.				
1992							
18Jun92		372					Presentation of Proposed Seismic Performance
00Jun92		9	Decision to use SEE instead of MCE				SEE to be used for scope of study preformed by Prof. Astaneh
1993							
05Nov93		81					Update given on performance criteria
03Dec93		303	Criteria for retrofitting SEE and FEE				Seismic performance criteria for retrofitting a Major bridge
1994							
14Jul94		81					Presentation of requirements for retrofitting
Fall94	R	263					
00Oct94		248	**Seismic Advisory Board** report to the Caltrans Director. "The Continuing Challenge".		Assessment of Bridge Performance during seismic events.		
1995							
30Mar95		303	Retrofit strategy changed				Performance criteria reduced

CHRONOLOGICAL TABLE

DATE	R/S	Doc#	Major Decisions	Cost	Seismic	Geotech	Performance Criteria / Lifeline
	Retro / Signa						
00Jun95		117	Performance level selected				Performance level will be higher than the minimum, but below that required for an Important bridge
05Jul95		303	Adjustments to be made for retrofit				Adjustments to be made to criteria but performance level will not drop below minimum performance level
Sum95	R	263	**SAB** raises issue for replacement due to predicted high cost of retrofit.				
Sum95	S	263					
5Jul to 14Aug95	R	84					
1996							
02Jan96	R	R					
12Jan96	R	326					Meeting initiates no drop retrofit strategy & stops other strategies for cantilever section.
Mid96	S	263	**Eng.& Design Advisory Panel (EDAP)** to advises against skyway & a 2nd alternative - a two-piered, cable-stayed bridge.				
01Sep96	S	252		Gray Report - Cost Estimates for 10% Design w/ 20% Contingency.			
28Oct96	R	168	Director Van Loben Sels - memo places top priority on completion of all toll bridge programs.				
06Nov96	R	168	Structural Design Chief Davission - memo places top priority on SFOBB retrofit projects.				

Appendix 1
Chronological Table

CHRONOLOGICAL TABLE

DATE	R/S	Doc#	Major Decisions	Cost	Seismic	Geotech	Performance Criteria / Lifeline
	Retro / Signa						
06Dec96	S	254	**SB60** Senate Bill 60 Introduced to fund replacement of SFOBB. Passed 8/11/97.				
10Dec96	S	329	**Caltrans Seismic Advisory Board** & **Caltrans Peer review Panel** recommend replacement. Letter received by Director Van Loben Sels on 12/09/96.				
16Dec96	S	23	Decision to replace rather than retrofit. Cable stayed is preferred in conjunction with concrete viaduct.	**Value Analysis Findings** by Ventry Eng. Replacement is less costly than retrofit.			Seismic performance is better for replacement than for retrofit (pg1.3). And, performance level is higher for replacement(pg4.3).
17Dec96	S	249		Cost Analysis and Decisions to replace instead of retrofit.			
1997							
29Jan97	R	219	Caltrans Management decides to go with Replacement Option				
07Feb97	S	329	**"GAR"**, Governor's Action Request. Decision to Replace instead Retrofit.				
After 2/7/97	S	276	**SFOBB Task Force** organized by **Metropolitan Transportation Commission (MTC)**. Task Force is to produce a consensus design recommendation. All members are MTC Commissioners.				
After 2/7/97	S	276	**Task Force** forms an **Eng.& Design Advisory Panel (EDAP)** to advise Task Force.				
27Mar97	S	276	1st of 4 public meetings is held to consider design concepts.				
01Apr97		250		Retrofit vs. New Bridge. Economic Analysis by Caltrans. Life cycle cost. Supports **GAR**			

Appendix 1
Chronological Table

CHRONOLOGICAL TABLE

DATE	R/S	Doc#	Major Decisions	Cost	Seismic	Geotech	Performance Criteria / Lifeline
	Retro / Signa						
00May97		321 & 263	**EDAP** presents concept design proposals to **MTC Task Force**. EDAP advises against continuous skyway bridge and 2 pier, cable stayed bridge.				
30Jul97	S	271, 321	**MTC** recommends a replacement bridge on a northern, adjacent alignment.				**MTC** recommends "lifeline" service.
21Jul97	S	321, 267	San Francisco / Mayor Brown "support the northern alignment" which includes a signature bridge cost (replacement). [Doc 267, App. A]				
11Aug97	S	254	**SB60** Senate Bill 60 passes Senate and Assembly. Introduced 12/06/96.				
14Aug97	S	305	**EDAP** receives definition to be used for "30% Design" in making a Type Selection for the Replacement Bridge. Prepared by CALTRANS.	To include Life Cycle costs and probable maintenance.	States the general requirements for seismic analysis.	Calls for additional drilling and qualitative analysis of boring data.	Calls for Performance expectations to be developed.
15Dec97		372	Major bridge projects to use uncorrelated ground motions				For Major bridge projects where site specific ground motions and time history analysis are used three sets of ground motions should be employed
1998							
29May98	S	263	30% Type Selection by EDAP to MTC Task Force.	Developed to identify costs impacted by Seismic Design (9.1).	Ground motions continue to be re-evaluated and changes are anticipated to impact Costs.		
22Jun98	S	264	30% Supplemental to Type Selection by EDAP to MTC Task Force.				
04Jun98	S	272	**MTC** recommends single tower, self anchored suspension.				

Appendix 1
Chronological Table

CHRONOLOGICAL TABLE

DATE	R/S	Doc#	Major Decisions	Cost	Seismic	Geotech	Performance Criteria / Lifeline
	Retro / Signa						
24Jun98	S	377					Astaneh: Seismic Safety letter about Replacement (uses 45% plans).
24Sep98	S	276	Draft **Environmental Impact Statement (EIS)**.				
1999							
15Jan99	S	260 & 261					
05Apr99	R	267	San Francisco / Mayor Brown "new design has . seismic flaws" and "retrofitting is the immediate answer". [Doc 267, App A] (changes position)				
15May99	S	256					
02Aug99	S	277					
23Aug99	S	258					
2000							
31Mar00	S	259					
15Feb00	S	257					
End							

Appendix 2

13 June 2000

Scope of Work
for Services provided by the U.S. Army Corps of Engineers
to Evaluate and Compare Proposed Alternatives to Retrofit and Replace
the East Span of the San Francisco-Oakland Bay Bridge

BACKGROUND

In response to the 1989 Loma Prieta earthquake, the State of California Department of Transportation (Caltrans) began a program to seismically retrofit all bridges in the state, including the damaged east span of the San Francisco Oakland Bay Bridge. As its plan to retrofit the east span progressed, Caltrans concluded preliminarily that it would cost little more to replace the structure altogether. Caltrans considered several designs for a replacement span and in 1997 selected a "skyway" design as the best alternative. Based on the cost of that design, Caltrans formally decided to replace rather than retrofit the east span. Subsequently, the Metropolitan Transportation Commission, representing nine Bay Area counties and acting under authority granted it by the California legislature, decided to pay the cost of adding "amenities" to the replacement span. These amenities included a self-anchored suspension bridge design, which the Commission felt was more distinctive than the skyway design, and a bicycle/pedestrian path.

The City and County of San Francisco (the City) and the State of California have asked the U.S. Army Corps of Engineers (COE), as a body of independent experts, to evaluate key technical decisions made by Caltrans. Specifically, the purpose of the COE's assessment is to examine two broad concerns raised by the City, outside experts, including Professor Abolhassan Astaneh of the University of California, Berkeley, and others. First, the City believes that, from the standpoint of cost and public safety, it is preferable to retrofit the east span than to replace it with the currently proposed design. Second, the City believes that the self-anchored suspension design that Caltrans is currently proposing for the replacement span is not seismically safe. To expedite the COE's study of these two concerns, the Federal Highway Administration (FHWA), in cooperation with the U.S. Navy, will facilitate the COE's communication with appropriate Federal, State and Local Agencies, consultants to those agencies and other outside experts. The U.S. Coast Guard will participate as well.

SCOPE OF WORK

Approach and Major Questions: The COE will evaluate technical assumptions, engineering analyses and cost estimates as contained in ***existing sources of data*** -- specifically reports, backup data, and other analyses provided by Caltrans, the City, their consultants, other Federal and State Agencies, and relevant outside experts. (The parties must provide 4 copies of ***all*** pertinent information--including information that has not yet been made public-- to the COE at least one week prior to "kickoff" meetings, as discussed below.) The aim is to address the two broad concerns identified above: whether retrofit is preferable to the currently proposed replacement alternative; and whether that same replacement alternative is seismically safe. Specifically, the COE will answer the following four major questions, to which the City, the State and key Federal agencies (Federal Highway Administration, Navy, Coast Guard and the National Economic Council) have all agreed:

Appendix 2

1. Was Caltrans' selection of the proposed retrofit alternative reasonable -- i.e., was it based on appropriate criteria and sound analysis, including consideration of realistic, accurate and complete cost figures?

    a. Did Caltrans adequately consider/evaluate other retrofit alternatives, including a West Span-type retrofit and other steel retrofits, and did this evaluation include consideration of realistic, accurate and complete cost figures?

    b. Did Caltrans adequately consider/evaluate the ability of other retrofit alternatives, including a West Span-type retrofit and other steel retrofits, to meet lifeline criteria? Which (if any) retrofit alternatives meet lifeline criteria?

    c. Did Caltrans adequately consider/evaluate the costs of retrofitting the span to meet lifeline criteria?

2. Was Caltrans' cost-benefit analysis comparing the originally proposed replacement alternative vs. the proposed retrofit alternative reasonable -- i.e., was it based on appropriate criteria and sound analysis, including consideration of realistic, accurate and complete cost figures?

3. How does the currently proposed replacement alternative, including as well any work in progress, compare to various retrofit alternatives in terms of a) cost and b) seismic reliability (including ability to meet lifeline criteria)?

4. Is the currently proposed replacement alternative seismically safe? How will the currently proposed replacement alternative perform in a maximum credible earthquake? Specifically, does the currently proposed replacement alternative meet lifeline criteria? To what extent and how quickly could it accommodate passenger vehicles?

Assumptions:

- The COE will rely on *existing sources of information*; it will not generate any new data or analyses.
- If the parties to the study do not provide information or data to the COE, the COE will assume that it does not exist.
- The parties providing analyses to the COE will review them for quality and accuracy.
- Caltrans has supporting documentation for its identification of the currently proposed replacement alternative as the preferred alternative. Experts and others who share the City's concerns have supporting documentation for their concerns.
- If any party withholds critical data or documentation, the Federal Government may cease further activity related to this scope of work.

Appendix 2

Definitions:

- The *originally* proposed replacement alternative refers to the skyway design that Caltrans initially proposed; it does not include the amenities (the self-anchored suspension structure and the bicycle/pedestrian path) that were subsequently added.

- The *currently* proposed replacement alternative refers to the N-6 alignment in the "San Francisco-Oakland Bay Bridge East Span Seismic Safety Project Draft Environmental Impact Statement," September 24, 1998. It includes the self-anchored suspension structure and pedestrian/bicycle path.

- The proposed retrofit alternative refers to the retrofit approach described in the "San Francisco-Oakland Bay Bridge East Span Seismic Safety Project Draft Environmental Impact Statement," September 24, 1998.

- "Lifeline criteria" are above-average standards for bridge (or other infrastructure) construction. A bridge constructed to meet lifeline criteria could accommodate emergency response vehicles and heavy equipment immediately following a maximum credible earthquake. By contrast, most bridges are constructed to meet no-collapse criteria; these are lower standards that ensure against catastrophic failure or loss of life. (For more detail on lifeline criteria, see the San Francisco-Oakland Bay Bridge East Span Seismic Safety Project Draft Environmental Impact Statement, September 24, 1998.) The "lifeline condition" of a bridge is a measure of the degree to which the structure meets lifeline criteria.

TASKS AND SCHEDULE: PHASE ONE

The COE will conduct its evaluation in two distinct phases. During Phase One, which is scheduled to take four weeks, the COE will a) receive from the parties all reports, data and analyses that pertain to the four major questions, and b) assess the completeness and quality of that information. At the conclusion of Phase One, the COE will meet with the parties to review this information and identify any significant gaps in the information needed to answer the major questions. If such gaps exist, the Corps may decide not to proceed to Phase Two. If the available information is adequate to answer the major questions, the COE will begin Phase Two.

Task I. Hold Kickoff Meetings (Week 1)

The COE will begin Phase One one week after a Memorandum of Agreement has been signed. It will hold two meetings in the first week to receive briefings from, and ask questions of, the parties. The first meeting will feature the City and its experts. The second meeting will feature the State and its experts. Both meetings will include FHWA, Navy, Coast Guard and other relevant Federal agencies.

In advance of these meetings, the COE will receive specific reports, backup data, and analyses from the parties. Parties must provide at least 4 sets (original and 3 copies) of this and any other material (including pertinent information that has not yet been made public) to the COE at least one week prior to the meeting. Each party also will provide the COE with information on a primary point of contact and an alternative point of contact, including name, telephone number, mailing address and e-mail address. FHWA, in cooperation with the Navy, will schedule these meetings and in other ways facilitate the study.

Task II. <u>Assess Data (Week 2)</u>

The COE will catalog the reports, data, analyses and design review processes that the parties provide. This data catalog will include the COE's initial assessment of the quality and completeness of the data for answering the major questions. Each member of the COE team will catalog the data within his or her area of technical expertise

Task III. <u>Identify Data Gaps (Week 3)</u>

Using this data catalog, the COE will determine whether or not sufficient data is available to address the major questions. The COE will document any data deficiencies and contact the relevant parties to determine whether additional data is available.

Task IV. <u>Determine Significance of Data Gaps (Week 3)</u>

The COE will assess how significant the remaining data gaps are to its ability to answer the major questions. Each data gap will be rated as having a low, moderate, or high degree of significance.

Task V. <u>Prepare Interim Letter Reports (Week 4)</u>

The COE will prepare an interim letter report summarizing the availability and quality of data for each of the two broad concerns addressed by its evaluation: retrofit vs. the proposed replacement alternatives, and the seismic safety of the currently proposed replacement alternative. In addition, the letter reports will indicate whether sufficient data is available to answer the four major questions and recommend whether to undertake Phase Two. One day in advance of the meeting to conclude Phase One (see Task VI), the COE will provide copies of these interim letter reports to the City, Caltrans and key Federal agencies.

Task VI. <u>Hold Review Meeting to Conclude Phase One (last day of Week 4)</u>

At the end of Phase One, the COE will meet with the City, Caltrans and key Federal agencies to brief them on the status of its work and the significance of any data gaps. If the data gaps are not significant, the parties will adjust and/or finalize the scope of work for Phase Two. If the data gaps are significant, the COE may recommend that it terminate the study.

Appendix 2

TASKS AND SCHEDULE: PHASE TWO

During Phase Two, the COE will review the information it has collected so as to answer the four major questions. As part of that process (and to the extent necessary to answer the four major questions), the COE will evaluate key design alternatives -- the originally proposed replacement alternative, the currently proposed replacement, and various retrofit alternatives (including the one proposed by Caltrans and another championed by Professor Astaneh). The criteria for evaluating these alternatives, as reflected in the major questions, include cost-effectiveness, seismic safety, and lifeline condition.

Task VII. Evaluate Alternatives

To determine whether the data/analyses support key conclusions by Caltrans and the City, the COE will look at the relevant alternatives in terms of three major criteria:

Cost-effectiveness reflects life-cycle costs associated with the construction, maintenance, and operation of the relevant alternative. These include initial construction costs, costs to maintain traffic during construction, construction-related accidents, traffic delay, on-going maintenance and operations costs (including inspection, painting, replacement and servicing of structural elements, and resurfacing), and the time value of money.

Seismic safety refers to the performance and reliability of the relevant alternative during mild, moderate and maximum credible earthquakes. This measure takes into account the exposure of bridge users (drivers, passengers, maintenance crews, etc.) to risk, the extent of damage, the costs of having the bridge closed following a seismic event, costs of repair, and loss of life or injury to motorists.

Lifeline condition reflects the degree to which the relevant alternative meets lifeline criteria. This criterion takes into account whether, immediately following a maximum credible earthquake and during the post-earthquake recovery, the structure could accommodate emergency vehicles, heavy equipment, and other vehicles transporting critical supplies.

Task VIII. Answer Four Key Questions

Based on its evaluation of key design alternatives in terms of the major criteria, the COE will answer the four major questions identified on page two.

Task IX. Identify Remaining Concerns

The COE will identify which, if any, of the four major questions cannot be answered because of insufficient information.

Task X. Prepare Final Letter Reports (Week 12)

The COE will prepare two final letter reports: The first, due week 12, will summarize the evidence comparing the retrofit alternative with the originally proposed replacement alternative in terms of cost-effectiveness, seismic safety, and lifeline condition; and with the currently proposed replacement alternative in terms of cost-effectiveness. The second letter report, due week 16, will summarize the evidence on the seismic safety/reliability of the currently proposed replacement alternative. The letter reports will state whether the major questions are adequately answered. If any questions are not answered, the letter reports will explain why and identify what actions are needed to answer the questions. One day prior to the final assessment meeting, the COE will provide copies of its final letter reports to the City, Caltrans and key Federal agencies.

Task XI. <u>Hold Final Assessment Meeting (last day of Weeks 12 and 16)</u>

The COE will meet with the City, Caltrans and key Federal agencies to brief them on the final results of its study, as reflected in the final letter reports. The COE will present the results of its "retrofit study" on the last day of week 12, and the results of the "replacement study" on the last day of week 16. The COE anticipates that this meeting will conclude its involvement in the study.

SFOBB Document Inventory, Table for Data Catalog

Date: 11Oct00

Doc	Subject			Provided by	Description	Date	REMARKS		Rev	CT Eng
##	Grp	Grp	Type				R/S		init	name
							(Retrofit / Signtr)			
			Univ Research							
1	S		Testing	CT / UCB	Cyclic Tests of Existing & Retrofitted Sway Frames of SFOBB - Astaneh --(288 Ft Section)	27May98	R		Ma	Akinsanya
2	S		Testing	CT / UCB	Cyclic Behavior & Seismic Design of Steel H-Piles - Astaneh --(Test results compared w/design)	20May98	R		DG, Ma	Akinsanya
3	S		Testing	CT / UCB	Final. Proof-Testing of Latticed Members & Their Connections on SFOBB - Astaneh --(Research not completed)	00Jun98	R		Ma	Akinsanya
4	S		Testing	CT / UCB	Cyclic Tests of Rivets for SFOBB Sway Frame Specimens - Astaneh --(288ft section)	00May96	R		Ma	Akinsanya
5	S		Testing	CT / UCB	Cyclic Tests of Riveted & Bolted Angle Connections of SFOBB - Astaneh --(288ft section)	00May96	R		Ma	Akinsanya
6	S		Testing	CT / UCB	Cyclic & Monotonic Tests of Truss Verticals of SFOBB - Astaneh --(288ft section)	00Dec96	R		Ma	Akinsanya
7	S		Testing	CT / UCB	Final. Proof-Testing of Latticed Members & Their Connections on SFOBB - Astaneh --(Work terminated by Caltrans)	00May98	R		Ma	Akinsanya
8	G		Analysis	CT / UCB	Analysis of Cyclic Behavior of Existing & Retrofitted Sway Frames of SFOBB - Astaneh --(288ft section)	00Jan97	R		Ma	Akinsanya
9	S		Concepts Report	CT / UCB	Seismic Retrofit Concepts for the Bay Bridge - Astaneh	24Aug92	R	Need detailed eval. by CT	RT	Akinsanya
10	S		Research	CT / UCB	Latticed Research Vol III. Various correspondence and reports on lattice research. 1 & 2 of 2. --(288ft section)	96-'97			Ma	Akinsanya
11	S		Research	CT / UCB	Research on Truss Sway Frames. Correspondence between Caltrans/UCB. Report : Analysis of Cyclic Behavior & Retrofitted Sway Frames of SFOBB, Report Number: UCB/CEE-Steel-96/05. Also, invoice info for UCB.	93-'98	R	Retrofit stopped before report final.	Ma	Akinsanya
12	S		Short Course	CT / UCB	Vol 1: Business Correspdnc. - Astaneh. 96-99. Vol 2: Seismic Design of Components of the East Bay Crossing --(by Astaneh - Handout for short course to Caltrans engineers by Astaneh. Anal. and testing of lattice members. Discussion for test program and setup.)	00Apr95	R		Ma	Akinsanya
13	S		Analysis	CT / UCB	Seismic Condition of the East Bay Bridge, Results of the Elastic 3-D Dynamic Analysis of The 288-ft Spans From E11 To E23. - Astaneh -- (SAP 90 anal. output, linear analysis.)	00Apr95	R		YG	Akinsanya
14	S		Analysis	CT / UCB	Seismic Condition of the East Bay Crossing of the SFOBB. Volume 5: 3-D Modeling & Analysis Using The Facts Program. - Astaneh --(volume 5/12)	00Dec93	R	Missing 11 volumes?	Ma, YG	Akinsanya
15	S		Analysis	CT / UCB	Seismic Condition Assessment of the East Bay Bridge, Results of the Elastic 3-D Dynamic Analysis of The Cantilever Span. - Astaneh -- (SAP 90 anal. output only, linear analysis.)	09Jun93	R		YG	Akinsanya
16	S		Analysis	CT / UCB	Seismic Condition Assessment of the East Bay Bridge, Results of the Elastic 3-D Dynamic Analysis of The 500-ft Spans from E4 to E11. - Astaneh -- (SAP 90 anal. output only, linear analysis.)	09Jun93	R		YG	Akinsanya
17	S		Testing	CT / UCSD	Structural Systems Research Project, Cyclic Performance of As-Built Latticed Members for the SFOBB. - Uang / Kleiser --(concentric and eccentric seismic loads, by subcontractor to UCB.)	00Jun97	R		Ma	Akinsanya
18	S		Testing	CT / UCB	SFOBB Cyclic & Montonic Tests of Truss Verticals of SFOBB, Report No.: UCB/CEE-Steel-96/04	01Dec96		?Missing		Akinsanya
19	S		Testing	UCB, others	Proof-Testing of Latticed Members and Their Connections on SFOBB, Final Summary of Report Report Number: UCB/CEE-Steel-98/03 --(contract & admin. correspondence.)	01Jun98	R	Compare to Doc 7; Concerns about Isolation Devices. See comments by Ma	Ma	Akinsanya

SFOBB Document Inventory, Table for Data Catalog

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Doc ##	Subject Grp	Grp	Type	Provided by	Description	Date	REMARKS R/S (Retrofit / Signtr)		Rev init	CT Eng name
20	S		Testing	CT / UCB	Cyclic Tests of Existing and Retrofitted Sway Frames of SFOBB, Report No.: UCB/CEE-Steel-98/02	27May98		?Missing		Akinsanya
21	S		Testing	CT / UCB	Cyclic Tests of Riveted & Bolted Angle Connections of SFOBB, Report No.: UCB/CEE-Steel- 96/03	01May96		?Missing		Akinsanya
			Value Analysis							
22	A	C	Value Analysis	Ventry	Value Analysis Summary of the SFOBB East Bay Spans Foundation, Contract No. 53Y286 --(VE cost proposal)	16Aug96	R	See cmnts by	RT, AC, BF	Akinsanya
23	A	C	Value Analysis	Ventry	SFOBB East Bay Crossing Replacement Value Analysis Findings --(life cycle cost matrix)	00Dec96	S	no detailed cost on retro. See comments by Ma.	RT, Ma	Akinsanya
			Other							
24	A		various		memos, newspaper articles, magazine articles, invoices, letters --(info for retrofit & original construction)	var	R	Good History	PS	Akinsanya
25	S	G	Seismic Evaluation	G&E EngSys Inc	Seismic Evaluation for SFOBB - deterministic and probabilistic approaches -- (nonlinear stick model of existing bridge w/ seismic input)	00Dec94	R	Model recmndtn for EQ. See comments by Ma.	RT, Ma	Akinsanya
26	A		Articles	ENR	Series of articles published in ENR about SFOBB written by Chief Engineer, Bridge Engineer, & Engr of Design for the Bay Bridge --(Descript of original design, constr, & seismology)	3/34 - 4/37		Historical Description, good geotech reference.	PS, MR	Akinsanya
			Caltrans Plans							
27	G	C	LOTB	Caltrans	East Span of the SFOBB Log of Test Borings (Preliminary Geologic Report to the MTC EDAP. Projects 3, 4, 5 & 6. EQ Retrofit.	18Apr97	R	Includes Retrofit Summary Costs, good geotech reference. Same as 95.	DG, MR, BF	Akinsanya
28	S		Plans	Caltrans	Project No. 4 GP and other plan sheets (E6 - E9) EQ Retrofit.	00Dec96	R	Foundation retro.	Ma	Akinsanya
29	S		Plans	Caltrans	Project No. 10. Plan sheets - lower chord retrofit details E9-E23, vertical member retrofit details, various other detail sheets --(compliment to design / analysis & cost)	05Dec96	R	not checked or signed	PS	Akinsanya
30	S		Plans	Caltrans	Project No. 3 GP and other plan sheets (unchecked details) EQ Retrofit. --(compliment to design / analysis & cost)	00Jun96	R	Enlarge caisson	PS	Akinsanya
31	S		Plans	Caltrans	Project No. 9 GP and other plan sheets (unchecked details) Seismic Retrofit --(compliment to design / analysis & cost for 500ft section)	30Jul96	R	not checked or signed	PS, Ma	Akinsanya
32	S	G	Plans	Caltrans	Project No. 5 GP and other plan sheets. EQ Retrofit. --(compliment to design / analysis & cost; foundation plans)	31Dec96	R	not signed	PS, AC	Akinsanya
33	S	G	Plans	Caltrans	Project No. 2 GP and other plan sheets. EQ Retrofit. (34 pages) --(compliment to design / analysis & cost; encase steel tower, add piles; foundation plans- YB)	26Oct95	R	not signed	PS, Ma, AC	Akinsanya
34	S	G	Plans	Caltrans	Project No. 2 GP and other plan sheets. EQ Retrofit. (49 pages) --(compliment to design / analysis & cost; foundation plans - YB)	31Dec96	R	update Doc 33	PS, Ma	Akinsanya
35	S		Plans	Caltrans	504' truss span sheets. (Interim Seismic Retrofit.) --(compliment to design / analysis & cost, including steel tower retrofit)	06Jun97	R	not signed	PS	Akinsanya
36	S	G	Plans	Caltrans	Project No. 6 GP and other plan sheets (P&Q). EQ Retrofit. --(compliment to design / analysis & cost; enlarge footing and add piles.)	10Mar97	R	not signed	PS, Ma	Akinsanya
37	S	G	Plans	Caltrans	Cofferdam at Caisson E3 details. Electrical (PS&E)	30Dec95	R		DG	Akinsanya
38	S	C	Plans	Caltrans	Project No. 7 Structure PS&E plans. EQ Retrofit. --(compliment to design / analysis & cost backup)	31Dec96	R	not signed	PS, MR	Akinsanya

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##	Grp	Grp	Type				R/S		init	name
							(Retrofit / Signtr)			
39	A	C	Estimate Submittal	Caltrans	Cantilever superstructure, retrofit project No. 8, E1, E4 anchorage. Retrofit release analysis, retrofit concept development, quantity & cost summary and detail generation	22Oct96	R	not signed	PS, MR, BF	Akinsanya
40	A		Report	CA DPW	General Safety of the SFOBB From Earthquake Point of View & Geologic and Seismic Conditions Affecting SFOBB	24May38		historical interest, good geotech reference.	GC, MR	Akinsanya
41	A		Report	Caltrans	SFOBB & Port of Oakland Overcrossing, October 17, 1989 Loma Prieta Earthquake Damage/Repair Report (Robert Bidwell) --(no cost estimate)	01May90		Good Description and geotech reference. M65+M78	PS, MR	Akinsanya
42	A		Report	Caltrans	Post Earthquake Report for the SFOBB (Robert Bidwell)	29Jun92	R	historical data on Loma P	RT	Akinsanya
43	S		Permit Application	Caltrans	Draft #1, Permit Application to BCDC for SFOBB Retrofit	00Jan97	R	Discussion of rejected alternatives; numerous mini-contracts. Later ver. at #70.	PS, MR	Akinsanya
44	S		Plans	Caltrans	Connection Team: various plans, meeting notes, memos, details, alt. Concept, retrofit. -- (Details of 'drop span' joint mod. @ added piers & steel Jt design criteria)		R	Ref. to concrete encasement, but no supporting data.	RF, Ma, BF	Akinsanya
			Contract related							
45			Contract	Caltrans, UCB	Various contract monitoring documents by Caltrans for UCB research	95-98		?missing?		Akinsanya
46			Contract	Caltrans	Various contracting out documents for UCB research	95-98		?missing?		Akinsanya
47			Contract	Caltrans, UCB	Various contract and research correspondence between Caltrans and UCB	95-98		?missing?		Akinsanya
			Design Related							
48	S			Caltrans	04-00434GI - East Anchor Arm - Checked Details. Retrofit.	8/96-1/97	R		GK	
49	S			Caltrans	04-00434GI - East Cantilever - Checked Details. Retrofit.	8/96-1/97	R		GK	
50	S			Caltrans	04-00434GI - West Anchor Arm - Checked Details. Retrofit.	8/96-1/97	R		GK	
51	S			Caltrans	04-00434GI - West Cantilever - Checked Details. Retrofit.	8/96-1/97	R		GK	
52	S			Caltrans	04-00434GI - Suspended Span - Checked Details. Retrofit.	12/95-1/97	R		GK	
53	S			Caltrans	SFOBB East Bay Member Survey - 288' truss YB3 to E1 Volume II, Book 3 of 3; with Section 5 and Section 6		R	?3of3 missing? See 188 & 199	GK	
54	S			Caltrans	SFOBB Hand Calculations - Binder No. 3 -Cantilever Structure - Truss Spans -East Bay Connector; 1, 2, & 3 of 3.	00Aug93	R	4 copies 1of2; 1 cop. 2of2; ? Not all copies present	GK	
55	S			Caltrans	SFOBB Hand Calculations - Binder No. 5 - Cantilever Structure, East Bay Connector - Truss Spans -- (hand calcs / section props / DL & LL)	00May93	R		CC, GK	
56	S			Caltrans	SFOBB Dead Loads and Section Props - 504' Spans -- (DL / Section props / static anal.)	92-96	R		CC	
57	S			Caltrans	SFOBB Demand / Capacity -- (DL and inertia)	11/95 - 5/96	R	Need force demand calcs	CC	Avila
58	S			Caltrans	SFOBB East Bay - Model Geometry and Loading Cases -- (500' span truss model)	yr1995	R	limited and incomplete (YG)	CC, YG	
59	S			Caltrans	SFOBB Isolation - [504' Truss Frame / Heel Stiffener Install SEQ] (Friction Pendulum Bearing w/ Tension Load Capacity) -- (vendor cost est / Exp. Joints; includes FPS isolation)	00Jun96	R		RF, BF	
60	S	C		Caltrans	SFOBB 504' Spans Notes + Various (Scheduling, Estimates, Tasks, etc.)	yr1996	R	Good retrofit cost brkdwn	CC	
61	S			Caltrans	SFOBB Peer Review - 504' Truss Spans	13Dec96	R	500' Span retrofit	CC	
62	S			Caltrans	SFOBB Jacking Operation and Heel Strengthening (Book 2)	00Nov96	R	Old Drawings (1934)	CC	
63	G			Caltrans	SFOBB New Alignment - Foundation Design	00Apr96	S		DG	
64	G			Caltrans	SFOBB Analysis of Existing Foundations - Book 1, 7/17; Book 3, 8/28/95	17Jul95	R		MR	
65	S			Caltrans	SFOBB Member Typical Sections	06Jun96	R	Extension of Doc 55	CC	Siemers
66	S			Caltrans	SFOBB UC3 to UC7 Portal Frame Analysis / calculations --(Cantilever)	00Jul96	R			
67	S			Caltrans	Cantilever Truss - Final -- (Strudl, ADINA runs for existing only??)	00Jul95	R	FE model input / plots	CC	
68	S			Caltrans	YB Spans - Final	00Jun95	R	FE model input / plots	CC	

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##	Grp	Grp	Type				R/S		init	name
							(Retrofit / Signtr)			
69	A			Caltrans	SFOBB Survey Control - Dist 04 R/W Engineering	00Jun96		Surveying - not needed	CC	
70	A			Caltrans	Draft #1 - SF Bay Conservation and Development Commission for SFOBB --(Permit Appl., Retrofit desciption)	00Jan97	R	Good geotech reference. See 43 for earlier version.M51	CC, MR	
71	G			Caltrans	SFOBB Retrofit Strategy for the Foundations of Piers E-17 to E-22		R	Good calcs on Foundation	BF	
72	S	G		Caltrans	SFOBB Modeling Guidelines, Assumption - Notes and Meeting Summaries -- (ADINA design of retro approach)	5/95-2/97	R	Focus on pier rehab	CC, MR	
73	S	G		Caltrans	SFOBB Hand Calculations E-17, E-22, E-23, Segment E-17 to E-23 --(Retrofit calcs, including Finite Element)	3/94-3/95	R		CC, MR	
74	S	G		Caltrans	SFOBB Correspondence --(Geotech Info)	95 - 96	R	Various Retro Schemes	CC	
75	S			Caltrans	SFOBB East Crossing 288' Double Deck Truss - Spans E9 - E22 - Misc Calcs	yr1993	R			
76	S			Caltrans	GT STRUDL INPUT 288's and Cantilever - Section Properties and Beta Angles	00Jun95	R			
77	S			Caltrans	GT STRUDL INPUT 504's		R			
78	S			Caltrans	SFOBB East Crossing 504' Through Truss - Spans E4 - E8 - Misc Calcs	93-95	R	Useless Info	CC	
79	A			Caltrans	SFOBB Miscellaneous Notes and Calculations -- (Seismic, cost est, 504' span - for Retro)	00Jun93	R	Historical Criteria	BT	
80	S	C		Caltrans	SFOBB East Crossing Double Deck - Cantilever Truss - Spans E1 - E3 - Misc. Calcs	yr1993	R	F.E. input, not much use	CC	
81	A			Caltrans	SFOBB BCDC - East Bay Approach - General Information - Region Wide Permit 9	9/93-11/94	R	Some criteria discussion	PS	
82	A			Caltrans	SFOBB Project Planning Descriptions	13Apr95	R	W & E Info & alternative designs	BF, PS	
83	S			Caltrans	Capacities	2/93-3/96	R	Hand calcs, not much use	CC	
84	S			Caltrans	Special Analysis Loads and Miscellaneous --(modeling E6 to E23, memos, DL cant. Deflection)	00Dec94	R		MR, RT	
85	S	G	Internal Rpt Eval.	Caltrans	Predicted Large Earthquake Response Scenario for the East Spans of the SFOBB in its Current (12-1996) State and Condition Seismic Life Safety Evaluation	06Jan96	R	by Maroney to Davison. Good reference.	RT, CC, BF	
86	S	G		Caltrans	Seismic Evaluation SFOBB Progress Meeting Presented to Caltrans of Structures on Contract No. 59U064	21Mar94	R	Results show need for retrofit	CC, MR	
87	S			Caltrans	Miscellaneous Retrofit Binder	95-96	R	Good Info, disorganized	CC	
88	S			Caltrans	SFOBB Substructure Misc. Binder	00Nov94	R	Not very useful	CC	
89	G			Caltrans	SFOBB Geotechnical Misc. Binder	00Aug94	R	Seismic Geot, pile survivability. Good geotech reference.	BT	
90	S			Caltrans	SFOBB Member Typical Sections F, K, N, Q, R, S, X, AB, A, B, W, E, P, T, C, O, I, D, H, Y, L, V, Z, J, M	00Dec96	R	Backup data, no use.	CC	
91	S			Caltrans	Member Capacity Hand Calculations --(Cantilever span, backup data)	00Jun96	R	no use.	CC	
92	S			Caltrans	SFOBB Cantilever Structure East Bay (Binder 2) includes Section Properties, Truss	00Aug93	R	See Docs 54 & 55		
93	S			Caltrans	~~Misc. Calcs~~(Suspended Span;Lower Chords & Bottom Laderals;North & South Truss;Portal Frames,etc.) --(I.D. members in model, no calcs)	1995	R		CC	
94	S			Caltrans	Program CAPP Binder; Member Database (runs with "CAPP")		R	No use.	CC	
95	G				East Span of SFOBB Log-of-Test-Borings Apr. 18, 1997	18Apr97	R	Good geotech reference. Same as 27.	MR	
96	S			Caltrans	Stick Model (E1-E5)	00Jun96	R	Backup data.	CC	
97	S			Caltrans	SFOBB - E2 (Retrofit analysis for E2)	00Mar96	R	Good document	CC	

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98	S			Caltrans	SFOBB/Jacking Operation & Heel Strengthening Book 1	00Nov96	R	See Docs 62	MR	
99	G			Caltrans	SFOBB Analysis of Existing Foundations	00Nov99	R	See Doc 64. Good geotch ref.	MR	
100	G			Caltrans	SFOBB Analysis of Existing Foundations Book 1	17Jul95	R	Missing Original. Good geotech reference.	MR, AC, DG	
101	G			Caltrans	SFOBB Analysis of Existing Foundations Book 2	00Sep95	R	Ref #64 for book 3. Good geotech reference.	MR, DG	
102	G			Caltrans	SFOBB Analysis of Existing Foundations Book 4	17Jul95	R	Good geotech reference.	MR	
103	G			Caltrans	SFOBB Analysis of Existing Foundations Book 5	18Nov95	R	Good geotech reference.	MR	
104	S			Caltrans	SFOBB Analysis of Existing Tower Shoes Book 1	17Jul95	R	not much use	CC	
105	S			Caltrans	Analysis & Design Concrete Encased Steel Towers -- (details & x-section runs for pushover)	28Aug95	R	No summary (CC)	RF, BF	
106	S			Caltrans	Towers E13, E14, E15 m/s/ Steel Calculations Binder #1 --(demand capacity calcs)	2/95 - 3/95	R		PS	
107	S			Caltrans	1st Binder for Towers E13 to E16 Models & Section Properties --(computer analysis)	00Feb95	R		PS	
108	S			Caltrans	SFOBB E13-E16 Tower Binders --(computer analysis)	00Dec94	R		PS	
109	S			Caltrans	SFOBB Towers E13 - E15: Binder #1; Binder #2 "as-built" Capacities & Joint Capacities --(analysis of existing towers)	00Mar95	R		PS	
110	S			Caltrans	SFOBB E13 & E14 Binder #3 Steel Capacities --(analysis of existing towers)	00Jul95	R		PS	
111	S			Caltrans	SFOBB Binder #4, E15 & E16 Steel Capacities --(calcs & analysis backup info)	00Jul95	R		PS	
112	G			Caltrans	Pier E23 -- (Time histories, spring supports, ductility & strength)	00Apr96	R		AC	
113	S	G		Caltrans	Pier E17 to 22 -- (Modeling analysis & design assumptions, etc.)	00May95	R		AC	
114	S	G		Caltrans	Pier E17 to 22 -- (value analysis & recommendation. ADINA model assumptions, analysis, calcs, discussions)	00Aug96	R		RT	
115	S			Caltrans	Pier E7 -- (Drawings & calcs, computer input+F147)+F169+F193+F169	00Apr95	R		PS	
116	S			Caltrans	Rocking Analysis E2 --(ADINA model I/O - no roadmap or conclusion)	00Jun96	R		RT	
117	G	S		Caltrans	SFOBB - Cantilever Project Engineer Binder 04-0434GI	00Jan97	R	Steel alternative strategy	BF	
118	S			Caltrans	SFOBB - East Bay Member Survey, Volume II, Book 2 of 3, 288' Truss, YB3 to E1, Anchor Arm, Cantilever Arm, Suspended Span --(detailed list of Cant. truss members)	00Dec94	R	See Doc 53	PS	
119	S			Caltrans	SFOBB - East Bay Member Survey, Vol. II, Book 3 of 3, 288' Truss, YB3 to E1, Anchor Arm, Cantilever Arm, Suspended Span --(detailed list of Susp. Truss members)	00Dec94	R	See Doc 53	PS	
120	S			Caltrans	SFOBB Concrete Retrofit Binder #1 --(Tower E13, development of computer model)	00Sep95	R		PS	Soon
121	S			Caltrans	SFOBB Concrete Retrofit Binder #2 --(Towers E10, E12 to E16, design of Conc. Encasement alternative.)	00Oct95	R		PS	Soon
122	S			Caltrans	SFOBB Concrete Retrofit Binder #3 --(Towers E13 to E16, design of Conc. Encasement alternative.)	00Nov95	R		PS	Soon
123	C			Caltrans	SFOBB Towers E13-16 Quantities --(Steel retrofit alternative; Quantity backup w/ no cost data.)	00Dec95	R		BF, PS	Soon
124	S			Caltrans	Cantilever -- (Struct Calcs)	00Feb96	R		PS	Soon
125	S			Caltrans	SFOBB Towers E10-12 -- (struct calcs, as built)	94-95	R		PS	Soon

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126	S			Caltrans	Supplemental Revised Sheets: 288' Spans - East Bay Member Survey --(as built section properties)	00Nov92	R	See doc 53	RF	
127	S			Caltrans	Capacity Binder: Types I, J, L, K, M, N, O, Q, R, S, V -- (Cantilever section properties & member capacities - as built)	00Jul96	R		RF	
128	S			Caltrans	Misc.: Adina Results; YB3 to E10 Demands to Capacities; E4 Moment Frame; Various Pier Calcs. Superceded. Parts 1 & 2 of 2	95-97	R		RF	
129	A		Plans	Caltrans	Structure Plans 1933	1933	R	copies missing	RF	Slocum
130	A		Plans	Caltrans	Retrofit Plans 1960 (portion of)	1960		all missing	RF	Slocum
131	A		Plans	Caltrans	Contract 04-043434 Plans for Retrofit (Sheets 38 to 157) (As-built 1999 East Approach)	1995	R	1 orig + 5 copies	RF	Slocum
132	A	C		Caltrans	Quantity Estimates --(East approach beyond pier 23; Quantity / Cost summary)	1994	R		BF, RF	Slocum
133				Caltrans	Contract 04-043434 Special Provisions --(East Approach beyond pier 23)	1995	R		RF	Slocum
134	A			Caltrans	Engineering Criteria Review Board - Retrofit Presentation to BCDC	27Jul94	R	Summary w/o Recommnd.	BT	Slocum
135	S			Caltrans	SFOBB East Approach - Design Notes (3 binders) (East Approach beyond pier 23; encompasses Bents 22-39)	93-94	R	Retrofit Strategy	RF, MR	Slocum/Lian
136	C			Caltrans	Estimating File (3 folders) --(East Approach beyond pier 23; EQ retrofit; Bid cost seismic retrofit;)	94-95	R		BF, RF	Slocum
137	G			Caltrans	Indicator Pile Test Program - Contract 04-043494 - Final Report --(East Approach beyond pier 23)	30May95	R		RF, MR	Slocum
138	S			Caltrans	Super Structure Strengthening Design Notes (& calcs for West Bay)	00Sep96	R	YBI	BT	Slocum
139	S			Caltrans	Yerba Buena Island (YBI) Existing Column Capacities, Designs & Calcs. --(for West Bay)	00Mar95	R		RF	Masoor
140	S			Caltrans	Retrofit Design Bent 39-52 -- (Loads & calcs, for West Bay)	00Apr95	R		RF	Masoor
141	S			Caltrans	Check Super-Structure Capacities Base Isolation Alternative --(for West Bay)	00Feb97	R		RF	Masoor
142			Plans	Caltrans	Contract 04-043001 Interim Retrofit - East Bay Yerba Buena Island Viaduct --(for West Bay)	1997	R		RF, BF	Masoor
143	S			Caltrans	Retrofit Options (various) --(struct analysis for West Bay, detail calcs for YBI approach, retrofit)	00Jun96	R		BF, RT	Masoor
144	C	S		Caltrans	Bay Bridge E23 SFOBB --(Details & Quantities only, also for E23 & trestle data)	00Feb95	R		RF, BF	Hight/Adams
145	S			UC-SD	Cyclic Testing of Latticed Member UCSD-1 for SFOBB, Progress Report, & Misc. Items	00Nov96	R	w/ envrnmntl assmn't. See Doc #3 for Final Report.	RF	Hight/Adams
146	S			Caltrans	Isolation Bearing Data File	1995	R	Same as Doc #6	RF	Hight/Adams
147	A			Caltrans	Original SFOBB Reports & Post Earthquake Reports (various)	1934-1990			AC	Hight/Adams
148	S			Caltrans	Cyclic & Monotonic Tests of Truss Verticals of SFOBB - Astaneh	00Dec96	R		RF	Hight/Adams
149	G			Caltrans	Designer's File E17-E22 Contract 6 - Bay Bridge Foundations	00Apr96	R		RF	Hight/Adams
150	A			Caltrans	Designing & Building SFOBB (13 Articles appearing in Engineering News & Record 1934-1937)	1934-1937		See Doc 26	RF	Hight/Adams
151	C	G		Caltrans	Bay Bridge PE File SFOBB Seismic Retrofit Cost Estimate (Piers E17 - E 22, mtg mins, environmental, etc.)	00Nov96	R	Mostly memos for foundations. Little cost estimate #'s.	RF, MR	Hight/Adams
152	G			Caltrans	SFOBB Retrofit Strategy for Foundations of Piers E17-E22	00Feb97	R	Good geotech ref.	RF	Hight/Adams
153	A	C		Caltrans	SFOBB East Bay Spans Summary of Meetings Memos, Piers E9-E23, 288 ft. Trusses	00Feb96	R	Note: Attachement 'F'	RF	Hight/Adams
154	S	C		Caltrans	SFOBB Research & Testing Data File -- (Lattice testing - Astaneh	2/95-7/96	R	See Doc #3	RF	Hight/Adams

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Doc	Subject			Provided by	Description	Date	REMARKS		Rev	CT Eng
##	Grp	Grp	Type				R/S		init	name
							(Retrofit / Signtr)			
155	S			Caltrans	SFOBB Design & Calculations, Design Section 4 -- (EQ Retrofit, 208' Truss spans E9 to E23, calcs w/no map)	00Feb97	R		RF, AC	Hight/Adams
156	S			Caltrans	Design Notes for Steel Members (SFOBB) Notebook #1 --(copies of tech. Papers)	00Oct95	R	Mtg w/ Astaneh. Retrofit 5/94	RF	Hight/Adams
157	A			Caltrans	Project Eng. Binder SFOBB -- (Laced Mbr. - Astaneh, work schedule data)	00Oct94	R		RF	Hight/Adams
158	S			Caltrans	SFOBB Retrofit Project 10 Spans E9-E23 --(retrofit plans)	00Jul96	R		RF	Hight/Adams
159	S			Caltrans	SFOBB (Gen. Plan) Deck Joints - Seismic Revisions E17-E23 --(&89' post EQ retrofit plans)	00Jun93	R		RF	Hight/Adams
160	S			Caltrans	Analysis of Cyclic Behavior of Existing & Retrofitted Sway Frames of SFOBB -- (by Astaneh; ABACUS modeling & results discussion)	00Jan97	R	Same as Doc #8	RF	Hight/Adams
161	S			Caltrans	Towers E5 - E16 Retrofit Summary of Results -- (Sect prop. of existing & axial cap calcs. AND alternative for all piers shown)	11/95-1/97	R	inclds summary of work	RT, RF	Sadek
162	S	C		Caltrans	SFOBB E5 - E16 Memos & Estimates -- (Includes - Interim Retrofit Strategy; encasement vs. steel, FPS bearings, etc.; Cost summary)	00Mar00	R	Good Insight to Retrofit progrsn	RF,RT	Sadek
163	S			Caltrans	E11 - E17 Existing Tower Analysis -- (all Calcs)	11/94-12/95	R		RT	Sadek
164	S			Caltrans	Towers E5 - E8 Retrofit Analysis	10/95-1/97	R	Need verif. of encsmnt strtgy.	RT	Sadek
165	S	C		Caltrans	Piers E5 - E8 Quantities --(quantity takeoff)	00Dec95	R		WH	Sadek
166	S			Caltrans	SFOBB Project File Vol. 1 - memo's, minutes, reports, isolators, seismic loads	1995	R		WH, RF	Asnaashaari
167	S			Caltrans	SFOBB Project File Vol. 2 - towers YB-1, YB-2, & YB-4 retrofit	8/95-10/95	R		RF	Asnaashaari
168	S			Caltrans	Various folder: memos, final 288' Truss Strudl, isolation model, specifications,E1 & E4 anchorage. --(Full model w/ retrofit changes)	9/95-1/97	R		RF	Asnaashaari
169	A	C	Value Analysis	Ventry	Value Analysis Summary of SFOBB East Bay Replacement Contract No. 53Y 286, Oakland, CA	7/8-8/23/96	S	Compares retrofit to replcmnt. Very Important.	BF,MR	Maroney
170	A		Value Analysis	Ventry	Value Analysis Summary of SFOBB East Bay Replacement Bridge Retrofit Project "Structural Report"	00Sep96	S	Compares retrofit to replcmnt. Very Important.	RT, MR	Maroney
171	A			Caltrans	Peir Construction Data As Built	28Jul37		Historical	MR	Moran
172	S			Caltrans	Modeling of E2-E5, 3-D Stick, 1-D, Check Calculations -- (prelim analysis)	00Apr95	R		PS	Moran
173	A			Caltrans	Papers and Reports by Others for information only	00Mar97			MR	Moran
174	A			Caltrans	SFOBB- Final Report Drawings			Historical	MR	Moran
175	G			Caltrans	Steel Sheet Piling -- (Catalog)				DG	Moran
176	G			Caltrans	Report on Cofferdams and Caissons -- (by Ben Gerwick) -- (copy of text chapter)				MR	Moran
177	S	G		Caltrans	SFOBB- Seismic Analysis of existing West Bay Span	00Nov94			MR	Moran
178	S			Caltrans	Design Criteria- Section 5 Concrete Structures (Specifications)		R	/S (replacement)	PS	Moran
179	G			Caltrans	Design Criteria- Section 10 Foundations (Specifications, standard design manual)				PS	Moran
180				Caltrans	ADINA- Software Information	94-95			PS	Moran
181	A			Caltrans	SFOBB- Annual Progress Report 1 and 2	7/34-7/35			PS	Moran
182	A			Caltrans	SFOBB- Annual Progress Report 3 and 4	7/36-7/37			PS	Moran
183	A			Caltrans	SFOBB- Annual Progress Report 5 and 6	7/38-7/39			PS	Moran
184	S			Caltrans	Structural Steel Information -- (Various Sources)	23Sep93			PS	Moran
185	S	G	Plans	Caltrans	SFOBB Drawing- Substructure East Crossing	00Jul33	R	Original Foundation Plans	PS	Moran
186	G	S	Plans	Caltrans	SFOBB Drawing- Misc. -- (EQ Retrofit, plans, specs & estimates; west span costs)	00Oct95	R		PS	Moran
187	S			Caltrans	SFOBB Seismic Retrofit Pier E-2 Modeling 2-D --(prelim analysis)	24Mar95	R		PS	Moran

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##	Grp	Grp	Type				R/S		init	name
							(Retrofit / Signtr)			
188	G	S		Caltrans	SFOBB- Foundation Analysis and Design for New Piers of the East Span	00Feb97	R	Good geotech ref.	MR	Moran
189	G			Caltrans	SFOBB- Soil Columns and Foundation Stiffness	00Jul95	R	Good geotech ref.	MR	Moran
190	A	C		Caltrans	SFOBB- East Span Caisson Information: Vol I -- (Quantities & costs)	00May96	R		MR	Moran
191	A	C		Caltrans	SFOBB- East Span Caisson Information: Vol II -- (Quantities & Cost)	00May96	R		MR	Moran
192	G	S		Caltrans	SFOBB- West Span Misc Files by EM	00Jul95	R		DG	Moran
193	G	C		Caltrans	SFOBB- Cofferdam Contract files by EM --(Quantity takeoff, no cost data)	94-96	R		DG	Moran
194	G	S		Caltrans	SFOBB- Caisson Engineering Analysis and Design	94-96	R		MR	Moran
195	A			Caltrans	SFOBB- Peer Review file by EM -- (Presentation slides)	95-96	R	Good summary	MR	Moran
196	A			Caltrans	SFOBB Misc. files by EM --(foundation and soil struct interaction info)	94-97	R		PS	Moran
197	A		Info	Caltrans	Computer Software Information -- (Adina, Microstation; soil spring info)	4/95-7/95		Reference only	PS	Moran
198	S		Plans	Caltrans	Various as-built packages for all aspects of the cantilever -- (Drawings from 1930's)			Historical	PS	VanDe Pol
199	S			Caltrans	Design Guidelines binder --(mostly Dead Load calcs and detailed backup)	yr1993	R		PS	VanDe Pol
200	S			Caltrans	Replacement Alternative No. 1 binder -- (FEA of original Alternative)		S	FEA of original Retro	AC	VanDe Pol
201	S			Caltrans	Replacement Alternative No. 2 binder -- (FEA of original Alternative)	00Aug96	S	FEA of original Retro	AC	VanDe Pol
202	A	C		Caltrans	Replacement Alternative No. 2 cost estimate --(Quantity takeoff & cost summary)	00Mar96	S	Also Retrofit?	BF	VanDe Pol
203	S	C		Caltrans	Replacement Alternative Presentation Binder --(cable stayed alternative, double deck steel; Quantity takeoff & cost summary)		S		BF, MR	VanDe Pol
204	A	C		Caltrans	Replacement Alternative Planning Estimate Binder --(cable stayed alternative)	26Aug96	S	Alternate #2 w/ Quantities	BF	VanDe Pol
205	A			Caltrans	No-Drop E2 and E3 Anchorage Top Analysis Binder --(other retrofit investigation)	06Feb96	R	Fixing E2 & E3	AC	VanDe Pol
206	S			Caltrans	No-Drop E4 Tower/Caisson Interface Analysis Binder	00Feb96	R	Fixing E4, Lead Paint	AC	VanDe Pol
207	S			Caltrans	No-Drop E2 and E3 Tower/Caisson Interface Analysis Binder	00Feb96	R	GTStrudl Run	AC	VanDe Pol
208	C			Caltrans	E1 Anchorage Retrofit Estimating Binder -- (Stringer Seats) -- (other retrofit)	00Nov96	R	Quant. Take-off and estimate	BF	VanDe Pol
209	S			Caltrans	No-Drop Strategy No. 1 Binder -- (Cantilever Spans) -- (2 new piers)	00Sep96	R	Piers to support Cant span. Outline of anal. Valuable Eng notes.	YG	VanDe Pol
210	S			Caltrans	Longitudinal SuperStructure Push No. 1 Binder -- (Modeling Data)	00Jun96	R	Modeling outline	YG	VanDe Pol
211	S			Caltrans	Bay Bridge Notes Binder	00Aug95	R	non-linear analysis	AC	VanDe Pol
212	S			Caltrans	Cantilever Structure Member Properties Binder --(handwritten)	00Aug93	R	Refer to 199. Hard to read.	AC	VanDe Pol
213	S			Caltrans	No-Drop E4 Anchorage Top Analysis Binder	00Jan96	R		AC	VanDe Pol
214	S			Caltrans	E1 Shoe Analysis Binder	00Feb96	R		AC	VanDe Pol
215	S			Caltrans	Revised Edge Truss Analysis Binder	00May96	R		AC	VanDe Pol
216	S	G		Caltrans	Replace Alt. 2 Presentation -- (Items for Cost Savings) --(cable stayed alternative)	00Aug96		Estimate	BF	VanDe Pol
217	S			Caltrans	East Bay -- West Anchor Arm Properties	00Aug96	R		AC	VanDe Pol
218	S			Caltrans	East Bay -- West Cantilever Arm Properties	00Sep96	R		AC	VanDe Pol
219	S			Caltrans	Retrofit Project No. 8 Wrap-up Binder	00Feb97	R	Decision to Replace	BF	VanDe Pol
220	S			Caltrans	E1 and E4 Articulated Barrier Binder	00Feb97	R		AC	VanDe Pol
221	S			Caltrans	Total Cantilever Structure GTS Model Binder		R		AC	VanDe Pol
222	S			Caltrans	E1 and E4 Anchor Replacement Model Binder -- (Retro Concepts)	00Oct96	R		AC	VanDe Pol
223	S			Caltrans	Articulated Barrier Analysis Binder -- (Barrier Rail Comparisons)	00Jan97	R	Concept Comparisons	AC	VanDe Pol
224	S			Caltrans	E1 Anchorage Presentation Binder (3 binders)	10/95-4/97	R		AC	VanDe Pol
225	S			Caltrans	Edge Beam 8 and 9 and Combined Binder -- (Calcs and Data)	00Jan96	R		AC	VanDe Pol
226	S			Caltrans	Edge Beam 1 to 7, BR1 and BR2 GTS Model Binder -- (Calcs, analysis and data)	00Nov95	R		PS	VanDe Pol

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##	Grp	Grp	Type				R/S		init	name
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227	S	C		Caltrans	Edge Truss Notes and Discussion Binder -- (Quantity takeoff & no cost data; Lead Paint; computer analysis and discussion)	00Jan96	R		PS	VanDe Pol
228	S			Caltrans	Top Existing + Edge Truss Push Models Binder -- (Calcs, analysis, data and discussions)	00Jan96	R		PS	VanDe Pol
229	S			Caltrans	Replacement Option Cap X-Section Binder --(computer analysis)	00Apr96	R		PS	VanDe Pol
230	S			Caltrans	E1 Anchorage Retrofit Conclusions Binder -- (Peer Review; Cant. Truss Alt Development)	00Oct95	R	Retro - Pros and Cons; good summary	BF, RT	VanDe Pol
231	S			Caltrans	Revised Longitudinal Structure Push Binder --(Computer Models, summary of analysis)	00Jun96	R		PS	VanDe Pol
232	S			Caltrans	E1 Anchorage Analysis Binder No. 4	00Jun97	R	Retro Conclusion	PS	VanDe Pol
233	S			Caltrans	E1 Anchorage Analysis Binder No. 1 & No. 2 --(analysis discussion)	00Mar97	R		PS	VanDe Pol
234	S			Caltrans	E1 Anchorage Analysis No. 3 Binder	00May97	R	details included	PS	VanDe Pol
235	S		Plans	Caltrans	E1 Anchorage Retrofit Structure Plans --(detailed plans)		R	This portion was deleted frm retrofit	PS	VanDe Pol
236	S		Plans	Caltrans	Articulated Barrier Preliminary Details			Includes 237. Need backup.	PS	VanDe Pol
237	S		Plans	Caltrans	E1 and E4 Anchorage Hold-down Preliminary Details --(Information)			Includes 236	PS	VanDe Pol
238	S			Caltrans	Binder of GTS Models -- 1, 2, & 3 of 3 --(detailed computer input)	00Nov95	R		PS	VanDe Pol
239	A		Plans	Caltrans	Tower Retrofit Details -- (Doc 239 to 241)	95-97	R	Includes Replacement	PS	VanDe Pol
240	A		Plans	Caltrans	Various Preliminary No-Drop Retrofit Details -- (Doc 239 to 241)		R	Includes Replacement	PS	VanDe Pol
241	A		Plans	Caltrans	Miscellaneous Details -- (Doc 239 to 241) -- (Doc 239 to 241)		R	Includes Replacement	PS	VanDe Pol
242	S			Caltrans	East Anchor Cantilever Arm Properties --(Detailed calcs for section properties)	00Aug96	R	Used for computer input	PS	VanDe Pol
243	S			Caltrans	Suspended Span Properties --(Detailed calcs for section properties)	00Aug96	R	Used for computer input	PS	VanDe Pol
244	S			Caltrans	East Cantilever Arm Properties	00Sep96	R			VanDe Pol
			Schedule, Reports							
245	A		Schedule	Caltrans	Retrofit construction schedule	05May00			BF	Toan
246	A		Schedule	Caltrans	Replacement construction schedule -- Govenor's Schedule	Yr2000			BF	Toan
247	A		Report	Govnr's Brd	Competing Against Time	31May90		General References	RT	Toan
248	A		Report	SAB	The Continuing Challenge, The Northridge Earthquake of 1/17/94 (prepared by the Seismic Advisory Board)	00Oct94			MR	Toan
249	A		Report	Caltrans	"The Yellow Report" Replacement Study for the East Spans of the Bay Bridge Seismic Safety Project	00Dec96	S	Maroney to Davison	RT, MR	Toan
250	A	C	Report	Caltrans	Retrofit vs. New Bridge, An Economic Analysis for the East Span of the Bay Bridge	00Apr97	R	Investment Analysis	RT, AC	Toan
251	A		Strip map	Caltrans	Strip map showing the location of individual retrofit contracts along the Bay Bridge			Ref. Only	RT	Toan
252	C		Plans	Caltrans	"The Gray Report" Cost Estimate Investigation for the East Spans Replacement	00Sep96	S	Summary Replcmnt Options	RT	Toan
253	C		Estimate	Caltrans	Seismic Retrofit Cost Summaries	30Dec96		Abridged replaced w/ expanded 7/18	BF	Traina
254	A		Legislation	Caltrans	Senate Bill 60 (SB 60) Transporation Funding for Toll Bridges	06Dec96		Ref. Only	BF	Toan
REPLACEMENT DOCUMENTS										
			New East Span							
255	A		Plans	Caltrans	65% YBI structure plans -- Transition E. Bound on Ramp	15Sep99	S	no calcs	RT	Hulsebus
256	A		Plans	Caltrans	65% Main Span structure plans -- (Suspension)	15May99	S	See 277 for later revision		Hulsebus
257	A		Plans	Caltrans	85% Skyway structure plans -- (super and sub - structures)	15Feb00	S	no calcs	RT	Hulsebus
258	A		Plans	Caltrans	65% In-Progress YBI structure/Oakland Approach structure plans	01Sep99	S			Hulsebus
259	A		Plans	Caltrans	65% Skyway structure plans (Cast in Place Alternative)	01Apr00	S	Superceded by 255 - 258	RT	Hulsebus
260			Plans	Caltrans	45% Main Span structure plans	15Jan99	S	Superceded by 255 - 258	RT	Hulsebus

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Doc ##	Subject Grp	Grp	Type	Provided by	Description	Date	R/S	REMARKS (Retrofit / Signtr)	Rev init	CT Eng name
261			Plans	Caltrans	45% Skyway structure plans	15Jan99	S	Superceded by 255 - 258	RT	Hulsebus
262			Plans	Caltrans	45% YBI/Oakland Approach structure plans	15Jan99	S	Superceded by 255 - 258	RT	Hulsebus
263	A		Report	Caltrans	30% Type Selection Report -- (Summary for replcmnt, w/ costs)	01May98	S	nice Doc. - no calcs	RT	Hulsebus
264	C		Report	Caltrans	Supplement to 30% Design Report -- (Cost summary, EDAP directives)	22Jun98	S		RT	Hulsebus
265	A	C	Report	Caltrans	Contractors Information Session -- (East bay; Estimates, Geology, Environmental, Description, Demolition)	23Jun98	S	Overview, Ref 263. Good ref.	RT, MR	Hulsebus
266	A	C	Report	Caltrans	Contractors Outreach Information -- (East bay; Estimates, Geology, Geotech, Substr info .)	01Mar99	S	Ref 263. Good Ref.	RT, MR	Hulsebus
267	A		Report	Caltrans	Replacement vs. Retrofit for East Span -- (Summary of history)	01Apr00	S	Also Retrofit. No backup	RT	Hulsebus
268	A		Minutes	Caltrans	Seismic Safety Peer Review meetings 2 Volumes. Nov 98 to Feb 00; Add 18May00 Minutes. Add 19Mar98. X-Ref Doc 303. -- (includes mtg minutes & handouts for design)	Var	S	Need calcs. Add on 9/28/00.	YG	Hulsebus
269			Plans	Caltrans	95% roadway/electrical plans for Skyway	01Jun00	S		RT	
270	A		Quantities	Caltrans	Preliminary contract quantities. Structure: SAS Oakland Approach (CIP), YBI Transition, YBI Temporary Detours, Skyway (segmental box girder). Roadway: Skyway, YBI/SAS.	Var			RT, BF	
271	A		Letter	Caltrans	MTC Planning & Design recommendations for replacement bridge	01Jul97		MTC position paper	RT	
272	A		Letter	Caltrans	Additional replacement recommendations from EDAP for Single Tower Design	04Jun98	S	MTC decision paper	RT,BF	
273	G		Geotechnical repor	Caltrans	Geotechnical related material for replacement (transferred from COE SF District) - 21 documents	var		See Doc for references	MM	
274	S	G	Plans	Caltrans	Pile Installation Demonstration Project plans -- (Boring data)	01Aug99	S		RT	
275	A		Plans	Caltrans	West Span contracts 15, 16, and 18	33-99		Clarify Dates? Titles Cntrct#'s		
276	A		Document	Caltrans	Draft Environmental Impact Statement/Statutory Exemption (DEIS)	24Sep98		minimal tech. Value	RT	
277			Plans	Caltrans	65% Main Span Suspension Bridge -- (Revised to Mar 2000)	02Aug99	S	Latest 65% after Doc 256. No Calcs	RT	
278			Plans	Caltrans	SFOBB East Viaduct, Seismic Retrofit, Description & Summary	18Jul00	S			Masroor
279			Specs	TY Lin	85% Specs, Skyway Structures, East span w/ Transmittal Letter	15Feb00	S		RT	
280			Chart	CT	SFOBB Staff Chart by Span & Project for Retrofit	19Jul00				Mac Leay
281			Plans	Caltrans	SFOBB GP/ Planning study - Alternative #1 Viaduct - Seismic analysis by Mario Velado, lead design engineer (see gray report Doc 252) (Vols. A - C)	01Oct96	R			Akinsanya
282			Plans	Caltrans	SFOBB East Span, Log of Test Borings, Projects #3 - #6. Seismic Retrofit.	00Jul97	R			Akinsanya
283			Plans	Caltrans	SFOBB Skyway Structures, 100% in Progress (01) Plans.	02Aug00				Akinsanya
284			Plans	Caltrans	65% Skyway structure plans (Precast Segmental Alternative)	15Jul99	S		RF	Akinsanya
285			Plans	Reserved	Reserved					
286			Document	TY Lin	Design Criteria, East Span. -- (ref to AASHTO and Caltrans design guides)	30Jun00	S	Incomplete. Team declares obsolete. See 367, Vol 1.	RT	
287	G		Document	Earth Mchncs	Time Histories for Conc. Piles (by Earth Mechanics)	22Feb99	S		RT	
288	G		Document	Fugro	Preliminary Pile Drivability Evaluation. East Span.	13May98	S	/retrofit	RT	
289	G		Document	Fugro	Pile Installation Demonstration Project. East Span. --(soil condition summary)	25Jun99	S	/retrofit	RT	
290	G		Document	Fugro	Executive summary - Seismic Hazard Ground Motion Criteria	15Jun99	S		RT	
291	G		Document	Earth Mchncs	Design ARS Spectra, Lateral Pile Cap & Drivability @ Oakland Mole Approach	01Jun99	S		RT	
292	G		Document	Fugro	Factual Soils Data, Borings 98-49 & 98-82	12Jun99	S		RT	
293	G		Document	Fugro	Large Offshore Hammer Inventory and Manufacturers	00Jun99	S		RT	

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294			Document	Fugro	Dynmaic t-z & p-y Method for Bay Bridge SSI Model for 2.5m Diameter Piles	09Mar99	S	Additional work for refined input & motion?	RT	
295			Letter	Astaneh	Transmittal Letter w/ copies of slides, articles, and references to information.	03Jul00	R		RT	
296			Proposal	Mayor - SF	Proposed Scope of Work w/ 17 Questions	27Jun00		Presented at 6/27 Mtg.	MR	
297			Information	Astaneh	Information to Facilitate Document Review	27Jun00				
298			Report	Astaneh	Seismic Design, Evaluation and Retrofit of Steel Bridges. Report # UCB/CEE-STEEL-96-09 ; Printed November 1996	05Jul00				
299	G	S	Information	Quest	2 Papers: 1st - 3-D Structural Influences . 1989 Loma Prienta . 2nd - Calcs of Broad Band Time Histories of Ground Motion: Comparison of 1994 Northridge	06Jul00		Ref Only		
300	A		Information	Caltrans	Caltrans Deputy Directive to Minimize Motorist Delay	15Jun00				
301	A		Information	Caltrans	Power Point Slides for Presentation, 6/28 by Steve Hulsebus & Maroney	28Jun00			BF	
302	A		Information	Caltrans	CD-ROM for Power Point Slides, Presentation, 6/28 by Hulsebus & Maroney	28Jun00				
303	A	S	Document	Caltrans	Peer Review Minutes, E. Span Retrofit, 12/3/93 to 12/13/99; X-Ref Doc 268.	03Jul00	R	Minutes reference other docs	YG	
304			Video Tape	Corps	SFOBB / Corps Kickoff meeting, 6/27 & 28/00	28Jun00				
305	A	C	Document	Caltrans	30% Design Definition for Type Selection of the Bay Bridge East spans Replacement Project	14Aug97	S	Used for Quantity Estimates	BF	
306	S		Specs	Caltrans	Structural Specifcations for 65% Main Span	25Aug99	S			Hulsebus
307	S		Specs	Caltrans	Structural Specifications for 85% Sky Way	15Feb00	S			Hulsebus
308	S		Design	Caltrans	**INDEX** of Main Span (Suspension) Design Calcs	11Aug99	S			Hulsebus
309	S		Design	Caltrans	SFOBB East Spans Retrofit-504' Truss Seismic Retrofit-Design Loading and Truss Analysis w/details (w/disks)	00Jan96	R	Strategy Notes	PS	
310	S		Design	Caltrans	SFOBB East Spans Retrofit-YBI Tower and Foundation Retrofit Seismic Model and Strategy Notes (w/disks & strategy report) --(Summary of Steel vs Conc)		R		PS, AC	Asnaashari
311	S		Design	Caltrans	SFOBB East Spans Retrofit-Steel Tower E5-E-8, E10-E16 Encasement in the Bay	00Oct 95	R	Composite Discussion	PS	Sadek
312	A		Value Analysis	CT/TVI	San Francisco-Oakland East Bay Bridge Replacement EA04-10200K, Contract 53A0005	00Jun98	S		BF	Falsetti
313			Report	TYL/DOKKEN	SFOBB East Span Seismic Safety Project-Demolition Technical Memorandum-Bridge Removal	24Apr98	S		BF	Falsetti
314			Report	TYL/DOKKEN	SFOBB East Span Seismic Safety Project-Demolition Technical Memorandum-Bridge Removal-Supplement to Section 5, 290' Span Steel Truss	24Jun98	S		BF	Falsetti
315			Report	TYL/DOKKEN	SFOBB East Span Seismic Safety Project-Cost Report-Bridge Removal	11May98	S		BF	Falsetti
316	A		Display	Caltrans	SFOBB Plan and Elevation for East Bay Spans, Display		R			Maroney
317	A		Display	Caltrans	SFOBB East Bay, Cantilever Retrofit, Seismic Deflected Shape by Adina Modeling		R			Maroney
318	A		Display	Caltrans	SFOBB East Bay Spans, Retrofit-Elevation & Global Dynamic Seismic Model		R			Maroney
319			Report	CT	Geotech Design Report, Oakland Mole Touchdown, Seismic Safety Report	00Dec99				Hulsebus
320			Minutes	CT	EDAP Meeting Minutes from 4/9/97 to 9/24/99	17Jul99				Hulsebus
321			Report	CT	Caltrans Presentations to COE on 06/28/00, 2 Vols., A & B (submitted previously on CD-ROM, Doc 302)	17Jul99				Hulsebus
322	C		Design	CT/Traina	Bid Analysis and Summaries on various contracts from 03/20/97 to 09/08/99, Add 8/08/00	11Aug00			BF, WH	Traina

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							(Retrofit / Signtr)			
323		G	Design	CT/Adams	Retrofit Strategy for the SFOBB Foundations E17 - E23 - summary --(strategy discussion and brief analysis summary)	18Jul00	R	good discussion	PS, DG	Adams
324	G		Design	CT/Maroney	Executive summary - Seismic Hazard Gound Motion Criteria	01Jul99	R	/S Design Criteria	RT	Maroney
325	S	G	Design	CT/Maroney	Adina - Global Baseline Model --(Brief summary of model. See Doc 318, Display)		R	need more detail, good starting point.	PS, AC	Maroney
326	S		Design	Caltrans	SFOBB East Bay Cantilever Retrofit, EQ Retrofit Strategy Summary Report --(detailed summary of cantilever.)	11Jul00	R	good summary	PS, BF	Van de Pol
327			Design	Caltrans	SFOBB East Bay Spans, Retrofit-Elevation & Global Dynamic Seismic Model - Diskettes(4 copies of same)	13Jul00	R			Mitchell
328	S	G	Design	CT	SFOBB East Bay Cantilever Retrofit, Seismic Summary Report at Cease Work	22Nov96			PS, BF	Lynch
329	A		Document	Governor	Governor’s Action Request (GAR) Decision to Replace Existing with New	07Feb97			MR	Hulsebus
330			Report	CT	Study - Rock Slope Stability @ YBI Piers E1 & E2 Retrofit	16Jun97	R		MR	Beck
331			Report	Earth Mech	Executive summary - Seismic Hazard Gound Motion Criteria	15Jun99	S			Lam
332			Report	Earth Mech	Revised Draft, Main Span & Skyway, Axial Pile Design & Drivability, Seismic Safety Project	01Aug99	S		DG	McNeilan
333			Memo	Earth Mech	Effects of Lateral Spreading on Pile @ Oakland Mole, Response to Comments For SFOBB East Span Seismic Safety Project	20May99	S			Law
334			Seminar	UCB	UCB Engineering Seminar - Computational Simulation of the Transient Response of Long Span Bridges, McCallen, Larsen, Astaneh.	23Nov98				
335			Report	Earth Mech	Seismic Ground Motion for SFOBB East Span Seismic Safety	24Dec98				Law
336			Report	Earth Mech	Final Marine Geotech Site Characterization, SFOBB East Span Seismic Safety Project, Vol. 1A- Main Test & Vol. 1B-Plates	00Feb99	S		DG	McNeilan
337			Report	Earth Mech	Final 2-D Marine Geophysical Survey Report, SFOBB East Span Seismic Safety Report	00Jun98	S			McNeilan
338			Report	Earth Mech	Draft-Final 3-D Marine Geophysical Survey Report , SFOBB East Span Seismic Safety	00Aug99	S			McNeilan
339			Report	Earth Mech	Final-Oakland Shore Approach, Geotech Site Characterization Report, SFOBB East Span Seismic Safety: Vol. 1-Main Text; Vol.2A-Appendices Land Borings 98 - 51 to 60; Vol. 2B-Appendices Marine Borings 98 - 39 to 44; Vol. 3-Appendices CPT Soundings; Vol. 4-Additional Reports, Memos & Testing	00Aug99	S			McNeilan
340	G		Memo	Earth Mech	Summary of Studies For Lateral Spreading of Fills at Oakland Mole, SFOBB East Span Replacement	25Aug99	S			Law
341			Vacated	Vacated	Duplicated at 294	25Jul00				
342			Report	Earth Mech	Rock Slope Stability Report For West Pier & Main Pylon, SFOBB East Span Seismic Safety	00Oct99	S			Law
343	C		Directive	Caltrans	Deputy Directive - Transportation Management Plans - Traffic Delays & contingencies.	15Jun00			BF	
344			Document	Caltrans	SFOBB East Spans Seismic Safety Project, Seismic Design Criteria, Replacement, ver12.	27Jun00	S		AC, RT	Maroney
345			Index	TYL	SFOBB East Bay - YBI Transition - **Index** for 65% Structural Design Calcs, 4 Vols. & Appdx.	??				
346			Index	TYL	SFOBB East Bay - Skyway - **Index** for 65% Design Cacs & Analysis	??	S		BF	Abbas
347			Diary	TYL	SFOBB East Bay - Seismic Safety Project - Weekly Diaries 3/97 to 6/97	19Jul00				
348			Index	Caltrans	SFOBB East Bay Spans - Testiing Programs - 8 count list	19Jul00				

SFOBB Document Inventory, Table for Data Catalog

Doc	Subject			Provided by	Description	Date	REMARKS		Rev	CT Eng
##	Grp	Grp	Type				R/S		init	name
							(Retrofit / Signtr)			
349			Document	Caltrans	SFOBB East Bay Spans - Testiing Program - Scope of work.	08Apr00				
350			Document	TYL	SFOBB - Quality Assurance / Quality Control Plan - Update	17May99				
351	C		Document	Caltrans	SFOBB East Bay Summary of Cost Estimates, Comparison of Retrofit to 30% Replacement	20Jul00			BF	
352			Letter	CT	Bolt/Fish Response To COE - Concerning 10" Displacement - SFOBB East Spans Seismic	11Jul00				Fish/Bolt
353			Specs	Caltrans	SFOBB Skyway Structures, 100% in Progress (01) Specs	02Aug00	S			Akinsanya
354			Report	CT	SFOBB East Spans Retrofit - Project Description 7/96 & Performance Criteria Statement 1/97 - by Zelinski	21Jan97	R		BF, MR	Zelinski
355			Vacated	Vacated	Vacated to Doc 25 due to duplication	25Jul00	R			
356	A		Search	Maroney/CT	Work Product by Maroney - for Astaneh Document 1992	19Jul00	R		BF	Maroney
357	S		Document	Caltrans	SFOBB East Spans YB1-E23, Analysis procedure from linear to non-linear	30Jun94	R		MR	Maroney
358			Document	Caltrans	SFOBB Cantilever Segment -Seismic retrofit, strategy selection.	yr96	R		MR	
359	A		Photos	Caltrans	SFOBB Photos w/Captions showing Interim Retrofit examples of difficulties, issues, dangers, environmental problems, and others.	31Jul00	R			Hulsebus
360			Document	Caltrans	SFOBB Retrofit Strategy 288' Trusses, E9-E23, Exit Report for Peer Review	30Oct95	R		BF	Hight
361			Document	Caltrans	SFOBB Cable Stay, Alternatives #1-#3, Cost Backup for "GAR" Alternatives, Structure cost only.	00Aug96	S		BF	Traina
362			Document	Caltrans	SFOBB Viaducts, Alternatives #1-#3, Cost Backup for "GAR" Alternatives, Structure cost only.	00Aug96	S		BF	Traina
363	G		Report	Caltrans	SFOBB East Span Seismic Retrofit Geotechnical Reports, 14 Items per Memo, 8/03/00. - Add to - Rock Motions for San Andreas Event by Norm Abrahamson, 1994	03Aug00	R	Best geotechnical reference document	MR, AC, GC	Abghari
364	S		Report	Caltrans	SFOBB Cantilever Truss Retrofit, Project #8, Edge Truss Investigation Report -- Modeling	18Jan96	R		RT	Van De Pol
365	C		Report	Caltrans	SFOBB Cantilever Truss Retrofit, Project #8, Edge Truss GP Estimate Elements (for Existing Structure Strengthen) Contract Costs and Discussion	30Jan96	R		RT	Van De Pol
366	C		Report	Caltrans	SFOBB Cantilever Truss Retrofit, Project #8, Edge Truss GP Estimate Elements (for Existing Structure Strengthen) Individual Responsiblities	16Jan96				Van De Pol
367	S		Calcs	CT/TYL	SFOBB East Bay, Suspension Span Design Calculations 65% Submittal - 41 Volumes. Vol. 1 w/ Design Criteria 4/9/99, Rev. 6.	09Apr99	S		CW, RT	
368	S		Des Code	AASHTO	Proposed LRFD Guide Specifications for Design of Segmental Concrete Bridges, January 1997	00Jan97	S			Akinsanya
369	S		Des Code	AASHTO	Guide Specifications for Design and Construction of Segmental Concrete Bridges, Proposed 2nd Edition, January 1998	00Jan98	S			Akinsanya
370	C		Document	Caltrans	SFOBB Cost-Estimate Comparison. Compares between Retrofit, 30% Replacement, and 65% Replacement.	11Aug00	R	/S	BF, WH	Traina
371	G	S		Caltrans	SFOBB GP/ Planning study - Alternative #1 Viaduct - Seismic analysis by Mario Velado, lead design engineer (see gray report Doc 252) (Vols. A - C)	12Mar96	S		BF, RP	Velado
372	A		Minutes	Caltrans	CALTRANS Seismic Advisory Board Meeting Minutes, 1/03/93 thru 10/10/95 and 12/05/95 thru 5/30/00.	17Aug00			BF	Gates
373	S		Manual	Caltrans	SFOBB Seismic Retrofit Evaluation & Design Manual for Latticed Members & Connections. Final Draft by Latticed Members Task Group.	00Apr95	R			Sadek
374	S		Calcs	Caltrans	SFOBB Retrofit for Cantilever & Towers E3, E2 - Design Calcs & Exit Notes	28Jan97	R		BF	Soon
375			Document	Boyle/CT	Exit Report for Contract 3 of SFOBB East Span, Retrofitting Cassions w/email from Moran --(Also refer to Docs 172, 180, 189, 194, 195)	22Aug00	R			Moran

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Doc	Subject			Provided by	Description	Date	REMARKS		Rev	CT Eng
##	Grp	Grp	Type				R/S		init	name
							(Retrofit / Signtr)			
376	G		Documents	Caltrans	Geotech and Foundation Reports for Proposed New East Span of SFOBB. List shows item #4 with twenty one bullets.	96 & 97	R		DG	Akinsanya
377	A		Letter	Astaneh	Astaneh Letters, to MTC 6/24/98 - Seismic Safety of Replacement; to MTC Task Force 6/20/98 - Seismic Safety w/ Attachments	24Aug00	S			Astaneh
378	S		Calcs	CT/TYL	SFOBB East Bay, Skyway Superstructure Design Calculations 65% Submittal - 17 Volumes. Vol. 1 w/ Design Criteria 7/16/99, Rev. 7.	15Jul99	S			Akinsanya
379	A		Document	Caltrans	SFOBB Retrofit, Cantilever Strategy meeting minutes from 4/95 to 11/96	24Aug00	R			Zelinski
380	A		Document	Caltrans	SFOBB Retrofit, General Team Strategy meeting minutes w/ Design Engineers from 7/94, 8/95, 1/96	24Aug00	R		PS	Zelinski
381	S		CD	Caltrans	SFOBB Suspension Span, East Pier Analysis. 2-dimensional site response, input, output, and time history info using QUAD4. 4 CD copies w/ Transmittal Letter.	06Sep00	S		CW	Hulsebus
382	C		Letter	Caltrans	SFOBB East Bay Retrofit Estimate, Explanation for Cost Calcs, "Division by 2" As shown in Doc 249.	28Aug00	S		WH	Maroney
383	S	A	Document	TYL	SFOBB SAS Bridge, Q & A for meeting at COE w/ Caltrans / TYLI. See Doc 384 for comprehensive description.	25Sep00	S		MM	Akinsanya
384	S	A	Document	TYL	SFOBB SAS Bridge, Summary Description of Design And Analysis w/ Appendix. (Add errata 10/9 and addenum 10/10.)	29Sep00	S		MM	Akinsanya
385	G		Report	Earth Mech	SFOBB East Span Lateral Pile Design for Main Span Pier E2 & Skyway Structure, Draft.	00Oct99	S		DG	Thorne
386	G		Report	Earth Mech	SFOBB East Span Geotechnical Foundation Report for Struct. Alternative, Oakland Approach, Draft.	00Jul00	S		DG	Thorne
387	G		Report	Earth Mech	SFOBB East Span Foundation Design Report, Draft Preliminary.	00Aug98	S		DG	Thorne
388	G		Calcs	TYLI	SFOBB SAS Bridge, Main Span Foundation Calcs of Input Data, Independent Check of for Adina Model #1089-03, 2 Volumes	17Apr00	S		YG	Akinsanya
389			Calcs	TYLI	SFOBB Skyway Substructure, Pile Cap, Design Calculations	04Oct00	S		RF	Akinsanya
390			Calcs	TYLI	SFOBB Skyeay Substructure, Pier Casing, Design Calculations	04Oct00	S		RF	Akinsanya
391			Report	Caltrans	SFOBB - Seismic Soil Foundation Structure Interaction by Caltrans Seismic Advisory Board Ad Hoc Committee on . Ref Doc 372, 3/26/99	26Feb99	S		JS	MacLeay
392			Specs	TYLI	SFOBB East Bay, Specification, Reference Special Provision for Modular Joint Seal Assemblies.	11Nov00	S		RF	Toan
393			Data	CT/UC-B	Tests, Mauer Expansion Joint, by U. of C. Berkeley. Dynamic Tests	12Nov00	S		RF	Akinsanya
394			Report	TYLI	SFOBB East Bay, Seismic Soil-Foundation Structure Interaction Study for Piers E2 & E7 Summary Final Report	10Oct00	S		YG	Akinsanya
395	S		Memo	TYLI	SFOBB SAS Bridge, Live Load Analysis for Maximum Deflection and Stability Analysis. Ref. Doc 384 - Summary Description.	10Oct00	S		MM	Akinsanya
396	A		Document	Governor	Executive Order No. D86-90. Requires performance criteria to be applied to bridges. Requires review by external, seismic experts.	17Oct89	R		JS	MacLeay
397	A		Document	Caltrans	List, Statewide Life Line routes w/ map. Revised Dec. 1997.	00Dec97	R	/S	JS	MacLeay
HISTORICAL DOCUMENTS										
			Historical					?Check Doc #'s and revise		
1					First to Sixth Annual Progress Report SFOBB	7/34 - 7/36		Docs 181 to 182		
2			Plans	Caltrans	Final Report drawings for SFOBB	1930's		Find & Clarify		
3			Specifications	Caltrans	SFOBB Specifications Contract No. 3 - San Francisco Anchorage	yr1933		Find & Clarify		

SFOBB Document Inventory, Table for Data Catalog

Date: 11Oct00

Doc	Subject			Provided by	Description	Date	REMARKS		Rev	CT Eng
##	Grp	Grp	Type				R/S		init	name
							(Retrofit / Signtr)			
4			Specifications	Caltrans	SFOBB Specifications Contract No. 4 and 4a - Substructure East Bay Crossing	yr1933		Find & Clarify		
5			Specifications	Caltrans	SFOBB Specifications Contract No. 5 - Yerba Buena Crossing	yr1933		Find & Clarify		
6			Construction	Caltrans	SFOBB Construction Report Contract No. 2 - West Bay Crossing	yr1933		Find & Clarify		
7			Specifications	Caltrans	SFOBB Specifications Contract No. 7- Substructure East Bay Crossing	yr1933		Find & Clarify		

Record of Revisions For Data Catalog

Date: 11/14/00

All	Mills	Data file reconciliations for initial "Table . Catalog"	30Jun00
Groupings	Schwenk / Chudgar	Input groupings	30Jun00
Assignment	many	Assign document numbers, 277 to 294	03Jul00
1 to 54	Mills/ Gianelli	Corrections, revisions, updates, remarks.	03Jul00
Assignment	Astaneh	Doc 295	03Jul00
	Mills/ Gianelli	Corrections, revisions, updates, remarks thru 192.	04Jul00
Assignment	Mills	Docs 296 & 297	05Jul00
192 to End	Mills/ Gianelli	Corrections, revisions, updates, remarks thru end	05Jul00
Assignment	Mills	Doc 298 (after printing)	05Jul00
Revision	Mills	Remove "E _Seismic" from footer	05Jul00
Assignment	Mills	Docs 299 & 300	06Jul00
Revision	Mills	Docs 256, 265, 266, & 277, 252	06Jul00
Assignment	Mills	Docs 301, 302, 303, 304	07Jul00
Revision	Wong	Docs 255, 256, 257, 263, 267, 268, 277, 286	07Jul00
Revision	Team1	Docs 3 - 7, 44, 55 - 61, 65, 67, 68, 105, 151, 163, 200, 201, 205 - 209,	07Jul00
Revision	Ghanaat	Docs 13, 15, 16, 25, 209, 210, 268, 286, 291	07Jul00
Revision	Ramsbothan	Docs 176, 195, 266	07Jul00
Revision	Turton	Docs 9, 22, 23, 25, 42, 44, 67, 72, 79, 85, 89, 117, 134, 138, 151, 161 - 164, 170, 247, 249, 250, 252, 255 - 272, 274, 276, 277, 279, 286 - 295, 299	
Assignment	Mills	Doc 305	12Jul00
Revision	Ferguson	Change initials from BF to RFish and add initials BF to Docs 27, 39, 82, 117, 123, 132, 136, 143, 169, 202-204, 208, 216, 219, 230, 245, 246, 253, 254, 301, 305	13Jul00
Revision	Fish	Rev per Fish. Docs 44, 59, 71, 105, 126, 127, 131-145, 148, 151, 153-162, 168-170, 198, 209,	14Jul00
Revision	Sauser	Docs 24, 26, 29-39, 41, 43, 81, 82, 106-111, 115, 172, 178-188, 196-199, 203, 204, 216, 226-234,	15Jul00
Revision	Grey	Docs 2, 27, 32-37, 40, 63, 175, 179, 185, 186, 190, 192, 193, 203, 268, 287-293,	15Jul00
Assignment	Mills	Docs 306 to 318	17Jul00
Revision	Chasten	Docs 55-58, 60, 65-73, 78, 80, 83, 86-88, 90, 91, 93, 94, 96, 97, 104, 105,	17Jul00
Revision	Sauser	Docs 13, 14, 19, 26, 118-125, 216, 235-243,	17Jul00
Revision	Ghanaat	Docs 57-59, 303	18Jul00
Assignment	Mills	Docs 319 to 325	19Jul00
Revision	Mills	Add Col for Caltrans Eng. Docs 53, 142, 253	19Jul00
Revision	Ramsbothan	Docs 26, 27, 38-43, 70, 85, 95, 99-103, 152, 169, 170, 188, 189, 265, 266,	19Jul00
Assignment	Mills	Docs 326 to 346	19Jul00
Revision	Ma/Heely	Docs 1 to 44, 22, 23, 27, 39, 123, 132, 136, 144, 162, 165, 193, 202, 203, 227, 264,	20Jul00
Assignment	Mills	Docs 347 to 358	20Jul00
Revision	Mills	Remove Doc 334 to place at duplicate 319.	21Jul00
Revision	Schwenk/Fish	Doc 355, moved to Doc 25 due to duplication; same -- 341 to 294	25Jul00
Revision	Schwenk	Doc 353, a draft is eliminated to final copy at Doc 335	25Jul00
Revision	Schwenk	Doc 70 is later version of 43.	25Jul00
Assignment	Mills	Doc 334	25Jul00
Assignment	Mills	Doc 359	31Jul00
Revision	MacLeay	Docs 232 change to Binder No. 4. Doc 233 change to Bndr No. 2 and add No. 1	03Aug00
Assignment	Mills	Docs 360 to 362, 364, 365	03Aug00

Record of Revisions For Data Catalog

Date: 11/14/00

Assignment	Mills	Doc 363 received but of no present use.	03Aug00
Assignment	Mills	Doc 366 reserved waiting for organization by Caltrans	03Aug00
Assignment	Mills	Docs 367 to 369	10Aug00
Revision	Mills	Add material to Doc 322 from CT/Traina	11Aug00
Revision	Mills	Add material to Doc 363 from CT/AbGhari	15Aug00
Assignment	Mills	Doc 370	16Aug00
Assignment	Mills	Docs 371 to 374	17Aug00
Revision	Mills	Add material to Doc 268, and X-Ref to Doc 303.	17Aug00
Assignment	Mills	Docs 281, 282, 375, 376, 378	21Aug00
Assignment	Mills	Docs 377, 379, 380; Combined 375 w/ 376 then added 375.	24Aaug00
Assignment	Mills	Doc 381	06Sep00
Assignment	Mills	Docs 283 & 353. Note elimination of 353 to 335 on 25Jul	07Sep00
Revision	Ferguson	Docs 123	10Sep00
Revision	Sauser	Docs 124	10Sep00
Revision	Sauser	Docs 199	10Sep00
Revision	Turton	Docs 290	10Sep00
Revision	Ramsbothan	Docs 363	10Sep00
Revision	Ghanaat	Added as reviewer to Docs 14, 58,	10Sep00
Revision	Ferguson	Added as reviewer to Docs 22, 27, 39, 44, 59, 85, 105, 142, 270, 272, 312, 313, 314, 315, 322, 326, 328, 343, 346, 351, 354, 360, 361, 362, 370, 371, 372, 374,	10Sep00
Revision	Chudgar	Added as reviewer to Docs 22, 32, 33, 100, 112, 113, 155, 211, 212, 213, 214, 215, 217, 218, 220, 221, 222, 223, 224, 225, 250, 310, 325, 344, 363,	10Sep00
Revision	Gray	Added as reviewer to Docs 100, 101, 332, 336, 376,	10Sep00
Revision	Fish	Addded as reviewer to Docs 128, 132, 134, 136, 161, 166, 167,	10Sep00
Revision	Sauser	Added as reviewer to Docs 328, 380	10Sep00
Revision	Ramsbothan	Added as reviewer to Docs 329, 330, 354, 357, 358, 363	10Sep00
Revision	Turton	Added as reviewer to Docs 344, 364, 365, 367,	10Sep00
Revision	Poeppleman	Added as reviewer to Docs 371	10Sep00
Assignment	Mills	Doc 382	12Sep00
Revision	Mills	Doc 303, Per YG, revised remark.	13Sep00
Revision	Mills	Doc 286 declared obsolete by team.	26Sep00
Revision	Mills	Doc 268, Add more minutes including dates Nov 98 back to Mar98	28Sep00
Assignment	Mills	Doc 383, 384	03Oct00
Assignment	Mills	Doc 385, 386, 387	04Oct00
Assignment	Mills	Docs 284, 388, 389, 390	05Oct00
Assignment	Mills	Doc 391	09Oct00
Revision	Mills	Doc 384, add note for errata and addenum.	11Oct00
Assignment	Mills	Doc 392	11Oct00
Assignment	Mills	Doc 393, 394, 395	13Oct00
Assignment	Mills	Doc 396, 397	16Oct00

Appendix 4. Table of Contents

Appendix 4. Retrofit Support Documents

Executive Summary for the Proposed Retrofit Alternative

This appendix discusses the proposed retrofit for the East Span of the SFOBB. The retrofit was divided into 11 separate contracts that included various portions of the proposed retrofit. Seismic and geotechnical considerations as they pertain to all contracts are discussed, followed by independent evaluation of each contract. For each contract, geotechnical and seismic issues and analysis and criteria are described and evaluated. Conclusions drawn for individual contracts do not necessarily reflect conclusions for the entire retrofit. Comments in this summary are based on evaluation of the entire retrofit proposed.

Although various alternatives had been considered, only a conceptual design had been completed for the cantilever portion. With exception of the cantilever portion of the bridge, the proposed retrofit alternative was based on an isolation strategy. Foundations and towers were to be strengthened and stiffened, isolation bearings were to replace existing bearings and the superstructure was to be strengthened at various locations. The cantilever portion of the bridge was to be modified by reinforcing the pier foundations, modifying the towers, adding two new towers, and separating the trusses into three spans.

A consistent and formal definition of performance and design criteria that pertains to the bridge as a whole has not been identified. It appears that the initial goal was to provide lifeline conditions, and ensure elastic behavior; however, this criterion was not clearly defined for all portions of the structure, and criterion is not consistent for all portions of the structure. The design criteria for the cantilever portion of the bridge were apparently relaxed and a no-collapse or no-drop criteria was considered. Analysis to substantiate the performance level was not complete.

Adequate and appropriate subsurface and physical property investigations were carried out to determine the soil properties. Appropriate procedures were employed to develop five ground motions for the site, and the maximum earthquake magnitudes and rock ground motions are appropriate. However, kinematic interaction of the foundations and soil for retrofitted foundations was not considered, and the effects of possible differential permanent displacements that could occur between the adjacent piers for a situation where one support is founded on rock (Pier E1) and another on soil (Pier E2) were not considered.

Analysis and design calculations are included in various documents, most of which are incomplete and unorganized, with few narratives, plots, figures, or tables describing the actual procedures or results. The documentation does not provide a clear statement on specific requirements regarding the type and sequence of analysis, or how the various analyses are inter-related. Several types of models of varying complexity were generated, but it is not explained how results of these various analyses were coordinated in determining retrofit strategies (Data Gap 9). For certain contracts (Contracts 4 and 5 and

9), there is very little evidence of time history analyses, and summaries of the meaning of results are not available. Verification of various models was not fully determined (Data Gap 5).

It is evident that a considerable effort was given to development and utilization of a global model of the full structure employing ADINA. There is, however, no detailed description of the model or discussion of results. Furthermore, there are questions regarding validity of the analyses that were carried out. In various analyses, it was reported that the structure damping was increased from 5% damping to 10% damping, apparently to reduce the isolator displacement demand. The level of damping used in the analysis was never justified and is inappropriate for a nonlinear dynamic analysis. In analyses that include isolation bearings (i.e. retrofitted structure) it is apparent that the properties of the retrofitted cantilever structure were not included.

Various other retrofit alternatives were considered on a local basis for given contracts. There were no other global retrofit alternatives that were considered with any level of detail.

Conclusion Statement

- The validity of a base isolation strategy has not been demonstrated and is questionable.
- The basis for a demand capacity ratio (D/C) limit of one to satisfy lifeline performance criteria has not been demonstrated and is questionable.
- Analyses were not adjusted to reflect the proposed retrofit concepts for final design efforts.
- The retrofit design was not developed to a level that substantiates the validity of the retrofit strategy.

Given the items noted above, the seismic reliability and reasonableness of the retrofit cannot be assumed. This conclusion does not imply recommendation of either retrofit or replacement. Rather, it is not clear that the decision to replace was based on a substantially completed engineering effort.

1. Introduction

This appendix describes the proposed retrofit alternative. It is organized first to cover those factors relating to the East Span retrofit as a whole, followed by a summary of the retrofit contracts and evaluation of each contract.

Considering the East Span as a whole, the current retrofit strategy considered relies heavily on one item — isolation of the superstructure. This concept resulted in modification of a long period structure that resulted in a structure with similar period. The concept, therefore, does not appear to improve the overall performance, and analyses were not conducted to substantiate the viability of this chosen retrofit alternative.

Appendix 4.

Criteria

The SFOBB has been designated as a lifeline route on the State Highway System. As such, it has been deemed critical that the bridge remain open immediately following a major earthquake for emergency response/life saving activity use. This project has been subject to a two-part performance criteria as described in the following paragraphs.

A formal definition of lifeline and associated performance and design criteria that pertains to the bridge as a whole has not been identified. It is evident, however, that various qualitative and quantitative criteria were established. In a letter from James Gates, Chief, Office of Earthquake Engineering (1993), requirements for a functional evaluation earthquake (FEE) and a safety evaluation earthquake (SEE) are specified (a similar, somewhat refined version, is provided in the 1997 GAR, [Document 329]). For important bridges including the SFOBB, immediate service level with minimal damage following the FEE event is required. Terminology such as minimal damage is ambiguous and requires further definition to define design requirements. In a performance criteria statement from January 21, 1997, the following is stated [Document 354]:

- The original performance goal was to provide full serviceability immediately following the SEE, (the maximum earthquake that the retrofit bridge is designed for). This goal was in accordance with the recommendation from the Governor s Inquiry Board following the 1989 Loma Prieta Earthquake.
- However, a series of analyses for varying strategies (apparently related to the cantilever span) and cost-benefit studies caused Caltrans management to retreat from this idealistic goal.
- The current goals will allow access to emergency vehicles within hours of the event; limited public access within one month; 3 lanes of public traffic each direction after 6 months; and full traffic after one year. (This criteria is not substantiated by quantifiable criteria.)
- The goal is designed to be consistent with expected damage on both sides of the bay in a SEE event, and the limited need for public traffic to cross the bridge under those circumstances.

On this basis, Caltrans engineers developed the following qualitative criteria:

- deck system and supports must remain elastic
- damage of service load carrying members limited to minor yielding
- minor buckling of service load carrying members allowed if capacity not reduced
- local buckling of wind bracing allowed
- permanent deflections must be less than a few inches
- expansion joints and seats will be designed for 1.25 times the maximum calculated displacement
- damage to foundations, piles, and all portions of structure below water is not allowed.

These criteria are somewhat vague, and a summary of how these criteria were met with the given retrofit is not provided in any of the documents. This statement did provide a direction during the early parts of the studies (mid-year 1995).

Criteria regarding structure response to the ground motion were not clearly defined for all portions of the structure, and consistent criteria were not utilized for all portions of the structure. A 1994 (Caltrans) letter provided preliminary guidelines on determining capacity of various steel and concrete members and specified a limiting D/C of 1.2. These criteria were provided as a guide to develop a preliminary cost estimate. Later documents pertaining to superstructure and tower retrofit show that this limit was taken as 1.0. This was consistent with the original lifeline performance goal and was followed for most of the bridge excluding the cantilever portion. The relaxation of the criteria from the lifeline criteria is apparently a result of the studies for the cantilever section. Similar specific performance criteria were never developed for foundations.

It is evident that the goal of retrofit was to provide a structure that would remain elastic under the SEE. The criteria, however, is not consistent, may be over-conservative, and were not fully developed for foundations. Elastic D/C was used for the 288 and 504 spans and these elastic criteria were apparently abandoned for the cantilever portion of the bridge. A no-drop strategy was specified and specification criteria were not developed. It appears that Caltrans relaxed the elastic requirement to a realistic goal for the cantilever spans as alternatives were identified and analyzed. Regarding the requirements to remain elastic, it is arguable as to whether such conservative limitations (D/C < 1.0) are necessary to satisfy the lifeline or service level requirements. Studies to pursue structural behavior with allowance of D/C greater than 1.0 were not conducted. For foundations, specific performance criteria in terms of acceptable D/C ratios were not developed. Designers were given the latitude to develop retrofit strategies that would accomplish the overall lifeline performance objective.

2. Seismic Evaluation

2.A. Description

The SEE ground motions for the existing and retrofitted East Span were developed deterministically for two maximum credible earthquake events on the Hayward and San Andreas faults. The maximum magnitudes for these events were based on a Geomatrix Consultants study of seismic hazard for the Northern California bridges and the 1992 study by Bolt and Gregor for the East Span. The 84th percentile ground motions were developed for each event. According to the Geomatrix probabilistic hazard assessment of ground motions for the Northern California bridges, the 84th percentile level of ground motion is between 1000- and 2000-year return period equal hazard spectra. The estimated peak bedrock ground accelerations for these events ranged from 0.55 g to 0.65 g [Documents 72, 96, 189, 325, 363, 375].

Appendix 4.

2.B. Earthquake Ground Motion Criteria

The safety evaluation earthquakes adopted for the East Span of Bay Bridge were two maximum credible earthquake events on the Hayward and San Andreas faults. A moment magnitude of M_w =7.3 was assigned to the Hayward fault located 8 km from Pier E23 and a M_w = 8 to the San Andreas Fault located 19 km from Pier YB1. The ground motions were characterized in terms of two response spectra at Pier E23 for the Hayward event and at Pier YB3 for the San Andreas event. These response spectra were developed for the 84th percentile level of ground motions using a composite of the most recent attenuation relationships. The average horizontal response-spectra obtained in this manner were adjusted to obtain fault-normal and fault parallel components. The rock motion acceleration time histories for each event were developed to match the respective response spectra and then were computed at each pier location by applying the spatial variation effects.

2.C. Rock Motions

Rock motion target response spectra were developed at Pier E23 for the Hayward event and at Pier YB1 for the San Andreas event using a composite attenuation relationship at the direction of the Peer Review Panel. Two sets of three-component acceleration time histories were developed for the Hayward event. The first set developed using the 1993 procedures was conservatively not attenuated along the length of the bridge and was judged deficient in the long period energy. Learning from the 1994 Northridge earthquake, the second set was developed to include sufficient amount of long-period energy and the recommended composite attenuation relationship. These modifications increased the target response spectra by 12% at periods greater than 1 second. The initial time histories used as a seed for the Hayward event were the Corralitos recording from the 1989 Loma Prieta earthquake. The initial seed records were then modified to be compatible with the target spectra.

Three sets of three-component acceleration time histories were developed for the San Andreas event. Again, the first set developed using the 1993 procedures was conservatively not attenuated along the length of the bridge and was deficient in the long period energy. The second and third sets developed after the 1994 Northridge earthquake were modified to include sufficient amount of long-period energy and the recommended composite attenuation relationship. The long-period energy was increased up to a period of 3 seconds for the second set and up to a period of 10 seconds for the third set. Since there are no recordings at close distances from a magnitude 8 strike-slip event, numerical simulation procedures were used to generate initial seed motions for the San Andreas event. The initial seed motions were then modified to be compatible with the respective target response spectra.

The response-spectrum-compatible time-histories at Pier E23 for the Hayward event and at Pier YB1 for the San Andreas event were further modified using the spatial variation effects to generate rock motion time histories at the location of each pier. For each event, the resulting multi-support sets of time histories are similar in waveforms and frequency contents and only differ with respect to long-period energy and attenuation along the length of the bridge.

Appendix 4.

2.D. Site Response Effects

The rock motions discussed above were further modified by propagating through soil columns to obtain site-specific ground motions at appropriate foundation levels for the structural analysis. Three-component free-field acceleration and displacement time histories were generated at Piers E1 to E23 and YB1 to YB4 using the program SHAKE. The soil properties were based on 10 borings and were interpreted for piers between the test borings. For E6 to E23, the ground motions for pile foundations were developed at the bottom of pile caps. Where the pile caps were located in the Bay Mud, the ground motions were generated at a depth just beneath the Bay Mud. The ground motions for spread footings at YB2 to YB4 were generated at the bottom of the footing. The motions at caisson piers E3 to E5 were generated at dense sand directly underlying the Bay Mud. The ground motions at pier YB1 were computed at a depth of 23 feet from the ground surface. The resulting acceleration time histories were then used to develop acceleration and displacement response spectra at these locations for the response-spectrum analysis of various segments of the bridge.

It should be noted that the foundation ground motions developed in this manner do not account for the kinematic interaction effects of the pile foundation. The kinematic interaction analysis was conducted only for existing caissons beneath the cantilever section as discussed below. The kinematic interaction analysis was also not performed for the retrofit pile foundations, which are substantially larger than the exiting timber piles.

2.E. Soil Structure Interaction

The first step of the soil structure interaction analysis consisted of the performance of seismic soil response analysis for the soil profile specific to each pier location, in the absence of the foundation structure, i.e. it constituted an analysis of the free field conditions. Our review of these analyses indicated that they were performed using reasonable assumptions and a commonly used computer program (SHAKE). Specifically the shear wave velocity profile assumed at each location was consistent with field-measured values, which are reasonable for the soil types present at each pier. The earthquake records used in the analysis were consistent with the bedrock motions selected for the SFOBB project and were appropriately input at the bedrock level.

For piers E2 through E5, which are supported on caissons, an analysis of the seismic interaction of the caisson with the adjacent soils was performed. A stick model was used for the caisson, connected to the adjacent soils by means of a series of both horizontal and vertical springs (referred to as p-y and t-z respectively). The vertical and horizontal ground motions computed from SHAKE were then applied to the end of the corresponding springs opposite the caisson at various elevations. The movements obtained for the top of the caissons were then used as input to the bridge piers with springs representing the caisson foundation. The springs were uncoupled (e.g. coupling between horizontal and rocking modes was not considered). The analyses indicated overstressing of the caissons and thus a discussion was presented on alternatives for strengthening the caisson foundations. However, an actual design for retrofitting the caissons was not included in the information available to us.

For piers E6 through E23, which are pile supported, we found no information as to whether a seismic soil-structure interaction (SSI) analysis was performed. It appears that the seismic input to the superstructure was chosen as the SHAKE output at the base of the pile cap elevation. Thus the potential change in the seismic motion caused by the presence of the piles was ignored. While this assumption would be reasonable if only the existing timber piles were present, it is not apparent that the presence of the much stiffer 60-in. diameter piles included in the retrofit can be ignored.

There was no information available to us that indicated an analysis was performed of the stresses in the piles due to the ground movements computed from SHAKE, often referred to as kinematic effect. In our opinion, these effects may be significant for the 60 in diameter piles and for any batter piles.

Permanent ground movements. The potential for permanent ground movements associated with accumulation of seismically induced strains in the soils surrounding and/or beneath the caissons and the pile foundations was not specifically addressed in the documents made available to us. Note that if such movements were to occur they may be additive to the tectonic differential movements that occur between piers. In response to this issue, the Caltrans seismic advisory board offered an estimate of less than 1 cm differential permanent bedrock movement between two adjacent piers. Although this estimate is appropriate for supports founded on rock, it may not be suitable for a situation where one support is founded on rock (Pier E1) and another on soil (Pier E2).

2.F. Appropriate Criteria

The Caltrans criteria for the maximum earthquake magnitudes and rock ground motions are appropriate. The criteria follow standard procedures that were available at that time and were later modified to account for the long-period energy observed in the 1994 Northridge earthquake. However, the effects of possible differential permanent displacements that could occur between the adjacent piers were not considered.

3. Geological and Geotechnical Site Investigations

3.A. Description

Subsurface investigation and field and laboratory testing was performed to provide data for evaluation of retrofit schemes and design. The historic data (prior to Loma Prieta) was supplemented by investigations in 1994 through 1996. Downhole geophysical measurements were made to establish and/or confirm the seismic soil properties. Additional and more extensive investigations were performed starting in 1998 for the replacement structure, but these were not available during the retrofit study.

Except for the portion of the bridge on Yerba Buena Island, the generalized subsurface profile consists of the following stratigrafic sequence: Young Bay Mud underlain in succession by the Merritt/Posey/San Antonio Formation, the Yerba Buena (Old Bay) Mud, the Upper and Lower Alameda Formation, and lastly the Franciscan Complex (bedrock). Bedrock slopes steeply from the east side of Yerba Buena Island to approximately elevation —300 ft and then slopes gently down to the east to approximately elevation —440 ft. in the vicinity of the Oakland touchdown. Yerba Buena foundation conditions consist of alluvial deposits over the Franciscan Formation or just the Franciscan.

3.B. Geotechnical Considerations [Documents 27, 267, and 363]

Sufficient subsurface exploration, insitu testing and laboratory testing exists to adequately characterize foundation conditions along the SFOBB alignment. The subsurface and physical property investigation appears adequate to support a retrofit design, including new larger and longer piles and seismic characteristics.

3.C. Condition of Existing Foundations

Only the condition of the exposed portions of the foundations was apparently checked following the Loma Prieta earthquake. No documentation of the condition of the buried foundations, in particular the timber piles, was found in the documents. Research and load tests of old timber piles was planned but was abandoned due to deterioration of the wood outside of its saturated environment [Document 2]. Caltrans notes speculate that some battered timber piles may have been damaged during the Loma Prieta earthquake [Document 373].

The assumption of minimal or insignificant deterioration of the timber after 65+ years is not unreasonable based on experience in Bay Area and previous timber pile research. Whether physical damage to the piles has occurred as a result of excess lateral loads is unknown. Caltrans apparently assumed no damage. This is a potentially significant data gap in the retrofit scheme since the intent was to add new Cast-In-Steel-Shell (CISS) piles to supplement the capacity of the timber piles. The net effect of confirmed or assumed damage to the timber would likely be to increase the cost of the retrofit.

4. Global Model

4.A. Description

Documentation of the ADINA global baseline model is incomplete and vague [Documents 72, 325]. It includes a computer-generated plot of the model, a few presentation viewgraphs, and some notes, but no description of the assumptions and discussion of the results. The notes indicate four global models, which are a fixed base model, a rocking model, a spherical isolator model, and a cylindrical isolator model. The isolators are friction pendulum bearings with a period of 5 seconds and friction coefficient of 6%. The purposes of the various global models and their relationship to local models were not defined. This was identified as Data Gap No. 9 in our Phase 1 report. From the limited output data it appears that the global model was intended to provide an estimate of the maximum displacements and the maximum loads exerted on the footings to assess the base isolation retrofit scheme. Apparently, displacement histories from the global model were used as the input for a more detailed analysis of the 504 truss system.

Appendix 4.

The superstructure, in the global model, was represented using frame elements with lumped masses. The properties of the frame members and masses were derived from separate GTSTRDL models developed for 288 , 504 , and cantilever truss spans. The lumped properties of the superstructure included axial and bending stiffness, and total mass of the truss. The mass of the truss was located at superstructures center of gravity.

The effects of pile foundation were represented by uncoupled translational and rotational springs included at the base of towers. The tower was fixed in the vertical direction, thus no vertical and torsional foundation springs were included. Mass of tremie, pedestal, and enclosed water was represented as a point mass at the center of gravity. Tremie and pedestal were considered rigid due to expected retrofit.

It appears that contact surfaces were included at the tower-foundation interface for towers YB2-YB4 and E2-E16 to model rocking response. Expansion joints at towers YB3, E4, E11, and E17-E23 were modeled using gap elements. Elements to represent friction pendulum bearings were included at Piers E4 to E23, but the adequacy of these elements in capturing the actual behavior of the bearings was not discussed.

4.B. EQ loading and Application

Earthquake loading consisted of multi-support displacement histories. The input displacement histories were obtained from the SSI analysis. Such a SSI analysis was not referenced, but we assume it is referring to the kinematic interaction analyses conducted for the caissons [Document 325]. We did not find any kinematic interaction analysis for the timber piles, which implies that input at Piers E6 to E23 were free-field displacement histories from the SHAKE analyses.

4.C. Sound Analysis

Nonlinear time-history analysis using multi-support excitation was performed for the global model. Initially a Rayleigh damping of 5% with an 8-second isolator having a friction coefficient of 6% was used, which resulted in a maximum isolator displacement of 60 in. at Pier E5. In a subsequent analysis a 5-second isolator having a friction coefficient of 6% with a 10% Rayleigh damping was employed, resulting in a reduced maximum displacement of 40 inches at Pier E4 [Document 325]. At 5% damping, the computed maximum transverse displacement (60 in.) exceeds the isolator displacement capacity of 44 inches, while at 10% damping, the maximum displacement drops below the isolator capacity. A nonlinear dynamic analysis, which explicitly models the isolators and gaps, does not justify increasing the elastic structural damping from 5% to 10%. The energy dissipation due to sliding of isolators and opening and closing of the gaps has already been accounted for through their nonlinear force-displacement relationship, and that the foundation damping effects were considered separately in development of the seismic input using SHAKE and SSI analyses.

The results of analysis are limited to tables of maximum displacements for superstructures, isolators, towers, soil springs, gaps, and footing reactions [Document 325]. No plots of displacement and force histories were provided to examine the results. The isolator permanent displacements, possible steel yielding, and the demand-capacity ratios of pile foundations were not determined. The computed force-displacement relationships for the isolators were not provided to check the accuracy of the ADINA modeling procedures.

In summary, the validity of the global baseline model and its results could not be determined. The foundation springs and the seismic input used in the model are those developed for the existing timber pile foundation and not for the retrofitted pile foundation. At a reasonable level of 5% damping, the maximum displacement demand of 5 ft exceeds the isolator displacement capacity of 44 in. It appears the isolator retrofit would work only if a damping value of 10% can be justified. Possible permanent displacements of isolators were not determined.

5. Base Isolation Considerations

5.A. Description

Seismic isolation bearings were proposed and incorporated into the retrofit scheme in response to concerns with the superstructure steel truss elements. The intent was to reduce relative horizontal displacements between each corner of a truss span thus relieving stresses caused by warping of the truss as well as reducing inertial forces of the truss mass [Documents 59, 72, 98, 146, 168].

Seismic isolation bearings were to be located at the top of each column (leg of tower) at all piers with the exception of the original support points of the cantilever structure, E1, E2, E3, and E4. The final retrofit concept that was being considered prior to termination of the retrofit project was to add two new piers, E2A and E2B, between E2 and E3. These new piers would support the joints attaching the drop-in truss segment between the two cantilever sections. The retrofit scheme is to separate the two superstructure types with the end of the cantilever sections fixed to the new pier and the drop-in section supported by isolation bearings.

5.B. Sound Analysis

The computer program ADINA was used for the overall global modeling of the SFOBB. This program has the capability of evaluating nonlinear (such as bearing stiffness and friction) and one directional effect (such as expansion joint gaps) under a dynamic time history. Typically in such an analysis, a global damping of 5% is used for the elastic range of behavior, and damping due to the nonlinear behavior is accounted for by explicit modeling of the nonlinear mechanisms.

Caltrans increased damping for the global ADINA model from 5% to 10% [Document 325]. This increase of damping reduced displacements across the bearings from about 60 inches to about 40 inches. The bearings are designed for a maximum displacement of 44 inches [Document 146] and, therefore, it is assumed the isolation design is based on the 10% damped model. The justification to increase the damping is apparently based on using a higher global damping that is attributed to soil yielding, sliding of bearings (friction pendulum type), concrete cracking, and steel yielding [Document 72]. This is inconsistent with the expected linear-elastic performance of the structural members and other considerations discussed below.

As stated in the "Description" above, the isolation strategy was to reduce stresses in the superstructure spans, caused by warping of the truss geometry and due to the inertial forces of the truss mass. As such the superstructure elements, towers and foundations, were designed to remain within the elastic range and should not be analyzed for greater than 5% damping.

The bearings were intricately modeled using the friction and restoring characteristics of the bearings. The isolator damping, whatever its value, was therefore directly accounted for in the nonlinear ADINA model. No additional damping based on isolator bearings can be justified.

Damping generated by soil deformations also seems to have been accounted for in the SHAKE and kinematic interaction analyses that generated seismic input motion and foundation springs for the ADINA model. The SHAKE analyses employed strain-dependent damping curves to produce free-field motions. Kinematic interaction analyses were based on nonlinear p-y and t-z springs to account for the nonlinear soil behavior. Therefore, no further damping for the soil-pile foundation is warranted.

In conclusion, an increase in damping above the typically accepted global application of 5% is not prudent for the retrofitted structure designed to respond elastically. An increase of damping based on inclusion of isolation bearings for the soil structure interaction effects has not been adequately justified in the documents provided.

5.C. Seismic Reliability - Lifeline Criteria or No Collapse:

The use of seismic isolation bearings is an attempt to increase the overall reliability of the structure by reducing damage to the members and joints of the existing steel truss elements. However, the bearings themselves cannot be directly evaluated under this criterion. One must look at the results of their use on various portions of the structure and evaluate the various portions as to their seismic reliability. One such area of concern is the expansion joints between the superstructure spans. Seismic isolation bearings trade off a reduction in force for increased displacement. It was noted in the Description above that the bearings would reduce the relative displacement within a truss span. However, the inter-truss displacement at expansion joints would increase. Therefore, the extent of damage to the expansion joint elements may dictate the overall time delay in allowing safe passage of emergency vehicles. Another concern is that the isolation scheme works for a 10% damping but not a 5% damping. Overall the seismic isolation strategy may not be as effective for a long period bridge as it is for short period structures, as described in 4.D. In fact the notes of the 30 October 1995 Peer Review Meeting reference a statement by Professor James Kelly of UC Berkeley, an expert in base isolation, stating "Caltrans may be making a mistake using the isolation strategy for SFOBB." We could neither find a Caltrans response to Professor Kelly's concerns nor any documentation that would support the effectiveness of such a large isolation unit for

SFOBB. Similar concerns were also raised by the seismic advisory board on several occasions [Document 372].

5.D. Validity of Isolation Strategy

Seismic base isolation is typically used to reduce seismic force demands by shifting the period of structure away from the peak of the earthquake response spectra and providing additional damping through friction or other means. As such the base isolation concept is both cost effective and technically sound for stiff structures, where seismic force demands are high.

The as-built periods of vibration for the long-period piers such as Piers E7 [Document 115] and E13 [Document 106] of the East Span of SFOBB are 6.3 and 4.2 sec, respectively. At these periods of vibration the seismic force demands for most of the as-built truss systems are not much different from those for the retrofitted bridge with the 5-sec friction pendulum isolators. On this basis the validity and effectiveness of isolation strategy is questionable because the as-built flexible steel piers transmit approximately the same level of seismic forces to the truss system. Shortening the period of vibration by stiffening piers with the concrete encasement, and then using isolators to elongate the period approximately to the same level of the as-built condition does not appear prudent.

6. Summary of Retrofit Contracts

The proposed retrofit includes work between pier YB1 on Yerba Buena Island and Pier E23 in Oakland. The SFOBB East bay span retrofit project was divided into various separate contracts organized by Caltrans. Each of these contracts is described and discussed in the following sections. The overall retrofit strategy was to reinforce the piers and foundations to provide pure elastic response, and to isolate the superstructure from the substructure by replacing the existing bearing shoes with isolator bearings. In all portions of the bridge except the cantilever, such a scheme was followed. The proposed retrofit as interpreted by the COE design team (the proposed retrofit is not defined anywhere for the bridge as a whole) is summarized by the following.

(1) Span between Pier YB1 and Pier E1 (Contract No. 2 and Contract No. 10). The YB2, YB3 and YB4 foundations were to be reinforced by enclosing the existing spread footing foundations with a new pile cap and new cast-in-drilled-hole concrete piles placed around the perimeter of the existing spread footing. The existing steel towers were to be encased in concrete and a bearing support was to be constructed at the top of the tower. Isolation bearings were to be installed, and various truss members and connections of the four 288 superstructure trusses were to be reinforced or replaced. Expansion joints were to be modified to accommodate differential truss displacements.

(2) Span between Pier E1 and Pier E4 (Contracts No. 3 and 8). This portion of the bridge includes the cantilever superstructure and supporting piers. Various strategies had been considered, and it is not clear that a final retrofit strategy had been selected for this portion of the bridge. The retrofit design was not complete

for either the superstructure or the foundations (see Sections 6.D and 6.G below). It appears that the following scheme was the selected retrofit used for cost estimating purposes.

- New piers were proposed to support the structure where the cantilever and suspended spans join. Foundations had not been designed for these piers.
- The suspended span was to be separated from the cantilever and was to be supported on isolation bearings mounted on top of the new piers. The ends of the existing cantilever spans were to be pinned to the new piers.
- The anchor span anchorages at Pier E1 and Pier E4 were to be modified such that the superstructure would be tied down and damped transverse and horizontal release would be provided.
- Superstructure retrofit was to involve reinforcement of various members and connections and a stiffening edge truss were to be added to the outside faces of the trusses (details are not known).
- Foundations at E2, E3 and E4 were to be reinforced with large diameter concrete filled steel pipe piles surrounding the caissons and precast concrete pile caps.
- Towers were to be modified by removing the lateral bracing and encasing the existing steel towers with new concrete box towers. A large concrete crossbeam was to be constructed between the tops of the box towers.
- Articulated joints were to be necessary to accommodate large differential displacements between truss segments.

(3) Span between Pier E4 and Pier E23 (Contract Nos. 4, 5, 6, 7, 9 and 10). This portion of the bridge includes five 504 truss spans (between Pier E4 and Pier E9) and fourteen 288 truss spans. The overall retrofit strategy is to stiffen the substructure and isolate the superstructure.

- The retrofit strategy for all of the foundations excluding E5 includes the installation of large diameter steel pipe piles around the perimeter of the existing foundations and pile caps and construction of new pile caps. The piles will be driven open ended, partially reamed, and the upper ends will be backfilled with reinforced concrete.
- The foundations at E5 will be reinforced with large diameter concrete filled steel pipe piles surrounding the caissons and precast concrete pile caps.
- For towers E5 through E16 (with exception of Tower E9), tower members (legs, cross braces, joints) will be encased with a reinforced concrete jacket, and a permanent concrete platform will be provided at the top. Tower E9 will include strengthening of various members by replacing lacing with perforated plates, and adding jacking platforms.
- Isolation bearings will be installed at each pier.
- Various members and connections of the 288 trusses will be reinforced or replaced.
- Various members and connections of the 504 superstructure trusses will be reinforced. In addition, an edge beam will be constructed and mounted to each

truss. The beam is a box beam that is mounted horizontally along the entire length of each truss just below the mid panel points.

6.A. Contract 2, YB2-YB4 Towers.

Description: This contract includes YB2, YB3, and YB4 towers and foundations supporting the 288 trusses on the east side of the Yerba Buena Island. YB2 and YB4 are single towers; YB3 is a double steel tower. The tower legs are made of built-up steel sections and all of the cross bracing are laced members. All of the towers are supported on spread footings. [Documents 310, 354]

The existing steel tower will remain in place and will be encased in concrete. The isolator bearings will be installed on a large ledge at the top of the towers. The new pile cap and cast-in-drilled hole concrete piles will enclose the existing footings. All existing loads and expected seismic loads will be transferred to the new foundations.

Appropriate Criteria: Caltrans established a seismic retrofit performance objective for this contract. The objective was to keep the foundation for all the towers in the elastic range during the Safety Evaluation Seismic Event. Meets lifeline criteria as stated in Section 1, Criteria.

***Geotechnical Considerations*:** Sufficient subsurface exploration, insitu testing and laboratory testing exists to adequately characterize foundation conditions along the SFOBB alignment. Information was obtained from historical sources as well as recent project specific data acquisitions.

Yerba Buena foundation conditions consist of alluvial deposits over the Franciscan Formation or just the Franciscan. Adequate foundation conditions exist to support the retrofitted SFOBB.

Existing foundations for YB1 through YB4 and E1 are shallow or deep footings bearing on rock (YB1, YB4 and E1) and soil (YB2 through YB3). Notes indicate the existing foundation capacities are not adequate and will be replaced. The new foundations at YB2 through YB4 consist of five and six-foot diameter drilled piers (CIDH piles) that will develop the required vertical (tension and compression) and lateral capacities. Detailed evaluation of existing foundation capacities and design of new CIDH piles was not found. The retrofit scheme for E1 and YB1 was not found except notes indicating the need for increased uplift capacity.

Rock slope stability on YBI at Piers E1 and E2 was evaluated using field mapping, borings, and laboratory data. A two-dimensional analysis was performed using bridge loads and earthquake accelerations. A static three-dimensional analysis was performed using a Steronet. Some of the input data for the stability analyses was not contained in the document we reviewed. Thus, we can evaluate general conclusions and not detailed results. We could not determine if a sensitivity analysis was performed. Caltrans concluded the rock slopes at Piers E1 and E2 will be stable during a maximum credible earthquake with factors of safety against sliding of about 3.5. The scope of the study and the conclusions appear reasonable.

Seismic Considerations: Refer to Sections 2.B. Earthquake Ground Motion Criteria, and 2.C. Rock Motions for seismic and earthquake considerations on this contract. Due to proposed installation of isolation bearings, it was assumed the isolation device would only transmit 0.1g longitudinally and 0.2g transversely at the top of the towers.

Sound Analysis: The SFOBB retrofit design is complex. The design presents numerous challenges and requires considerable engineering judgement for this contract. Decisions were based on a sound design strategy and appropriate analyses.

From the review of various documents, it was assumed that computer modeling was accomplished using recognized software (WFRAME, XSECTION, GTSTRUDL). Analyses were performed using typical methodology and common references and procedures. An equivalent static analysis was coordinated with a global dynamic nonlinear analysis (ADINA model).

The as-built baseline model was developed using foundation springs. The as-built model revealed that the tower legs, the laced member bracing, the anchor bolts, and the concrete pedestals and footing did not have sufficient capacity to meet the established performance criteria of D/C of less than 1.0 the D/C greater than 3.0 was reported for various structural elements.

A Push Over" model was used to determine the seismic demand on the retrofitted towers. A Caltrans in-house program was used in the push over analysis. The towers were assumed fixed at the base and allowed 1 to 1.5 inches movement of the foundation. The results from static analysis compared favorably with results from the ADINA global model analysis

Seismic Reliability: The foundation retrofit design is essentially elastic and damage to towers and foundations would not be expected. Therefore, design meets lifeline criteria.

Other Alternatives: There were two seismic retrofit alternatives considered for the existing towers and foundation. Alternative 1 was a steel strengthening and Alternative 2 was concrete encasement of the existing tower.

Cost Analysis: The cost estimates were developed to GP level. This included 10% mobilization and 20% contingencies as is standard with GP estimates. At the time the retrofit study was discontinued the cost estimate was realistic and accurate for the design level completed. It was not complete. There is adequate pricing data with backup. The towers were at a PS&E level and the foundations were at a GP level.

Appendix 4.

6.B. Discussion of Contract 3, E2-E5 Caissons

Description: This contract includes the retrofit of caissons at Piers E2 to E5. The foundations for the two new piers, E2A and E2B, supporting the cantilever spans, were apparently to be designed by the Contract 8 team.

E2 is basically a large spread footing founded on rock at the edge of Yerba Buena Island. E3 to E5 are deep caissons extending into the bay mud. The concept for the new piers, E2A and E2B, is to use large diameter steel pipe piles protruding down into the bay mud and underlying soils, with a reinforced concrete cap at the Bay s water surface.

Appropriate Criteria: Refer to Section 1. Criteria.

Geotechnical Considerations**:** The generalized subsurface profile consists of the following stratigrafic sequence Young Bay Mud underlain in succession by the Merritt/Posey/San Antonio Formation, the Yerba Buena (Old Bay) Mud, the Upper and Lower Alameda Formation, and lastly the Franciscan Complex (bedrock). Bedrock slopes steeply from the east side of Yerba Buena Island to approximately elevation —300 ft and then slopes gently down to the east to approximately elevation —440 ft. in the vicinity of the Oakland touchdown. Adequate foundation conditions exist to support the retrofitted SFOBB. See Contract 2 for geotechnical comments for E2.

Seismic Considerations: Refer to Section 2. Seismic Evaluation.

Sound Analysis: For Caissons of Piers E3, E4 and E5 - The as-built baseline model was developed using Com624P to provide an initial rough look at the behavior of the caissons. Patran and ADINA were then used to provide more refined analysis. Existing bearing capacities and settlement behavior were determined to be unacceptable in order to add load to caissons E2 through E5.

It is stated in the calculations that the resulting displacement time histories from the local caisson ADINA models were then used as the input for the global (entire bridge including superstructure) model. This poses a problem in that this procedure would not have resulted in the capture of the actual behavior of the caissons as the effect of the pier (towers) and superstructure mass were not included in the local caisson model.

The initial concept of using post-tension tendons anchored into the underlying rock layer was rejected by Caltrans Office of Structural Foundations (Geotechnical engineering). This concept would relieve stresses induced by overturning moments by redirecting the overturning forces via tension into the rock. However, the additional prestressing forces resulted in an effective increase in the vertical loading on the caissons to the point of overstressing the underlying soil. This scheme was abandoned [Document 375]. The revised and current scheme is to internally stress the caisson by coring through the outer concrete walls of the caissons, installing prestress tendons, and locking the bottom end of the tendons off against the bottom of the caissons. This would add moment capacity to the existing concrete structure but would not reduce the rocking displacements, as would the rock anchors. In addition, this alternative had several remaining concerns. First, a construction method for locking off the tendon end against the bottom of the caisson had not been developed. Second, this alternative would not be adequate to support additional dead load from retrofits of the towers and superstructure. It was the designers conclusion that an additional external support system, perhaps using large diameter pipe piles would have to be developed to carry these additional forces. No calculations or plans were provided for this concept.

In summary, it is apparent that a valid retrofit scheme had not been reached. For Caissons of Pier E2: The analysis procedure and initial retrofit scheme is similar for that described above for Piers E3, E4, and E5. However, this pier is founded on the outcropping rock of Yurba Buena Island and the prestress force of the tiedowns was not a problem. Actually the inverse was determined in that if the pier were allowed to rock, the edge stresses under the footing could overstress the rock. Therefore, the use of rock anchors to prevent rocking and control stresses at the edge of the footing was acceptable. The analysis and details provided indicate that the rock anchors would be placed around the exterior perimeter of the existing foundation. They would be attached to the existing caisson with a reinforced concrete cap surrounding the caisson.

The ADINA models provided indicate that for this pier, the towers and superstructure mass were included.

Additional Foundation for new Piers E2A and E2B: Refer to the section covering Contract 8.

Seismic Reliability: A viable solution had not been obtained as of the date the retrofit project was abandoned in favor of the complete bridge replacement alternative.

The foundations (caissons) for E2, E3, E4, and E5, with the internal prestress retrofit, are designed to remain fully elastic:

Demand < Elastic Capacity

Therefore, damage to the caissons would not be expected, which could be interpreted as meeting lifeline criteria at least for these elements. However, this is irrelevant as the internal prestressing represents only a partial retrofit. It does not address other concerns such as excessive rocking which could result in excessive displacement at the top of the towers, and settlement caused by the additional weight of the tower and superstructure retrofits.

Other Alternatives: There were two general retrofit alternatives considered for the existing caissons. Alternative one is to internally strengthen the caisson for overturning moments by installing tiedown anchorages. Within this scheme, several different methods ranging from internal stressing of the existing reinforced concrete caisson to external (protruding below the bottom of the caissons) tiedown anchorages into bedrock.

Alternative two is similar to that used for piers 6 through 23, which is an external strengthening using large diameter pipe piles around the perimeter of the existing caisson with a concrete cap tied to the caisson to produce monolithic behavior of old and new.

Alternative one, using external tiedowns into bedrock was the original concept chosen. All calculations and plans supplied by Caltrans are for this scheme. However, late in the design process a geotechnical review concluded that the additional vertical load induced by the additional tiedowns may result in unacceptable settlement due to overstressing of the underlying soils. At this point Alternative one was modified to eliminate the rock anchors and use internal prestressing. After it was concluded that this was insufficient to address all of the deficiencies associated with the existing structure, Alternative two was discussed. No calculations or plans were provided regarding this alternative.

Cost Analysis: The cost estimates were developed in a GP level. This included 10% mobilization and 20% contingencies as is standard with GP estimates. At the time the retrofit study was discontinued the cost estimate was realistic and accurate for the design level completed, GP. It was not complete.

6.C. Discussion of Contract 4, Contract 5 and Contract 6 E6-E23 Foundations.

Contract 4 Description: This contract includes the retrofit of the existing foundations at Piers E6 through E9 supporting the 504 trusses and towers. The existing steel towers will remain in place and will be encased in concrete under separate contract. The footing block rests on top of an existing non-reinforced concrete seal/pile cap and consists of numerous timber piles.

The retrofit strategy is to isolate the 504 superstructure spans from the towers with isolation bearings. Thus the substructure was designed to withstand the seismic force of approximately 20% of the superstructure mass and 100% of its self-weight.

The retrofit consists of constructing a sheet pile cofferdam, dredging mud from within the cofferdam, driving a single row of large diameter piles to a specified tip elevation around the perimeter of the existing foundation, and constructing a pile cap enlargement. A reinforcement cage will be inserted in shells and the shells will be concrete filled. The new piles were to be designed to a strength and stiffness criteria sufficient to provide composite load and displacement resistance in combination with the existing timber piles [Document 354].

Data Gap: A significant data gap exists (Date Gap 5) for Contracts 4 and 5. Without this information it is difficult to determine if sound analysis and appropriate criteria including lifeline were used for this portion of the bridge. However, it is reasonable to assume that the criteria and analysis for these contracts were similar to that used for Contract 6. Therefore, for the purpose of this study we have assumed the same conclusions on geotechnical consideration, seismic and earthquake consideration, appropriate criteria, sound analysis, seismic reliability - lifeline criteria or no collapse, and other alternatives as those concluded on Contract 6.

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Contract 5 Description: This contract includes the retrofit of the existing foundations at Piers E10 through E16 supporting the 288 trusses and towers. The existing steel towers will remain in place and will be encased in concrete under separate contract. The footing block rest on top of an existing non-reinforced concrete seal/pile cap and consist of numerous timber piles.

The retrofit strategy was to isolate the 288 superstructure spans from the towers with isolation bearings. Thus the substructure was to be designed to withstand the seismic force of approximately 20% of the superstructure mass and 100% of its self-weight.

The retrofit consists of constructing a sheet pile cofferdam, dredging mud from within the cofferdam, driving a single row of large diameter piles to a specified tip elevation around the perimeter of the existing foundation, and constructing a pile cap enlargement. A reinforcement cage will be inserted in the shells and the shells will be concrete filled. The new piles will be designed to a strength and stiffness criteria sufficient to provide composite load and displacement resistance in combination with the existing timber piles.

Data Gap: A significant data gap exists (Date Gap 5) for Contracts 4 and 5. Without this information it is difficult to determine if sound analysis and appropriate criteria including lifeline were used for this portion of the bridge. However, it is reasonable to assume that the criteria and analysis for these contracts were similar to that used for Contract 6. Therefore, for the purpose of this study we have assumed the same conclusions on geotechnical consideration, seismic and earthquake consideration, appropriate criteria, sound analysis, seismic reliability - lifeline criteria or no collapse, and other alternatives as those concluded on Contract 6.

Contract 6 Description: This contract includes the retrofit for existing foundations and pedestals at Piers E17 through E23, which support the 288 trusses. The towers and footings are hollow walls three to four feet thick. The footing block rest on top of an existing 12.5 ft. thick non-reinforced concrete seal/pile cap, supported by approximately 300 timber piles.

The retrofit strategy is to isolate the 288 superstructure spans from the towers with isolation bearings. Thus the substructure was designed to withstand the seismic force of approximately 20% of the superstructure mass and 100% of its self-weight.

The retrofit includes the construction of a cofferdam, excavation to the bottom of the existing seal course, driving large diameter steel piles, partially filled with concrete to limit the deflection in the timber piles to acceptable levels. The construction of footing and buttress walls are to connect the retrofitted and existing footing together.

Due to the reduced clearances below the trusses, dredging was planned to precede the cofferdam construction. The cylindrical steel piles will be driven to the side of the existing pile cap. The piles were to be filled with reinforced concrete. The isolation bearings will isolate the 288 superstructure spans from the towers. The foundation and tower at E17 would no longer act as an anchor pier for spans E11 through E16 [Documents 323, 354, 152, 36].

Appropriate Criteria: Caltrans established a clear seismic retrofit performance objective for this contract. The objective was to keep the foundation for all the towers in the elastic range during the Safety Evaluation Seismic Event. Localized exceptions to the totally elastic philosophy would be acceptable as long as the serviceability goal was not compromised. The serviceability goal is the often referred to lifeline criteria of being able to accommodate immediate post earthquake emergency relief access and public access within a short time frame.

Capacities of timber piles were determined using soil data at each tower, published properties of wood piles, and plans/specifications for the original construction. Vertical capacities were based on skin friction using the minimum specified pile butt and tip diameters. Some calculations used nominal pile diameters rather than the minimum [Document 74]. No uplift (tension) capacity was recommended because of the lack of a good structural connection of the pile to the cap [Document 73]. However, some calculations assumed tension capacity. We do not know if these discrepancies were resolved. Lateral capacities were determined using p-y curves. Supplemental CISS piles were designed to augment the existing timber piles by limiting deflections and bending [Document 323]. No reduction in the timber pile capacities appears to have been taken to account for possible historic structural damage.

Vertical, 5-foot diameter steel pipe piles were selected to augment the timber pile foundation capacity and control loads and deflection in the timber piles. Lateral capacities were determined using p-y curves. Drivability studies for these piles were performed and pile load tests were planned to confirm capacities. Various retrofit schemes were selected. The selected scheme appears reasonable.

Geotechnical Considerations**:** The generalized subsurface profile consists of the following stratigrafic sequence: Young Bay Mud underlain in succession by the Merritt/Posey/San Antonio Formation, the Yerba Buena (Old Bay) Mud, the Upper and Lower Alameda Formation, and lastly the Franciscan Complex (bedrock). Bed rock slopes steeply from the east side of Yerba Buena Island to approximately elevation —300 ft and then slopes gently down to the east to approximately elevation —440 ft. in the vicinity of the Oakland touchdown [Documents 27, 74, 89, 189].

Adequate foundation conditions exist to support the retrofitted SFOBB.

Seismic Considerations: A significant effort was exerted in defining the seismic setting for the SFOBB. Rock motions were determined and propagated through developed soil columns to provide a series of site specific time histories and response spectra at appropriate foundation levels. Rock motion was propagated using equivalent linear (SHAKE) and nonlinear analysis. Soil-Foundation-Structure Interaction analyses were not performed on retrofitted foundation. Foundation stiffness and force-deflection and moment-rotation curves were developed for each foundation. The kinematic interaction analysis was not performed for as-built and retrofit foundation.

Sound Analysis: The SFOBB retrofit design is complex. The design presents numerous challenges and requires considerable engineering judgment. Decisions were based on a sound design strategy and appropriate analyses. From the review of various documents, it was assumed that computer modeling was accomplished using recognized software (ADINA, COM624, GROUP, SHAKE, GTSTRUDL, etc.). Analyses were performed using typical methodology and common references and procedures. The COM624 and GROUP programs were used for the analyses of both as-built and retrofitted foundations. The demand and capacity were calculated from the above and equivalent static analysis for lateral displacement, axial, moment, shear. These results were coordinated with a global dynamic nonlinear analysis. The closing of Data Gap 9 would have confirmed our assumptions.

The stresses in the timber piles to the ultimate capacities, F_b = 5000 psi and F_v = 450 psi were used in the design of the retrofit foundation [Document 323]. The foundation needs to be retrofitted to withstand the seismic force of approximately 20% of the superstructure mass and 100% of its self weight. The large diameter steel piles were used to limit the deflection in the timber piles to acceptable level.

Design assumption for timber piles, footing stiffness, design assumption for the concrete pier foundations were reviewed [Document 152].

Strut and tie were analyze and designed using AASHTO LRFD bridge design specifications.

Seismic Reliability: The foundation retrofit design is essentially elastic and damage to towers and foundations would not be expected. Therefore, design meets lifeline criteria.

Other Alternatives: Alternative foundation retrofit strategies were considered. These alternatives included ground improvement (grouting) schemes, small diameter piles, and large diameter vertical and battered steel piles with a new pile cap / load transfer structure above the water surface.

Micropiles [Document 323] — This retrofit concept was to drill through the lightly reinforced footing block and concrete seal (elevation +8 to —45) and install micropiles to take uplift and compression. Caltrans determined the micropiles would not significantly increase the lateral capacity of the foundation and therefore this strategy was dismissed.

Floating Cofferdam - This retrofit concept was to dredge out to the bottom of the footing block and float precast concrete cofferdams into place. Then the precast units would be connected, the bottoms knocked out and large diameter steel piles driven. After the piles were driven, a new pile cap would be stressed to the existing footing block. Due to large amount of dredging and the uncertainty involved in construction, this option was not selected.

Cost Analysis: The cost estimates were develop by Caltrans represented PS&E level estimates. At the time the retrofit study was discontinued the cost estimate was realistic and accurate for the PS&E level of design. However, 100% plans and specification and estimates were not complete.

6.D. Discussion of Contract 7, E5-E16 Towers

Description: Contract 7 includes the steel towers supporting the 504 and the 288 truss spans from piers E5 to E16, with the exclusion of the steel tower at pier E9. All steel towers are supported on concrete pedestals. The steel towers at piers E5 to E8 have double X-braces that are 120 ft to 140 ft tall, whereas steel towers at piers E10 to E16 have single X-braces that are 60 ft to 90 ft tall (see Contract for E9). In addition, pier E11 has double steel towers to allow for thermal expansion and contraction. All steel towers consist of built up steel sections and rest on timber pile foundations with the exception of the steel tower at pier E5, which is supported on a concrete caisson foundation [Document 354].

The isolation retrofit strategy involves decoupling the top of the towers from the trusses by installing friction pendulum isolation bearings in between at each existing truss shoe location. First, the tower members (tower legs, X-braces and joints) are encased with a reinforced concrete jacket, and a permanent concrete jacking platforms is constructed at the top of the towers [Document 354]. The trusses are then to be jacked from the top of the towers, and the top of the tower members are to be cut. This would be followed by attaching the isolation bearing to the top of the tower members and to the bottom of the existing shoe base plate. For the low height towers at piers E13 to E16, the space between members is to be filled to form a solid in-fill wall [Documents 38, 354].

Appropriate Criteria: The portions of the criteria referenced in Section 1 that pertain to the evaluation and design of the steel towers include discussions on the allowable tensile, compressive, and shear capacities of the steel and concrete members, as well as gusset joint, shear and tensile rivet capacities. Acceptable D/C for both evaluation and design of steel and concrete members were also defined.

In 1995, more refined guidance for the evaluation and retrofit design of latticed members, rivet and gusset plate connections were published by Caltrans, based on the work of its Steel Committee and other researchers (see also "Seismic Design of Components of the East Bay Crossing" by A. Astaneh, 1995) in the so called white paper report [Document 373].

Geotechnical Considerations: Refer to Section 3. Geological and Geotechnical Site Investigations.

Seismic Considerations: See Section 2, Seismic Evaluation, for general considerations. For the local tower models, ARS curves (see Caltrans Bridge Design Specifications) were generated for each pier by Caltrans' Structure Foundations Section for pushover analyses [Documents 38, 311].

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Sound Analysis: Caltrans' analysis approach is to start with a simple model and then add complexity to it gradually until the model captures all of the system's response to an acceptable degree [Document 357]. For the stand alone steel tower analysis, this involves a simple plane frame model with in plane and out of plane motions in both the longitudinal and transverse directions. If required, foundation flexibilities, plastic deformations, rocking motions, softening of soil modulus and etc. can also be included in the analysis.

For all steel towers in this project, some or all of the following types of the analysis were performed,

- static push over analysis;
- linear elastic 3D response spectrum modal analysis;
- linear elastic time history analysis;
- nonlinear time history analysis with geometric and/or material nonlinearities.

Modeling Assumptions: Global baseline GTStrudl and ADINA models of all the existing East Bay Bridge spans were created to assess the overall seismic behavior and to capture interactions between adjacent spans, using different (rocking, fixed, soil spring) boundary conditions [Documents 180, 197]. The global results are useful for comparison with results from the local tower models. In some cases, global data were extracted and used as input for the local tower models.

For the stand alone tower models, the steel tower legs were assumed to be connected to the foundation by rigid links with translational and rotational springs. Rigid links were also used to account for connection stiffness (gussets). The contact surfaces were included at the bottom of the tower legs to allow for rocking motions. The pedestals supporting the tower legs were modeled as beam elements using actual stiffness. The masses of the foundation, pedestal, tremie, and enclosed water were modeled as a point mass, and this mass was attached to the end of a vertical rigid link. Gap elements were used at the top of the towers to model expansion joints. For retrofit analysis, base isolation bearings were modeled as springs with additional masses placed at the top of the piers.

Damping is used to dissipate energy through deformation during seismic motions. The values used for the SFOBB studies range from 5% to 15% of the critical damping depending on the type of system being analyzed and the level of peak deformation assumed [Document 72]. Each tower model was subjected to full static and dynamic loads with 3 dimensional (fault normal, parallel, and vertical) time histories. Time history analyses were carried out to 10, 20 and/or 40 seconds, with 0.02 increments. Different displacement time histories were applied to the base of each tower, accounting for wave passage, coherency, attenuation, and other local effects. Questions on the validity of global analysis regarding damping and verification of the selected strategy are discussed in Section 4, Global Model.

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In analyzing the retrofitted steel towers, the demand loads were obtained from the worst case of two methods, which were the global model using ADINA with foundation springs and displacement time histories applied at each support, and the stand alone local pushover models using GTStrudl and ADINA, with assumed isolation bearing transfer loads of 10%-20% g applied at the top. Member geometric properties were based on the cracked section, and the inertia forces were based on maximum accelerations from ARS curves generated by Caltrans' Structure Foundations Section.

Capacities of retrofitted tower members were analyzed using computer program "X-Section" and are based on the combined existing steel section and the new reinforced concrete encasement as a composite section [Document 122]. Capacities of the concrete encased joints were analyzed with "X-Section" using the combined yield capacities of the existing gusset plates and the new joint reinforcement. The main steel, which would be continuous across the jacking platform, was checked using the strut and tie method as suggested by the peer review panel [Document 311]. Buckling and slenderness of gusset plates were not considered since the joints are fully encased in concrete [Documents 161, 311].

Analyses conducted by Caltrans indicated that many members of the existing steel towers are overstressed. Some of these have the potential to buckle and induce instability. Connections were identified as possessing capacities below the capacities of the adjacent members. Given the large demand-to-capacity ratios reported in Caltrans' as-built analyses, Caltrans felt that it would not be economical to retrofit all deficient bridge members such that they will behave elastically during major seismic events.

The soundness of analysis cannot be determined independently by the COE Team. All analysis and design calculations are scattered in different documents provided by Caltrans. Most of these are incomplete and unorganized, with few narratives, figures, or tables describing what was actually done. Given the limited amount of time and the large volume of material that are available, it is not possible to thoroughly review each of these documents. However, it can be stated qualitatively that, in general, the analyses and design follow accepted current procedures and practice.

Seismic Reliability: Given acceptance of the propose retrofit (not justified by analysis as described in Section 4, Global Model), retrofit work of this contract satisfies the lifeline criteria.

Other Alternatives: Different ways to strengthen the deficient structural members were evaluated by Caltrans to various degrees. Some of those that were considered are listed as follows [Document 311].

- boxed section steel strengthening - remove lacings and rivets and replace with steel plates and high strength bolts and strengthen existing gusset plates and connections;
- hollow concrete encasement - encase existing tower with hollow concrete pier wall;
- solid concrete encasement - encase existing tower with solid concrete pier wall;

- solid X-bracing encasement - encase individual tower members with reinforced concrete jacket.

The following are reasons listed by Caltrans, however many lessons were not substantiated by analysis and the COE Team does not necessarily agree with justification. Concrete encasement was chosen by Caltrans as the preferred method for member strengthening after comparing different alternatives. The biggest advantages are to provide increased stiffness of the tower and reduce rotation at the top; minimize local buckling problems of the existing steel members, lacings, and gusset plates; avoid structural solutions for which little physical testing exists; eliminate the need to retrofit the existing steel tower anchorage by providing a continuous connection between the new base of tower longitudinal steel and the new foundation retrofit steel; provide a permanent jacking platform at the top of tower for jacking the trusses for the installation of the isolation bearings or for future replacement, and to eliminate lead paint removal and future painting and maintenance costs [Document 161]. Retrofit of the existing steel members, on the other hand, will require replacing the existing rivets with high strength bolts in addition to building up the existing steel tower legs in the longitudinal direction, which would also require staging the retrofit process in order to remove existing rivets and lacings while maintaining traffic on the bridge [Documents 311, 354].

Cost Analysis: The cost estimates were developed by Caltrans in a PS&E level estimate. At the time the retrofit study was discontinued the cost estimate was realistic and accurate for the PS&E design level completed. It was not 100% complete as design was not completed, finalized, checked, or stamped.

6.E. Discussion of Contract 8, Cantilever Towers and Superstructure.

Description: Contract 8 consists of three separate contracts. Contract 8A consists of installation of two new towers and foundations supporting the suspended span between the east and west cantilever spans (Document 188). The towers are reinforced concrete moment frames with inclined legs. The foundation consists of a precast hollow cellular pile cap with large diameter pipe piles. The piles at pier E2A are anchored 10 feet into bedrock while piles at pier E2B are driven into dense sand. Tie-downs are included to reduce tension loads on piles due to longitudinal loads. The towers support isolator bearings installed on top of the tower cross beams.

Contract 8B consists of separating the cantilever spans into 3 independent, long period structures and retrofit of towers E1, E2, E3, and E4. The proposed retrofit of pier E1 includes installing a reinforced concrete jacket around tower columns and web wall, and installing prestressed tie-down anchors through the piers into rock. Retrofit of the remaining piers includes encasing tower columns in reinforced concrete, removing tower cross bracing, installing a stiff prestressed bent cap between top of tower columns, and strengthening the connection between tower legs and pier. Connections of the suspended spans at piers E1 and E4 are released and replaced with a vertically restrained bearing.

Contract 8C consists of seismic retrofit of the three-span steel truss superstructure spanning from Pier E1 to Pier E4 to include installation of edge trusses, minor retrofit of suspended spans, Pier E1 knee joint strengthening, Pier E1 and Pier E4 release and hold-down strategies, extensive deck joint construction possibly with large dampers where joint impact cannot be avoided, installing tiedown bearings (to keep I-bars in tension) at Piers E1, E2A, E2B, and possibly E4, strengthening connections (rivet and gusset plate replacement), strengthening portal frame members, top lateral members, top and bottom cross bracing, vertical truss members, floor beams, and, anchor shoes, and extending stringer seats with steel plates [Document 227]. The investigations were in various stages of completion when the cessation directive was issued.

Appropriate Criteria: Design criteria were presented in the form of allowable stresses, material strengths, strain limits, capacity equations, and acceptable demand-capacity ratios. Performance criteria were presented in the form of expected performance of the retrofitted bridge. The initial performance criterion (1994) was to achieve full serviceability (full access to traffic immediately following the earthquake) after the design earthquake. This was to be achieved by limiting stresses to or below the elastic limit and demand-capacity ratios to unity. This goal became increasingly difficult to achieve as the analyses progressed and complexity and cost of the retrofit increased. Acceptable performance at the time the project was terminated included allowing some damage and yielding of material (1997) [Documents 224, 354]. Under this scenario, it was expected that three lanes would be serviceable in six months, and full service would be available in one year. Documentation identifying the level of excedance (D/C) of the affected components of the respective alternatives has not been identified to date.

Member capacities were based on an upper bound yield strength, e.g. Fy of 37 ksi for steel yield strength of 33 ksi. Capacities for steel members in combined axial bending or compression, flexure, shear, and torsion were based on AISC or AASHTO LRFD design criteria with increased resistance factors. Acceptable stresses in as-built truss members were limited to 60% of yield [Documents 44, 230]. This limiting value was to account for the expected controlling mode of failure (buckling) of compression members. It was reported that 94% of truss members failed under this limitation. Concern was also expressed regarding behavior of riveted connections and their impact on the capacity of truss members. Concrete and reinforcement strains for concrete towers were limited to 0.003 and 0.002 respectively. Concrete and reinforcement strengths for concrete towers were limited to 5 and 60 ksi respectively.

Geotechnical Considerations: Not applicable.

Seismic Considerations: Several computer models were developed to analyze the components of Contract 8. A global baseline model was develop for the entire structure using the ADINA computer program. A detailed model of the cantilever span trusses (from pier E1 to Pier E4) was developed using GTSTRUDL [221] and ADINA [224]. Several local models, including the E1 anchorage [Documents 222, 224], vertically restrained bearings, truss portal frames [Document 228], and pier towers and foundations [Documents 97, 116], were also developed. The separate ADINA and GTSTRUDL results were compared to develop a level of accuracy and confidence in the models [Documents 215, 228]. The documents reviewed did not indicate that a level of confidence required for this type of analysis was achieved in the models. Source of input loads was never well defined [Document 219].

Earthquake loads were represented in the form of time history displacements and accelerations used in linear and non-linear dynamic analyses and acceleration-response spectra used in linear static pushover analyses. It appears that for the time-history analyses, input loads are applied at the foundation (at least for tower and foundation analyses). Loads are applied to tops of towers for tower and foundation analyses and to truss members in the superstructure analyses (in three dimension) for pushover analyses. The origin of loads was not clear in the documents reviewed, except that accelerations of 0.1g and 0.2g for longitudinal and transverse directions respectively were generally applied to tops of towers with isolation bearings to account for bearing effects.

Sound Analyses: Reasonable procedures using well-established criteria were employed in the evaluation of steel member capacities. Material properties appear reasonable. An upper bound material strength and resistance factor is a reasonable approach. Computer modeling, analysis, and design efforts were undertaken using commercially available software and recognized procedures.

Confidence in model development and performance did not appear to be fully achieved. The requirement to achieve fully elastic behavior may have been too restrictive for a loading event expected to occur no more than once in the life of the structure. The relaxation of this requirement further into the evaluation process seems more reasonable and perhaps would lead to different conclusions if this strategy were pursued in the beginning. Results from testing of lattice members may have provided more confidence in structural performance of the existing members.

Several advantages of concrete encasement over steel retrofit of towers were presented. However, little support for these contentions was provided. The argument that concrete towers can be sized for strength and ductility, does not require testing, does not require highly specialized labor and careful staging, requires shorter construction time, and provides easier installation for isolation bearings can be made for steel as well.

Seismic Reliability: The seismic reliability of the retrofit of the cantilever superstructure cannot be assessed because Caltrans did not conclude their efforts to establish a viable retrofit strategy prior to the cessation of work on the retrofit effort.

Other Alternatives: Several other strategies for support of the cantilever truss spans were evaluated including a cable system, an edge arch system, and external strut truss system. These alternatives included retrofitted towers and in some cases additional towers. These alternatives were evaluated qualitatively, with minimal analysis, and eliminated from further study.

Cost Analysis: Not applicable.

Appendix 4.

6.G. Discussion of Contract 9, 504 Trusses From E4 to E9, Tower E9 and 50 ft Long Deck Slabs Above E9

Description: The East Span of the SFOBB includes five 504 truss spans between Pier E4 and Pier E9. Project 9 includes retrofit of the 504 trusses, tower E9, and deck slabs above tower E9, and installation of isolation bearings. All five of the trusses are to be connected to form a truss train. Tower E9, 50-ft slab supports, and truss heel areas must be modified to accommodate installation and support of the isolation bearings. The proposed retrofit for the 504 spans is shown by the General Plan — 504 Span plans (Document 31).

The proposed retrofit of the trusses involves installation of a horizontal edge beam along the length of each truss, reinforcement of vertical members, strengthening of the heel area adjacent to bearings, and strengthening of floor beam connections. The edge beam is located at approximately mid-height of the truss and is composed of a built-up 2 -8 by 2 -5 box beam. The beam is to be installed in the same plane as truss members, so the beam intersects each vertical and lower diagonal member requiring a significant bolted connection at each intersection. Four vertical members on each truss will be retrofitted by replacing the existing lacing with new perforated cover plates. Rivet removal and temporary support of un-laced members will be required. Significant reinforcing of the heel area with plates and stiffeners and local strengthening of adjacent end posts and diagonal members is required. All connections between floor beams and the supporting truss will be strengthened.

Tower E9 would have to be modified to provide a platform for jacking the trusses and installing the isolation bearings. Upper connections at each corner are to be strengthened by adding gusset plates and stiffeners resulting in significant bolted and welded connections. Tower diagonals are also to be reinforced by replacing lacing with perforated cover plates.

Due to overall structural modifications, an alternate support system is to be provided for the 50 ft span deck slabs located above tower E9. Cantilever corbels are to be mounted to the adjacent floor beam to provide an extended seat for the stringers. In addition, deck expansion joints must be installed at each end of the upper and lower deck slabs, and sidewalks must be modified.

Prior to installation of the isolation bearings, the top portion of the existing towers must be removed, anchor bolts for the bottom portion of the bearing must be installed, and adjacent truss heels must be connected. After the truss is jacked, the bearings are to be slid into position and the upper portion of the bearing is to be attached to the truss heels, and the lower portion bolted and grouted to the towers. Much drilling and specialized construction is involved.

Appropriate Criteria*:* Although a formal statement of criteria is not provided by a single document, there is evidence in several documents [Documents 60, 61, 79, 309] that specific criteria for capacity determination and D/C limits were developed for retrofit of the 504 spans. The criteria have evolved over time with the most recent criteria

[Documents 61, 354] supporting a near lifeline condition (fully elastic response in the 504 trusses to a maximum event with some offsets expected at the ends of isolated trusses). As outlined in Section 1, qualitative definitions of performance criteria have been provided. It is also evident that member capacities were well defined using appropriate criteria and capacities were calculated for all primary structural members in the 504 trusses. Concise and well-defined analysis guidelines, however, are not included.

Criteria for determining capacity of tension, compression and flexural members were developed through an extensive effort [Document 61]. Design criteria of AISC, AASHTO and CALTRANS were each considered in development of guidelines. In addition, a specific study on the strength of latticed members [Document 373] was carried out to determine truss member strengths. Extensive efforts were undertaken to determine the capacities and section properties of each structural member. The need for retrofit was determined based on a D/C limit of 1.0.

Geotechnical Considerations: Not Applicable.

Seismic Considerations: For the superstructure, earthquake loads are based on accelerations of 0.2g in the transverse direction and 0.1g in the longitudinal direction. This is justified on the basis that friction pendulum isolation bearings are used to support the structure.

Sound Analysis: It is stated in various documents that linear, nonlinear and time-history analyses (THA) have been conducted. A global model of the entire east portion of the SFOBB, local truss models and detailed finite element models of various connections were developed and utilized. It is not clear, however, how these analyses were coordinated. The documentation does not provide a clear statement on specific requirements on analysis type, description of required sequence for type of analysis, or how various analyses are inter-related.

In a December 1996 presentation to the pier review panel, it is stated that linear (GTSTRUDL), nonlinear (ADINA), and dynamic time history analyses (THA) were conducted to determine the as built and retrofitted performance of the structure [Document 61]. A single span space frame model and a plane frame model of all five spans were developed. Additionally, several detailed models of various connections were developed. The time-history analysis (THA) is not described and no summary of results is provided.

Special analyses were conducted for jacking and temperature. Finite element models were developed to model gusset plates at the truss support heel [Document 309]. Additionally, a temperature analysis was conducted. In the temperature analysis, it was assumed that the truss was fully expanded when the towers were retrofitted, and then after retrofit of the towers, the truss was allowed to contract with temperature. Shears at support points were checked to determine if sliding of the joint would occur. The document states that all shears were under the limit of 0.7 times the dead load reaction, so sliding would not occur. This is inconsistent with the friction value of the isolation bearings, which is between 0.1 and 0.2.

Static push analyses using a 3-D model were conducted for longitudinal and transverse loading [Document 58]. Static loads that are equivalent to 0.2 times the dead load were applied at end nodes for longitudinal analysis and at truss panel points for lateral analysis. Member force results were provided; however, the results do not correspond with any other results presented. Push analyses were not used to determine ultimate capacity and failure mechanisms of the truss.

Analysis Results: In a January 1996 strategy meeting, existing structure and retrofitted D/C plots for a general 504 truss model were presented [Document 309]. In a December 1996 peer review meeting plots for a general 504 model and for models that represent spans E4, E6 and E8 were presented [Document 61]. These apparent most recent results show that the retrofitted structure would have all members with a D/C of less than 1.0 with exception of four vertical members in span 8 (all are less than 1.3). In early to mid 1996, results for capacity calculations and demand to capacity ratios are shown by spreadsheet type printouts [Document 57]. Results in each document are consistent for the general case 504 model; however, results for spans E4, E6 and E8 are not consistent. In no case are the input loads described. This presents some confusion in the interpretation of results and it is not clear exactly what type of analysis was conducted to determine the demands used in formulation of the results.

There is evidence that appropriate analysis was carried out, but due to the inconsistencies and lack of summary, a definite conclusion cannot be made.

Seismic Reliability: The retrofit strategy involves isolation of the superstructure so the superstructure demands are dependent on the performance characteristics of the isolation bearings. The reported analysis loads have been based on these characteristics; however, displacement demands are not well summarized. Given the conservative definition of capacity and conservative D/C limit, high seismic reliability is likely considering force effects on the structure. Regarding displacement effects, however, a solution to account for the effects of displacement demands and interaction with adjacent truss trains (impact between unattached segments of structure) has not been developed or is not documented appropriately. Furthermore, the global analysis was not conducted using retrofitted tower and foundation properties, so the isolation strategy was not verified.

Considering only the structural aspects of the 504 truss spans (not including end of truss displacements), a lifeline condition is apparently met. The D/C limitation of 1.0 ensures elastic behavior in the superstructure trusses.

Other Alternatives: Document 60 presents two alternative schemes. Both schemes would employ additional towers placed adjacent to existing towers. In the first alternative, the truss would be isolated from the existing tower and new additional towers would be constructed to catch an un-seated truss. These towers would be located under the bridge to the east and west of each pier. The second alternative would include a set of towers placed on either side of the bridge adjacent to each existing tower. These would not support the trusses, but would restrain lateral movement of the trusses during an earthquake. Analysis of towers was conducted and a planning budget level design was completed. These alternatives were examined to provide cost information for alternative schemes.

Cost Analysis: The cost elements were developed by Caltrans in a GP level. Included are 10% mobilization and 20% contingencies as standard at the GP level. At the time retrofit study was ordered discontinued, the cost estimates appear to be realistic and accurate for the GP level. It was not complete.

6.H. Discussion of Contract 10, 288 Trusses

Description: Contract 10 includes all the superstructure work from piers YB1 to E1 and from piers E9 to E23. These spans include four 288 trusses between pier YB1 and pier E1 on Yerba Buena Island and fourteen 288 trusses between pier E9 and pier E23.

The retrofit strategy selected by Caltrans is to substitute isolation bearings for existing bearing shoes at various locations from pier E9 through pier E23 to reduce excessive relative truss displacements and to reduce shock due to impact. The trusses from pier E9 to pier E11 and from pier E11 to pier E17 are to be connected as in the existing condition. The trusses from E17 to E23 are also to be connected in similar manner. The thermal expansion joints at piers YB1, E1, E9, E11, E17, and E23 at the ends of the truss train will be modified to allow for the significant displacement associated with the isolation bearings [Documents 146, 354, 360].

The truss vertical hanger members are to be strengthened to prevent excessive drift. New lateral bracings will be added and existing diagonal and vertical bracings will be strengthened to increase load carrying capacity and to allow load transfer to deck without inducing bending of the floor beams about their weak axes. Existing knee bracings will be upgraded to improve shear and torsion resistance. In addition, longitudinal bracings are to be added, reinforcement of connections between stringers and floor beams, diagonals, lower chords, upper chords, gusset plates, sidewalks, barriers, floor beams are to be strengthened, I-bar assemblies are to be added and new diagonal kickers will be installed to restrain upper deck floor slab movement [Documents 29, 79, 155, 158, 159, 360].

Appropriate Criteria: The portions of the criteria referenced in Section 1, Criteria, that pertain to the evaluation and design of the 288 span superstructure include discussion on the allowable tensile, compressive, and shear capacities of the steel members, as well as gusset joint, shear and tensile rivet capacities. Acceptable D/C for both evaluation and design of steel members were also defined.

In 1995, more refined guidance for the evaluation and retrofit design of latticed members, rivet and gusset plate connections were published by Caltrans, based on the work of its Steel Committee and other researchers (see also "Seismic Design of Components of the

East Bay Crossing" by A. Astaneh, 1995) in the so called white paper report [Document 373].

Geotechnical Considerations: Refer to Section 3, Geological and Geotechnical Site Investigation.

Seismic Considerations: Refer to Section 2, Seismic Evaluation.

Sound Analysis: Caltrans' analysis approach is to start with a simple model and then add complexity to it gradually until the model captures all of the system's response to an acceptable degree [Document 357]. For the stand alone segment analysis, this involves a series of simple plane frames with beam elements in between subjected to seismic motions in both the longitudinal and transverse directions. If required, lumped masses, foundation flexibilities, plastic deformations, rocking motions, softening of soil modulus and etc. can also be included in the analysis.

For all segment models in this project, some or all of the following types of the analysis were performed,

- static push over analysis;
- linear elastic 3D response spectrum modal analysis;
- linear elastic time history analysis;
- nonlinear time history analysis with geometric and/or material nonlinearities.

Modeling Assumptions: Global baseline GTStrudl and ADINA models of all the existing East Bay Bridge spans were created to assess the overall seismic behavior and to capture interactions between adjacent spans, using different (rocking, fixed, soil spring) boundary conditions [Documents 180, 197]. The global results are useful for comparison with results from the local segment models. In some cases, global data were extracted and used as input for the local segment models.

The 288 spans in the stand alone segment models were modeled by interconnected beam elements with lumped masses at the quarter points along the superstructure length between steel towers [Documents 76, 113]. The steel tower legs were assumed to be connected to the foundation by rigid links with translational and rotational springs. Rigid links were also used to account for connection stiffness (gussets). The contact surfaces were included at the bottom of the tower legs to allow for rocking motions. The pedestals supporting the tower legs were modeled as beam elements using actual stiffness. The masses of the foundation, pedestal, tremie, and enclosed water were modeled as a point mass, and this mass was attached to the end of a vertical rigid link. Gap elements were used at the top of the towers to model expansion joints. For retrofit analysis, base isolation bearings were modeled as springs with additional masses placed at the top of the piers.

Damping is used to dissipate energy through deformation during seismic motions. The values used for the SFOBB studies range from 5% to 15% of the critical damping depending on the type of system being analyzed and the level of peak deformation assumed [Document 72]. Each segment model was subjected to full static and dynamic loads with 3 dimensional (fault normal, parallel, and vertical) time histories. Time history analyses were carried out to 10, 20 and/or 40 seconds, with 0.02 increments. Different displacement time histories were applied to the base of each tower, accounting for wave passage, coherency, attenuation, and other local effects. Questions on damping and verification of selected strategy regarding the validity of global analysis are discussed in Section 4, Global Model.

In analyzing the retrofitted superstructure, the stand alone local structural models were extracted from the global baseline model with all spans assembled as trains, assuming that all the 288 trusses are coupled both transversely and longitudinally [Document 146]. These segment models were analyzed as a series of lollipops, including refinements such as material nonlinearities, banging between adjacent spans and superstructure damping [Document 113]. Member geometric properties were based on the cracked section, and the inertia forces were based on maximum accelerations from ARS curves generated by Caltrans' Structure Foundations Section.

The basic steps in the analysis procedures are:

- establish demands using results from the global baseline model and perform stand alone analysis on the segment models;
- establish member capacities using criteria discussed above;
- strengthen member or redistribute loads to other members when the demand is greater than its capacity.

Analyses conducted by Caltrans indicated that many members of the existing steel superstructure are overstressed. Some of these have the potential to buckle and induce instability. Connections were identified as possessing capacities below the capacities of the adjacent members. Given the large demand-to-capacity ratios reported in Caltrans' as-built analyses, Caltrans felt that it would not be economical to retrofit all deficient bridge members such that they will behave elastically during major seismic events.

The soundness of analysis cannot be determined independently by the COE Team. All analysis and design calculations are scattered in different documents provided by Caltrans. Most of these are incomplete and unorganized, with few narratives, figures, or tables describing what was actually done. Given the limited amount of time and the large volume of material that are available, it is not possible to thoroughly review each of these documents. However, it can be stated qualitatively that, in general, the analyses and design follow accepted current procedures and practice.

Seismic Reliability: Given acceptance of the isolation strategy (not justified by the global analysis, see Section 4, Global Model), retrofit work of this contract satisfies the lifeline criteria.

Appendix 4.

Other Alternatives: Different ways to strengthen the deficient structural members were evaluated by Caltrans to various degrees. Some of those that are listed below were eventually incorporated into the project:

- "surgery" - remove existing members and joints one by one and replace with new ones [Document 358];
- modification - attach new plates, angles, beams and/or other structural shapes to stiffen existing members;
- addition - add new, but separate plates, angles, beams and/or other structural shapes to existing structural system to redistribute demand loads.

Cost Analysis: The cost elements were developed by Caltrans in a GP level. Included are 10% mobilization and 20% contingencies as standard at the GP level. The cost estimates appear to be realistic and accurate for the GP level achieved at the time retrofit study was ordered discontinued. It was not complete.

6.I. Other Alternatives

Even though significant additional work would be necessary to complete the proposed retrofit scheme, Caltrans has adequately considered other retrofit alternatives. There is no other global retrofit alternative defined using a consistent retrofit strategy for the entire bridge. There are, however, many local alternatives that were explored in the development of each individual contract. Many of these alternatives were disregarded for various reasons prior to developing a detailed alternative design. In many cases, it was not possible to develop a complete and accurate cost estimate, and it is not possible to determine whether or not many of these alternatives would satisfy lifeline criteria. The design process is an evolving process in which many alternatives are given due consideration without completing a design and cost estimate. Because many factors control design decisions, complete and accurate cost analysis is not always necessary in the decision process. Some of the considered alternatives are summarized below.

(1) Regarding the towers, a steel strengthening and various concrete encasement seismic retrofit alternatives were considered. Concrete encasement was selected based on several factors.

(2) Alternative foundation retrofit strategies were considered. These alternatives included ground improvement (grouting) schemes, small diameter piles, and large diameter vertical and battered steel piles with a new pile cap / load transfer structure above the water surface.

(3) For the 504 span trusses, two alternative schemes that employ additional towers placed adjacent to the existing towers were considered. In the first scheme, the additional towers serve to restrain lateral movement of the trusses, and in the second scheme, the towers provide platforms to catch an unseated truss.

(4) Several strategies for support of the cantilever truss spans were evaluated including a cable system, an edge arch system, a superstructure frame, a

substructure frame, and an external strut truss system. These alternatives included retrofitted towers and in some cases additional towers. These alternatives were evaluated qualitatively, with minimal analysis, and eliminated from further study.

(5) There were two general retrofit alternatives considered for the existing caissons (E2 to E5). Alternative one is to install tiedown anchorages and the second alternative was to add large diameter pipe piles around the perimeter of the existing caisson with a concrete cap tied to the caisson.

Appendix 5. Review of Originally Proposed Replacement Alternative (Skyway or Viaduct Type Bridge)

Purpose of Review: Review of the originally proposed replacement design alternative assists in answering Question 2 from the scope of work. Question 2 as stated in the scope follows:

Was Caltrans' cost-benefit analysis comparing the originally proposed replacement alternative vs. the proposed retrofit alternative reasonable -- i.e., was it based on appropriate criteria and sound analysis, including consideration of realistic, accurate and complete cost figures?

This alternative essentially is the least cost replacement alternative that provides the required seismic performance (lifeline). It was used by Caltrans in late 1996 and early 1997 to compare cost of the retrofit alternative to a replacement alternative and was the primary basis for recommending replacement over retrofit. Replacement was recommended by Caltrans and the State of California in early 1997. **This review primarily focuses on the appropriateness of the design effort as related to the alternative costs that were used in the comparison. A discussion of lifecycle costs is also given.**

Documents Reviewed and Chronology of the Skyway Design

Doc No.	Provided by	Description or Title	Date
371B	Caltrans	Skyway Design Calculations — 300ft spans	03/96
371C	Caltrans	Skyway Design Calculations — 500ft spans	08/96
169	Ventry Engr	Value Analysis Summary of SFOBB Replacement	08/96
252	Caltrans	The Gray Report Cost Estimate Investigation for the East Spans Replacement	09/96
170	Ventry Engr	Value Analysis Summary of SFOBB Replacement Bridge Retrofit Project — Structural Report	09/96
23	Ventry Engr	San Francisco-Oakland Bay Bridge East Bay Crossing Replacement Value Analysis Findings	12/96
249	Caltrans	The Yellow Report Replacement Study for the East Spans of the SFOBB Seismic Safety Project	12/96
329	Caltrans	Governor s Action Request (GAR)	02/97
250	Caltrans	RETROFIT VS. NEW BRIDGE	04/97
263	Caltrans	30% Type Selection	05/98
276	Caltrans	Draft Environmental Impact Statement (DEIS)	09/98
267	Caltrans	Replacement vs. Retrofit	04/00

Appendix 5

Description of Alternative: The originally proposed replacement alternative is generally described as haunched girder skyway structure and would follow the same alignment as any other replacement bridge. This type of structure makes up a significant portion of the SAS and Cable Stayed alternatives with the difference being in the main span. Document 276 describes the Skyway Design as a structure constructed of either concrete or steel, supported from under the bridge by piers. With this structure type, each bridge would be constructed as a separate, independent structure. Under the skyway design alternative, spans over the navigation channel area could be a maximum of 490-550 feet in length which would require 3 spans as compared to the 2 spans for the self anchored suspension and cable-stayed alternatives. It is noted that there were several variations of the skyway alternative over time and subsequently several different cost estimates. Indeed, the cost estimate used in two of the primary reports (docs 249 and 250) that demonstrated the lifecycle cost superiority of the replacement over the retrofit approach was a composite of several different replacement alternatives including cable stayed.

Geotechnical Consideration: Geotechnical information for the 1996 designs was extrapolated from the retrofit alternative. Site-specific geotechnical data was developed for the 1998 design.

Seismic and Earthquake Consideration: Site-specific seismology for the retrofit alternative was utilized for the 1996 replacement designs. This essentially included the response spectras for six different ground motions. The 1998 design also utilized the retrofit ground motions as the updated ground motion data was not yet complete. A significant effort was exerted in defining the seismic setting for the SFOBB. Rock motions were determined and propagated through developed soil columns to provide a series of site-specific time histories and response spectra at appropriate foundation levels. Rock motion was propagated using equivalent linear (SHAKE) and nonlinear analysis. Soil-Foundation-Structure Interaction analyses were also performed.

Sound Analysis/Design Criteria: The following documents were reviewed that contain design information and/or calculations related to the originally proposed replacement alternative (skyway/viaduct).

1) Documents 371B and 371C contain design calculations that appear to be for the viaduct designs contained in document 252 with 371B including designs of 300 ft spans and 371C including designs of 500 ft spans.
2) Document 252 contains design criteria, drawings and cost estimates for the preliminary replacement alternatives. The originally proposed replacement alternative is not actually included in this document though a similar skyway/viaduct alternative is.
3) Document 170 contains seismic design calculations (pushover analyses, column design, foundation evaluations), drawings and cost estimates for the VA alternatives.
4) Document 263 contains 30% design level drawings, design criteria, costs, geotechnical data and selected analysis results and discussions. The Skyway was analyzed using a SAP2000 global model that included nonlinear springs and

beams to represent the pile foundations. Response spectrum analysis was used with this model. ADINA was used to perform 2D and 3D parametric analyses of the pile foundations (soil-foundation-structure interaction). This included nonlinear soil properties and inelastic properties of the piles and columns.

The design and analysis for the originally proposed replacement appears to be reasonable and appropriate for the level of design. The more detailed 1998 design confirms the adequacy of the earlier designs.

Seismic Reliability - Lifeline Criteria or No Collapse: The design intent of the originally proposed replacement alternative was to meet lifeline criteria, with seismic events defined and the expected performance for the events given [Document 252]. Ductility and displacement goals are also given [Document 252]. The 1996 versions of this alternative were based on preliminary design efforts (appropriate) and therefore the design to lifeline criteria is not actually demonstrated; however, based on the provided design documents it is clear that Caltrans and various consultants felt very comfortable that this bridge type could be designed to lifeline standards and that this bridge type would provide the best seismic performance, i.e. better than cable stayed or suspension bridges. Caltrans 30% design (1998) describes a more refined design effort with performance criteria, site-specific data and selected design/analysis results though no actual calculations/analyses were provided [Document 263]. It is concluded, based on the documents provided and engineering judgment, that this bridge type would provide better seismic performance than the other alternatives due to its relatively simple design and construction. The designs used for the comparison to the retrofit are reasonable and appropriate for the intended purpose and are representative of a lifeline bridge. The skyway design, which was completed after the replacement versus retrofit decision, appears to confirm the earlier designs and cost estimates [Document in 263].

Cost Figures — Realistic, Accurate and Complete: Though there were variations in the alternative over time and different reported costs, it does appear that reasonable first cost figures were developed for the originally proposed replacement alternative and the cost figures used in the primary decision reports may have actually been conservative.

Reported Costs for Skyway/Viaduct Alternative

Doc No.	Provided by	Description or Title	Date	Reported Cost (1)
169	Ventry Engr	Value Analysis Summary of SFOBB Replacement	08/96	605
252	Caltrans	The Gray Report Cost Estimate Investigation for the East Spans Replacement	09/96	NA (2)
170	Ventry Engr	Value Analysis Summary of SFOBB Replacement Bridge Retrofit Project — Structural Report	09/96	660
23	Ventry Engr	San Francisco-Oakland Bay Bridge East Bay Crossing Replacement Value Analysis Findings	12/96	797
249	Caltrans	The Yellow Report Replacement Study for the East Spans of the SFOBB Seismic Safety Project	12/96	987 (3)

Appendix 5

Doc No.	Provided by	Description or Title	Date	Reported Cost (1)
329	Caltrans	Governor s Action Request (GAR)	02/97	1,075
250	Caltrans	RETROFIT VS. NEW BRIDGE	04/97	990
263	Caltrans	30% Type Selection	05/98	1,100 (4)
276	Caltrans	Draft Environmental Impact Statement (DEIS)	09/98	1,200 (4)
267	Caltrans	Replacement vs. Retrofit	04/00	1,170

(1) Millions of $, Includes Construction and design costs, including approaches, demo, interim retrofit, temporary structures.
(2) Report did not include original replacement alternative, however a similar viaduct structure was presented with a $531/SF cost.
(3) Average of 4 different replacement alternatives including 2 cable-stayed and 2 viaduct types.
(4) 1998 dollars, DEIS included rounding otherwise the same as 30% Type Selection

Summary: The design for the originally proposed replacement was based on appropriate criteria and sound analysis, which results in realistic, accurate, and complete costs.

Appendix 6. Currently Proposed Replacement Support Documents

Appendix 6

The COE Team has reviewed numerous documents related to the currently proposed replacement (SAS) alternative. The reviews are summarized in this narrative description and attached detailed review sheets.

1. Design and Performance Criteria

1.A. Identification of Performance and Design Criteria [Document 367, Volume 1]

Virtually all U.S. state highway departments use the AASHTO standard code for bridge design and construction. Because of special considerations in California, Caltrans BDS code is used; it is based on the AASHTO code, but with exceptions. The ATC-32 guide (code) augments the BDS by addressing various seismic concerns. For a more comprehensive steel code, the AISC manual of steel construction has been used. There are other codes, which are detailed in the Design Criteria, and each of these codes, guides, authorities, or standards of applicable practice have been used where appropriate.

The Design Criteria set out design loads, material strengths and performance criteria. The loads include dead, live, wind, thermal, stream flow, etc. More importantly, the Design Criteria sets out the Seismic Design Criteria. The seismic loads of the SFOBB are unique and require a unique definition. This unique definition is in contrast to the various codes of applicable practice. These codes generalize situations, which are modified by the individual engineer to fit the specific application. Given the importance of the SFOBB, seismic performance and design criteria have been developed by several committee processes.

The Design Criteria includes the Seismic Design Section [Document 367, Section 8.0]. Two seismic events are identified: the Functional Evaluation Earthquake (FEE) and the Safety Evaluation Earthquake (SEE). Compared to FEE, the SEE event places higher demands on the SFOBB. Concurrent criteria address performance in terms of ductility, strains, displacements, and damage.

The COE Team has reviewed the Design Criteria for the 65 percent design submittal. It is noted that this Design Criteria appears incomplete, it is in draft form and is dated April 9, 1999, Revision 6.

Additionally, the COE Team has reviewed design calculations, plans, specifications and other documents for conformance with the Design Criteria, including the Seismic Design Section [Document 367, Section 8.0]. Conformance of the SAS components to the Design Criteria is noted in the following text.

1.B. Exploration Program

General. Geologic and geotechnical studies for the SFOBB are being conducted by Fugro-Earth Mechanics Inc., (F-EMI) a joint venture between Fugro West Inc., and Earth Mechanics Inc. Geologic and geotechnical studies include onshore site and marine site foundation characterizations to support the design of the Oakland Shore Approach, the

Skyway, the Main Span (Signature, Self Anchored Suspension Span), and the Yerba Buena Island (YBI) Transition.

Marine Explorations. The marine exploration and testing program, which is conducted in support of the designs for the Main Span and Skyway designs, has been described and summarized in Document 336 A and B. The descriptions of the subsurface conditions are based primarily on historical drilling information, previous Caltrans borings completed from 1994 through1996, 44 marine borings drilled in 1998 specifically for the northern alignment, and 2-D and 3-D geophysical surveys conducted in early 1998. Interpretations are based primarily on the site-specific conditions obtained in 1998 and the subsurface geometry imaged with the geophysical methods. The marine exploration program was conducted and completed using standard methods and procedures applicable to characterizing a bridge site. The exploration program is of sufficient detail to support the subsequent detailed bridge and foundation design.

Onshore Explorations. The onshore exploration and testing program was accomplished in two parts, one for the Oakland Shore Approach and the other for the Yerba Buena Island Transition.

The Oakland Shore Approach exploration and testing program results are summarized in Document 339. The descriptions of subsurface conditions are based on 25 soil borings, 9 Cone Penetrometer Test (CPT) soundings, 53 offshore Seascout CPT soundings in the tidal flat, 15 all terrain CPT soundings in the toe of the mole, two exploration trenches and two exploration pits. Explorations were conducted in 1998 and 1999. The exploration program has been completed using standard methods and procedures applicable to characterizing the site. The exploration program is of sufficient detail to support the subsequent detailed bridge and foundation design.

The exploration and testing program for the Yerba Buena Island Transition contained in the Yerba Buena Island Site Characterization Report has not been provided to the COE Team.

1.C. Site Characterization, N-6 Alignment

The soil stratigraphy and geology along the N-6 alignment for the Yerba Buena Island structures, the Main Span, the Skyway, and the Oakland Shore approach are thoroughly described in the catalog documents [Document 387]. Representative subsurface profiles and soil/rock properties have been developed [Documents 387 and 394). In addition, an evaluation of rock (Franciscan) slope and soil slope stability at Yerba Buena Island for the West Pier and Main Span Pylon has been conducted. The liquefaction potential, instability, and lateral spreading potential at the Oakland Shore Approach have been evaluated. Preliminary foundation recommendations and considerations were prepared. The work to date indicates the final level of geotechnical and geologic detail will be sufficient to adequately define subsurface conditions and support subsequent design of the bridge foundation.

However, a major concern is the identification and thorough characterization of faults in the vicinity of Yerba Buena Island. Document 338 identifies a possible fault located directly beneath the main pylon for the Main Span. Other documents note the potential for faults in the area of Yerba Buena Island. Considering the potential for earthquake energy to progress through other fault channels, it is important that a thorough geologic analysis of this area be performed including diagonal/inclined drill holes with oriented rock coring and accompanying geophysics.

1.D. Seismic Criteria

Seismic Motion Criteria. The Seismic Design Criteria adopted for the replacement design are based on two levels of ground motions, the FEE and the SEE. The ground motions for the FEE have been obtained deterministically for a Magnitude 6.5 event on the Hayward Fault at 10 km from the toll plaza [Document 335]. The seismic criterion for the FEE is to check SFOBB against this event to insure that the level of damage, if any, would be minimal with limited minor concrete cracking and no permanent set [Document 384].

The ground motions for the SEE scenario have been developed probabilistically using a return period of 1,500 years [Document 335]. The SEE spectra has also been developed for Yerba Buena Island and for the toll plaza. The Yerba Buena spectrum is considered adequate to be adopted as the target spectra for the entire bridge. Reference rock-motion time-histories for the SEE consist of six sets of recorded motions. These motions have been modified such that their response spectra closely match the target rock spectra for the SEE event. Three sets of the motions correspond to the San Andreas event and the other three to the Hayward event. The six sets of time histories include a variety of directivity pulses such as a one-sided pulse, two one-sided pulses, two-sided pulses, and multiple pulses. Currently these have been developed for the Main Span only. The reference rock-motion time-histories have been used to generate multiple-support rock-motions at each pier of the Main Span by applying specified coherency functions and wave passage parameters. The multiple-support rock motions have been used in free-field site response analyses to develop input motions at various soil layers for the soil-pile interaction analysis. These motions have been used as input to a soil-pile interaction model to obtain kinematic motions at the pile cap. The pile cap motions are then used to develop acceleration response spectra (ARS) for structural analysis.

According to Caltrans under the SEE scenario, damage to SFOBB is anticipated for pre-determined structural components that can be readily inspected and repaired. Repairs are expected to proceed without significant disruption to general traffic and no disruption to emergency vehicles [Document 384]. The proposed SEE ground motions appear conservative for periods up to 2 seconds, the period range for which the SEE response spectrum exceeds the 84th percentile Maximum Credible Earthquake (MCE) spectra for both the Hayward and San Andreas events (see Section 1.E.). However, for periods longer than 2 seconds, the SEE response spectrum falls below the 84th-percentile MCE spectrum for the San Andreas event. In fact, for the period range significant to this bridge (i.e., 2 to 5 seconds), the San Andreas 84th-percentile MCE spectrum shows 1 to 30 percent higher ground displacements than the proposed SEE. In other words, the bridge design is for the lower SEE spectral motions rather than the higher MCE motions. Furthermore, the generated free-field time histories, at Pier E2, are deficient at a period of 2 seconds; the period that approximately matches the pile foundation period at this location [Document 367, Volume 37]. In summary, if subjected to the MCE ground motions, the bridge response would be higher than that computed for the SEE. As discussed below in section 1.E, the bridge has not been evaluated for the MCE event

Seismic Motion, Permanent Ground Movements. The potential for permanent ground movements associated with accumulation of seismically induced strains in the soils surrounding and/or beneath the pile foundations, has not been specifically addressed in the documents reviewed. Such movements may add to the tectonic differential movements that occur between piers. In response to this issue, the Caltrans Seismic Advisory Board offered an estimate of less than 1 cm differential, permanent, bedrock-movement between two adjacent piers. Although this estimate is appropriate for supports founded on rock, it may not be suitable for the self-anchored span where the main tower and Pier W2 supports are founded on rock but the Pier E2 support is founded on soil. Document 384 refers to nonlinear site response analyses to account for the effects of permanent ground displacement. However, such nonlinear site response analyses and the associated bridge response could not be found or verified.

Seismic Ground Motions, Non-Linear Site Response Analysis. Document 384 notes that multiple-support seismic excitation was generated at each of the bridge pier locations, based on the latest advances in earthquake engineering. It also points out that nonlinear site response analyses were conducted in addition to the conventional equivalent linear site response analyses, and that free-field displacement time histories from the nonlinear analyses were used as seismic input to evaluate the effects of permanent ground displacement. The conventional equivalent site response analyses presumably correspond to the QUAD4M analyses presented in Document 381. Data for the nonlinear site response analyses cannot be found. It cannot be determined what analytical technique or computer program was used to implement the nonlinear site response analyses and how they are different from the QUAD4M analyses.

1.E. Maximum Credible Earthquake

Question 4 references the MCE by asking, How will the currently proposed replacement alternative perform in a maximum credible earthquake?" As discussed above, the replacement alternative is being designed based on the 1500-year SEE ground motions (developed probabilistically) and not for MCE ground motions.

The deterministic MCE ground motions were used in evaluating the existing bridge and its retrofit design. For the existing bridge two MCE events, one from the Hayward Fault and another from the San Andreas Fault were considered. The maximum magnitudes for these events were based on a study of seismic hazard for Northern California bridges. A moment magnitude of Mw = 7.3 was assigned to the Hayward Fault located 8 km from the east end of the East Span and an Mw = 8 to the San Andreas Fault located 19 km from the west end of the East Span. The 84th percentile MCE ground motions were developed for each event. According to Geomatrix s probabilistic hazard assessment

[Document 335] the 84th percentile MCE ground motion is between 1000- and 2000-year return period, equal hazard spectra. As discussed in Section 1.D., for the period range significant to the replacement bridge (2 to 5 seconds), the MCE ground motions are larger than the proposed SEE ground motions.

With this short introduction, the bridge has not been evaluated or designed for the larger MCE event. Previously, Caltrans and other authorities decided not to use the MCE, but to use the SEE instead.

Document review, by the COE Team, has not determined why Caltrans decided to exclude the MCE event from the Design Criteria [Document 9]. In fact, for this long period bridge, Caltrans has designed for an SEE event, which is less forceful than the MCE event. This misses the intent stated by Bruce Bolt for a more forceful event. [Document 9, Page 7].

1.F. Lifeline Criteria

A general description of lifeline objectives is described in the Scope of Work. This description does not include a detailed discussion of "Lifeline Criteria" as it specifically applies to this bridge. More importantly, design documents, and in particular the Design Criteria [Document 367, Volume 1], do not address "Lifeline Criteria." Other documents provide a description of the lifeline objectives in terms of traffic access to the bridge, which are not consistent. For instance, the access for emergency vehicles varies from "immediate" to some undefined time and the access for normal traffic varies from "immediate" to "within 72 hours" [Documents 81, 134, 303,and 372]. Furthermore, none of the documents provides a detailed description in terms of specific engineering parameters, which are required for assessment of accessibility of the bridge. The Design Criteria [Documents 367, Volume 1], includes a "Performance Criteria" for the SEE event, but makes no link to a "Lifeline Criteria." Such a link is made in Document 384 dated 29 September 2000 by stating that:

"*The new replacement East Span of the San Francisco - Oakland Bay Bridge (SFOBB) is designed as a lifeline structure under the Safety Evaluation Earthquake (SEE).*"

"The San Francisco Oakland Bay Bridge is designated as a Lifeline connection as defined in the Environmental Impact Statement (EIS). The Lifeline criteria in the EIS is satisfied by designing the Self Anchored Suspension Span (SAS) in accordance with the seismic Design Criteria (Lifeline) developed by Caltrans (San Francisco-Oakland Bay Bridge East Spans Seismic Safety Project, SEISMIC DESIGN CRITERIA, Draft - BHM ver 9, April 9, 1998) and the project specific design criteria developed by the Joint Venture.

However, it is noted that Document 384 is not a design document. It was only prepared in response to COE questions regarding the "Lifeline Criteria" and other issues, and still does not provide specific engineering parameters. Similarly, the EIS is not a design document and only refers to "lifeline" as the route classification for the SFOBB. The second document referenced, "SEISMIC DESIGN CRITERIA", was only provided to the

COE Team as, Version 12*,* dated June 27, 2000, and it also classifies the SFOBB as a Lifeline Route. It further provides a description of performance for an "Important Bridge" but not for the higher level of a "Lifeline Route .

Lifeline Route is noted only as a Facility Classification for Interstate 80, as of December 1997. The criteria referenced to the SEE event include parameters for design, performance, damage, repair, and for both daily and emergency operations. The SEE performance criteria were developed over the course of this project by various committees. It is for these reasons, that the COE Team s answers to the questions are based on the performance criteria for the SEE event, as stated in Document 367, Volume 1.

Appendices 1 and 8 of this report provide a chronology of events and discussions in relation to "Lifeline Criteria" and seismic performance issues.

In summary Caltrans has not presented a document, which clearly declares Lifeline Criteria . Further, the Scope of Work for the COE Team defines lifeline criteria in anticipation of a maximum credible earthquake (MCE). After reviewing Document 344, Full Service becomes an ambiguous term. These various terms have conflicts that make it difficult to communicate not only among engineers, but also with the taxpayers. Additional documents [i.e., Documents 354 and 352] present other conflicting expectations for lifeline criteria.

2. Foundations

Foundation designs reviewed by the COE Team generally consist of the Main Span (Piers W2, 1 and E2), and the Skyway (Piers E3 through E16) completed to the 65 percent design level. Documents have not been provided that describe the foundation system(s) proposed for the transition structure between W2 and the tunnel on YBI. Geotechnical characterization and foundation design reports, and 65 percent design drawings have been available for the Oakland Approach structure and fill, east of Pier E16. However, these documents have not been reviewed in detail and no evaluation has been performed due to a lack of time.

2.A. Foundations - Suspension Span

Axial Capacity. No cyclic degradation or rate-of-loading effects are considered for the bond strength of piles socketed or embedded in rock (Piers W2 and 1) [Document 367, Volumes 33 and 34]. The potential on Pier E2 for cyclic degradation of the skin friction along the pile due to multiple cycles of reversal of axial loads during earthquake loading, is computationally addressed based on offshore structure experience in the Gulf of Mexico. No experimental pile test data are available to describe the progressive degradation in axial shear transfer capacity in the San Francisco Bay sediments (Bay Mud) [Document 332]. Therefore, the behavior of these materials is assumed to be similar to that observed in the Gulf of Mexico clays. It is advisable to confirm the appropriateness of these data to the Bay Muds by means of a testing with an instrumented pile.

The cyclic degradation effects are modeled in a detailed analysis of the soil stratigraphy using the DRIVE computer program [Document 332]. The results of the analysis are used to develop simplified t-z side shear curves, which are then used in the global ADINA bridge model. It is our opinion that the method used to model cyclic degradation effects of pile skin friction is reasonable.

The rapid application of earthquake loads mobilizes undrained shear strength that are higher than those measured from in-situ and laboratory tests. These higher values have been used in computing static axial capacity. This increase, in skin friction capacity from rate-of-loading effects, was modeled using an equivalent linear viscous damping coefficient. The resulting equivalent linear damping coefficient is used to perform a DRIVE simulation of pile dynamic load test results. These results were obtained for Caltrans in 1992-1993 as part of a comprehensive study of the behavior of various pile types for a deep deposit of Bay Mud. The calculated pile behavior agrees well with the dynamic loading measurements.

Lateral Capacity. The lateral load behavior of piles cast in rock has been modeled using the Finite Element Model (FEM) and assuming elastoplastic constitutive relationships (elastic properties of the rock mass combined with the Mohr Coulomb failure criteria). Different values for the elastic and strength parameters have been adopted for intensely weathered, moderately weathered, and fresh rock mass conditions. The FEM results are then used to calibrate a simpler beam column type analysis, which in turn was used to calibrate p-y curves. The p-y curves describe lateral resistance for the three rock conditions mentioned above and do not depend on depth below top of rock [Document 367, Volumes 33 and 34]. No cyclic degradation effects are considered in development of the p-y curve for piles embedded in rock. Likewise, no rate-of-loading effects are identified in the calculations.

Unanticipated lateral loads may be applied to the Pier 1 foundations if the rock cut slope on the west side of the pylon fails during an earthquake. Document 342 recommends this slope be inclined at 30 degrees. The 65 percent design drawings [Document 277] show this slope as 45 degrees. Final design should resolve this discrepancy or slope stabilization may need to be provided for adequate seismic performance of the Pier 1 piles.

For Pier E2, lateral pile capacity is defined using p-y curves developed in general agreement with the guidelines provided in the American Petroleum Institute s (API s) guidelines, *Recommended Practice 2A*, dated July 1993. The API guidelines are modified to increase the stiffness of soil response at small displacements; this modification is justified based on published data [Documents 332 and 367, Volumes 36 and 37].

Document 385 indicates that a reduction factor of 0.5 (relative to the API recommendations) is applied to the p-y springs to account for degradation, gapping and pile group effects. The present state-of-the-art is such that there is no clear justification for the 0.5 factor and thus no comment can be made on whether it is an appropriate value. Use of this factor should be reviewed. Sensitivity studies should be performed to ascertain the effect of the 0.5 factor on the results of the analysis. No dashpots are included because the hysterectic nature of the p-y curves would cause the desired damping effects. Strain rate effects are not included, but no justification is given.

It is our opinion that the method used to develop p-y curves for rock and soil methods are reasonable.

2.B. Foundations - Skyway

Axial Capacity. Pile capacity is achieved primarily through skin friction with a small end-bearing component, using analyses similar to Pier E2 for the Main Span discussed above [Document 332].

Pile batters changed from 1:6 to 1:8 prior to the 65 percent design [Document 378, Volume 15]. It is not clear that the analysis has been revised to confirm the capacities and deflections for the changed batter.

Cyclic loading effects on the soil and soil strength degradation are evaluated using the same procedures describe above for Pier E2. Static axial pile capacities and deflections are presented but had not been modified by F-EMI at the 65 percent design stage. Sample methodologies for the modification appear reasonable and appropriately documented.

2.C. Foundations, Soil Structure Interaction (SSI) Model and Displacements

The following comments are based on our review of the 65 percent design calculations and supporting documents. We understand, from meeting with TY Lin, that revisions to the model and input parameters are currently being implemented and will continue to occur as the design progresses. Such revisions may have already addressed some of our comments. Because the design is in progress not all documents reviewed are consistent with one another (for example, the design computations are 65 percent complete while some of the supporting documents are based on 45 percent design completion [e.g., Document 332]).

The key to the model for the pile supported piers lies in the appropriateness of the springs representing its interaction with the surrounding soil. The spring constants are generally determined following API s guidelines. Modifications are introduced to account primarily for the effect of rate-of-straining and degradation with repeated loading.

In the absence of data specific to the Young and Old Bay Mud we find the approach to consider cyclic degradation reasonable. However it would be advisable to perform cyclic pile load tests on piles installed in Bay Mud to verify whether the degradation occurs at a similar rate as the Gulf of Mexico clays. Consideration should also be given to degradation of the stiffness as well as of the resistance in the springs in the clay, and as to whether there is a possibility of degradation of the springs in the sand. Failure to adequately represent the soil degradation and incorporate its effects into the design will negatively impact seismic performance.

The effect on rate-of-loading is considered for the skin friction in clays. We agree with the need to consider the increase in resistance caused by rapid strain rates relative to static values. However, it is questionable whether the introduction of the dashpots in the analysis may unduly increase the damping in the SSI part of the global model. Radiation damping is not specifically considered, but it may be substantially lower than the damping introduced to account for strain rate effects. The increase in skin friction with strain rate is real but it is not necessarily associated with an increase in hysterectic damping. Since the effect of rapid loading tends to offset cycle degradation it must be adequately modeled or seismic deformations may be under estimated.

Kinematic SSI studies presented in Document 335 consider both vertical and battered (1H: 6V) piles. Free field motions are applied to the piles through p-y springs. Four typical profiles are analyzed. A sensitivity analysis shows that by increasing and decreasing the stiffness of the p-y curves by a factor of two, there is a very small effect on the computed displacements at the pile cap. A comparison of vertical versus battered pile cases indicates small differences in pile cap displacements at small periods and practically no change at long periods. The 65 percent design incorporates battered piles for the Skyway structure, which are intended to reduce displacements and to make the pile cap displacements less sensitive to variations in soil conditions along the alignment. Properly designed battered piles can provide adequate seismic safety. In view of the results of the kinematic interaction studies in Document 335, the need for battered piles should be reviewed, given the installation challenges and higher cost of battered piles (when compared to vertical piles). A comparative study has not been provided for the feasibility of vertical versus battered piles.

The preliminary analyses of Pier E10 indicate a permanent post-earthquake tip settlement of .0008 m [Document 332]. This appears very small for the magnitude of the expected loads and has not been checked. We understand that final analyses are planned, but documents are not available. Likewise the permanent vertical settlement during an earthquake at Pier E2 (0.2 inches) appears small and has not been checked [Document 367, Volumes 36 and 37].

3. Computer Analysis

The structural analysis of the bridge was performed using ADINA, SAP2000, and ANSYS software. The ADINA time-history analysis accounts for nonlinear geometry, nonlinear material, and multiple support excitation. Nonlinear geometry is important because the geometric stiffness is a function of displacements (not small displacements as were assumed in conventional analysis) and P-delta effects are significant for slender structures. Nonlinear material was modeled to account for the change in stiffness of the structure with increasing deformation, and the actual capacity of the elements (piers, main tower shear links, and the soil surrounding the piles), which are expected to yield. Multiple support excitations were incorporated into the ADINA models to properly capture the variability of the seismic input motions at each of the bridge foundations with different soil characteristics.

SAP2000 was used to evaluate service load conditions and provide a basis for comparison with ADINA analysis results under service loads. The features of the SAP2000 model are similar to the ADINA model, except for some simplifications made for idealizing the foundations, YBI Transition, and Skyway structures.

ANSYS was used to perform linear static analysis of the multi-tiered models. The Level 1 (global) ANSYS model was intended to review the overall static equilibrium and basic behavior of the box girder deck system. The Levels 2, and 3 ANSYS models, which zoom into refined localized regions of interest, allow detailed fatigue studies of the typical structural details. The Level 2 model was also used to supplement the ADINA models by verifying and calibrating the stiffness of the beam elements used to model the deck.

Documents 383 and 384 give a more comprehensive description of the SAP2000 and ADINA global analytical models, as compared to the very brief description provided in Volume 5 of Document 367. Based on this description and a cursory review of the sample SAP2000 and ADINA input files [Document 367, Volumes 3 and 5], it is fair to say that the finite element modeling techniques used are very sophisticated and represent state-of-the-art analytical procedures. However, the description is not detailed enough for the reviewer to verify all the critical features of the analytical models which include but are not limited to the following:

Expansion Joint Modeling. In the case of the ADINA global model, it is not clear how the expansion joints were modeled using rigid beam elements and elastic beam elements or what properties were used. In the case of the SAP2000 global model, it cannot be determined what degrees of freedom were considered for the equivalent stiffness matrix for modeling the expansion joints.

Boundary Conditions. In the case of the ADINA global model, it cannot be determined what boundary conditions were applied at the end of the boundary frames representing the structures for the YBI transition and the Skyway.

In the case of the SAP2000 global model, it cannot be determined what degrees of freedom were considered for the equivalent stiffness springs used to model the structures for the YBI transition and the Skyway.

Damping. There is no discussion of damping used for the ADINA global model, except for a plot showing Rayleigh damping with $\alpha = 0.94248$, $\beta = 0.002387$ in Volume 5, Section 4.4.2.12 of Document 367. No basis was provided for selecting these values of α and β.

Documents 383 and 384 provide descriptions for the ANSYS Level 1, Level 2, Level 3 models and explain how the analysis results were used to supplement the ADINA models. Appendices L and M of Document 384 present some plots of finite element meshes and stress results. However, the information is not detailed enough for the reviewer to determine the adequacy of the ANSYS models.

4. Suspension Span

The provided documentation consists of in-progress plans and calculations at various levels of completion. Review of the in-progress efforts indicates that the designers are working towards satisfying the stipulated performance levels, but have yet to converge on the solution. Given that the designs are not complete and evidence of a detailed independent check has not been observed, it cannot be determined if the currently proposed replacement alternative can satisfy lifeline criteria [Documents 277, 367 and 384].

Document 384 has been submitted and reviewed near the end of the COE Team s Phase 2 efforts. It is a summary description of the suspension bridge and is the designer s statement or narrative of design work to date. It is not a work product such as design calculations. Work has continued and not all revisions noted in Document 384 have been incorporated in the 65 percent submittal.

4.A. Suspension Span - Tower

According to the designer, the tower for the suspension span is designed to remain essentially elastic during the SEE event and to be compact per ATC-32 ; the goal is to avoid buckling before yielding occurs [Document 384, Page 28]. This paraphrased goal is consistent with the Design Criteria [Document 367, Volume 1].

The design calculations for the tower [Document 367, Volumes 19, 20, and 21] have been reviewed by the COE Team. An extraordinary amount of work has been presented, but the work is in progress and is not yet been finished.

In general, member sizes are selected or assumed, and without references. The strength of the member, the connections, and secondary members are developed, and are based on the selected member and its factored yield strength. The demand on the members is either based on yield, or on computer output, which is without explanation.

In Document 367, Volume 19, (no page numbers) a demand capacity (D/C) ratio curve is presented for the tower. In the height range of 80 to 100 meters, the D/C ratio peaks at 1.2. In Document 384, Appendix C, a similar curve is presented with peak ratios at 1.05 and located near the base. This example points out that work is ongoing and is not yet complete. The design calculations are generally without explanation and are open to interpretation by the reviewer.

In Document 384, Appendix I, a buckling analysis of the tower is presented. It consists of a short description, then several plots showing the conditions that have been variously described. The analysis is lacking any conclusions and is open to interpretation by the reviewer. However, it does give insight into the completed design.

It appears that the designer has determined member sizes and properties for input to a computer model. This has <u>not</u> been verified. From the model analysis, summary outputs are developed after which design is altered and the tower re-analyzed. The designer is obviously exploring the behavior of the tower beyond its capacity to carry prescribed loads. He has explored its various modes of failure and in so doing has developed an engineering judgment that should conclude in a reliable design. The work is not yet complete.

A tower stability (buckling) analysis is referenced in Document 384. The referenced analysis is found in Document 367, Volume 22, Section 8.82; where good correlation is observed between two buckling models. As presented in Document 367, the reviewer is not able to verify input data such as member sizes, loads, boundary conditions, etc. Lacking verification, the reviewer can only note the brief summaries, which require his interpretation.

4.B. Suspension Span - Tower Shear Links

Shear links connect the individual shafts of the tower and are designed to absorb seismic energy that would otherwise cause plastic behavior (damage) in the tower shafts. The absorbed-energy forces the links into plastic rotation (permanent bending) after which they can be replaced. Since the tower shaft is designed to remain elastic during the SEE event, the designer does not expect any permanent deformation for the tower. Thus, the links can be removed with the tower returning to its original position as new links replace the old. According to this description [Document 384], the repairable damage criteria for the SEE event is met, as required in the Design Criteria.

On the design drawings, the connections for the shear links show ASTM-A325 bolts. The design calculations call for both A325 and A490 bolts. The drawings also show 70-yield steel, whereas the design calculations show 55-yield steel in the respective parts of the links. Page numbers are not available for reference.

For the shear links, design calculations are presented [Document 367, Volume 20], but begin with assumed sizes. The strength calculated for the links is based on the yield (F_y) stress of the steel. Given this beginning assumption, the connections, associated members, etc, are proportioned according to the strength of the plastic hinge, which is a correct method. No comparison is made to the computed or ultimate load. Lacking this comparison or necessary references, no statement can be made concerning conformance to the SEE criteria.

4.C. Suspension Span — Tower cross Bracing

The design calculations for the cross bracing of the tower are presented in Document 367, Volume 21. However, Section 8.3.1 on Design Loads is blank. These loads would be considered in the computed or ultimate load, and, like the shear links, are not available to compare against the computed strength of cross bracing member. D/C ratios are at acceptable levels to indicate elasticity. However, no references are cited for the seismic demands listed. Similar descriptions are used for the tower diaphragms, and tower base. Lacking these references, the reviewer is not able to determine if the design meets the SEE criteria.

4. D. Suspension Span - Tower Grillage and Saddle

At the top, the tower grillage supports both the saddle for the cable and the tower head. The grillage is very stiff and is a massive piece of steel. Its mass contains more tonnage than is found in the entirety of many bridges. It is designed to remain elastic and to force plastic hinging to occur immediately below in the tower shaft [Document 384]. In the ADINA computer analysis the grillage is rigidly connected to the 4 tower shafts. Three plots are available to summarize computer output for the grillage. In Appendix D of Document 384, these plots are simply identified as 90mm wall plate with 85 percent shell thickness; no further explanation. Deflection and stresses are without units. Beyond the above narrative, conformance is subject to the reviewer s interpretation. The design calculations state an assumed design load at 500MN for dead load, with no further reference. Live load has not been stated and constructibility is presently undetermined.

The saddle design uses design loads from the ADINA analysis [Document 367, Volume 21]. Strength design, per AASHTO-LRFD, has been used to size the members. Geometry, angles, and forces for the cable, on the saddle, are based on given coordinates. The coordinates do not have backup references. Conformance to SEE criteria has not been found.

4.E. Suspension Span - Tower, Constructibility

The proposed structural steel tower construction is primarily comprised of a base, trapezoidal shaped tower shafts, cross-bracings, struts, grillage, and saddle all formed from plates. The design and detailing of these elements is such that questions exist regarding the feasibility and reasonableness of the fabrication and constructibility demands. Specifically, the thickness and configuration of the plates required to accommodate the designs may contribute to significant cost and seismic reliability issues. As a result, it cannot be determined if the design will provide a seismically reliable solution over the design life of the structure [Documents 277 and 367].

5. Suspended Superstructure

Document 384 has been submitted near the end of the COE Team s Phase 2 efforts. It is a summary description of the suspension bridge and is the designer s statement or narrative of design work to date. It is not a work product such as design calculations. Work has continued and not all revisions have been incorporated into the 65 percent submittal.

Appendix 6

5.A. Suspended Superstructure - Box Girder with Deck

The box girder forms a spine, extending from the east and west piers, and without a connection at the tower. The girder frames a fixed-end joint with the west pier, otherwise called a monolithic joint. At the east pier, the girder bears on, and is keyed to the pier, otherwise called a pinned end joint. Longitudinal movement is compensated through the flexibility of the east pier. Transverse movement is reacted from each of the piers. The suspension cable and its suspenders provide vertical support to the box girder. In turn, the suspension cable anchors into the box girder and results in a compressive force on the spine. As the girder deflects under vertical loads, the displacement creates a bending and/or buckling moment plus secondary moments, which are commonly termed P-delta effects.

The superstructure has been analyzed using ADINA, SAP 2000, and ANSYS. In Document 384, a computer plot indicates that the SAS bridge is globally modeled using ADINA. Global modeling is inferred for SAP. It is inferred that ANSYS is used to analyze only components of the bridge by means of submodeling. In the design calculations [Document 367], the various approaches and uses of programs are not readily presented or explained.

The box girder has been modeled as a spine in the global analysis. By so modeling, the analyst captures the interaction between cables, piers, tower, and adjoining structures. ADINA provides the non-linear, time history, seismic analysis. SAP provides a linear, service load analysis, which is used as a check for the ADINA output. ANSYS provides a linear, static, multi-tiered analysis for the bridge components.

The box girder has been designed using criteria from referenced codes such as BDS, ATC, and AASHTO-LRFD [Document 367, Volume 14]. It has been checked for compactness. Strength of the section(s) has been computed based on yield (F_y), and critical buckling (F_{cr}). Connections are designed using $\varnothing R_n$ and $1.1F_y$. By designing per these codes, performance becomes predictable. However, no comparisons have been found presenting these strength designs against the actual or ultimate loads, such as the SEE event. Design work is incomplete and much of the presentation lacks the designer s interpretation or conclusion.

The orthotropic deck, floor beams, splices, shear frames, cross beams, etc. have been designed using criteria from referenced codes [Document 367, Volumes 15 and 16]. The various elements are designed for appropriate forces, such as shear, bending, and axial stress. The design notes use of Group VII seismic loads and the application of associated design methods. Thus, performance becomes predictable. In most cases, design loads are without reference or explanation.

The bike path is cantilevered from the south face of the eastbound bridge lane. Design calculations are presented for the bike path and associated structures [Document 367, Volume 18]. It is not evident that the components have been designed for Group VII (seismic) loading, per BDS.

5. B. Suspension Span — Hinges

The structural adequacy of the hinges could not be confirmed, as the provided documentation was incomplete. Specifically, little to no design calculations have been provided. As a result, it cannot be determined if the design satisfies the structural integrity and operational aspects of Lifeline criteria. [Documents 277 and 367]

5.C. Suspension Span - Deck Joint Assemblies

Deck joint assemblies with operational and SEE movement ratings ranging to 450mm and 1500mm respectively are stipulated [Documents 259 and 283]. Calculations supporting the determination of the specified joint movement requirements were not observed in the information provided. As a result, the functionality of the design and performance of the proposed deck joint assemblies cannot be confirmed, as the movements that need to be accommodated exceed currently available product capabilities.

Additionally, the anticipated replacement of the deck joint assemblies as a result of damage from the SEE event may result in traffic restrictions for 6 months or more. This assessment is collaborated by the post-earthquake scenario included in Document 344 and conversations with major deck joint manufacturers. Given this duration of operational intention, it appears that the performance criteria for the SEE event will not be satisfied. [Document 259, 263, 283, 353, 367, and 384]

6. Connections

This section includes bolted and welded connections for the structure. It also includes bearing type connections for the main cable in the various saddle locations.

6.A. Connections - Main Cable

The main cable connects to bulkheads at the eastern end of each box-girder. The cable is splayed to many swadged-and-bolted connections, which create an internally redundant system. Details are presented in Document 367, Volume 13. Design is based on strand count, not load and with out reference to SEE criteria.

6.B. Connections - Suspenders

The suspenders support the superstructure by connection to the main cable. Design is presented to include the ropes, cable bands, etc. [Document 367, Volume 13]. This work does not reference Design Criteria or SEE criteria.

7. Suspension Span, Piles and Foundations

The piles are a composite design comprised of curved plates up to 95mm thick to form 2.5m diameter welded steel tubes that are filled with reinforced concrete. The pile caps are referred to as steel moment resisting frames comprised of heavy welded plates encased in concrete . [Document 384] While the provided documentation indicates consideration of loss of section in the pile design, it cannot be ascertained if similar consideration was given to the design of the pile cap. The constructability and serviceability of the design are of concern due to the design concept and the nature of the detailing. Specifically, there is a potential for degradation of the structural integrity due to the maritime environment and the effects of loading. As a result, it cannot be determined if the proposed design will provide a seismically reliable solution over the design life of the structure. [Documents 277, 367, and 384]

The piles supporting the tower appear to be designed for plastic hinging at 5m above the steel casing cut-off elevation. Available information indicates the founding material consists of rock and weathered rock with inclined bedding planes. While analyses reportedly account for reduced stiffness in the weathered rock, it has not been observed whether a potential for inclined bedding plane failures has been considered. As a result, it cannot be determined if the foundation design will perform as intended. [Documents 277 and 367]

The foundation design calls for battered piles that significantly increase the difficulty of construction. Documentation indicating a benefit from the use of battered piles has not been observed. As a result, if cannot be determined if the additional cost associated with the use of battered piles is justified. [Documents 277 and 367]

8. Skyway Design Criteria

8.A. Document 378, Vol. 1 Comments on Design Criteria

Sections 2.3.5 and 2.8.7, covers combination of live load with seismic. The specified reduction coefficient is 0.17. This is supposed to reflect the estimated peak hour traffic predictions for the year 2025. This seems very low and backup data, such as the traffic estimates, have not been located in the documents provided.

Section 6.3.2 specifies that the weld between the deck plate and ribs shall be 80 percent penetration weld. If welds are transverse, partial penetration welds are not acceptable.

Sections 6.3.5 and 8.5 state that for lifeline, bearings and expansion joints must accommodate SEE displacement during and after the event. No guidance is provided in 6.3.5 for SEE. In Section 8.5, criteria are provided based on estimated permanent displacement (i.e. AFTER the event). The joints must also function for the possible differential displacement that might occur during the SEE.

Section 8.1.2 does not indicate that the design criteria, as described for SEE event, satisfy the Lifeline condition (i.e. the precise definition for lifeline is not provided). See Section 1.F, Lifeline Criteria for additional comments.

The data that describes the global model is not always consistent with the contract drawings [Document 257]. For example, the data for pier caps included in Section 4.2.3.2 does not reflect what is shown on the contract drawings.

In summary, the reduction coefficient of 0.17 applied to live load for combination with seismic seems too low. Given the volume and long hours of peak traffic on a daily basis a higher value is expected. Partial penetration welds, transverse across plates, are not allowed per typical welding practice. The design criteria do not cover the requirements for expansion joints under the SEE event. A clear meaningful definition, with specific performance requirements for which the design of this structure can be compared to, is not provided.

9. Skyway Analysis

9.A. Skyway Foundations

Per Document 283, plans for 100% in Progress Submittal , Skyway Structures , a substantial change has been made in the geometry of the foundations when compared to Document 259 the 65 percent plan set. The pile caps for Piers 7 through 16 have been lowered below the water line by as much as 12.5m, (41ft). The reason this was done is not given in the 65 percent calculation documents provided. It is assumed that this change occurred after completion of these calculations based on additional analysis not provided to the COE Team. It is also assumed that the apparent reason is to add flexibility to the substructure. The profile grade descends as it nears the Oakland Approach Structure, [Document 258] and the piers become very short.

This affects all frame models, SAP2000 for service loads (Volume 1), SAP2000 response spectra analysis (Volume 2), and ADINA non-linear time history analysis (Volume 4). It will also affect the foundation push-over analysis (Volume 3). The final calculations must reflect this change.

The lower pile caps have an additional change. A pier casing has now been added to the now submerged pile cap, per Document 390, and as shown on the plans, Document 283. It is assumed that these casings act as a dry well around the submerged height of the piers to provide for inspection and maintenance of the piers. The mass (hydrostatic and dynamic) and hydrodynamic affect on the pile, pile cap and pier have not been evaluated in the documents provided to the COE Team.

9.B. Document 378, Volume 1 Comments

This volume includes a section on Design Criteria . See Skyway Design Criteria above for comments specific to the Skyway structure and Part 1.A. for general comments. Section 4.4.2.3, Foundation Stiffness Matrixes, lists the foundation stiffness parameters used in design. The origin of these design parameters is not indicated.

Appendix 6

9.C. Document 378, Volume 2 Comments on Elastic Response Spectra Analysis Using SAP2000

ARS SAP2000 used p-y, t-z, and q-u secant stiffness springs applied along the length of the shafts, which are each independently modeled. Linearized to spectral displacements with 5 percent damping. Under part 7 of Modeling Assumption superstructure effective moment of inertia, $I_e = 0.7\ I_g$ (gross moment of inertia, weak axis only). This reduction was used in Section 4.6.1.2. Frame 1 Models . Validation of using the I_e in the analysis, rather than I_g, such as a comparison of allowable tension in the deck for seismic loading to actual stresses and evaluation of the meaning of the actual D/C ratios, which are all less than 1.0, has not been provided. p-y iterations should include direct comparison between assumed stiffness and actual stiffness (v/u), where v is output force and u is output displacement. Although a non-linear time history analysis using ADINA was performed (see Volume 4 comments below), its use in evaluating the design of this structure is not evident.

For Frame 1 Models the cross-sectional area of the superstructure used in the SAP2000 analysis seem to be up to 8 percent less than presented in Volume 1, Section 4.2.1.1 Box Girder , which post dates the SAP2000 analysis. Data described for model input does not consistently represent features shown on contract drawings [Document 257]. It is not clear that analyses represent the final selected design. (For example, Document 253 shows Piers E3 through E14 as nearly identical, and the input description shows different cross sections. Pile batters are shown as 1:8 [Document 253] and are described as 1:6 in the model description). It is not clear how cracking and yielding effects are incorporated in the analyses. The design earthquake is not defined. The SEE is assumed by the reviewer. Specific results of analyses, although necessary in a seismic design, are not summarized, and it is not clear how results are used in the design.

9.D. Document 378, Volume 3 General Comments

Pile force results are provided, but the displacement at which the results are obtained is not identified. Resulting pile forces could be used for pile cap service design also, if the forces provided correspond to the target displacements. It is assumed that the SEE level earthquake RSA were used, but this is not stated.

For push-over Analysis Procedures, per Note 1, soil spring stiffnesses are reduced to 50 percent of their ultimate strength for revision of the RSA displacements for the full Skyway model. A rationale for doing this is not provided. Pile force results are provided, but the displacement at which the results are obtained is not identified. Resulting pile forces could be used for pile cap service design also, if the forces provided correspond to the target displacements. It is assumed that the SEE level earthquake RSA were used, but this is not stated or verified.

For pile cap linear elastic RSA demands, page 8 indicates that the output is for a tension model. However, a table of results for compression has no explanation and is confusing. On page 13 (as shown at the bottom of the sheet), time history results are tabulated, but their use in design is not explained or demonstrated.

9.E. Document 378, Volume 4 Comments

The COE Team cannot locate in the design volumes where this non-linear time history analysis is used. This volume includes numerous displacement and force demand graphs, but does not make comparisons to either the element capacities or the ARS demands generated by SAP2000.

In summary, numerous inconsistencies in analysis have been noted above. Without verification of these concerns, conformance to the lifeline criteria and goals cannot be demonstrated.

10. Skyway Superstructure

As shown by the contract drawings [Document 257], the Skyway superstructure consists of two separate structures (eastbound and westbound). They are precast concrete segmental haunched box girders. Typical span lengths are 160 meters with shorter spans down to 95 meters as the structure nears the Oakland Approach structure. A steel box cantilever span is used at the west end of the Skyway structure to tie into the steel box superstructure cantilever span from the suspension span.

10.A. Document 378, Volume 5 Comments

The calculations are incomplete for Section 5.3.1.2, Ultimate Loads , for Longitudinal Design Westbound Structure . Some sections are difficult to follow and evaluate as they are unlabeled and the purpose of their inclusion is not stated.

10.B. Document 378, Volume 6 Comments

Work seems to be comprehensive. However, clear descriptions of many calculation sections are not provided and therefore, completeness cannot be determined. There should be a written description for and with each new output/ spreadsheet format.

In Section 5.2.1.2.2 Demand to Capacity Ratios for Longitudinal Moments, under Group VII load combination (seismic), superstructure behaves elastically (D/C ratios are under 1.0). However, it is not clear whether live load, per Design Criteria, Sections 2.3.5 and 2.8.7, has been combined with seismic in this analysis. For the D/C ratio check, demand (D) is from Volume 2, Section 4.6.1.2. Frame 1 Models . This model uses an effective moment of inertia $I_e = 0.7\ I_g$ (gross moment of inertia, weak axis only). It is not clear if the moment capacity calculations, beginning on calculation page 16 of 106, have been based on this reduced section property, to be consistent with the demand side.

10.C. Document 378, Volumes 7 and 8 Comments

Work for this volume seems to be comprehensive. However, clear descriptions of many calculation sections are not provided and therefore completeness cannot be determined. Should have a written description for and with each new output/ spreadsheet format.

In summary, the combination of a realistic live load, given the long hours of peak commute traffic across this structure, with seismic loading is critical to ensure that conformance to the lifeline criteria and goals has been met. This has not been demonstrated. Also, the use of an appropriate effective moment of inertia for the superstructure is critical to ensure that a realistic and conservative performance of the structure is captured in the analysis. If the actual effective moment of inertia exceeds the 70 percent of the gross moment of inertia assumed for design, the resulting forces and displacements would likely exceed those calculated. Without verification of this assumption, conformance to the lifeline criteria and goals cannot be demonstrated.

11. Skyway Superstructure Deck Expansion Joint Seals

The Skyway superstructure deck expansion joint seals are the modular joint seal assembly type. A special, or modified, design to accommodate the SEE event movements in the longitudinal direction is required. These seals are placed in, and at the same level as, the deck, to bridge the superstructure hinge gap for vehicles and to provide a watertight seal. See Skyway Superstructure Hinge below for comments on the structural design for transference of superstructure forces across the hinge.

11.A. Document 378, Volume 13 Comments

No details on the plans set, even the 100 percent complete set [Document 283], or calculations, for the joint seals, have been provided to the COE Team for review. The following comments are derived from the requirements of the design criteria and of the special provisions (specifications) [Document 392], and from the reviewer s experience with this type of deck joint.

For the FEE event, the mechanical opening capacity of the joint seal assemblies must accommodate the FEE displacement demand. Therefore, it is expected that no damage, including loss of the rubber compression seals, would occur under such an event. However, the possibility of pounding of the joint seal assemblies due to closing of the joint gap during the FEE has not been addressed.

For the SEE event, the support beams span across the joint between ends of the superstructure, which support the transverse joint seal beams. They are extended to provide excess opening capacity, without unseating, during the SEE event. It seems that the intent, or goal, of the design is to develop a joint system that can accommodate the full displacements of the SEE event without failure of any kind. To be consistent with other design methodologies used for the other components of this structure, even if this goal is theoretically attained, the joint failure mechanism should still be determined. Such an evaluation addressing what would happen if the mechanical opening operation limits of the joint are exceeded is not provided. Failure could occur in several ways. The joint seal beams or their welds to the support beams could fail, an edge beam could pull away from the concrete, or the edge beam could be designed as a fuse to allow pullout away from the concrete. Again, pounding of the joint seal assemblies due to closing of the joint gap has not been addressed.

Appendix 6

The joint seal may be the weak link in the concept of providing a lifeline route across this bridge. It is recognized that Caltrans is working with the modular joint seal industry to develop a joint seal to meet the demands of this project. However, several concerns still arise:

1. The original development of this type of joint is for slow thermal movements. The behavior of the joint under the high velocity of seismic motion has not been proven to be satisfactory. Rapid cycles of movement could cause major damage to the transverse seal beams.
2. As discussed elsewhere in this report, the maximum credible earthquake (MCE) event could result in displacements up to 30 percent greater than that anticipated under the design SEE event. This is well beyond the 10 percent factor of safety applied to the extended length of the support beams.
3. The impact force under joint closing cycles could cause major damage to the transverse seal beams.
4. Per Document 392, Reference Special Provision for Modular Joint Seal Assemblies , the joint is to be designed to accommodate both SEE $_{opening}$ (maximum movement of adjoining frames during the SEE event, which opens the expansion joint gap) and SEE $_{closing}$ (maximum movement of adjoining frames during the SEE event, which closes the expansion joint gap). Therefore, the total joint movement capacity must be the sum of these two movements. That is, the initial joint setting would be near the midpoint of its full travel range. It is assumed that the joint would be field adjusted from this point to accommodate thermal movement and the remaining portion of concrete shortening. The design movement as shown in the calculations range from 1,252mm (combined opening and closing) at joint E06E/E07E to 314mm at joint E14E/E15E. The maximum allowable opening at initial setting and after all shortening and thermal contraction is 80mm. This equates to about 12 seals for the largest joint based on a rough assumption of thermal and shortening movements.

In summary, the above concerns could result in severe consequences.

Concerns 1. or 2: The consequences of either occurring could be that the joints are rendered impassible. Large gaps between transverse seal beams and / or severely bent bars protruding above the deck could prevent the quick evacuation of vehicles from the bridge as well as delaying the accessibility of open lanes to emergency vehicles. Also, even if such damage does not occur, the violent movement of the joint seal assemblies and transient occurrence of large gaps, could result in numerous accidents. This would further disrupt the use of this bridge as a lifeline route.

Concern 3: The consequence of this occurring is self evident. The unseating of the support beams could result in the total loss of the joint seal assembly. The unseated assembly would become hung up in the hinge beams below. This could cause severe damage to those items also. However, the greatest concern is the resulting very wide gap that would result in the deck. The 100 percent plans show dimension a for the gap between faces of concrete segments, but the actual width is not provided. The table is left blank. This would cause a severe danger to vehicles, as well as create a major obstruction to egress of the bridge.

Concern 4: Such a joint design, which is much larger than the typical upper range of conventional modular joint seal assemblies and must be capable of handling high seismic accelerations and translations and rotations, is a very large step in the state of the art for this item. A physical test of the new joint design is described in a test report from the University of California, Berkeley [Document 393]. The report includes two paragraphs that describe the test parameters such as input motions, velocities, and displacements. The results consist of graphs only showing the actual tests in terms of displacement, force, or velocity vs. time, and force vs. displacement. A description of the joint design tested, summary of the results, and conclusions regarding its performance, including fatigue testing for service loadings, is not included. Therefore, the feasibility of using such a joint remains only theoretical, even though it is relied upon heavily to accomplish the lifeline objective set for this structure.

Therefore, conformance to lifeline criteria and goals has not been demonstrated.

12. Skyway Superstructure Hinges

As shown by the contract drawings [Document 257], the Skyway superstructure hinge consists of four horizontal beams aligned parallel to the longitudinal axis of the superstructure to transfer vertical forces, and a set (one below the deck and one above the soffit) of wide beams turned on their sides to transfer horizontal forces. One end of each beam is fixed into the end face of a superstructure segment, the opposite end is free to slide longitudinally over a set of double bearings. This double bearing arrangement also allows for the transference of moment about the plane of the bearing sets. See Skyway Superstructure Deck Expansion Joint Seals above for comments on the deck joints to carry live loads across the hinge.

12.A. Document 378, Volume 13 Comments

Design covering the hinges contained in this volume seems to be comprehensive. However, clear descriptions of the intent of the hinge beams and particularly a clearly written description for each new output/ spreadsheet format, are not provided. Therefore, completeness cannot be determined.

In summary, the actual performance of the hinge beams, under seismic loading has not been demonstrated. Therefore, conformance to lifeline criteria and goals has not been demonstrated.

13. Skyway Pier Caps

As shown by the contract drawings [Document 257], the Skyway pier caps are heavily reinforced concrete superstructure segments, which provide a starting point for placement of precast segmental superstructure sections. The cap is cast-in-place, prestressed transversely and vertically, as well as providing anchorages for longitudinal prestress tendons. It will utilize similar lightweight precast panels to support the overhangs to the segmental superstructure units.

A closure pour is used at both faces to join the first precast segments. The main longitudinal (vertical) bars and welded hoop confinement reinforcement, from each of the four corner columns of the pier are projected into, and embedded into, the cap.

13.A. Document 378, Volume 13 Comments

Section 5.5.1, Pier Cap states that all pier caps are fixed in their final configuration to the top of the piers. However, temporary movement capacity is provided in the longitudinal direction for the end pier of each frame. This will accommodate superstructure shortening due to shrinkage, and elastic and creep from post tensioning force. This movement capacity is to remain for the first twelve months. See Skyway Piers below for comments on this issue.

Strut and tie models were used for the analysis. A separate model for a number of force path mechanisms was developed. The combination of reinforcing requirements from each of these models is not clearly documented. This may have resulted in an overlap of requirements, which would in turn result in excess reinforcement. See concern under Document 283, 100 percent plans (note that Document 259, 65 percent plans, does not show the pier cap reinforcement).

13.B. Document 283, 100 Percent Plans Comments

The reinforcement details seem highly congested. The vertical, longitudinal, and transverse mild-reinforcement form a three dimensional grid that is typically 200mm (8inches) on a side. Projected through this mesh from the pier is a double circular pattern of large diameter bars with a center to center spacing of approximately 165mm (6.5), and a hoop pattern at 100mm (4). In addition, both vertical and transverse post tensioning tendons are included, as well as numerous main longitudinal superstructure tendons, which cross through the cap.

In summary, pier caps seem extremely congested. A discussion of the constructibility of this cap has not been provided. Statements on the plan sheets allowing the cutting of bars to facilitate tendon placement seem open ended and do not protect against excessive and possibly detrimental removal by the contractor.

14. Skyway Piers

As shown by the contract drawings [Document 257], the Skyway piers consist of four closely spaced, heavily reinforced and confined columns in a rectangular pattern, interconnected with reinforced concrete walls. The center of the pier is void. Stairs are provided within the void for access to the superstructure and for inspection of the pier.

Appendix 6

14.A. Document 378, Volume 14 Comments

Section 6.1, Piers: Description , states that the top of the piers at each end of a frame are detailed to allow temporary longitudinal sliding of superstructure (to accommodate creep, shrinkage, and PT shortening). These connections will not be made fixed until after 12 months. It is further stated that all service and seismic load analysis are performed with these joints fixed. It is not clear whether both the pier analysis, and analysis of the superstructure were subject to this check. Also, a discussion of the analysis and design requirements for the interim period of 12 months, when the pier tops are pinned and free to translate, is not provided. It appears that plastic hinges are expected to form at top and bottom of the piers under certain events. This document does not describe in which event this will take place; however, the design criteria of Document 378, Volume 1 describes performance objectives for the FEE and SEE. Limited damage to piers, including yielding of reinforcement and spalling of concrete cover are accepted under the SEE. Also, the columns must have a clearly defined plastic mechanism for response to lateral loads. The design must at a minimum account for the following in order that plastic hinges will be allowed to form:

- Adequate flexural strength at plastic hinge, i.e., adequate section and reinforcement.
- Adequate ductility in plastic hinges to accommodate rotations and displacements, i.e., proper detailing of plastic hinge area including regions in pile caps and pier tables.
- Adequate design for shear in plastic hinge regions, and
- Adequate design of regions outside of plastic hinges to ensure hinges will be confined to regions expected.

Design Approach. It appears that the design approach is a combination of displacement-based and force-based designs. Displacement demands are determined for the design input load from non-linear time history. Z factors are applied to RSA results. Capacities are based on Moment-Rotation relationships at limiting strains for the actual cross-section. D/C ratios are then computed and assessed. It appears D/C ratios less than 1.0 are desired in most cases although it is unclear which design event is evaluated (SEE, FEE, MCE). Limiting concrete strains of 0.004 are reported in various aspects of the design. This would imply the FEE by section 8.13.1 of the design criteria. Most D/C ratios are below the allowable limit in this analysis, except that displacements in a diagonal direction exceed capacities, based on assumed fixity conditions, at several locations. Therefore, the design criteria are not met.

Flexural Design of Plastic Hinge Regions. Flexural strength can be determined by applying a Z factor to an elastic response and sizing the member accordingly (force-based design) or from required displacements and corresponding member stiffness (displacement-based design). The design process for the plastic hinge regions is not discussed. Most of the flexural design of the piers is devoted to the service limit state. The table provided presents an assumed plastic hinge capacity, demands (from RSA and THA), and D/C ratios (with a maximum allowable ratio of 1.25). There is no explanation of the design process or how these numbers are achieved. The assumed plastic hinge capacity may come from pushover analyses with limiting capacity based on strain limits. The design procedure for the piers and the origin of the assumed plastic hinge capacity allowable D/C ratios could not be located in the design documents provided.

Ductility Design of Plastic Hinge Regions. The pier must provide sufficient inelastic rotation and ductility to meet performance requirements in the plastic hinges, i.e., remain ductile. Ductility is attained through adequate confining reinforcement. The reinforcement required is a function of the magnitude of hinge rotations, curvatures, and strains (displacement based design) and material strengths, axial load, cross-sectional area, and longitudinal reinforcement (force based design). The confinement reinforcement must be provided along the entire length of plastic hinge and sufficiently into pile cap and pier table, and must prevent buckling of the longitudinal reinforcement as well.

The ultimate or maximum curvature is based on the ultimate or maximum allowable strains from pushover analyses. The plastic hinge length can be determined from empirical equations. The plastic rotation is a function of plastic hinge length and hinge curvatures. Ductility capacity is determined from ultimate curvatures or displacements versus yield curvatures or displacements and expressed in terms of a ductility factor. Ductility factors are about 3.0 or lower for the analyses conducted. However, the event for which these factors are achieved is not described. Ductility should be analyzed for the FEE and SEE.

No calculations are included for prevention of buckling of longitudinal reinforcement. No calculations are shown for plastic hinge lengths.

Shear Design of Plastic Hinge Regions. The description of shear design in this volume states that piers have been detailed to carry shear after a plastic hinge has formed. No calculations or discussions are provided that specifically describe shear design in these regions. A reduced section should be used for concrete shear capacity to account for degradation of the cross-section due to cracking and reduction of shear transfer during the formation of a plastic hinge. An effective section is shown, but not described. A more detailed description of shear design is required.

Design of Regions Outside the Plastic Hinge Regions. This topic is not specifically addressed in this volume. Since maximum moments are at top and bottom of pier, remainder of pier should be adequate as long as reinforcement is consistent along the length of the pier and proper hinge detailing is provided. Pile cap and pier table regions are not covered here. Reinforcement details should be checked to ensure adequate reinforcement is present in regions outside plastic hinges. Proper detailing of plastic hinge zones into pier tables and pile caps must be shown and documented.

Pier Casing. The pier casing adds additional mass loads to the pier and redirects hydrodynamic loads in the form of reactions from the casing to different locations on the piers. This change in loading condition must be addressed in the pier design. See comments under Analysis above. Also, the calculations address water pressure, but do not analyze for the effects of soil. It should be assumed that the dredged area around the foundation will fill back in over time after construction.

In summary, hinge regions appear to be adequately detailed and flexural capacity adequate for this stage of design. However, there are a number of uncertainties expressed in this review:

1. It is unclear to what extent plastic hinges are expected to form. It appears that ductility is achieved through limiting strains although this is not well documented. The ability of the massive transverse section to form a true plastic hinge is questionable. While material strains can be predicted and allowable capacity limited accordingly, it is unclear how expected behavior used in global and local models reflects the actual behavior.
2. The applicability of plastic hinge detailing equations to this type of section is not demonstrated.
3. The design criteria require testing of hollow cross-section members. Testing, if it has been done, or is intended to be done, is not represented as such in the design documents provided.
4. Concluding remarks that summarizes pier behavior and governing design requirements are not provided in this document.
5. Interaction curves for the SEE event, which allows for concrete strains up to 2/3 of the ultimate strain, are not provided. Therefore, the use of the curves generated using an allowable strain of 0.004, as allowed for the FEE event, would be conservative for the SEE event.
6. Per section 6.1.5, Flexural Design , this section covers comparison of service and seismic load demands from the SAP2000 static and seismic response spectra analysis to the capacities determined in section 6.1.3. The results show that capacity is exceeded by up to 33 percent for service loads. A complete comparison for seismic loads is not provided. The partial results reviewed are for one direction only and do not include multi-directional combinations as required by the design criteria. Time history demands are shown, but only the response spectra demands are used in the capacity checks.

Therefore, conformance to lifeline criteria and goals has not been demonstrated.

Part 15 Skyway Pile Caps

As shown by the contract drawings [Document 257], each Skyway pile cap (footing unit) consists of a stiffened steel shell with numerous stiffeners and pipe sleeves for the pile connections, which form many compartments. These compartments are filled with lightweight concrete in perimeter voids and normal weight concrete in the interior voids. The pile cap encases the upper portion of the piles and provides a foundation anchor for the piers. Foundations for Piers E3 through E14 include six 2.5-m diameter battered CISS piles and the pile caps are octagonal in shape. Four piles found Piers E15 and E16 and the pile caps are rectangular.

Major components of the pile cap include:

- Top plate, bottom plate and edge plate and associated stiffeners.
- Pile sleeve and associated shear plates.
- Pier socket casing and associated reinforcement.
- Concrete in-fill.
- Precast concrete perimeter walls around the sides of the shell.
- Cast-in-place reinforced concrete cap over the top plate of the shell.

15.A. Document 389 Comments: (Supplemental Calculations to Document 378)

The basis of these supplemental calculations, which cover the steel alternative pile cap, is finite element analysis. Intricate, detailed models have been developed using ADINA. The results are depicted with color-coded stress distribution plots through various cross-sections. Both SEE displacement demands, from other analysis, and application of the plastic over-strength forces from the piers, from other analysis, are used as the input loads. In addition, a push-over displacement of 1.0 meter is applied to determine the probable failure mechanism. All analyses have been performed with two separate structural conditions: steel shell and plates only; and steel shell and plates with concrete in-fill.

Although stresses, as depicted in the stress plots, seem reasonable, with only a few locations being just above yield, a written summary of results is not provided. Also, input is not provided thus plate thickness, etc. cannot be verified.

Design and analysis is not entirely consistent with the criteria. The design calculations do not consider the overall effect of pier and pile forces acting simultaneously. Only the components involving pile and pier connections are designed for the individual pile and pier ultimate loads.

Performance under the FEE event is never considered. The performance under the SEE event with actual pier forces is not analyzed. In the finite element analysis for the SEE, the pier over-strength plastic moment is applied with the SEE displacement. The pier will not maintain its over-strength plastic moment simultaneously with the SEE displacement, and analysis with a realistic (lesser) pier load will show a different stress distribution that may yield different results.

The finite element results [Document 389] for the SEE condition and the ultimate condition indicate that critical members in the cap remain essentially elastic. This does not necessarily show conformance to the SEE criteria since strength reduction factors must be considered.

Concerns Regarding the Finite Element (FE) Analysis

The pier will not maintain its over-strength plastic moment simultaneously with the SEE displacement. With a lower moment applied, the stress distribution in the cap would change with certain elements having a higher stress. This may or may not affect the overall result.

The pile cap displacements assumed for the analyses were 0.275 m and 0.2 m for longitudinal and transverse SEE demand, respectively. The SAP 2000 linear elastic response spectrum analysis indicated longitudinal and transverse demands of between 0.5m and 0.6m for the pile caps at Piers 13 through 16. These results are indicated for the Frame 3, 4, and 5 - revision 6 analysis (dated 6/30/99), and the full Skyway model analysis (dated 7/16/99) [Document 378, Volume 2].

As shown by the FE model results, the concrete in-fill is necessary when considering the ultimate design condition. The concrete will include shear and tensile forces that should warrant some reinforcement. No reinforcement is shown in the drawings. [Document 257]

Although a brief written summary of finite element results [Document 389] indicates that critical members in the cap remain essentially elastic, a complete set of design calculations was not provided. Given the stated performance criteria, a factor of safety (strength reduction factor) regarding steel yielding must be provided. The FE results do not necessarily show conformance to this criteria.

<u>Concerns regarding pile-to-pile cap connection design</u>

- The pile-to-pile cap connection analysis consists of separate hand calculations. See discussion of three alternatives considered, located at the front of that section in Document 389.
- The design forces (pile moment and axial force) are not consistent through the calculations. The shear plate design and shear stud design include different forces.
- The contract drawings [Document 257] call for 480 studs per pile, while the design calculations show a minimum of 1478 studs.
- There are no calculations regarding the concrete reinforcement between the pile shell and the pile sleeve. Significant reinforcing is shown on the contract drawings.

<u>Concerns regarding the pier-to-pile cap connection design</u>

- The slab thickness is 1.13 meters in the calculations and is shown as 1 m in the contract drawings.
- Stiffeners are not sized for the bottom shear component, and no check is made for resisting the bottom shear component given transverse bending of the pier.

<u>Concerns regarding the perimeter wall design</u>

- No calculations are provided to show the affect of filling the interior of the footing unit with wet concrete. The concrete must be placed in specified lifts to ensure that the lateral load from the wet concrete is not too high.
- The outside concrete should extend to the bottom of the plate per calculations.
- The dimensions used in the calculations and shown on the contract drawings are not consistent (i.e. bottom plate).

- The contract drawings show the concrete skin to be a precast concrete wall. The calculations assume that the steel edge plate and concrete skin are a composite section. It is assumed that the concrete skin will be precast onto the unit after the steel unit is assembled.

Fabrication Review

This section in the calculations is empty. Not coincidentally, this is an area of great concern. The constructibility of this item has not been addressed and the following issues are presented:

- The possibility of a shaft placement not being within tolerance to allow the prefabricated steel shell to slip over the 4 and 6 shaft group pattern is very likely. This would require considerable field modifications, which could bring the original analysis into doubt.
- Design of the welding for the individual plates used to fabricate the steel shell, though detailed in the plans, is not provided in the calculations.
- Corrosion protection of the steel plate components has not been discussed, particularly in regards to the exposed bottom plate.
- Refer to above comments under Analysis , General above regarding the changes in pier cap elevations. Per the 100 percent design plans, the pile caps will be situated well into the bay soil. This will probably necessitate the use of a cofferdam for installing the piles, cap and pier. This seems to eliminate any perceived construction advantages, as might be the case in open water, for the precast steel shell pile cap.

15.B. Document 257 Comments (65 Percent Project Plans)

The selected design is shown on the 65 percent contract drawings [Document 257]. In general, it appears that there are many unnecessary steel components that involve heavy welding. The advantage of additional stiffness and strength imposed by stiffeners, collars, etc. is offset by the difficulty in fabrication and construction of the footing unit.

The reliability of the pile cap and its connection to the piles is related directly to the seismic performance of the bridge. Although construction may be possible, it is not likely that the cap would be constructed to the level of workmanship required. There are concerns regarding welding requirements, congested work area for the connection of the pile and pile cap, and impractical tolerances required.

Welding. The stiffened box structure that makes up the footing unit of the pile cap is composed of intersecting steel plates that are interconnected with full penetration welds and large fillet welds. Most of the plates range in thickness from 38 mm to 68 mm. There are many locations where welds must intersect from three orthogonal directions. Due to high tensile residual stresses that develop after welding, and adverse metallurgical effects of welding, these locations will be prone to brittle fracture**.**

In the pile-to-pile cap connection, all pile head shear plates (eight plates per pile) must be custom fitted and field welded between the piles and pile sleeves. For each plate, the connected steel must be pre-heated. AWS D1.5, Section 4.2 requires that preheat and inter-pass temperature be 175 degrees F, when welding involves 2-1/2 thick steel plate (thickness of the pile steel shell). To maintain this temperature will be difficult if not impossible and will certainly impose worker safety concerns.

Congested Work Area. The worker space available to construct the pile-to-pile cap connection is very limited, especially if the pile sleeves and piles are not exactly parallel and centered. For each pile, eight shear plates must be installed by field welding. Considering the location of stiffeners and prestressing bars, a work area no greater than approximately 750 mm by 900 mm wide in a height of approximately 5000 mm will be available. To maintain the required welding preheat and then to perform the welding in these enclosed areas is not practical. Construction of the concrete reinforcing steel that is placed between the pile shell and the footing pile sleeve will also be very difficult. This must be completed after prestressing bars (installed for temporary support), welded anchor studs, and pile head shear plates are installed.

Precise Tolerance Required. It is critical that the pile cap fit on the piles. For most of the caps, six different piles, each having a different batter, must be accommodated. Even if the pile sleeves are located perfectly, the corner piles battered at 1:8 will have only 200 mm of clearance with the bottom steel of the pile cap. If one pile is 200 mm off or two corners are out a combined 200 mm, the unit will not fit. Final fabrication of the footing unit is to be completed after piles are driven to final elevation and field measurements are taken. This will involve a large percentage of the overall fabrication and will require a significant amount of time. This will ensure that the pile sleeves are positioned correctly with respect to the piles provided field measurements are accurate. The measuring and fabrication are critical steps.

Pile head shear plates are to be 900 mm wide and approximately 5000 mm long. To fillet weld, the edge of the stiffener must be within 1.5 mm of the connected plate along its entire length (AWS D1.5, 3.3.1) (or the weld size must be increased). Because it is likely that the piles will not be perfectly centered and parallel to the cap sleeve, each plate will require custom cutting and fitting. It is not practical to expect that these plates could be fabricated to the correct geometry and then fit into position while maintaining the necessary tolerance.

Additional Remarks. With some of the volumes of concrete being up to 5000 mm by 10000 mm by 5000 mm, special considerations for massive concrete will be necessary. Massive concrete issues are not addressed. It is not clear why all of the stiffeners in the pile cap footing unit are necessary. It seems that much less steel could be used. There are many steel stiffeners and a significant amount of concrete reinforcement in the pile head connection. If this is necessary for shear transfer, reinforcement on the outside (away from pile) of the footing pile sleeve is also necessary to provide a transfer of the shear beyond the pile sleeve. The contract drawings show the concrete skin to be a precast concrete wall. The calculations assume that the steel edge plate and concrete skin are a composite section. It is assumed that the concrete skin will be precast onto the unit after the steel unit is assembled and prior to installation.

In summary, the calculations are not complete, specifically in regards to plate thickness and materials used and for welding. The extent of detail developed for the finite element models is impressive, and the color stress charts provide for an expedient cursory review of the results. However, verification that the item modeled accurately represents the details shown on the plans, and analysis of the joints, is not provided. In addition, numerous constructibility concerns have been noted above. This precludes the COE Team from establishing that conformance to lifeline criteria and goals have been demonstrated.

16. Skyway Piles

As shown by the contract drawings [Document 257], each pile consists of a steel shell filled with reinforced concrete. A pile cap encases the upper portion of the piles and provides a rigid connection, which is assumed to provide fixity. The pile is directly connected to the internal steel plate system of the pier cap with a perimeter of fin type connecting plates. Composite action between steel shell and concrete is achieved with a combination of welded studs at the top of the casing and rings of shear lugs at the bottom end. The steel shell is full length and driven open ended. Clean out of the soil, within the casing is not full length, leaving a soil plug at the bottom end. The pile, then, has four distinct sections:

1. Top: Embedment into pile cap.
2. Just below bottom of cap: Heavily reinforced concrete filled, plastic hinge zone.
3. Mid-length: Nominally reinforced concrete filled.
4. Bottom: Steel shell only driven into soil.

16.A. Document 378, Volume 15 Comments

Refer to above comments under Analysis , General above regarding the changes in pier cap elevations. Per the 100 percent design plans, the pile caps have been lowered well into the bay soil. This has been apparently done to lengthen the piers in order to increase the flexibility of the foundations. However, the piles have been left as battered. This seems inconsistent and vertical piles should be evaluated.

Section 6.3.4 Pile Section Properties (Stiffness and P-M Diagrams) states that the computer program ADRIANNA-M is used to develop Moment-Curvature relationships for the piles under various axial loads (-100MN to 180MN). This analysis has been performed at three main x-sections of the pile. The three main x-sections are top-cased and heavily reinforced, center-cased and mildly reinforced, bottom-uncased and unreinforced. They have been further divided based on the degree of assumed casing corrosion. These Moment-Curvature relationships are then used as input in the ADINA models as the section properties of the piles, along with soil springs. Based on calculated moment and axial force, ADINA uses this input to determine the deflected shape of a pile and, hence the pile cap displacement.

Appendix 6

Section 6.3.5.4, Dynamic Component of Axial Resistance (per last paragraph on first page of March 9, 1999 memo) indicates that further update in the input motion and / or p-y and t-z soil spring models have been suggested. However, such an update has not been found in the documents provided.

In summary, it has been suggested that the input motion and soil springs should be updated. However, no documentation has been provided to show that this has occurred. This precludes the COE Team from establishing that conformance to lifeline criteria and goals have been demonstrated. The piles have a relatively deep point of effective fixity resulting in the behavior of long flexible columns. It appears that the slight batter of 1:8 will not significantly alter the foundation stiffness, as it will still be controlled predominantly by flexure. However, the batter of the piles detract significantly from the constructibility in regards to accurate placement and to the tie into the pile cap. Also, it does not seem rational to both batter piles for an <u>increase</u> in lateral stiffness and, at the same time, lower the pile caps up to 12.5meters, (41feet) to increase the length of piers to <u>reduce</u> lateral stiffness.

San Francisco-Oakland Bay Bridge Data Review
Replacement Evaluation

Reviewed by: Cameron Chasten **Review Date:** 9/29/00
Discipline: Structural **Document I.D. #** 257

Answers Question 1, 2, 3, or 4? 4

Description of Data Reviewed: Structural details of the pile cap. This information is provided on Sheets 054 through 069.

Answers what part of Question? Describe.
Is the currently proposed replacement alternative seismically safe? No. However, if the following concerns are addressed, the pile cap performance will be acceptable.

The reliability of the pile cap and its connection to the piles is related directly to the seismic performance of the bridge. Although construction may be possible, the pile cap as described can not be practically and safely constructed to the level of workmanship required. There are concerns regarding welding requirements, congested work area for the connection of the pile and pile cap, and impractical tolerances required.

1. Welding.

The stiffened box structure that makes up the footing unit of the pile cap is composed of intersecting steel plates that are interconnected with full penetration welds and large fillet welds. Most of the plates range in thickness from 38 mm to 68 mm. There are many locations where welds must intersect from three orthogonal directions. Due to high tensile residual stresses that develop after welding, and adverse metallurgical effects of welding, these locations will be prone to brittle fracture.

In the pile cap to pile connection, all pile head shear plates (eight plates per pile) must be custom fitted and field welded between the piles and pile sleeves. For each plate, the connected steel must be pre-heated. AWS D1.5, Section 4.2 requires that preheat and inter-pass temperature be 175 degrees F, when welding involves 2-1/2 thick steel plate (thickness of the pile steel shell). To maintain this temperature will be difficult if not impossible and will certainly impose worker safety concerns.

2. Congested work area.

The worker space available to construct the pile-to-pile cap connection is very limited, especially if the pile sleeves and piles are not exactly parallel and centered. For each pile, eight shear plates must be installed by field welding. Considering the location of stiffeners and prestressing bars, a work area no greater than approximately 750 mm by 900 mm wide in a height of approximately 5000 mm will be available. To maintain the required welding preheat and then to perform the welding in these enclosed areas is not practical. Construction of the concrete reinforcing steel that is placed between the pile shell and the footing pile sleeve will also be very difficult. This must be completed after prestressing bars (installed for temporary support), welded anchor studs, and pile head shear plates are installed.

3. Precise tolerance required.

It is critical that the pile cap fit on the piles. For most of the caps, six different piles, each having a different batter, must be accommodated. Even if the pile sleeves are located perfectly, the corner piles battered at 1:8 will have only 200 mm of clearance with the bottom steel of the pile cap. If one pile is 200 mm off or two corners are out a combined 200 mm, the unit will not fit. Final fabrication of the footing unit is to be completed after piles are driven to final elevation and field measurements are taken. This will involve a large percentage of the overall fabrication and will require a significant amount of time. This will ensure that the pile sleeves are positioned correctly with respect to the piles provided field measurements are accurate. The measuring and fabrication are critical steps.

Pile head shear plates are to be 900 mm wide and approximately 5000 mm long. To fillet weld, the edge of the stiffener must be within 1.5 mm of the connected plate along its entire length (AWS D1.5, 3.3.1) (or the weld size must be increased). Because it is likely that the piles will not be perfectly centered and parallel to the cap sleeve, each plate will require custom cutting and fitting. It is not practical to expect that these plates could be fabricated to the correct geometry and then fit into position while maintaining the necessary tolerance.

Additional Remarks.

1. With some of the volumes of concrete being up to 5000 mm by 10000 mm by 5000 mm, special considerations for massive concrete will be necessary. Massive concrete issues are not addressed.
2. It is not clear why all of the stiffeners in the pile cap footing unit are necessary. It seems that much less steel could be used.
3. There are many steel stiffeners and a significant amount of concrete reinforcement in the pile head connection. If this is necessary for shear transfer, reinforcement on the outside (away from pile) of the footing pile sleeve is also necessary to provide a transfer of the shear beyond the pile sleeve.

San Francisco-Oakland Bay Bridge Data Review
Replacement Evaluation

Reviewed by: Michael G. Mills **Review Date:** 9/25/00
Discipline: Structural **Document I.D. #** 268

Answers Question 1, 2, 3, or 4? 4

Description of Data Reviewed: Meeting minutes of SSPRP meetings.

Minutes reviewed include those for 30Apr99 and 17Feb00. These two were tagged by Caltrans/ A. Akinsanya and C. MacLeay.

17Feb00 makes note of several design issues: E. pier pile cap cross beam design; Tower stability w/ discussion of buckling mode, shear links, independent checks and attached graphs; Deck stability w/ discussion of buckling analysis, and attached graphs; Pile to pile cap connection; Pile to Pile Cap Connection; Box girder stiffener Requirements; Design Criteria; Skyway; and others.

Answers what part of Question? Describe. Does not answer question 4. It gives insight to design, but design calcs, plans and specs will take precedence and confirm the topics of these meetings.

SSPRP - Seismic Safety Peer Review Panel

Additional Remarks. Pile to pile cap connections are changed from these minutes to the present 65% design.

San Francisco-Oakland Bay Bridge Data Review
Replacement Evaluation

Reviewed by: R. Turton **Review Date:** 10/12/00
Discipline: Structural **Document I.D.** #277

Answers Question 1, 2, 3, or 4? 3 & 4

Description of Data Reviewed: *65% Submittal — Main Span Suspension Bridge Plans*

This document consists of preliminary construction plans for the main span of the currently proposed replacement structure. The plans are not annotated as having been checked. [Note that it was determined that there were several versions of Doc. 277 in the library. This particular version was dated 8/99 and is thought to be an in-progress set of plans subsequent to Doc. 256, which was printed in 5/99. This document was superceded by a version dated 10/99. On 09/05/00, Caltrans provided additional copies of Doc. 277 dated 10/99 to replace the above referenced interim document for further study.]

Answers what part of Question? Describe. *Question 4*

Is the proposed replacement alternative seismically safe?

While it is not determinable at this point in the development of the design if the design will satisfy the performance levels stipulated in the design criteria (Doc. 367), it does appear that the design is being developed to preclude collapse during a seismic event. Based on the review of work in progress to date, it appears that the proposed replacement alternative will be capable of resisting the assumed ground motion without collapse.

How will the proposed replacement perform in the maximum credible earthquake?

Performance of the proposed replacement in the MCE can not be determined from the information provided. The SEE is established as the design event for this project. While the development of the SEE is based on the MCE, satisfactory performance in the MCE can not be assumed.

Does the proposed replacement structure meet lifeline criteria?

It can not be determined if the proposed replacement structure meets lifeline performance criteria. Specifically, lifeline performance requires managing risk associated with the SEE (structural integrity) and ability to restore full traffic operations within a specified time frame (months). A quantification of the acceptable duration of traffic restriction has not been observed in documentation provided to date.

How quickly can the proposed replacement structure accommodate passenger vehicles?

While a post-earthquake scenario narrative is presented in Doc. 344, repair/replacement details and narrative on specific elements that are expected to be damaged (other than deck joints) have not been observed. Assuming that replacement of the deck joints is the only damage that will require repair efforts that impede the flow of traffic, it is estimated in Doc 344 that the duration of restricted operations will exceed three months. It is implied that emergency response vehicles can be accommodated almost immediately.

Additional Remarks.

Seismic Safety:

- It appears that the designers are working towards satisfying the stipulated performance levels, but have yet to converge on the answer (work in progress).

- It would be expected that at the 65% Submittal level that the viability of the main concepts would be confirmed, yet elements of the work appear to need additional study and documentation.
- The design should be fully developed and revisited at that time to insure that the mandated level of seismic reliability and safety has been achieved.

MCE:

- It appears that the target is to develop the design such that the main gravity load carrying elements remain elastic (nearly undamaged) during the SEE. (See Doc. 344)
- It appears that only secondary members (deck joints, struts, cross-bracing, etc.) are intended to sustain permanent deformation during a seismic event, and they are to be designed to be replaceable.
- It is stated that the design intent is to restrict inelastic behavior to the tower, piers, piles, and abutments. (See Doc. 367, Vol. 1)
- It is apparent that the designers are still working on refining the design to meet the seismic performance goals, as reported D/C ratios for some elements are in excess of 1. (See Doc. 344 and Doc. 367)
- There is concern that the tower foundation may not perform as intended due to existing geological conditions (sloping bedding plans and rock/weathered rock interface at the proposed tower foundation site.
- There is concern that the MCE may exceed the design demands of the SEE.

Lifeline Criteria:

- Lifeline performance criteria is established in Doc. 344, but the criteria is broad and contains subjective terminology with respect to the level of permissible damage and duration of operational impedance.
- While it appears to be the goal, it can not be determined if the goal has been met from the level of the design documentation provided.
- The design appears to include elements that push the envelope (extend beyond the limits of proven technology). This is contrary to Caltrans stated policy to avoid any high tech items, which could arguably include the proposed modular deck joints that require movement ratings in excess of 1m and accommodation of out-of-plane dynamic response displacement. Others concerns include large plate thickness curvature and weldments, grouting of the weathered rock at the tower foundation location, strut/tower interaction behavior under dynamic loading, etc., all which could impact seismic reliability.

Restoration of Traffic:

- Doc. 344 is understood to be the basis for the seismic design criteria for the replacement alternative.
- The design criteria is understood to be consistent with lifeline requirements whereby minor to moderate damage is anticipated.
- The level of anticipated damage is to be consistent with restoring traffic within hours and full operations within months.
- Given that deck expansion joints will need to be replaced (Doc. 344) after the SEE, there is concern regarding the ability to satisfy the presumed intent of the operational requirements of lifeline criteria whereby unimpeded traffic is restored in several months.

Other:

The tower caissons appear to be designed for plastic hinging at 5m above the steel casing cut-off elevation. Foundation design calculations indicate bedding planes and weathered rock that suggest possible sloped failure planes under lateral loading. Such a failure mode would result in piles with varying rigidities that may not have been modeled accordingly. It is not apparent if the potential for bedding plane failures and group effects were considered. It is also questionable if there is any benefit derived from the battered piles. [Consider load testing? The 3D FEM analysis accounts for reduced stiffness in the upper weathered rock, but does not appear to account for the potential for inclined bedding failure planes.]

It appears that the caissons are designed as composite columns. This design incorporates bent plates up to 95mm in thickness to form caisson diameters ranging to 2.5m. Additionally, the details require complete joint penetration (CJP) weldments in the casing, which may be subjected to tension under flexural or uplifting loads, and thus susceptible to fracture. The plate bending and welding requirements may be do-able , but appear to be beyond reasonable limits. [Brian Maroney indicated in the 9/20/00 meeting with TYLI that test piles were being constructed at the Mare Island facility.]

It is not apparent if differential foundation displacement is anticipated or was considered.

The articulation of the proposed replacement structure is not clearly established. It appears that Pier W2 is intended to provide vertical, transverse and longitudinal restraint (fixed). Since the deck is not attached to the tower, it is assumed that only inertial effects (no global transverse and longitudinal movement) are intended to be resisted by the tower. The superstructure appears to be pinned longitudinally and transversely, and restrained vertically by the bearings Pier E2. The tower saddle is fixed to the top of the tower. The cables appear to be restrained by friction over the saddle. [If so, why is it that the saddles do not need to be checked for slip at the ultimate limit state (per Doc. 286)? Has the potential for slip during a seismic event evaluated?] Rockers appear to be provided to allow the saddle to move relative to the deck to accommodate movements at the east anchorage. (See Doc. 367, Vol. 1 and Doc. 286.) [TYLI clarified the intended articulation in the 9/20/00 meeting. TYLI is to forward graphics and a written statement to document the intended behavior.]

Are the towers to be designed to remain elastic (Demand/Capacity < 1+/-)? If so, won t the removal of the yielded struts release energy? Can the restoration energy upon release of the deformed struts be managed? If there is permanent deformation in the tower, what are the procedures for replacing yielded struts? Also, what provisions, if any, are there to restore or accommodate permanent deformations in the tower? [TYLI indicated that the permanent deformation in the tower and struts as a result of the SEE are anticipated to be minimal, and they do not anticipate a need for replacing the struts or correcting the alignment of the tower. Supporting documentation is to be forwarded.]

What is the target demand/capacity ratio? [TYLI advised that all gravity load-resisting elements are designed to 0.6Fy (D/C<1). Review of the calculations indicated that some elements are reported to have a D/C>1 (1.2+/-). TYLI indicated that there are still working on those elements. TYLI advised 9/20/00 that they would forward the latest D/C ratios with a qualifying statement.]

How is the progressive yielding of the struts modeled? Is this being considered to capture the minimally plastic response (Ductility = 4) anticipated for the transverse seismic load case? {TYLI indicated in the meeting on 9/20/00 the struts are for controlling the response of the tower, and not for the protection of the bridge. TYLI will forward additional documentation that illustrates the effects of yielding of the struts.]

Was the design evaluated for a post-earthquake condition whereby the struts are yielded and the facility is subjected to a large magnitude aftershock? [In the 9/20/00 meeting, TYLI indicated that the response of the structure subjected to the SEE is intended to be essentially elastic . Given that scenario, it is assumed that the structure will respond satisfactorily to an aftershock (of lesser magnitude).]

Was the bikeway and counterweight considered in the dynamic model? [Responses by TYLI to 9/20/00 questions provided in written format dated 9/7/00 states this was done. It was also stipulated in the Draft Supplement to Doc. 367 dated 9/29/00.]

The structural adequacy of the hinges could not be confirmed. (See Doc. 367, Vol. 41, which is not complete.)

Are the deck expansion joint movement requirements feasible? Does the proposed joint fall into Caltrans category of High Tech , which is to be avoided per Caltrans directive? (See Doc. 344.) The proposed movement rating, which is in excess of l meter, appears to only provides a factor of safety slightly greater than one. Is this adequate given the unpredictable nature of earthquake displacements? [TYLI indicated in the 9/20/00 meeting that D.S. Brown has developed and tested a swivel type modular joint that is capable of accommodating the anticipate joint movements (in excess of 1m with a nominal factor of safety). Watson Bowman Acme was also reported to be working on developing a joint that will meet the performance requirements. Caltrans indicated that this was of particular interest to minimize the potential for a sole source proprietary system.]

Would the hinge location be better situated at Pier E2 (in lieu of the proposed configuration that incorporates a substantial cantilever (45m), horizontal curvature, and significant suspended span arrangement)? (Note that Pier W2 is a fixed pier with only a 10m cantilever, where Pier 2E is an expansion pier.) It would appear that displacements at the joint would be more controllable under a dynamic excitation given a joint location at the pier. [It was explained by TYLI at the 9/20/00 meeting that the cantilever was incorporated to allow the tuning of the design of the deck and behavior of the system under dead load. The significant suspended span is capable of supporting itself as a cantilever, as it is reported to be designed for balanced cantilever construction.]

Consideration of uplift/rocking potential at the tower base during a seismic excitation due to the hold-down connections was not observed.

Documentation that justifies the use of a 5% damped site-specific response spectra was not observed. (See Doc. 367, Vol. 1.)

The design criteria stipulates that in the event that construction is interrupted, the structure is to be stabilized against seismic loads. The level of stabilization and how will it be accomplished was not observed.

Significant concerns exist regarding fabrication/constructibility of a number of primary elements of the bridge, such as the pile caps, piles, tower shafts, tower grillage, etc.

Tower leg construction requires complete and partial joint penetration weldments in the skin plates. Consideration for fatigue was not observed. Similar issues can be extended to other elements of the bridge. (See the previous item.)

All bolts are to be ASTM A325 per the General Notes. Plans show A490 bolts at the deviation saddles. [Is the use of load indicating washers or other means to be specified to insure proper installation tension? Is smart bolt technology warranted anywhere (at the saddle, etc.)?]

The maintenance traveler rail consists of a flange with a complete joint penetration weldment to the web that will be subjected to direct tension. [Is there a better detail?]

The geometry at the eastern end of the main span complicates the design and construction efforts. [Can the proposed horizontal curvature in the roadway alignment be relocated to the Skyway Approach?]

The suspension cables can not be replaced without re-supporting the bridge as it was in construction.

Note: [Bracketed text indicates commentary.]

Editorial Comment:

While the aesthetics of the self-anchored suspension span are arguably superior to the alternatives considered, there are trade-offs with respect to constructibility and serviceability that translate into potentially significant cost and seismic reliability issues. Clearly a viaduct type structure would provide a more reliable facility at a lesser (initial and life cycle) cost.

San Francisco-Oakland Bay Bridge Data Review
Replacement Evaluation

Reviewed by: Michael Premo **Review Date:** 29 SEP 00
Discipline: Structural **Document I.D. #** 283 & 353

Answers Question 1, 2, 3, or 4? None

Description of Data Reviewed: Document #283 - SFOBB 100% Skyway Structures (East Span) Plans and Document #353 - SFOBB 100% Skyway Structures (East Span)Technical Specifications.

Answers what part of Question? Describe.

Additional Remarks.

100% Plans:
1. Grout labeled as grout on some sheets such as No. 391 and labeled as non-shrink grout on other sheets such as No. 423.Recommend consistent labeling to ensure a consistent product is provided.
2. Section 39-11 of the Technical Specifications requires waiting until the grout below the bearings has attained 100% of the specified strength. If this is not addressed in the Caltrans Standard Specifications, recommend specifying the required grout strength on sheet No. 423.
3. Sheet 389: No details are provided for the elastomeric bearing pad. No specification(s) is/are provided for the bonding material.

100% Technical Specifications:
1. Page 127: Specify to what degree the contractor should remove any handling devices from the steel pipe piling. Is just torching off adequate? Do you want it ground flush? Also, field is misspelled in section 7.
2. Page 163: Is Section 12 required (is there SPTC involvement for this structure)?
3. Page 234: See Note 2 for 100% plans above.
4. Page 236: shelf is misspelled in the first full paragraph.
5. Page 309: Are there any other requirements for the PVC plastic pipethat might warrant having a separate section in the technical specifications.

San Francisco-Oakland Bay Bridge Data Review
Replacement Evaluation

Reviewed by: Yusof Ghanaat **Review Date:** 9/26/00 to 10/2/00
Discipline: Seismic Input **Document I.D. #** 290, 331, 335, 367 (Vol. 37)

Answers Question 1, 2, 3, or 4? 4

Description of Data Reviewed: Seismic Hazard Ground Motion Criteria

The listed documents describe seismic hazard ground motion criteria adopted for the SFOBB replacement alternative. Two levels of ground motions termed as Safety Evaluation Earthquake (SEE) and Functional Evaluation Earthquake (FEE) were considered. The events on the Hayward Fault and San Andreas Fault dominated the SEE ground motions. Briefly, the FEE ground motions were obtained deterministically for a Magnitude 6.5 event on the Hayward Fault at 10 km from toll plaza. The FEE ground motions roughly corresponds to a 92-year return period or two events with 50% probability of occurrence in 150 years. The SEE ground motions were developed probabilistically having a 1500-year return period. Various aspects of the SEE ground motions are discussed below.

1. Earthquake sources and MCE

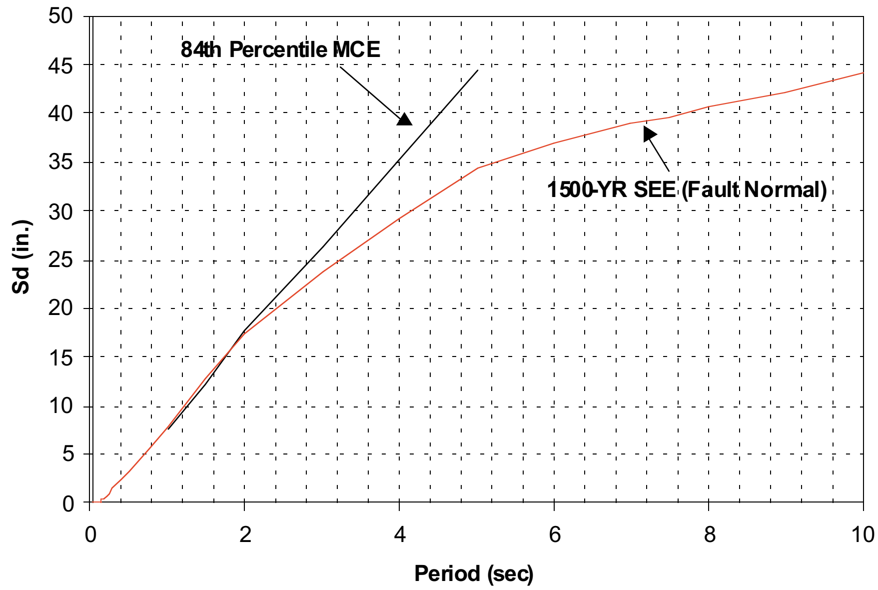
For the east span of SFOBB, the dominant earthquake sources are the Hayward fault at 9 km east of the Oakland Toll Plaza and the San Andreas faults at 18 km west of Yerba Buena Island. Other faults considered in the seismic hazard analysis include San Gregario, Roger's Creek Healdsbury, Green Valley, Calaveras, and Greenville.

The maximum credible earthquake (MCE) ground motions associated with the Hayward and San Andreas faults were used in the retrofit design and evaluation of the exiting bridge. The maximum magnitudes for these events were based on a study of seismic hazard for the northern California bridges. A moment magnitude of M_w = 7.3 was assigned to the Hayward Fault located 8 km from the east end of the east span and an M_w = 8 to the San Andreas Fault located 19 km from the west end of the east span. The 84th percentile MCE ground motions were developed for each event. According to the Geomatrix probabilistic hazard assessment [Geomatrix, 1992] the 84th percentile MCE ground motion is between 1000- and 2000-year return period equal hazard spectra.

2. Probabilistic seismic hazard analysis

An Ad Hoc committee comprising of Bruce Bolt, Joseph Penzien, Roger Borcherdt, and Norm Abrahamson recommended a probabilistic approach and a return period of 1500 years for development of ground motions for the Safety Evaluation Earthquake (SEE). A return period of 1500 years corresponds to a probability of 10% exceedance over the 150 years service life of the new bridge. The 1500-year ground motion spectra were developed at Yerba Buena Island (western limit) and the Toll Plaza (eastern limit). While the YBI motions are smaller than the TP motions for short periods (T< 0.5 sec), differences are negligible for the long periods (T >0.5 sec). Stating that the bridge is insensitive to short periods, the Ad Hoc committee selected the YBI spectrum (the lesser motion in the short period range) as a single spectrum for the entire bridge.

A comparison between the 1500-year spectrum and the deterministic spectra developed for the San Andreas and the Hayward faults [Figure 2-5, Doc # 335] indicates that even though the 1500-year spectrum exceeds the Hayward and San Andreas 84th percentile spectra at periods less than 2 sec, it falls below the San Andreas MCE spectrum at periods greater than 2 sec. In the range of 2 to 5 seconds, the San Andreas spectrum gives 1 to 30 percent larger ground displacements than the 1500-year spectrum, as shown in the figure below. In other words, at periods longer than 2 seconds the 1500-year SEE ground motion is less than the MCE ground motion and thus is not conservative in this range.



Comparison of displacement spectra for 1500-year SEE and San Andreas 84th percentile MCE

3. Spectrum-compatible reference rock motions

Reference rock ground motions consisted of six sets of recorded motions that were modified such that their response spectra would closely match the target rock spectra for the SEE event. Three sets of the motions corresponded to the San Andreas event and other 3 to the Hayward event. The six sets of time histories were selected to include a variety of directivity pulses such as a one-sided pulse, two one-sided pulses, two-sided pulses, and multiple pulses. Currently these have been developed for the suspension span only.

4. Coherency compatible multiple-support rock motions

The reference rock motions discussed above were used to generate multiple-support motions at each pier of the self-anchored section of the bridge by applying the latest techniques. The process involves consideration of the effects of traveling wave, scattering and wave propagation, attenuation of motion with distance, cross-correlation between horizontal components, and spectrum-compatibility over 360-degree rotations.

5. Site response and soil-pile interaction analyses.

The multiple-support rock motions were used in free-filed site response analyses to develop input motions at various soil layers for the soil-pile interaction analysis or for input to the bridge model at the pile cap location. Both one-dimensional (SHAKE) and two-dimensional (QUAD4M) free-field site response analyses were performed. The two-dimensional site analysis was performed to evaluate the potential basin edge effects for geologic conditions immediately east of Yerba Buena Island. The results of two-dimensional analyses showed an increase in long period (above 2-sec) motions over those obtained from one-dimensional analyses. Subsequently, the 2D procedure was used to generate free-field motions for analysis of the self-anchored bridge.

Section 6.0 of Document 335 describes a procedure for developing input response spectra from the multi-input multi-degree-of-freedom soil-pile-foundation. The procedure involves formulation of a global bridge model consisting of the superstructure, pile-cap, and pile foundation. Apparently, the input to this model is in the form of displacement time histories applied at the soil spring supports. However, it is not clear how the resulting equations of motion were solved and why the stiffness and mass of the superstructure were included in the kinematic interaction analysis. Was a model consisting of the superstructure, pile cap, and the pile foundation developed for this purpose? A description of this model and solution of kinematic motions could not be found.

Volume 37 of Document 367 discusses free-field motions applied at the foundation spring supports in the ADINA nonlinear analysis of the self-anchored bridge. According to this discussion, response spectra of the free-field motions at Pier E2 indicate a low valley at about 2 seconds. The computed bridge response therefore appears to have been underestimated due to deficiency of the input motion. This issue needs to be resolved and its effects on the design are assessed.

6. Site-specific and foundation-type specific design response spectrum (ARS).

Apparently, the kinematic motions at the pile cap were used to develop acceleration response spectra (ARS) for structural analysis. The effects of p-y stiffness and variation of soil parameters on ARS were also considered. However, as discussed above, the solution of kinematic motions is not fully described and thus its accuracy could not be assessed.

Section 7.3 of Document 335 states that "The proposed 5-percent damped ARS criteria for battered piles results in between 5 to 10 percent reduction in shaking from the vertical pile groups at short periods, but approaches the vertical pile ARS criteria at longer periods." Such finding appears reasonable considering that the top 20 to 30 m of the piles are subjected to none or relatively small lateral soil resistance. On this basis, the piles appear to be so flexible that the effectiveness and advantages of the battered piles over vertical piles diminish. In fact, dominant periods of the various sections of SFOBB are about 3 sec and longer, at which the shaking for the battered and vertical piles is approximately the same. In summary the battered piles offer no or very little advantages and their use does not justify their complicated connection design and construction.

Answers what part of Question? Describe.

The use of two levels of ground motions for the safety and functional assessments of the bridge is appropriate. The ground motions for the safety evaluation earthquake (SEE) were developed probabilistically using a return period of 1500 years. The ground motions for the functional evaluation earthquake (FEE) were obtained deterministically for a Magnitude 6.5 event on the Hayward Fault at 10 km from toll plaza. The SEE ground motions appear reasonable at periods up to 2 sec, the period range where the SEE response spectrum exceeds the 84 percentile MCE spectra for both the Hayward and San Andreas events. However, at periods longer than 2 sec the SEE response spectrum falls below the 84-percentile MCE spectrum for the San Andreas event. In fact, in the period range of significance to the bridge (i.e. 2 to 5 sec) the San Andreas 84-percentile MCE spectrum shows 1 to 30 percent higher ground displacements than the proposed SEE. On this basis the SEE ground motions used in the design of replacement alternative are lower than the MCE ground motions referenced in Question 4. Furthermore, the generated free-field time histories at Pier E2 are deficient at period of 2 sec, the period that approximately matches the pile foundation period at this location.

In summary, if subjected to the MCE ground motions, the bridge response would be higher than that computed for the SEE.

Additional Remarks.

Permanent ground movements. The potential for permanent ground movements associated with accumulation of seismically induced strains in the soils surrounding and/or beneath the pile foundations has not been specifically addressed in the documents presented. Note that if such movements were to occur, they may be additive to the tectonic differential movements that occur between piers. In response to this issue, the Caltrans seismic advisory board offered an estimate of less than 1 cm differential permanent bedrock movement between two adjacent piers. Although this estimate is appropriate for supports founded on rock, it may not be suitable for the self-anchored span where the main tower and W2 supports are founded on rock whereas the E2 support is founded on soil.

San Francisco-Oakland Bay Bridge Retrofit/Replacement Evaluation

Date Document Reviewed by COE: August 2000 Reviewed by: Chung Wong

Document I.D.#: 324

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(1) Structure

Suspension span and skyway structure

(2) Ground Motion Criteria

- Executive summary on seismic hazard ground motion criteria prepared by Earth Mechanics.

- A probabilistic approach was adopted to define the ground motion design criteria for the main span.

- The dominant earthquake sources are the Hayward fault at 9 kilometers east of the Oakland Toll Plaza and the San Andreas faults at 18 kilometers west of Yerba Buena Island.

- The Ad Hoc Committee on Ground Motions recommended that a 1,500-year return period for the Safety Evaluation Earthquake (SEE), corresponding to a 10% probability of exceedance over the 150-year expected life span of the East Bay Crossing structures.

- The Ad Hoc Committee recommended that the structures be checked against six sets of multiple-support ground motions with the rock motions spectrum compatible to the 1,500-year return period spectra: three sets for San Andreas earthquake scenarios and three sets for Hayward earthquake scenarios. The committee also expressed a strong preference that time histories be based on actual recordings rather than on synthetic motions.

- Near-fault rupture directivity effects were considered in design ground motions.

(3) Remarks

The document provides a good summary on seismic hazard ground motion criteria. However, all data presented pertain only to SEE. For more information on FEE, refer to Document 335. It should be noted from Figure 3 that the fault normal and fault parallel SEE design spectra completely envelop the deterministic MCE based on ATC-32 guidelines.

(4) Conclusions

This document will provide a basis for our answers to Questions 3 and 4. The seismic input motions for the ADINA models were based on the ground motion criteria specified in this document.

San Francisco-Oakland Bay Bridge Data Review
Replacement Evaluation

Reviewed by: **D. Gray/A. Pujol** **Review Date:9/7 &10/3/00**
Discipline: **Geotechnical** **Document I.D. #332**

Answers Question 1, 2, 3, or 4? 4

Description of Data Reviewed: Describes overview of bridge and foundation design concepts for the main span east pier and skyway sections. Includes a summary of field and laboratory investigation methods and results (stratigraphy and relevant soil properties). Describes the axial pile design considerations, design methodolgy and approach. A pile driveability study is presented for two possible hammer sizes to evaluate expected blow counts, driving stresses and installation concerns. The axial pile desing and drivabilty study is based on 45% design drawings developed by TY Lin/M&N.

Axial pile capacity is controlled by earthquake loadings. Pile performance considerations included permanent settlement, reduction in soil strength due to cyclic degradation, increase in skin friction due to rate of loading effects, variations in end bearing, soil setup and load transfer for large diameter piles. Cyclic degradation of SF Bay soils is estimated using experience and data from the Gulf of Mexico off-shore industy. Design is based on API guidelines for large diameter pile. Due to the variation in soil conditions, capacities are developed for each pier. Further analyses is planned to evaluate static and dynamic pile performance using updated structural loading imfomation. Details of this analyses and a preliminary evaluation are provided of one pier location. The approach and analyses appear reasonable and in general agreement with API guidelines. The preliminary analyses of one pier indicates a permanent post-earthquake tip settlement of .0008 m (0.3 inches). This seems very small for the magnitude of the expected loads and should be check during the final analyses.

Pile driveability studies indicate that both hammers would be capable of driving the CISS piles into the lower Alameda sands but that th smaller of the two hammers would not be capable of driving the Main Span East Tower pile through the Upper Alameda Paleochannel sand. The evaluation appears thorough and reasonable. Evaluation of additional pile hammers would be prudent to provide contractors additional equipment options. Since a pile driveability test program is planned we assume the results of this study will be revised using the actual field data.

Answers what part of Question? Describe. Begins to answer Part b of Question 4 regarding how the replacement bridge will perform in a SEE.

Additional Remarks. The analyses reviewed is based on 45% design concepts and future modifications and analyses are recommended in the document.

San Francisco-Oakland Bay Bridge Retrofit/Replacement Evaluation

Date Document Reviewed by COE: August 2000 Reviewed by: Chung Wong

Document I.D.#: 335

(1) Structure

Suspension span and skyway structure

(2) Ground Motion Criteria

- A probabilistic ground motion approach was adopted for the Safety Evaluation Earthquake (SEE).

- As a result of latest advancement in treatment for directivity effects, the design shaking level is increased by about 30% as compared to prior study conducted for seismic retrofit.

- A 1,500-year return period was adopted for SEE corresponding to a probability of 10% exceedance over the 150-year service life of the new bridge.

- A deterministic earthquake corresponding to a Magnitude 6.5 event on the Hayward Fault has been adopted for Functional Evaluation Earthquake (FEE) which roughly corresponds to a 92-year return period.

- Six sets of multiple-support motions are to be developed for SEE design (see Document 324) with 3 sets corresponding to the San Andreas event and 3 sets corresponding to the Hayward event. The six sets of motion incorporate a wide range of seismological features (near fault displacement and velocity pulse characteristics) into the seismic input motions.

- Free field site response analyses and kinematic soil-structure interaction analyses were conducted to develop the ARS criteria for the skyway structures.

- By designing for the envelope of six sets of multiple-support input motions, each of which is compatible with the 1,500-year equal-hazard spectrum, the effective return period is greater than 1,500 years. The recommended ARS curves are judged to be reasonable and regarded prudent, even they might have exceeded the 1,500-year definition.

(3) Remarks

Document 335 provides a rather comprehensive summary on seismic hazard ground motion criteria. This document is an interim report and data presented can be considered as good background information. However, Document 324 should be considered as the final report for seismic hazard ground motion criteria.

(4) Conclusions

Document 335 provides good background information, but it will not be considered in answering Questions 3 and 4.

San Francisco-Oakland Bay Bridge Data Review
Replacement Evaluation

Reviewed by: M. McCaffrey/ Gray/ Pujol (GEI) **Review Date:** 9-22-00
Discipline: Geotechnical **Document I.D. #** 342

Answers Question 1, 2, 3, or 4? 4

Description of Data Reviewed: Rock Slope Stability Report, West Pier and Main Pylon by F-EM dated 10-1-99

Answers what part of Question? Describe. The F-EM report concludes that the seismic stability factor of safety for the rock cut slope on the west side of the Main Pylon is less than one for a 45 degree rock cut. The report recommends a 30 degree slope. However, the 65% design drawings show a 45 degree cut. Failure of the cut slope during an earthquake could result in unforeseen lateral load on the piles. The 65% design calculations do not contain an analysis to verify that failure of the slope (and the resulting loads on the foundation) would result in minimal or no damage to the piles, pile cap, and pier. The design of the subject rock cut slope as presented on the 65% design drawings does not meet the recommendations of report 342. Because the implications of the seismic failure of the slope have not been investigated, the seismic safety of this foundation cannot be assured. The answer to Question 4 as regards this aspect of the main pylon foundation is we don t know.

Additional Remarks. The scope of work performed for the F-EM slope stability analysis is reasonable. The stability analysis was based on a reasonable evaluation of field mapping data, borings, data from previous studies, and modeling. Three-dimensional stability analyses were performed using the computer models Swedge for static stability and the Key Block Theory model that includes earthquake accelerations and resistance from rock bolts. In general, the F-EM conclusions seem reasonable. However, we have the following comments;

1. Sensitivity Analysis - The stability analysis did not include a sensitivity analysis, in which the stability input parameters (such as rock properties and the orientations of the discontinuities) are individually varied within the range of their data set to evaluate the most sensitive parameters. The values of the most sensitive parameters may require adjustment to reflect their sensitivity and range of data.

2. Joint Friction Angle - The same joint friction angle, 30 degrees, was used for all six joint sets in the analyses. This seems appropriate only if the joint roughness and rock type is the same for all joint sets. The weak interbedded siltstone and claystone, described in Section 5.3.5, have a lower friction angle (average of 25 degrees). These were not considered in the F-EM stability analyses, but could control stability along the bedding plane.

3. Short Bolts - Rock bolt lengths of 10 to 15 feet recommended by F-EM to support the potential wedges in the West Pier do not seem long enough in all cases to extend beyond the wedge near the top of the cut.

4. Lateral Pile Resistance - The report does not address the possible differences in lateral support provided at the top of the piles for the Main Pylon. On the west side, the piles are in Slightly Weathered to Fresh Bedrock and on the east side the piles are in Weathered Bedrock and Stiff Clay, as shown in Plate 4

5. Unstable Wedges - For the West Pier, F-EM found the stability of potential wedges on three sides of the temporary cut have a factor of safety below 1. Thus, these wedges would fail during excavation. If the wedges do not fail as predicted, removing unstable wedges could be less costly than supporting them with rock bolts. Rock bolts installed before excavation may be appropriate for the N-slope to minimize ground loss below the pier.

6. Clay-filled Joints - Steeply dipping clay-filled joints with slickensides, described in Section 5.3.2 of the F-EM report, do not seem to have been incorporated in the stability analyses and could reduce the stability of the rock slopes.

San Francisco-Oakland Bay Bridge Retrofit/Replacement Evaluation

Date Document Reviewed by COE: August 2000 Reviewed by: Chung Wong

Document I.D.#: 344

(1) Structure

Suspension span and skyway structures

(2) Performance Criteria

Table 1 of Document 344 provides seismic performance criteria for SEE and FEE. Comparing the performance criteria to those presented in Volume 1 of Document 367, the latter actually provides more detailed and specific performance criteria. However, Table 2 of Document 344 gives definition of minor damage and moderate damage which cannot be found in Volume 1 of Document 367.

(3) Design Criteria

- Basically the same as those listed in Volume 1 of Document 367.

- Under seismic acceptance criteria (Table 2) for 30% design, the following was specified for displacement — permanent displacement as small as reasonable but may be as large as _ foot at pilecap and 1 foot (0.3 meter) at deck level (this has been reduced to 0.2 meter according to the Executive Summary of Document 384).

(4) Analysis Methodology

- A very general discussion on analysis methodology, in particular, on demand model and capacity model (global and local).

- Brief discussion of different types of structural analysis (dynamic response spectrum analysis, nonlinear static pushover analysis, and nonlinear dynamic time-history analysis), features associated with each type of analysis.

(5) Remarks

This document focuses more on philosophy of analysis and design than on the actual techniques used to perform analysis and design. The seismic performance and design criteria presented in this document have generally been covered in Volume 1 of Document 367, except for the specific definitions of earthquake damage levels.

(6) Conclusions

This document will provide a basis for our answers to Questions 3 and 4. The design will be checked to ensure compliance with performance and design criteria specified in this document.

San Francisco-Oakland Bay Bridge Data Review
Replacement Evaluation

Reviewed by: R. Turton **Review Date:** 10/12/00
Discipline: Structural **Document I.D.** #344

Answers Question 1, 2, 3, or 4? 3 & 4

Description of Data Reviewed: *Seismic Design Criteria (Draft, Version 12, dated 6/27/00)*

This document establishes the seismic design criteria for the proposed replacement structure. It is primarily a qualitative document with some quantitative information relative to allowable strain limits for concrete and reinforcing steel.

Answers what part of Question? Describe. *Question 4*

Does the currently proposed replacement alternative meet lifeline criteria?

It does not appear that the replacement structure is capable of meeting lifeline criteria as it is understood. While the design of the currently proposed replacement structure may be arguably superior to a retrofit alternative with respect to managing risk associated with an SEE (structural integrity), it can not be determined if operational requirements can be satisfied. Specifically, lifeline criteria require the facility to be fully operational in months . A quantitative limit of acceptable duration of traffic restriction has not been observed in performance criteria documentation to date. Such a limit is necessary for a determination of conformance.

To what extent and how quickly could it accommodate passenger vehicles?

Restoration of full traffic operations may take 6 months or more. This assessment is based on an understanding of the anticipated damage as a result of the SEE. Specifically, that the deck joint assemblies will sustain damage and require replacement. [This estimate of the duration of restriction is collaborated by the post-earthquake scenario included in Doc. 344, conversations with major deck joint manufacturers, and experience.]

Additional Remarks.

Lifeline Criteria:

It appears that deck joint replacement efforts could easily exceed 6 months in duration. It seems that restoration of full traffic operations should be attained in significantly less time for a lifeline-designated facility.

[The Final Version of Doc. 384, received on 10/03/00, contains an assessment of the level of joint damage associated with the SEE that is contradictory with Doc. 344. Specifically, the Executive Summary states that minor damage to expansion joints, at the extreme edges, may occur . While minor damage is not defined, it is reasonable to assume that this implies restoration of unimpeded traffic could be accomplished within a time frame that would be consistent with the intent of lifeline performance. The post-SEE scenario presented in Doc. 344, dated 6/27/00, indicates that a deck joint replacement effort will be required. It also indicates that the duration of the joint replacement effort will exceed 3 months. It is understood (and implied on Sheet 425 of Doc. 283) that the replacement efforts will require phased construction, thereby further impeding restoration of full operations in a manner that is understood to be consistent with lifeline performance. This contradictory assessment needs to be addressed.]

Restoration of Traffic:

Doc. 344 — Seismic Design Criteria presents a post-earthquake scenario. It is assumed that the scenario presented relates to the SEE. The development of the design, as it is understood through review of plans and calculations, and meetings with Caltrans and TYLI, is based on the elements of the structure remaining essentially elastic (nearly undamaged) during the SEE. Given such a scenario, it is reasonable to assume that the post-event efforts will be primarily geared towards mitigating displacements due to localized yielding or foundation movement (as opposed to overstresses). Project documentation indicates that displacement damage is assumed to be limited to replacement of the deck joints. Any other distress is assumed to be able to be addressed without impacting traffic operations, or the damage is such that repair would not be warranted. [Doc. 344 is noted to be authored by Caltrans. Whether the document was a part of TYLI s initial or modified scope of work has not been ascertained. The document is noted as Version 12 dated 6/27/00 and the chronology and implementation of the document has not been established.]

The design criteria qualitatively establish minimum acceptable performances level for the replacement structure. Specifically, it identifies the structure as a lifeline facility. (From review of other documents, it is apparent that Interstate 80 was designated a State Lifeline Route resulting in the lifeline performance criteria directive in 1996 by BBDTF/MTC/EDAP. This directive is established in Doc. 263, which is the 30% Design Submittal developed by Caltrans. Doc. 344 also identifies that minor to moderate damage is anticipated, but the facility would be operational at slowed speeds within hours and fully operational within months . Moderate damage is characterized as visible and likely to require emergency contracts to repair. [Note that a direct link between the qualitative (Doc. 344) and quantitative criteria (Doc. 367) for satisfying lifeline performance levels has not been observed. Caltrans (Ade Akinsanya) indicated on 9/27/00 that Doc. 344 was an evolving document, but it was coordinated with TYLI as the design was being developed. He also indicated that the quantitative parameters (Doc. 367, Vol. 1, Design Criteria) representing the qualitative performance requirements were developed by Caltrans in concert with TYLI, and with review panel oversight.]

The assumed post-SEE scenario is such that steel plates would be immediately placed at failed deck joints to allow for maintenance of traffic at slowed speeds. The scenario then identifies construction activities to replace the deck joints would be underway by the end of the third month. Given the proposed scenario, it is reasonable to assume that to fully restored traffic would take a minimum of 4 months. The proposed scenario allows about 11 weeks to quantify deck joint damage and fabricate/deliver replacement joints to the site. Given the high-tech nature of the deck joints required to accommodate the anticipated range of motion, the quantity of deck joints, and the potential for permanent deformation (distortion, greater, and/or non-uniform gap), it is questionable that replacement units could be provided to the site in such a time frame. Also, given maintenance of traffic issues, it is anticipated that staged demolition and replacement of the deck joint units will require an extended duration of construction.

The deck joints in the roadway of the main span (total 4) need to accommodate an operational movement of 450mm and a SEE movement of 1500m. The deck joints in the roadway of the Skyway (total 8) need to accommodate operational movement ranges from 120mm to 500mm and SEE movements ranging from 250mm to 980mm (without unseating). (See Doc. 259.) [There is also a concern regarding the capability of the hinges to support gravity loads at the large seismic displacements. Doc. 367, Vol. 41 addresses the design of the hinges, but it is incomplete.] The Oakland Shore Approach, Oakland Slab Approach, and the Yerba Buena Island Approach have another 5, 6 and 4 deck joints respectively, for a total of 27 roadway joints between touchdowns. (See Doc. 259.)

Note that on Sheet 425 of Doc. 283, which is the 100% In-Progress Plans Submittal for the Skyway, shows that the deck expansion joint movement requirements have not been revised from the 65% Submittal (Doc. 259, Sheet 357). Both submittals stipulate that the assemblies be fabricated in 3 equal length units to allow for progressive replacement (staged reconstruction to allow for maintenance of traffic). The notes also indicate that beyond the operational limits of the joint, the joints will have extended support bars to prevent unseating of the support bars during the SEE.

Also note that the 100% In—Progress Technical Specifications Submittal (Doc. 353.) does not include any specifications for the deck expansion joints. Rather, it indicates that the joint seal assemblies are to be covered by a standard special provision (51JTAS_R04-14-00). (See Page 12.)

Related Documents:

259, 263, 283, 353, 367, 384

Note: [Bracketed text indicates commentary.]

San Francisco-Oakland Bay Bridge Data Review
Replacement Evaluation

Reviewed by: R. Turton **Review Date:** 10/12/00
Discipline: Structural **Document I.D.** #367

Answers Question 1, 2, 3, or 4? 3 & 4

Description of Data Reviewed: *Suspension Span Design Calculations, Volumes 1-41*

Vol. 1 contains a description of the structure and the design criteria. The description includes a brief discussion about the intended articulation of the bridge. The design criteria appear to have been superceded by Doc. 286 dated 6/30/00.

Vol.2 — Not reviewed.

Vol. 3 contains results of the dead load and service load analyses of the global model using ADINA (Dead Load and Group I). Reported demand/capacity ratios are limited to approximately 0.65 for dead load for the spine. Reported demand/capacity ratios range to slightly over 1 for Group I loads. Reported demand/capacity ratios range to slightly less than 0.8 for live load for the tower shafts.

Vol. 4 contains the Demand and D/C ratios for the service load analyses of global model using ADINA (Group II & IV). Torsion loading case due to transverse eccentricity of loads (bikeway, LRT dead load) with global dead load is not included. D/C ratios for LRFD Group I results are not included.

Vol. 5 — Not reviewed.

Vol. 6 contains the results of the ground motion analyses of the global model using ADINA. D/C ratios are in excess of 2 for the box section at the anchorages (2.10), west bent (4.96), east bent piles (2.07). Other elements are noted to have D/C ratios greater than 1. D/C ratios appear to be based on allowable stress levels. (The design criteria stipulates that inelastic behavior shall be restricted to columns, piers, piles, and abutments. See Doc. 367, Vol.1.)

Vol. 7-10 — Not reviewed.

Vol. 11 contains the construction sequence analysis (in reverse) of the global model using ADINA.

Vol. 12 contains the aerodynamic analyses of the global model using ADINA and the wind testing results.

Vol. 13 — Not reviewed.

Vol. 14 contains a description, geometry and cable profile, analyses, and design of the deck system. (Note that the dead load model of the deck assumes a pin at the west pier and a roller at the east pier.)

Vol. 15 contains a description, analyses (using ANSYS), and designs of the floor beams, shear frames, and box sections/splices for the deck system.

Vol. 16 contains a description, analyses (using ALGOR) and designs of the cross-beams and connections, and orthotropic plate for the deck system.

Vol. 17 contains the description, analysis, and design of ancillary items associated with the deck system such as the drainage system, barriers, bike path, utility supports, access provisions, lighting supports, provisions for light rail, and mechanical system supports. (Note that many of the items were not included or complete.)

Vol. 18 contains a description, analyses (using SAP90), and design of the bikeway.

Vol. 19-24 — No reviewed.

Vol. 25 contains the analysis results and design of the west anchorage tie-down. Demand/capacity ratios for the tie-down ranged to 1. The FEM analyses were accomplished using SAP2000.

Vol. 26 contains cable geometry calculations, loop anchor cable force and displacement determinations, deviation and jacking saddle analyses and design, and cable placement and jacking sequencing for deck erection.

Vol. 27 contains the SAP2000 input files for the west anchorage analysis.

Vol. 28 contains the analyses and design for the east anchorage, including cable and strand geometry, cable placement and jacking sequencing, and strand anchorage design. (Note that the text indicates that the vertical component of the cable force was assumed to be zero, impying that the validity of the assumption would be confirmed later.)

Vol. 29 contains a description, geometry determination, analyses and design for the box girders, cross-beams, cable saddles, saddle base, splay castings, cable bands, saddle supports, bearing connections, and cable housings at the east anchorage. (Note that the box girder design is annotated as being based on an early cable anchorage location and needs to be adjusted to reflect any revisions.)

Vol. 30 contains material (similar to that included in Vol. 31 for Pier W2) for the design of the east pier (E2). Corresponding capacity/demand ratios for the shear design of the pier range from approximately 1.5 to 2.0.

Vol. 31 contains the design procedure for the west pier (W2), analyses results using ADINA, X-Section analyses and development of P-M-Phi curves, and the shear design. (Note that the pier design has not been completed due to difficulties in determining the moment-curvature of the pier section under tension due to flexure.) The shear design of the pier results in a capacity/demand ratio (factor of safety) in the plastic hinge region that ranges from 1.6 to 2.5.

Vol. 32 contains the results of a study to determine the sensitivity of the analyses of the piers to biaxial bending considerations. It is reported that biaxial bending considerations are not accommodated by the parametric formulation of the non-linear beam elements in the ADINA program. This was deemed necessary to evaluate the reliability of the model to reasonably predict displacements. (Note that the study indicated that actual drift might vary up to 15% from the value predicted by analysis.)

Vol. 33-40 — Not reviewed.

Vol. 41 contains the hinge design. The level of documentation was significantly incomplete.

Answers what part of Question? Describe.

See Evaluation Form for Doc. 277, 344, and 384.

Additional Remarks.

San Francisco-Oakland Bay Bridge Data Review
Replacement Evaluation

Reviewed by: Michael G. Mills **Review Date:** 9/25/00
Discipline: Structural **Document I.D. #** 367, Vol 1

Answers Question 1, 2, 3, or 4? 4

Description of Data Reviewed: Introduction, Design Criteria, Alignment

0.0 Master Contents
1.0 1-page, Narrative description of SAS bridge. W/drawings.
2.0 Design Criteria (draft 4/9/99, Revision 6). Details criteria for design loads, material parameters special requirements, seismic design, and geotechnical / foundation design. Some parts are incomplete.
3.0 Alignment w/ coordinates presented.

Answers what part of Question? Describe. Answers question 4. Presents criteria to be used for design and performance. Presents seismic criteria. Does not define lifeline criteria unless the SEE (safety evaluation earthquake) criteria is to be taken as lifeline.

SAS - Self-anchored-suspension

Additional Remarks. Some parts of Design Criteria are incomplete.

San Francisco-Oakland Bay Bridge Data Review
Replacement Evaluation

Reviewed by: Michael G. Mills **Review Date:** 9/04/00
Discipline: Structural **Document I.D. #** 367, Vol 2

Answers Question 1, 2, 3, or 4? 4

Description of Data Reviewed: Section and Material Properties for ADINA model.

1. Summary of Section Properties. Included for Suspension System, Deck, Cross Beams, Bents, and Pile Caps.
2. For the above, various data is presented, including: Geometry. Material properties and with various relationships such as Moment — Curvature, Moment — Axial interactions, stress — strain strength of material curves,

Answers what part of Question? Describe. Does not answer question 4. There is no discussion relating to seismic safety and lifeline criteria.

Additional Remarks. Many of the tables presented lack a meaningful description or discussion; thus they are only tables of numbers. It is assumed that these numbers will be used as input to the structural analysis. Many of the hand written notes by LR of TYL are marginally legible.

San Francisco-Oakland Bay Bridge Data Review
Replacement Evaluation

Reviewed by: Michael G. Mills **Review Date:** 9/0600
Discipline: Structural **Document I.D. #** 367, Vol 3

Answers Question 1, 2, 3, or 4? 4

Description of Data Reviewed: Global Analysis -- Dead Load & Service Load Analysis.

1. Data presented for dead load calculations; box types are not distinguishable.

2. Cable profile w/ spine (deck) loads via suspenders. No info on method used to profile cable sag.

3. Dead Load State.

   a. ADINA model. Plots and tables show load variances by coordinate position. No explanation or discussion provided.
   b. SAP Model. Plots show D/C ratio for major components by coordinate. Spine has D/C greater than 1.

4. Input File — SAP2000. Geometry, elements, ... no discussion.

5. Live Load. Some paragraphs are not presented. Live load envelopes are presented and D/C is shown less than 1.. (See DL above.)

   a. Cover Sheet at 4.3.4.1 says w/ 6 Lanes No LRT. Plot says w/ 5 Lanes and LRT.
   b. Cover Sheet at 4.3.4.2 says w/ LRT. Plot says w/ 5 Lanes and LRT.

Answers what part of Question? Describe. Does not answer question #4. There is no discussion relating to seismic safety and lifeline criteria.

Additional Remarks. For Live Loads, given the noted heading terminology, the loads presented are ambiguous.

San Francisco-Oakland Bay Bridge Data Review
Replacement Evaluation

Reviewed by: Michael G. Mills **Review Date:** 9/04/00
Discipline: Structural **Document I.D. #** 367, Vol 4

Answers Question 1, 2, 3, or 4? 4

Description of Data Reviewed: Global Analysis: Service Load Analysis

1. Static Load Cases on Global Model for: Thermal Effects: Temp. Gradient and Uniform Loads; Wind Loads; . Presents output plots for Spine, Cross Beam, Main Cable, Suspender. Output includes demand capacity ratios, demand forces and moments. Load cases include Group IV, III, II and LRFD.

2. Global Displacements for 6 Lane Highway Loading and 5 Lane +LRT. These are presented in hand notations on plots. File names are shown in margins, but no correlation to preceding output in volume.

Answers what part of Question? Describe. Does not answer question #4. Output plots do not detail what input loads used to determine member forces for seismic safety, if any.

Additional Remarks. Static forces are necessary to determine member sizes and overall design of structure. Structural analysis is necessary to determine behavior of structure, its reliability, and its seismic safety. These must be correlated to input data including geometry, member properties, and loads; in order to make the output data significant.

San Francisco-Oakland Bay Bridge Retrofit/Replacement Evaluation

Date Document Reviewed by COE: July 2000 Reviewed by: Chung Wong

Document I.D.#: 367 Volume 4 of 41

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(1) Structure

Suspension span

(2) Structural Analysis Results

- Plots showing the demands (forces and moments) and demand/capacity ratios for main cables, suspenders, spines, cross beams, and tower shafts for Load Group IV in which thermal effects were considered. The demands are presumably based on a global model, but it is not clear whether it is a SAP2000 model or ADINA model.

- Plots showing the demands (forces and moments) and demand/capacity ratios for main cables, suspenders, spines, cross beams, and tower shafts for Load Group II in which wind loads were considered. The demands are presumably based on a global model, but it is not clear whether it is a SAP2000 model or ADINA model.

- Plots showing the demands (forces and moments) for main cables, suspenders, spines, cross beams, and tower shafts for LRFD Group I. The demands are presumably based on a global model, but it is not clear whether it is a SAP2000 model or ADINA model.

- A plot showing the tower axial force at the base vs. displacement at the top based on a stability analysis of the tower-only model to analyze longitudinal buckling. It is not clear what program was used for the buckling analysis.

- Some numbers for the global displacements of the suspension bridge were provided. It is not clear what program was used for the analysis.

(3) Remarks

This document provides a lot of plots for the demands and demand/capacity ratios. No analysis and calculations were provided for the demands and demand/capacity ratios. These data will be useful in design but not in addressing the seismic safety of the suspension bridge. There are no narratives describing how the data were generated and how they were used for the design.

(4) Conclusions

The information in this document cannot answer the following aspects of Question 4: (1) seismic safety, (2) performance in a maximum credible earthquake, and (3) meeting lifeline criteria.

San Francisco-Oakland Bay Bridge Retrofit/Replacement Evaluation

Date Document Reviewed by COE: July 2000 Reviewed by: Chung Wong

Document I.D.#: 367 Volume 5 of 41

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(1) Structure

Suspension span

(2) Performance Criteria

The bridge was designed for two levels of earthquake, a functional evaluation earthquake (FEE) and a safety evaluation earthquake (SEE). See section **SEISMIC PERFORMANCE CRITERIA** under Chapter **SEISMIC DESIGN PHILOSOPHY** for more details.

(3) Analysis Methodology

Three forms of seismic analysis were employed: (1) time history analysis of the global model, (2) pushover analysis, and (3) local detailed analysis. See section **ANALYSIS METHODOLOGY** under Chapter **SEISMIC DESIGN PHILOSOPHY** for more details.

(4) Modeling Assumptions

- The bridge deck was modeled as two spines of linear beam elements representing the axial, bending, and torsional behavior of the suspended structure.

- The suspenders were modeled with non-linear truss elements which cannot take compression. The dead load stress in the main cables and suspenders was modeled by means of an initial strain applied to the element. For instance, in the sample ADINA input file, an initial strain was specified which corresponds to a cable force of 200,000 kN.

- Tower shaft was modeled with nonlinear beam elements (moment-curvature data were input into ADINA).

- The shear links between the tower shafts were modeled with inelastic moment-curvature beam elements.

- Each pile in the tower foundation was modeled from the bottom of the pile cap to the pile tip using several nonlinear beam elements supported with nonlinear p-y and t-z springs. The ground motions were applied to each spring.

- The east and west piers were modeled with nonlinear beam elements. The properties of these elements are based on X-section (moment-curvature) analysis.

- The west pier was assumed to be founded on rock and the ground motions were applied directly to the bottom of the pier.

- Each of the east pier piles was modeled from the pile cap to the pile tip using non-linear beam elements supported with nonlinear p-y and t-z springs along its height. The model included t-z dampers to account for viscous damping. Depth varying ground motions were applied to the p-y and t-z springs.

- The first frame of the skyway structure was modeled as a boundary frame.

- A hybrid model was used for the foundations of the skyway piers. The hybrid model consisted of beam elements modeling each pile from the bottom of the pile cap to the mud-line. Below the mud-line, each pile was modeled with a 12-degree-of-freedom stiffness and damping matrices (impedance matrices) which can rigorously model the battering of the piles. Ground motions were applied at the bottom nodes of the pile springs. Note that the ground motion for the hybrid model is not the mud-line motion, but the motion at a firm soil layer below the Young Bay mud.

- Some sketches showing modeling of expansion joint without any narratives. It is impossible to figure out how expansion joint was modeled without any clarification.

- A plot showing Rayleigh damping was used for the suspension bridge with $\alpha = 0.94248$, $\beta = 0.002387$ (see Section 4.4.2.12).

(5) Structural Analysis Input Data and Supporting Calculations for Input Data

- Calculations for dead loads associated with the superstructure of the suspension bridge.

- A typical ADINA input file.

(6) Remarks

This document contains some useful information on seismic performance criteria, analysis methodology, plots of ground motion time histories, and a brief description of the ADINA analytical models (global models). However, the information is not comprehensive enough to provide a clear understanding of the analysis methodology and analytical models. Specifically, no basis was provided for selecting the α and β values for Rayleigh damping. Caltrans and TYLIN had a meeting with us on 13 September 2000 to answer the questions we had from the review of this document and others for the suspension bridge (see Documents 383, 384).

(7) Conclusions

The information in this document cannot answer the following aspects of Question 4: (1) seismic safety, (2) performance in a maximum credible earthquake, and (3) meeting lifeline criteria.

San Francisco-Oakland Bay Bridge Data Review
Replacement Evaluation

Reviewed by: Michael G. Mills **Review Date:** 9/11/00
Discipline: Structural **Document I.D. #** 367, Vol. 6

Answers Question 1, 2, 3, or 4? 4

Description of Data Reviewed: Global Analysis: Summary of Results for Six Ground Motions.

1. Results Summary. Demand / Capacity (D/C) table for cable suspenders, tower, box girder, cross — beams, east and west bents, and piles. D/C exceed 1.. also displacements for tower & bents.

2. Seismic Response to 30% ground motion. Plots and tables show load variances by coordinate position. No explanation or discussion given.

   a. D/C plots for various elements w/o explanation. Exceeds 1.0.
   b. Seismic response to ground motion set #2. Plots and tables for forces, moments and displacements. Structure locations are general.
   c. Seismic response to ground motion set #3. More plots and tables. General locations.
   d. Seismic response to ground motion set #4. More plots and tables. General locations.

Answers what part of Question? Describe. Does not answer question 4. There is no discussion relating to seismic safety and lifeline criteria. The summary table shows many D/C ratios greater than 1.0.

Additional Remarks. Per description #2 above: D/C is plotted for spline on 2 different plots. The only distinction is the date & time of plot, no explanation.
Overall ~ no explanation or reference to distinguish ground motions, only output.

San Francisco-Oakland Bay Bridge Retrofit/Replacement Evaluation

Date Document Reviewed by COE: July 2000 Reviewed by: Chung Wong

Document I.D.#: 367 Volume 6 of 41

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(1) Structure

Suspension span

(2) Structural Analysis Results

- Summary of D/C ratios for different structural elements of the suspension bridge subjected to 6 sets of ground motions for Model S.C.18.
- Suspender forces due to 30% ground motion.
- Spine forces and moments due to 30% ground motion.
- Cable forces due to 30% ground motion.
- Floor beam forces and moments due to 30% ground motion.
- Pylon forces and moments due to 30% ground motion.
- Pier E2 forces and moments due to 30% ground motion.
- Pier W2 forces and moments due to 30% ground motion.
- Forces and moments at the tower foundation and Pier E2 foundation due to 30% ground motion.
- Plots for time histories of nodal displacements, forces and moments at selected locations due to 30% ground motion.
- Pier E2 forces and moments due to ground motion set No. 2.
- Forces and moments at the tower foundation and Pier E2 foundation due to ground motion set No. 2.
- Plots for time histories of nodal displacements, forces and moments at selected locations due to ground motion set No. 2.
- Cable forces due to ground motion set No. 2.

- Suspender forces due to ground motion set No. 2.
- Spine forces and moments due to ground motion set No. 2.
- Floor beam forces and moments due to ground motion set No. 2.
- Pylon forces and moments due to ground motion set No. 2.
- Plots of D/C ratios for spine, floor beam, and pylon for ground motion set No. 2.
- Pier W2 forces and moments due to ground motion set No. 3.
- Similar sets of output (see output described above for ground motion set No.2) due to ground motion set No. 3.
- Pier W2 forces and moments due to ground motion set No. 4.
- Similar sets of output (see output described above for ground motion set No.2) due to ground motion set No. 4.

(3) Remarks

This document contains a lot of output extracted from the ADINA runs. The ground motion sets used in the ADINA time history analyses correspond to SEE (this has been confirmed in the meeting with Caltrans and TYLIN on 13 September 2000). All output data described above pertain to Model S.C.18. These data (plots, tables, and ADINA output) will be useful for design of the suspension bridge. However, there are no narratives describing how the data were used for the design.

(4) Conclusions

The information in this document cannot answer the following aspects of Question 4: (1) seismic safety, (2) performance in a maximum credible earthquake, and (3) meeting lifeline criteria.

San Francisco-Oakland Bay Bridge Retrofit/Replacement Evaluation

Date Document Reviewed by COE: July 2000 Reviewed by: Chung Wong

Document I.D.#: 367 Volume 7 of 41

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(1) Structure

Suspension span

(2) Structural Analysis Results

- Similar sets of output (see output described previously for ground motion set No.2) due to ground motion set No. 5.

- Similar sets of output (see output described previously for ground motion set No.2) due to ground motion set No. 6.

(3) Remarks

This document contains a lot of output extracted from the ADINA runs. The ground motion sets No. 5 and 6 correspond to SEE (this has been confirmed in the meeting with Caltrans and TYLIN on 13 September 2000). All output data described above pertain to Model S.C.18. These data (plots, tables, and ADINA output) will be useful for design of the suspension bridge. However, there are no narratives describing how the data were used for the design.

(4) Conclusions

The information in this document cannot answer the following aspects of Question 4: (1) seismic safety, (2) performance in a maximum credible earthquake, and (3) meeting lifeline criteria.

San Francisco-Oakland Bay Bridge Retrofit/Replacement Evaluation

Date Document Reviewed by COE: July 2000 Reviewed by: Chung Wong

Document I.D.#: 367 Volume 8 of 41

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(1) Structure

Suspension span

(2) Structural Analysis Results

- Summary of D/C ratios for different structural elements of the suspension bridge subjected to ground motion set No. 1 and No. 3 for Model S.C.20WH.

- Plots of P-M for tower piles.

- Suspender forces due to ground motion set No. 1.

- Spine forces and moments due to ground motion set No. 1.

- Cable forces due to ground motion set No. 1.

- Floor beam forces and moments due to ground motion set No. 1.

- Pylon forces and moments due to ground motion set No. 1.

- Pier E2 forces and moments due to ground motion set No. 1.

- Pier W2 forces and moments due to ground motion set No. 1.

- Forces and moments at the tower foundation and Pier E2 foundation due to ground motion set No. 1.

- Plots for time histories of nodal displacements, forces and moments at selected locations due to ground motion set No. 1.

- Similar sets of output (see output described above for ground motion set No. 1) due to ground motion set No. 3.

(3) Remarks

This document contains a lot of output extracted from the ADINA runs. The ground motion sets No. 1 and No. 3 correspond to SEE (this has been confirmed in the meeting with Caltrans and TYLIN on 13 September 2000). All output data described above pertain to Model S.C.20WH.

These data (plots, tables, and ADINA output) will be useful for design of the suspension bridge. However, there are no narratives describing how the data were used for the design.

(4) Conclusions

The information in this document cannot answer the following aspects of Question 4: (1) seismic safety, (2) performance in a maximum credible earthquake, and (3) meeting lifeline criteria.

San Francisco-Oakland Bay Bridge Retrofit/Replacement Evaluation

Date Document Reviewed by COE: July 2000 Reviewed by: Chung Wong

Document I.D.#: 367 Volume 9 of 41

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(1) Structure

Suspension span

(2) Structural Analysis Results

- Summary of D/C ratios for different structural elements of the suspension bridge subjected to 6 sets of ground motions for Model S.C.23.
- Suspender forces due to ground motion set No. 1.
- Spine forces and moments due to ground motion set No. 1.
- Cable forces due to ground motion set No. 1.
- Crossbeam forces and moments due to ground motion set No. 1.
- Pier W2 cap beam forces and moments due to ground motion set No. 1.
- Pylon forces and moments due to ground motion set No. 1.
- Pier E2 forces and moments due to ground motion set No. 1.
- Pier W2 forces and moments due to ground motion set No. 1.
- Forces and moments at the tower foundation, Pier W2 foundation, and Pier E2 foundation due to ground motion set No. 1.
- Plots for time histories of nodal displacements, forces and moments at selected locations due to ground motion set No. 1.
- Similar sets of output (see output described above for ground motion set No. 1) due to ground motion set No. 2.
- Similar sets of output (see output described above for ground motion set No. 1) due to ground motion set No. 3.

(3) Remarks

This document contains a lot of output extracted from the ADINA runs. The ground motion sets No. 1, No. 2, and No. 3 correspond to SEE (this has been confirmed in the meeting with Caltrans and TYLIN on 13 September 2000). All output data described above pertain to Model S.C.23. These data (plots, tables, and ADINA output) will be useful for design of the suspension bridge. However, there are no narratives describing how the data were used for the design.

(4) Conclusions

The information in this document cannot answer the following aspects of Question 4: (1) seismic safety, (2) performance in a maximum credible earthquake, and (3) meeting lifeline criteria.

San Francisco-Oakland Bay Bridge Retrofit/Replacement Evaluation

Date Document Reviewed by COE: July 2000 Reviewed by: Chung Wong

Document I.D.#: 367 Volume 10 of 41

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(1) Structure

Suspension span

(2) Structural Analysis Results

- Similar sets of output (see output described previously for Model S.C.23 subjected to ground motion set No.1) due to ground motion set No. 4.

- Similar sets of output (see output described previously for Model S.C.23 subjected to ground motion set No.1) due to ground motion set No. 5.

- Similar sets of output (see output described previously for Model S.C.23 subjected to ground motion set No.1) due to ground motion set No. 6.

(3) Remarks

This document contains a lot of output extracted from the ADINA runs. The ground motion sets No. 4, No. 5 and No. 6 correspond to SEE (this has been confirmed in the meeting with Caltrans and TYLIN on 13 September 2000). All output data described above pertain to Model S.C.23. These data (plots, tables, and ADINA output) will be useful for design of the suspension bridge. However, there are no narratives describing how the data were used for the design.

(4) Conclusions

The information in this document cannot answer the following aspects of Question 4: (1) seismic safety, (2) performance in a maximum credible earthquake, and (3) meeting lifeline criteria.

San Francisco-Oakland Bay Bridge Data Review
Replacement Evaluation

Reviewed by: Michael G. Mills **Review Date:** 9/04/99
Discipline: Structural **Document I.D. #** 367, Vol 11

Answers Question 1, 2, 3, or 4? 4

Description of Data Reviewed: Global Analysis for construction sequencing.

1. General details presented for erection procedure.
2. Analysis for Construction Sequence of Main Span. Uses ADINA model to simulate various stages of construction and in reverse order from end to beginning. Pictures only.
3. Cable system loads and displacements are shown plotted for various conditions.
4. Deck system loads and displacements are shown plotted for various conditions.
5.

Answers what part of Question? Describe. Does no answer question #4. No mention or demonstration of lifeline criteria and limits. During construction, workers need a safe design, and taxpayers need their investment protected for timely completion.

Additional Remarks. Angle vs. X-Station is plotted for the main cable and the plot appears to show decreasing rates of angle change when increasing rates should be seen. For the north spline deck, shear distribution is shown with a sudden decrease just to the west of the tower and with no explanation; changes at east and west ends are reasonable.

San Francisco-Oakland Bay Bridge Retrofit/Replacement Evaluation

Date Document Reviewed by COE: July 2000 Reviewed by: Chung Wong

Document I.D.#: 367 Volume 11 of 41

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(1) Structure

Suspension span

(2) Structural Analysis Results

- Selected analysis results for the cable and deck system based on construction sequence analyses using ADINA.

(3) Remarks

This document contains a description of construction sequence for the main span and does not provide a basis for addressing the seismic safety of the suspension bridge.

(4) Conclusions

The information in this document cannot answer the following aspects of Question 4: (1) seismic safety, (2) performance in a maximum credible earthquake, and (3) meeting lifeline criteria.

San Francisco-Oakland Bay Bridge Retrofit/Replacement Evaluation

Date Document Reviewed by COE: July 2000 Reviewed by: Chung Wong

Document I.D.#: 367 Volume 12 of 41

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(1) Structure

Suspension span

(2) Analysis Methodology

- Discussion of aerodynamic analysis is included.

(3) Structural Analysis Results

- Some aerodynamic analysis results are presented.

(4) Remarks

This document only pertains to aerodynamic analysis and does not provide a basis for addressing the seismic safety of the suspension bridge.

(5) Conclusions

The information in this document cannot answer the following aspects of Question 4: (1) seismic safety, (2) performance in a maximum credible earthquake, and (3) meeting lifeline criteria.

San Francisco-Oakland Bay Bridge Data Review
Replacement Evaluation

Reviewed by: Michael G. Mills **Review Date:** 10/03/00
Discipline: Structural **Document I.D. #** 367, Vol 13

Answers Question 1, 2, 3, or 4? 4

Description of Data Reviewed: Cable and Suspension Structural System.

5.1 Suspender Design. Table for cable property presented, in Japanese. Plots show suspender forces, maximum and minimum, and for Suspension Bridge model-a(s.c. 18.in); no other explanation.

5.1.1 Ropes. Suspender loads are stated, no references, for dead, live, lane, and sidewalk and are dated 3/99. Live and lane are illedgible. Seimic force is stated at 5000kN, no reference. Breaking strength is stated, no reference. Rope size is noted at 64mm and 70mm. Seismic demand is computed at 45% of breaking strength. Then for 7/99, suspender design is restated. Breaking strength is shown for 71, 77.5 and 80mm rope.

5.12 Suspender Assemblies. Socket is designed for 90mm rope, with a breaking strength at 6030kN and is stated against a required load of 4450kN. (the preceding section requires 5000kN for seismic.) Socket is then designed for 75mm rope, with a breaking strength of 3370kN, and finally computes a required 80mm size. (No reconcilliation of strength with previously stated loads.) Clearances and vibrations are checked.

5.2 Suspender Geometry and Clearances.

5.3 Cable Band Design. Suspender load is stated at 10MN, no reference. Friction force for band is computed in table for each suspender location. Uses allowable stress, but does not note is load is factored or service.

- 5.3.1 Cable Bands. Layout and clearances for many locations, with detail sketches.
- 5.3.2 Stanchions.
- 5.3.3 Handropes.

5.4 Suspender Anchorages and Cable Brackets (at ends of floorbeams.) Design is based suspender breaking strength of 11,120kN and dated 3/22. Design is then shown for bracket, date 7/99, and is based on 10MN breaking strength. Design continues by computing required plate thicknesses and weld sizes. These thickness and welds are different than those shown on the 65% drawings (Doc. 277). For shear strength, the web buckling coef., C, has not been included. This ommission may increase plate thicknesses. (Design calcs should be reconciled to drawing details.

5.5 Cable Design.

- 5.5.1 Cable Geometry and Clearances. Main cable profile is computed based on dead load of deck.
- 5.5.2 Cable Structural Design. Area of cable is computed based not on load, but on a count of 37 strands of 500 wires, etc.
- 5.5.3 Cable Anchorage Details. Shows sketches of clearances and profiles.

Answers what part of Question? Describe. Does not answer question 4. Calcs refer to a seismic load of 5000kN as a seismic load. Given enough time, this could be back traced to identify if it is a SEE load.

Additional Remarks. These calcs should be redone at a future date to verify completed drawing details. Designer would be better able to defend his design if he would note references and sources.

San Francisco-Oakland Bay Bridge Retrofit/Replacement Evaluation

Date Document Reviewed by COE: July 2000 Reviewed by: Chung Wong

Document I.D.#: 367 Volume 12 of 41

(1) Structure

Suspension span

(2) Structural Analysis Results and Design Calculations

- Cable and suspender design check for seismic loads.

- Suspender sockets and anchor rods were designed for breaking strength of rope.

- Design calculations for cable band — design for 10 MN suspender force (it is not clear where this force came from).

- Design calculations for stanchion — design for live load plus dead load.

- Suspender brackets were designed for breaking strength of rope.

- Cable brackets were designed for 10MN suspender force.

(3) Remarks

This document contains design calculations for the cable and suspension anchorages. The design considered seismic loads for most structural elements and it appears that the design criteria basically follow ATC-32 and Caltrans BDS. However, the sources of some of the design loads shown in the design calculations cannot be verified because of lack of cross-referencing and narratives describing the design philosophy.

(4) Conclusions

The information in this document cannot answer the following aspects of Question 4: (1) seismic safety, (2) performance in a maximum credible earthquake, and (3) meeting lifeline criteria.

San Francisco-Oakland Bay Bridge Data Review
Replacement Evaluation

Reviewed by: Michael G. Mills **Review Date:** 9/25/00

Discipline: Structural **Document I.D. #** 367, Vol. 14

Answers Question 1, 2, 3, or 4? 4

Description of Data Reviewed: Deck Structural System

6.1 Box Girder Design Criteria ~ ATC 32-10.63, BDS-10.xx, AASHTO LRFD 6.xx

a. Compactness
b. Strength Design ~ Per Fy, Fcr, uses time history for real seimic stress. Connections – (phi) R_n and $1.1F_y$.
c. General notes by Weidlinger.

6.2 Global Shell Model. – (Procedure is incomplete.)

a. General purpose models described for D|0, D|1
b. Boundary Conditions
c. Nodal Geometry: deck, cable profile.
d. Member Properties: Summary and explanation. (Copies are only partial.) for Deck mbrs (pages & Types duplicated).
e.

Answers what part of Question? Describe. Begins to answer Question 4. If seismic safety is based on design codes, BDS, AASHTO and ATC, then design is being completed per code and is seismically safe. Strength needs to be compared against ultimate loads. Lots of model information including geometry, member indices, and limited loading cases. Output plots are presented for Von Mises stresses but lack interpretation.

Additional Remarks. Boundary conditions for deck show supports which differ from design. Hand calcs for moment of inertia do not adjust for axis shift and are high.

Stress plots are copied in black and white from color prints are are thus illedgible.

San Francisco-Oakland Bay Bridge Retrofit/Replacement Evaluation

Date Document Reviewed by COE: August 2000 Reviewed by: Chung Wong

Document I.D.#: 367 Volumes 14, 15, 16 of 41

(1) Structure

Suspension span

(2) Design Criteria

¥ Design criteria for main span box girders and crossbeams in compliance with applicable provisions in Caltrans BDS, ATC-32, AASHTO LRFD, AASHTO service load design.

¥ For strength design, all combinations of peak seismic stresses as determined by the time history analysis are considered.

¥ Connections will be designed such that the design capacity, ϕR_n, will be greater than the strength of the connected members, computed assuming a yield strength of $1.1F_y$.

¥ A correspondence from Weidlinger Associates Inc. indicates that the 10m wide crossbeam was over-designed for seismic demands.

(3) Modeling Assumptions

¥ This document shows a global model composed of beam and plate elements for the superstructure. The model was developed for ANSYS. There is no discussion of modeling assumptions and how the analysis results were used for design.

¥ Several local submodels (Levels 2 and 3), created using ALGOR in conjunction with a suite of in-house Weidlinger software, were developed for different localized regions of the deck structural system. But there is no discussion of how the analysis results were used for design.

(4) Structural Analysis Input Data and Supporting Calculations for Input Data

There is no input data associated with the FEM. However, nodal point coordinates and element connectivity were generated by spreadsheets. Mathcad calculations for computing the orthotropic deck properties were found along with some hand calculations for other member properties.

(5) Structural Analysis Results and Design Calculations

¥ Stress contour plots in the deck structural system from ANSYS analyses.

¥ Design calculations for floor beams, floor beam connection details and bracing members.

San Francisco-Oakland Bay Bridge Data Review
Replacement Evaluation

Reviewed by: Michael Premo **Review Date:** 9/26/2000
Discipline: Structural **Document I.D. #** 367 Vol 15

Answers Question 1, 2, 3, or 4? 4

Description of Data Reviewed: Volume 15, Suspension Span Design Calculations – Deck Structural System Calculations for the following structural elements were provided: Floor beams, splices, shear frames, and box sections. A detailed analysis of the structural elements is presented. The various elements are analyzed for appropriate forces and reactions to include shear, moment, axial, tensile, and compressive. Various elements are also analyzed for deflections and compactness. Seismic design criteria were included in the of the various elements as well. Examples of this include the following:

1. For the floor beam design, the basis for the no. of fasteners included a bearing type connection using Group VII (seismic) loads.
2. The deck box girder design incorporated the following seismic design methods:
    a. Taking the seismic moment envelopes and combining plus and minus values of the DL + EQ moment with and without 20% live load. This demand was designedby BDS capacity, allowing 15% overstress.
    b. Taking the seimic moments and combining plus or minus the vertical axis moment, the negative value of the DL+EQ moment (downward swing), with and without 20% live load. This demand was designed by LRFD capacity with no overstress. This was done to estimate the likely combination of stress.
3. The crossbeam size was based on seismic demand.

It was also evident from the print outs that the various computer program runs included the Group VII (seismic) loadings.

Answers what part of Question? Describe.

The volume answers “Is the currently proposed replacement alternative seismically safe?” The design incorporated seismic design criteria from CalTrans BDS and AASHTO.

The volume does not address the remaining parts of Question #4:

How will the currently proposed replacement alternative perform in a maximum credible earthquake?
Specifically, does the currently proposed replacement alternative meet lifeline criteria?
To what extent and how quickly could it accommodate passenger vehicles?

For each of the above items, no presentation was provided.

Although there were seismic design parameters applied during the design, there was no evidence of a comprehensive application of the “lifeline” criteria. The volume does not address provisions for “lifeline” requirements such as almost immediate full service and extent of repairable damage expected.

San Francisco-Oakland Bay Bridge Data Review
Replacement Evaluation

Reviewed by: Michael Premo **Review Date:** 9/26/2000
Discipline: Structural **Document I.D. #** 367, Vol 16

Answers Question 1, 2, 3, or 4? 4

Description of Data Reviewed: Volume 16, Suspension Span Design Calculations – Deck Structural System This volume includes calculations for the following structural elements: Crossbeams, Box sections, Connections, and orthotropic deck. The various elements are analyzed for appropriate forces and reactions to include shear, moment, axial, tensile, and compressive. Seismic design criteria were included in the of the various elements as well. It was evident that the only the anchor plates and crossbeams calculations/computer printouts included the Group VII (seismic) loading.

Answers what part of Question? Describe.
The volume answers “Is the currently proposed replacement alternative seismically safe?” The design incorporated seismic design criteria from CalTrans BDS and AASHTO for some elements.

The volume does not address the remaining parts of Question #4:

How will the currently proposed replacement alternative perform in a maximum credible earthquake?
Specifically, does the currently proposed replacement alternative meet lifeline criteria?
To what extent and how quickly could it accommodate passenger vehicles?

for each of the above items, no presentation was provided.

Although there were seismic design parameters applied during the design, there was no evidence of a comprehensive application of the “lifeline” criteria. The volume does not address provisions for “lifeline” requirements such as almost immediate full service and extent of repairable damage expected.

Additional Remarks.

San Francisco-Oakland Bay Bridge Data Review
Replacement Evaluation

Reviewed by: Michael Premo **Review Date:** 9/26/2000
Discipline: Structural **Document I.D. #** 367, Vol 17

Answers Question 1, 2, 3, or 4? 4

Description of Data Reviewed: Volume 17, Suspension Span Design Calculations – Deck Structural System This volume includes calculations for the following structural elements: Drainage Details, Steel Barrier computer Analysis, Lighting Supports, and Provisions for Future Light Rail. It was evident that the only the future light rail calculations included the Group VII (seismic) loading.

Answers what part of Question? Describe.
The volume answers "Is the currently proposed replacement alternative seismically safe?" The design incorporated seismic design criteria from CalTrans BDS and AASHTO for some elements.

The volume does not address the remaining parts of Question #4:

How will the currently proposed replacement alternative perform in a maximum credible earthquake?
Specifically, does the currently proposed replacement alternative meet lifeline criteria?
To what extent and how quickly could it accommodate passenger vehicles?

for each of the above items, no presentation was provided.

Although there were seismic design parameters applied during the design, there was no evidence of a comprehensive application of the "lifeline" criteria. The volume does not address provisions for "lifeline" requirements such as almost immediate full service and extent of repairable damage expected.

Additional Remarks.
A seismic analysis for items such as drainage details, steel barrier, and lighting supports is not required.

San Francisco-Oakland Bay Bridge Retrofit/Replacement Evaluation

Date Document Reviewed by COE: August 2000 Reviewed by: Chung Wong

Document I.D.#: 367 Volumes 17 of 41

(1) Structure

Suspension span

(2) Design Criteria

- Design criteria for steel barrier — Caltrans BDS Section 2.

(3) Structural Analysis Input Data and Supporting Calculations for Input Data

- SAP90 input file for the steel barrier composed of shell elements.

(4) Structural Analysis Results and Design Calculations

- SAP90 output file and stress contour plots.

(5) Remarks

This document contains analysis and design for such miscellaneous details as drainage, steel barriers, and lighting supports. Information presented in the document does not address the seismic safety of the suspension bridge.

(4) Conclusions

The information in this document cannot answer the following aspects of Question 4: (1) seismic safety, (2) performance in a maximum credible earthquake, and (3) meeting lifeline criteria.

San Francisco-Oakland Bay Bridge Data Review
Replacement Evaluation

Reviewed by: Michael Premo **Review Date:** 9/27/2000
Discipline: Structural **Document I.D. #** 367, Vol 18

Answers Question 1, 2, 3, or 4? 4

Description of Data Reviewed: Volume 18, Suspension Span Design Calculations – Deck Structural System This volume includes the following: Brief of Bike Path Design, Bike Path structural calculations, deck plate capacity evaluation, stringer loading evaluation, rib beam and stringer depth verification, and computer analyses for cantilever rib beams, stringers, and deck plates. It was not evident that any of the areas analyzed included the Group VII (seismic) loading.

Answers what part of Question? Describe.

The volume does not address any parts of Question #4 shown below:

Is the currently proposed replacement alternative seismically safe?
How will the currently proposed replacement alternative perform in a maximum credible earthquake?
Specifically, does the currently proposed replacement alternative meet lifeline criteria?
To what extent and how quickly could it accommodate passenger vehicles?

For each of the above items, no presentation was provided.

Additional Remarks.

San Francisco-Oakland Bay Bridge Retrofit/Replacement Evaluation

Date Document Reviewed by COE: August 2000 Reviewed by: Chung Wong

Document I.D.#: 367 Volumes 18 of 41

(1) Structure

Suspension span

(2) Structural Analysis Input Data and Supporting Calculations for Input Data

SAP90 input files for analysis of bikepath rib beams, stringers, and deck cover plates.

(3) Structural Analysis Results and Design Calculations

- SAP90 output files for analysis of bikepath rib beams, stringers, and deck cover plates.
- Design calculations for bikepath rib beams, stringers, and deck cover plates.

(4) Remarks

This document contains analysis and design calculations for elements of the bikepath. Information presented in the document does not address the seismic safety of the suspension bridge.

(5) Conclusions

The information in this document cannot answer the following aspects of Question 4: (1) seismic safety, (2) performance in a maximum credible earthquake, and (3) meeting lifeline criteria.

San Francisco-Oakland Bay Bridge Data Review
Replacement Evaluation

Reviewed by: Michael G. Mills **Review Date:** 8/18/00

Discipline: Structural **Document I.D. #** 367, Vol 19

Answers Question 1, 2, 3, or 4? 4 & 3

Description of Data Reviewed: This volume, #19, presents the tower shaft for the Suspension Span of the Bridge. It includes:
a) Geometry Calculations,
b) Design Loads and Estimates for member sizes.
  i) Demand Capcity Calcs w/ curves and
  ii) Axial Force due to dx, Max as a type of pushover,
c) Open Section Analysis to compute Torsional stresses. Analysis uses unit forces. Shear demand is computed at various heights and due to V_S, V_t, M_r, and Torsion. "s" and "t" are local axes. An open section analogy is used to determine a worst case shear stress. Uses unit force of 100KN. Reviewer assumes that force is to be adjusted at a future date.
d) Tower shaft design. Design of stiffened Box Column for Seismic Ductility per California BDS, Section 10.
e) Connection Design. Designed for 125% F_y. Determines # of bolts, spacings, etc. for splice between upper and lower shaft. Welds are sized. Stiffeners are sized. Clearances are checked.
f) Tower Shaft Deisgn, typical sections. Computes "Seismic Ductility of Stiffened Box Columns".
g) Tower Shaft Design, Connection Plates (8.1.2.2), Seismic Demands P-M & P-phi Diagrams (8.1.3) are not available.
h) NE and SW Tower Shafts. Commentary on the calculated seismic D/C ratios of the tower shafts. Interaction diagrams are presented for the NE and SW tower shafts at various elevations and for ground motion sets 1 & 3. Evaluations show overstress at local points and near corners. Otherwise, Ductility is 1 or less.

Answers what part of Question? Describe. For Question 4, the tower is being designed per Caltrans BDS code and AWS. Analysis includes loads resulting in axial, bending and torsional forces. Stresses remain in the elastic range or within a small percentage. Work is in progress to fine tune design.

BDS – Bridge Design Specification
AWS – American Welding Society

Additional Remarks. Connection Design needs explanation due to design for forces equal to 125% F_y. Designer should exlain to COE team. Section 8.1.2.2, Connection Plates, is not available. A FEM analysis is in progress and has been introduced.

San Francisco-Oakland Bay Bridge Retrofit/Replacement Evaluation

Date Document Reviewed by COE: September 2000 Reviewed by: Chung Wong

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(1) Structure

Suspension span

(2) Design Criteria

¥ Caltrans BDS, supplemented by AASHTO, ATC-32, and AISC LRFD.

¥ In particular, the design criteria for bolted splice of the main span tower legs are provided.

¥ The design loads or demands are based on time history analyses subjected to six sets of ground motions (push-over analysis was performed to check the main tower capacity and failure mode sequence).

¥ Buckling of tower shafts (considered as stiffened box columns) was considered.

(3) Structural Analysis Results and Design Calculations

¥ D/C ratios for pylon for earthquake sets No. 1, No. 3 (Note that D/C ratio for pylon for earthquake set No. 1 exceeds 1 for some locations).

¥ Plots of pylon axial force vs. elevation for different conditions.

¥ Shear D/C ratios for tower shafts (less than 1).

¥ Torsional stress D/C ratios for main tower (less than 1).

¥ Design calculations for bolted splices/connections of tower leg skin plates and longitudinal stiffeners (block shear rupture in the skin plate was checked).

¥ Seismic ductility of stiffened box columns was checked in accordance with ATC-32.

¥ Plots of interaction diagrams for axial force-curvature and axial force-moment against the corresponding seismic demands. These plots were developed at several elevations for the NE and SW tower shafts, for ground motions sets No. 1 and No. 3. The capacity plots correspond to a ductility level = 1.

(4) Remarks

This document showed that relatively minor inelastic behavior was observed between E1. 85.0 and E1. 95.0 where D/C ratios were registered close to 1.2 for = 1. It also indicated that the demands were below the corresponding capacity curves for   = 4, the maximum allowable ductility capacity adopted for this project. This observation seems to contradict with Caltrans design assumptions that the tower shafts remain elastic and undamaged after a major earthquake.

The tower shaft cross section area has been increased (primarily between EL. 50 and EL. 120) in design beyond the 65% submittal. The improved design produced highest D/C ratios near the shaft s base slightly exceeding one (See Appendix C of Document 384). This is acceptable because the D/C ratios are calculated conservatively. Ther easons for conservatism are explained in Document 384.

(5) Conclusions

Assuming the ADINA analyses produce reasonable seismic demands, it can be concluded from this document that the tower shafts remain essentially elastic after SEE. The essentially elastic performance of the tower shafts can then be used as a basis for answering the following aspects of Question 4: (1) seismic safety, (2) performance in a maximum credible earthquake, and (3) meeting lifeline criteria.

San Francisco-Oakland Bay Bridge Retrofit/Replacement Evaluation

Date Document Reviewed by COE: July 2000 Reviewed by: Chung Wong

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(1) Structure

Suspension span

(2) Performance Criteria

Paragraph 8.1 (page 23) provides performance criteria for two levels of earthquake, i.e. Functional Evaluation Earthquake (FEE) and Safety Evaluation Earthquake (SEE). After a FEE event, the bridge will provide full service almost immediately and will sustain only minimal damage. Minimal damage implies essentially elastic performance and is characterized by:

¥ Minor inelastic response.

¥ Narrow cracking in concrete.

¥ No apparent permanent deformations.

¥ Minor damage to expansion joints that does not affect serviceability of the bridge.

After a SEE event, the bridge will provide full service almost immediately and will only sustain repairable damage. Repairable damage can be repaired with a minimum risk of:

¥ Minimal damage to the superstructure.

¥ Limited damage to piers, including yielding of reinforcement and spalling of concrete cover.

¥ Minimal damage to piles and pilecaps

¥ Small permanent deformations, not interfering with serviceability of the bridge.

¥ Damage to expansion joints.

¥ Other damage not requiring closure for repair.

(3) Design Criteria

¥ The bridge shall be designed in accordance with Caltrans Bridge Design Specifications Manual BDS (1995), modified or augmented by pertinent sections of standards or codes as specified in this document.

San Francisco-Oakland Bay Bridge Data Review
Replacement Evaluation

Reviewed by: Michael G. Mills **Review Date:** 8/23/00
Discipline: Structural **Document I.D. #** 367, Vol 20

Answers Question 1, 2, 3, or 4? 4

Description of Data Reviewed: Vol. 20, Main Tower Shear Links.

1. Geometric Properties. Sizes appear to be assumed and not related to rational design.
2. Connection Concepts are presented using AISC –LRFD criteria for design of links and their bolted connections. Where appropriate, lower AASHTO limits are imposed on criteria.
3. Recommendations are given by Consultant Krawinkler, PhD, PE
4. Design of Shear Links by proportionment from assumed sizes and based on the yield strength, F_y.
   a. Given an assumed size, the plastic shear strength, V_p, is computed per LRFD seismic. And factors of 1.1 and 1.25 are applied. This shear strength is then used for other componenets associated with the links.
   b. Connection design determines # of bolts and bolt sizes for splices & connections. Note: Design calcs call for A325 & A490. Drwgs call for A325.
   c. Numerous shear link designs are presented, none of which are consistent with 65% drawings, Doc 277. Notably, design calls for A490X bolts while drawings show A325; Design – 55 yield steel, drawings – 70 yield steel.
5. Design loads, displacements and rotations are presented in sectio 8.2.1 and are from ADINA. No correlation with member strength is presented or evident.
6. A parametric study is presented which compares stiffness of tower as diaphragms and links are added. Intent is to determine behaviors under monotonic and hysteretic loading of plastic shear links. Accuracy of models is presented.
7. A testing program is recommended which will verify the performance of the shear links.

Answers what part of Question? Describe. This volume begins to answer the seismic performance of the tower for the bridge. It does not compare the member strength to the ultimate forces on the shear links; lacking this comparison seismic safety cannot be stated.

Additional Remarks. No information is presented to describe replacement of shear links after permanent displacement of tower occurs. (At 8.1.3.1, D/C ratios of 1.20 are noted.)

San Francisco-Oakland Bay Bridge Retrofit/Replacement Evaluation

Date Document Reviewed by COE: September 2000 Reviewed by: Chung Wong

Document I.D.#: 367 Volumes 20 of 41

(1) Structure

Suspension span

(2) Performance Criteria

Performance criteria for shear links are available in the correspondence between Dr. Helmut Krawinkler and Mr. Chuck Seim dated 8 April 1998.

(3) Design Criteria

¥ Some specific design criteria were presented in the correspondence between Dr. Helmut Krawinkler and Mr. Chuck Seim dated 8 April 1998.

(4) Structural Analysis Results and Design Calculations

¥ Under the Design Loads section, there are a lot of plots showing the design loads for the tower shafts and shear links (tower struts) based on ADINA analysis.

¥ Design calculations checking the capacity of shear links for Strut Types 1, 2, and 3.

¥ Design calculations for stiffener plate weld connections to shear links for Strut Types 1, 2, and 3.

¥ Design calculations for end splice bolt connections for shear links for Strut Types 1, 2, and 3.

¥ SAP2000 analyses were performed to evaluate the effects of diaphragm / shear link connection flexibility on the overall behavior of the tower.

¥ ADINA finite element analyses were performed for Strut Types 1, 2, and 3 to evaluate the behavior of each strut type as an elastic beam and as a plastified shear link under monotonic loading. The struts were modeled as shell elements.

¥ Calculations for the shear link alternative connection design which was not used for the 65% plans.

(5) Remarks

This document includes the performance criteria for the shear links. One of the criteria is to be able to replace the shear links fast after a major earthquake. However, Caltrans failed to demonstrate that it is feasible to easily and quickly replace the shear links after they yield in shear with possible plastification in the web. SAP2000 analyses were performed to validate the rigidity assumptions associated with the tower/strut connections in the global nonlinear ADINA tower models. ADINA finite element analyses were or will be performed for Strut Types 1, 2, and 3 to accomplish the following three tasks:

Task 1. To determine the behavior of each strut type as an elastic beam or as a plastified shear link under monotonic loading.

Task 2. To determine the effect of tower diaphragm stiffness and tower shaft stiffness on the above behavior of each strut type.

Task 3. To determine the behavior of each strut type under hysteretic loading.

Task 1 was completed and had led to useful conclusions that also validate certain modeling assumptions for the global tower model. Works on Task 2 and Task 3 were in progress at the time the 65% design calculations were submitted.

Two testing programs were proposed for the shear links. The first testing program was proposed to Caltrans by Professor Ahmad Itani, and the second one by Professor Frieder Seible & Professor Nigel Priestley. The first program was mainly used to determine the cyclic behavior overstrength ratio, plastic rotation, behavior of the bolted connection at the ends of the shear link, and the failure modes. The second program was intended to validate the connection details between the shear links and the tower, evaluate deformation and shear capacity of the links, and to evaluate the damage of the connection details at various levels of earthquake intensity. Results of the two testing programs are not available as part of the 65% submittal.

There are no narratives or meaningful descriptions on the numerous plots showing the design loads for the tower shafts and shear links based on ADINA analysis. It is not clear what load cases the design loads correspond to and how the design loads were used for the design.

(6) Conclusions

The information in this document cannot answer the following aspects of Question 4: (1) seismic safety, (2) performance in a maximum credible earthquake, and (3) meeting lifeline criteria.

San Francisco-Oakland Bay Bridge Data Review
Replacement Evaluation

Reviewed by: Conrad Bridges / Mills **Review Date:** 9/04/00
Discipline: Structural **Document I.D. #** 367, Vol 21

Answers Question 1, 2, 3, or 4? 4

Description of Data Reviewed: Section 8.3 (Vol. 21) covers design of suspension tower cross-bracing, diaphragms, tower base, and tower head. Specifics include:

1. Design Loads – currently blank (8.3.1)
2. Cross Bracing Design – based on “seismic demands” which are listed for tension and compression, but no reference to source for these demands. A built-up section is designed for the bracing using these seismic demands. Capacity/demand ratios are calculated that indicate ratios of 2.55 and more. Capacities are based on reasonable strengths and overstrength factors. Critical buckling stress is calculated, but not compared to the demands. (Math Cad Error, Cap/Dmnd Ratio should be twice the amount shown.
3. Diaphragm design loads are currently bland (8.4.1)
4. Stiffeners added to box columns (which is an ambiguous description and may refer to diaphragms per BDS or similar).
5. Diaphragms designed. Based on assumed sizes for strength design; No comparison to ultimate loads.
6. Diaphragms are not presented for locations above and Below Elev. 89m, at 13m, and at 8.5m (8.4.3, 8.4.5, 8.4.6). At 89m elev., diaphragm is designed using Fy, average skin plate t, and overstress factor of 1.1.
7. Tower Base. Design loads are not available for this member design.
8. Tower Head Grillage. DL is assumed to be 500MN and other loads are proportioned. Analysis summary presented. Design per BDS. Note 100x stiffness for Top Plate.
   a. Simplified Grillage. Design based on assumed DL (500MN) and fully plastic tower section. No comparison to ultimate load.
   b. Analysis by FEM ~ ANSYS. Forced with moment due to 1.25 x Plastic Capacity.

9\. Tower Saddle Design. Design loads are from ADINA analysis. Strength design used to size members. Design per AASHTO LRFD. Checks cable friction in saddle. Forces, geometry and angles based on given coordinates, no backup.

Answers what part of Question? For question 4, the cross bracing is designed per Caltrans BDS and AASHTO. Seismic demands are not referenced to any analysis, so it is unknown whether the design meets SEE criteria.
Other member strength presented but not compared to ultimate load. Anchor bolts per AISC, BDS, and AASHTO codes. Tower-head grill strength is presented but not compared, and constructibility is undetermined. Saddle loads and member strength presented per code, no lifeline statement.

AISC – American Institute
BDS – Bridge Design Specifications
AASHTO – American Association of State Highway and Transportation Officials
LRFD – Load and Resistance Factor Design
FEM – Finite Element Model

Additional Remarks. Design has not been independently checked, and may be revised. Strength based on Fy. Anchor bolt design assumes a lesser wall thickness (t) than diaphragm design at 89m. Tower base plate assumes another plate t, for design. Tower saddle shows a flange plate on outside of vertical ribs but no means of fastening is presented. Angles for cable at saddles presented but no backup to basis.

San Francisco-Oakland Bay Bridge Data Review
Replacement Evaluation

Reviewed by: **D. Gray/A. Pujol** **Review**
Date: 10/11/00
Discipline: **Geotechnical** **Document I.D.** # 367, V. 33

Answers Question 1, 2, 3, or 4? 4

Description of Data Reviewed: Describes west pier foundation design approach. Foundation design was in progress as of the date of the document.

It is stated that the pile lengths are governed by axial tension resulting from global overturning moments.

Presents geotechnical data for borings 98-14 and 98-15.

Includes Fugro-EMI document detailing spring models consisting of p-y, t-z, and q-u curves for piles embedded in rock. Spring parameters are developed by calibrating to results obtained from finite element method and beam-column analyses. Elastic properties and shear strength parameters for these analyses were factored from laboratory and borehole test results to attempt to account for weathering and the effect of discontinuities on overall rock mass parameters. Bond strength is selected based on pile load tests for the Benicia-Martinez bridge and published data included in this volume. Characterization methods and results appear reasonable. However, load tests would seem warranted in order to confirm parameters for rock socketed piles.

Design load tables from the ADINA structural analysis are included, from which pile axial loads, moments and shears are apparently taken. Tables lack a meaningful description or discussion. It is assumed that these loads are used for the structural analysis of the piles.

Answers what part of Question? Describe. Begins to address Question 4 in that it appears that the foundation is being designed for the seismic loads obtained from the global model.

Additional Remarks. Calculations are work in progress, unsigned and unchecked. We assume that they will be completed, finalized and a thorough independent check provided.

San Francisco-Oakland Bay Bridge Data Review
Replacement Evaluation

Reviewed by: **D. Gray/A. Pujol** **Review Date:** 10/11/00
Discipline: **Geotechnical** **Document I.D.** # 367, V. 34

Answers Question 1, 2, 3, or 4? 4

Description of Data Reviewed: Describes main tower foundation design approach. Limited damage is characterized as minor inelastic response and small permanent deformation so that repair of the foundation will not be necessary. The limited damage criteria will be considered to have been met if the maximum concrete compressive strain does not exceed 0.01 for steel-cased piles and 0.004 for uncased piles, and if the tensile strains in steel pile shells and reinforcing bars do not exceed 0.02. Axial pile capacity and pile strains are checked using the results of the ADINA inelastic time history analysis. Supplemental pushover analyses are performed to determine the displacement capacity of the pile group based on the limiting strains. This capacity is compared to the displacement demands from the non-linear time history analysis.

Foundation design and analysis was in progress as of the date of the document. The document is taken as a snapshot of the design process. See hand written memo by TY Lin dated May 27, 99 in Section 12.2.2.2 of this volume.

Presents various structural analyses comparing load distributions for concrete and steel pile caps.

Presents cross-sectional analyses for corroded and uncorroded piles.

Includes Fugro-EMI document detailing spring models consisting of p-y, t-z, and q-u curves for piles embedded in rock. Spring parameters are developed by calibrating to results obtained from finite element method and beam-column analyses. Elastic properties and shear strength parameters for these analyses were factored from laboratory and borehole test results to attempt to account for weathering and the effect of discontinuities on overall rock mass parameters. Bond strength is selected based on pile load tests for the Benicia-Martinez bridge and published data included in this volume. Characterization methods and results appear reasonable. However, load tests would seem warranted in order to confirm parameters for rock socketed piles.

Design load tables from the ADINA non-linear time history analysis are included, from which pile axial loads, moments and shears are apparently taken and compared to the cross-sectional capacities. Tables and plots lack a meaningful description or discussion. It is assumed that these loads are used to verify the structural design of the piles. Tower pile cap displacements resulting in limiting strains are obtained from the pushover analysis and compared to the displacements computed using the time-history analysis.

Answers what part of Question? Describe. Begins to address Question 4 in that the foundation is being designed for the seismic loads obtained from the global model. In addition maximum strain criteria have been given to define limited damage; the maximum seismic loads and displacements are being checked against the static loads and associated displacements that would result in the limiting strains.

Additional Remarks. Calculations are work in progress, unsigned and unchecked. We assume that they will be completed, finalized and a thorough independent check provided.

San Francisco-Oakland Bay Bridge Data Review
Replacement Evaluation

Reviewed by: **D. Gray/A. Pujol** **Review Date:** 10/11/00
Discipline: **Geotechnical** **Doc. I.D. #** 367, V. 36 & 37

Answers Question 1, 2, 3, or 4? 4

Description of Data Reviewed: Describes E2 pier foundation design approach. Limited damage is characterized as minor inelastic response and small permanent deformation so that repair of the foundation will not be necessary. The limited damage criteria will be considered to have been met if the maximum concrete compressive strain does not exceed 0.01 for steel-cased piles and 0.004 for uncased piles, and if the tensile strains in steel pile shells and reinforcing bars do not exceed 0.02. Axial pile capacity and pile strains are checked using the results of the ADINA inelastic dynamic (time history) analysis. Supplemental pushover analyses are performed to determine the displacement capacity of the pile group based on the limiting strains. This capacity is compared to the displacement demands from the non-linear time history analysis.

Foundation analysis was in progress as of the date of the document. The document is taken as a snapshot of the design process.

Presents cross-sectional analyses for corroded and uncorroded piles. These are used to develop Axial Load vs Moment curves and oment-curvature curves for the pile sections.

Includes Fugro-EMI documents presenting preliminary pile axial capacity and lateral capacity evaluations. The axial capacity evaluation is an early version of the Draft Axial Pile and Drivability Report (see review of document 332). The lateral capacity evaluation is a preliminary memorandum that was subsequently superseded by Fugro-EMI's Draft Lateral Pile Design Report (see review of document 385). Characterization methods and results appear reasonable. However, load tests would seem warranted in order to confirm parameters for these piles, particularly given the dissimilarity between foundation types for the main span (foundations on rock for piers P1 and W2 but on soil for pier E2).

Pile cap displacements resulting in maximum allowable curvature are obtained from the pushover analysis and compared to the displacement capacity exceeds the displacement demand. It was noted that the acceleration response spectra associated with the ground motions used in the non-linear time-history analyses have a valley in the period range of the E2 foundation. If this valley does not exist in the actual ground motion, the shaking could be greater than calculated. To evaluate this possibility, the structure's seismic response also was checked using ground motion spectra for soil profiles C and D, as these have longer displacement demands at period longer than 3 seconds and may impart more severe displacements to this pier than the response spectra for profiles A and B. The computation summary also shows that displacement capacity from pushover analysis remains greater than demand from ground motion spectra for profiles C and D.

Seismic design loads from the ADINA non-linear time history analysis are included, from which pile axial, moment and shear demands are apparently taken and compared to the cross-sectional capacities.

Finally, the permanent set at the E2 foundation is summarized. The vertical settlement from the design seismic event is computed to be 0.2 inches. Without the benefit of a detailed check of the model, we feel that this settlement is at least an order of magnitude too small. The source of the error, if there is one, might be traced to how the the axial load-displacement relationships are introduced i the model. We recommend that the model and vertical settlement evaluations be thoroughly checked.
Answers what part of Question? Describe. Begins to address Question 4 in that the foundation is being designed for the seismic loads obtained from the global model. In addition, maximum strain criteria have been given to define limited damage; the maximum seismic loads and displacements are being checked against the static load and associated displacement capacities that would result in the limiting strains.

Additional Remarks. Calculations are work in progress, unsigned and unchecked. We assume that they will be completed, finalized and a thorough independent check provided.

San Francisco-Oakland Bay Bridge Retrofit/Replacement Evaluation

Date Document Reviewed by COE: September 2000 Reviewed by: Chung Wong

Document I.D.#: 367 Volumes 38 of 41

(1) Structure

Suspension span

(2) Structural Analysis Results and Design Calculations

¥ Design calculations for hinge beams at hinge between W2-W3.

(3) Remarks

Section 13.1.1 (Design Loads) and Section 13.1.2 (Hinge Between E2-E3) are missing from Volume 38 of Document 367. The design calculations for the hinge beams at hinge between W2-W3 are very brief. It is not possible to verify the sources of some of the numbers used in the calculations because of lack of narratives and cross-referencing. There are no design calculations for the hinge beam anchor bolts and the PTFE spherical bearings. Considering the missing sections identified above and the incomplete design calculations for the hinges and the associated components, the seismic safety of the bridge cannot be determined.

(4) Conclusions

The information in this document cannot answer the following aspects of Question 4: (1) seismic safety, (2) performance in a maximum credible earthquake, and (3) meeting lifeline criteria.

San Francisco-Oakland Bay Bridge Data Review
Replacement Evaluation

Reviewed by: **Bob Fish** | **Review Date:** 09-24-00
Discipline: **Structural** | **Document I.D.** # 378, V 1 of 17

Answers Question 1, 2, 3, or 4? 3

Description of Data Reviewed: Design Criteria (rev. 7, 7-16-99) and general Analysis

Introduction – ***This section is missing***
Design Criteria
Alignment – ***This section is missing***
4.2.1 and .2 Superstructure section properties (**program used not noted**)
4.2.3 Substructure section properties

- 4.2.3.1 Piers using **Section Properties Program, V.w.e.1**(**TY LIN proprietary?**)
- 4.2.3.2 Cap using **MathCAD**?
- 4.2.3.3 Piles using **MathCAD**?

4.3 Supplemental DL: LRT, barriers, bikeway, utilities
4.4 Static Time dependent Analysis to design post tensioning system using **SFRAME**

- Creep model: CEB for all but ACI for steel transition superstructure.

4.5 Elastic Analysis using **SAP2000**

- Model includes elements for deck joints, and all substructure, including individual piles.
- Multiple runs to account for many various LL cases, and other loads such as wind longitudinal, uniform and gradient temperature, and LRT nosing, derailment, and rail restrain loads.

Answers what part of Question? Describe.

Question 3:
Shows thorough static analysis (non-seismic loads) giving credence to the estimated construction cost.

Question 4:
Seismic loads control the issues of "seismic safety and meeting lifeline criteria" and cannot be judged from data in this volume.

Additional Remarks.

Design criteria:
Sections 2.3.5 and 2.8.7, covers combination of live load with seismic. The reduction coefficient specified is 0.17. This is supposed to reflect the estimated peak hour traffic predictions for the year 2025. this seems very low and back up data, such as the traffic estimates, have not been located in the documents provided.

4.4.2.3 Foundation matrix.
From where was it obtained?

San Francisco-Oakland Bay Bridge Data Review
Replacement Evaluation

Reviewed by: Cameron Chasten
Discipline: Structural
Review Date: 10/11/00
Document I.D. # 378, Volume 2

Answers Question 1, 2, 3, or 4? 4

Description of Data Reviewed:

Volume 2 describes the SAP 2000 elastic response spectrum global analysis.

Several analysis models were developed. Each skyway frame was modeled individually, and the full skyway was modeled. For each model, a tension and compression model is considered, and separate analyses are conducted for corroded and non-corroded piles. Results include dynamic characteristics of the entire structure, pier and pile forces and pier and pile cap displacements.

The following describes assumptions as noted in the document for the global frame models.

- Fault parallel, fault normal and vertical response spectra for 5% damping applied. It is assumed that these represent the SEE, accelerations up to 1.5 g are shown in the attached ARS curves. Section 4.6 Seismic Forces does not contain information. Results are combined as 0.3 F_n + 1.0 F_p or 1.0 F_n + 0.3 F_p.
- Models consist of 3D discrete stick elements with explicit modeling of spatial distribution of stiffness and mass.
- Cracking and yielding effects in individual members are to be accounted for by reducing member properties.
- SSI is accounted for by modeling full-length piles with soil represented by p-y, t-z, and q-u secant stiffness springs.
- Effects of adjacent structures are modeled by boundary frames or equivalent stiffness and mass matrices.
- Spectral response is determined using eigenvalue analysis with CQC modal combination.

Answers what part of Question? Describe.

This information helps to answer the following question. Sound analysis is described, and this analysis is a necessary step in developing a structure with adequate seismic performance.

Is the currently proposed replacement alternative seismically safe? Yes, provided the concerns listed below are addressed.

Additional Remarks.
The following are concerns.

1. Data described for model input does not consistently represent features shown on contract drawings [257]. It is not clear that analyses represent the final selected design. (For example, [253] shows piers E3 through E14 as nearly identical, and the input description shows different cross sections. Pile batters are shown as 1:8 [253] and are described as 1:6 in the model description.)
2. It is not clear how cracking and yielding effects are incorporated in the analyses.
3. The represented earthquake is not defined. The SEE is assumed.
4. Although such analyses are necessary in a seismic design, specific results are not summarized, and it is not clear how results are used in the design.

San Francisco-Oakland Bay Bridge Data Review
Replacement Evaluation

Reviewed by: Bob Fish **Review Date:** 09-26-00
Discipline: Structural **Document I.D.** # 378, V 2 of 17

Answers Question 1, 2, 3, or 4? 3 & 4

Description of Data Reviewed: Response Spectra Analysis

- 4.6.1 Elastic response spectra analysis using **SAP2000**
  - Analysis procedures, modeling and assumptions
  - Input
  - Description of iteration process to obtain p-y, t-z, & q-u secant soil spring stiffness to be used along the length of shafts for design.
  - Output for super, columns, shafts for various frame and entire structure, tension and compression models.

Answers what part of Question? Describe.

Question 3:

Shows thorough seismic load analysis giving credence to the estimated construction cost.

Question 4:

Describes the behavior of the bridge under earthquake loading based on a response spectra analysis, giving credence to seismic reliability.

Additional Remarks.

4.6.1 Elastic response spectra analysis using **SAP2000**

4.6.1.1.1: Analysis Procedures

- ARS SAP2000: used p-y, t-z, and q-u secant stiffness springs applied along the length of the shafts, which are each independently modeled.
  Linearized to spectral displacements with 5% damping.
- Under part 7 of "Modeling Assumption": Superstructure I effective = 0.7lg (weak axis only). This reduction was used in Section 4.6.1.2., "Frame 1 Models"
  Why reduced for prestressed, elastic super? Check allowable tension for seismic loading, used in design. D/C ratios are all shown to be less than 1.0
- p-y iterations should include direct comparison between assumed stiffness and actual stiffness (v/u), where 'v' is output force and 'u' is output displacement.

- Per Design Criteria,, Section 8.3, "Seismic Analysis":
  Inelastic dynamic (time history) is required for the viaduct (skyway) where mass is concentrated in the superstructure and pier caps. This certainly applies to this structure.
  Why was an ARS analysis performed for design?

4.6.1.2. Frame 1 Models

- The cross-sectional area of the superstructure used in the SAP2000 analysis seem to be up to 8% less than presented in Volume 1, Section 4.2.1.1. "Box Girder", which post dates the SAP2000 analysis.

San Francisco-Oakland Bay Bridge Data Review
Replacement Evaluation

Reviewed by: Cameron Chasten **Review Date:** 10/10/00
Discipline: Structural **Document I.D. #** 378, Vol 3

Answers Question 1, 2, 3, or 4? 4

Description of Data Reviewed:

This volume includes the pushover results using an ADINA nonlinear model of the piles and pilecap. Results are to be included as a check for the design of the CISS piles. The pushover analysis procedure is described as follows:

- Nonlinear beam elements are used with moment curvature relationships included for piles. Nonlinear springs model the soil (p-y, t-z, q-u).
- To simulate the pier, the axial DL and plastic moment of the pier is applied at the pile cap center of gravity. To bound the solution, M_p is applied in same directino and opposite direction of push. The plastic moment is calculated specifically for bending in the push direction.
- The local axis of piles are oriented in the direction of push.
- Pile cap is pushed to target RSA displacements using the 100%-30% combination rule for normal and parallel fault results.
- Solution is bound by pier plastic moment acting in two opposite directions.
- Curvature capacity of piles are limited such that strain in concrete is less that 0.01 and strain in steel is limited to 0.02.

In Section 4.6.2.2, Pilecap LE RSA Demands, pilecap displacement demands for normal and parallel fault, and vertical input, and SRSS displacements are listed. These results are taken from the SAP 2000 results as shown in Volume 2 of [378]. In Section 4.6.2.3.1, ADINA input files for Frame 1, Pier 6 with corroded and non-corroded piles are listed. Section 4.6.2.3.1 provides analysis results for the ADINA models that include:

a. Force-displacement curve for push analysis
b. Pile forces (axial, shear, moment) as a function of depth (at target displacement?)
c. Pile curvature demands are plotted vs. capacity and no problems are shown. Results show that the capacities are greater than the demands.

Similar Results are shown for Frame 2, 3, 4 and 5. These results are useful to check the pile design, and if these results correspond to the target displacements, then they could be used as pile cap service design forces.

Answers what part of Question? Describe.

This information helps to answer the following question. Sound analysis is described, and this analysis is a necessary step in developing a structure with adequate seismic performance.

Is the currently proposed replacement alternative seismically safe? Yes, provided the concerns listed below are addressed.

Additional Remarks.

The following are concerns.

1. Pile force results are provided, but the displacement at which the results are obtained is not identified. Resulting pile forces could be used for pile cap service design also, if the forces provided correspond to the target displacements.

2. It is assumed that the SEE level earthquake RSA were used, but this is not stated.

San Francisco-Oakland Bay Bridge Data Review
Replacement Evaluation

Reviewed by: **Bob Fish** **Review Date:** 09-26-00
Discipline: **Structural** **Document I.D.** # 378, V 3 of 17

Answers Question 1, 2, 3, or 4? 3 & 4

Description of Data Reviewed: Push-over Analysis using ADINA

4.6.2 Push-over analysis using **ADINA**

- Models are for substructure elements, pile cap and piles only. Piers are not included.
- Demand displacements and forces at the bottom of the piers, from RSA analysis using SAP2000, refer to volume 2, are used as input loads.
- A check of the failure mechanism is achieved by arbitrarily pushing the cap lateraly 1 meter. This is approximately four times the SEE event displacement.

Answers what part of Question? Describe.

Question 3:
Shows thorough seismic load analysis giving credence to the estimated construction cost.

Question 4:
Describes the behavior of the bridge under earthquake loading based on a non-linear push-over analysis, giving credence to seismic reliability.

Additional Remarks.

4.6.2.1: Push-over analysis Procedures
Per Note 1: Why are RSA displacements revised for the full skyway model based on soil spring stiffness, which are 50% of ultimate strength?

4.6.2.2 Pilecap LE RSA Demands
On first page (shown as pg. 8 at the bottom of the sheet): At top of page it is indicated that the outputis for a “tension” model. Why is there a table for “compression” results?
On page 13 (as shown at the bottom of the sheet): Time history results are tabulated, but apparently not used. Where are they used?

San Francisco-Oakland Bay Bridge Data Review
Replacement Evaluation

Reviewed by: Bob Fish **Review Date:** 10-10-00
Discipline: Structural **Document I.D.** # 378, V 4 of 17

Answers Question 1, 2, 3, or 4? 3 & 4

Description of Data Reviewed: ***Non-Linear Time History Analysis using ADINA***

4.6.3 Non-Linear Time History Analysis using **ADINA**

- The data base program ADIVA is used as a post-processor to extract results from the ADINA output files.
- Analysis procedure, modeling, and assumptions.
- These models include the foundation elements, pile cap and piles, similar to the ARS analysis using SAP2000 in Volume 2 and the push-over analysis contained in Volume 3. Similar pile modeling techniques are used. The soil springs are represented by multi-linear force - displacement curves similar to that used in the push-over analysis, rather than the single-linear secant stiffness curves used in SAP2000.
- The piers are also modeled using multi-linear curves to represent their stiffness. Moment – curvature curves for varying axial loads, rather than a force - displacement relationship was developed using the program ANDRIANNA. See Volume 14.
- The models consist of individual frame and several frames both with boundary conditions to represent the adjoining frames. Also, a full viaduct model, which includes boundary conditions to represent the suspension span, was analyzed.

Answers what part of Question? Describe.

Question 3:

Shows thorough seismic load analysis giving credence to the estimated construction cost.

Question 4:

Describes the behavior of the bridge under earthquake loading based on a non-linear time history analysis, giving credence to seismic reliability.

Additional Remarks.

The COE team can not locate in the design volumes where this non-linear time history analysis is used. This volume includes numerous displacement and force demand graphs, but does not make comparisons to either the element capacities or the ARS demands generated by SAP2000.

San Francisco-Oakland Bay Bridge Data Review
Replacement Evaluation

Reviewed by: **Bob Fish** **Review Date:** 10-02-00
Discipline: **Structural** **Document I.D.** # 378, V 5 of 17

Answers Question 1, 2, 3, or 4? 3

Description of Data Reviewed: ***Precast segmental Superstructure***

5.1 IDS (files on CDROM)
5.2 Longitudinal Design Eastbound Structure
- Flexure –
  5.2.1.1. Service Loads – moment and force
  - Moving (live: HS, LRT and Bike) loads from SAP2000 output
  - Static (temperature: drop, rise, gradients) loads from SAP2000 output
  - Force plots (individual and Group I & IV combinations) from IDS program
  - Load Combination Matrix
  - Stress checks for service loads (groups 1 to 6) and Construction Loads

  5.2.1.2. Ultimate Loads **See Volumn 6**

5.3 Longitudinal Design Westbound Structure
- Flexure – Service Loads
  5.3.1.1. Service Loads – moment and force
  - See above for eastbound.

  5.3.1.2. Ultimate Loads **Missing: Not included in volume 5 or 6**

Answers what part of Question? Describe.

Question 3:
Shows thorough seismic load analysis giving credence to the estimated construction cost.

Question 4:
Seismic loads control the issues of "seismic safety and meeting lifeline criteria" and cannot be judged from data in this volume.

Additional Remarks.

- The calculations are incomplete. Section 5.3.1.2, "Ultimate Loads", for "Longitudinal Design Westbound Structure".
- Some sections are difficult to follow and evaluate as they are unlabled and the purpose of their inclusion is not stated.

San Francisco-Oakland Bay Bridge Data Review
Replacement Evaluation

Reviewed by: Bob Fish
Discipline: Structural

Review Date: 10-09-00
Document I.D. # 378, V 6 of 17

Answers Question 1, 2, 3, or 4? 3, 4

Description of Data Reviewed: ***Precast segmental Superstructure***

5.2.1.2 Ultimate Loads - Longitudinal Design Eastbound Structure

- Flexure – Service Loads
  5.2.1.1. Service Loads **See Volume 5**
  5.2.1.2. Ultimate Loads
    - Load Combination Matrix
      - Codes used to describe combinations cannot be located.
    - Demand to Capacity Ratios
      - Force plots for load combinations
      - Summary of Capacity along length of superstructure.
      - Summary of Demand along length of superstructure.
      - Demand to Capacity ratios.
        Under Group VII(seismic) loading, all D/C ratios are less than 1.0. Therefore, elastic and no damage is expected. (note that 3.2 is shown for end joint #300. Assumed this may be ignored as analysis program for capacity apparently does not account for prestress force. Also, moment should be zero at face of end segment.

5.2.2. Shear and Torsion

- Discussion of design
- Forces from SFRAME, SAP2000, and ADINA are evaluated.

5.3.1.2 Ultimate Loads - Longitudinal Designn Westbound Structure

- Flexure –
  5.2.1.1. Service Loads **See Volume 5**
  5.2.1.2. Ultimate Loads **Missing: Not included in volume 5 or 6**

Answers what part of Question? Describe.

Question 3:
Shows significant analysis giving credence to the estimated construction cost.

Question 4:
Evaluates the performance of the bridge superstructure during an earthquake.

Additional Remarks.

General:
Work seems to be comprehensive. However, clear descriptions of many calculation sections are not provided and therefore, completeness cannot be determined. Should have a written description for and with <u>each</u> new output/ spreadsheet format.

5.2.1.2.2. “Demand to capacity Ratios for Longitudinal Moments”

- Under Group VII load combination (seismic) superstructure behaves elastically (D/C ratios are under 1.0). However, it is not clear whether live load, per Design Criteria, Sections 2.3.5 and 2.8.7, has been combined with seismic in this analysis.
- For the D/C ratio check, demand (D) is from Volume 2, Section 4.6.1.2., "Frame 1 Models". This model uses ann effective moment of inertia, Ie, for the weak axis of the superstructure = 0.7* Igross. Has the moment capacity calculations, beginning on calculation page 16 of 106, been based on this reduced section property to be consistent with the demand side?

San Francisco-Oakland Bay Bridge Data Review
Replacement Evaluation

Reviewed by: **Bob Fish** | **Review Date:** 10-09-00
Discipline: **Structural** | **Document I.D.** # 378, V 7& 8 of 17

Answers Question 1, 2, 3, or 4? 3

Description of Data Reviewed: ***Precast segmental Superstructure***

5.4. Transverse Design
- Loadings
- Analysis
  - Grillage Analysis
  - Finite Element Analysis
  - Comparison of Grillage and FEM analysis
- Service Load Design for Bridge Cross Section
- Transverse Prestressing Design for Top Deck
- Time Dependent Analysis Models for SFRAME
- Component Design
  - Top deck, Interior webs, Bottom slab, Edge beam and barrier, Precast concrete lightweight panel.
- Combination of Transverse and Longitudinal Reinforcement
  - Top deck, Interior webs, Bottom slab, Edge beam and barrier, Precast concrete lightweight panel.

Answers what part of Question? Describe.

Question 3:
Shows significant analysis giving credence to the estimated construction cost.

Question 4:
Seismic loads control the issues of "seismic safety and meeting lifeline criteria" and cannot be judged from data in this volume.

Additional Remarks.

General:
Work seems to be comprehensive. However, clear descriptions of many calculation sections are not provided and therefore, completeness cannot be determined. Should have a written description for and with each new output/ spreadsheet format.

San Francisco-Oakland Bay Bridge Data Review
Replacement Evaluation

Reviewed by: Cameron Chasten **Review Date:** 10/09/00
Discipline: Structural **Document I.D. #** 378, V1

Answers Question 1, 2, 3, or 4? 4

Description of Data Reviewed: This document includes much information. The data that is reviewed and described by this data sheet pertains to general criteria and the pile cap portion of the global model

Answers what part of Question? Describe.
The comments provided in this summary pertain to Question 4:
Is the currently proposed replacement alternative seismically safe?
Does the currently proposed replacement alternative meet lifeline criteria?

If the items noted are addressed as the design is completed, then the answer to these questions should be Yes.

Additional Remarks.

1. Reference the Criteria Section 6.3.2. The weld between the deck plate and ribs shall be 80% penetration weld. If welds are transverse, partial penetration welds are not acceptable.

2. Reference the Criteria Sections 6.3.5 and 8.5. For lifeline, bearings and expansion joints must accommodate SEE displacement during and after the event. No guidance is provided in 6.3.5 for SEE. In Section 8.5, criteria is provided based on estimated permanent displacement (i.e. AFTER the event). The joints must also function for the possible differential displacement that might occur during the SEE.

3. Does design for SEE as described by Criteria Section 8.1.2 satisfy the Lifeline condition? (i.e. Where is the precise definition for lifeline?)

4. The data that describes the global model is not always consistent with the contract drawings [257] (For example, the data for pier caps included in Section 4.2.3.2 does not reflect what is shown on the contract drawings.) Final geometric properties must be considered in the final analyses.

San Francisco-Oakland Bay Bridge Data Review
Replacement Evaluation

Reviewed by: Bob Fish **Review Date:** 10-09-00
Discipline: Structural **Document I.D.** # 378, V 6 of 17
And # xxx

Answers Question 1, 2, 3, or 4? 3, 4

Description of Data Reviewed: ***Pier Table Design, Tendon Anchorages, Blisters and Deviation Blocks, Hinges, Bearings, and Expansion Joints***

5.5 Pier Table Design

- Introduction
- Strut and tie analysis discription
- The analysis uses MathCADD or similar program to evaluate theeffect of various force / moment combinations on the faces of the joint (pier cap) and determine mild reinforcement and prestress rod requirements.

  5.5.4.1 Top Deck
  - Calcs indicate thatanalysis shall be by finite element analysis.
  - Refer to Section 5.4 in Volume 7 for finite element and grillage analysis.

5.6 Tendon Anchorages, Blisters and Deviation Blocks

- Typical strut and tie hnd calcs based onultimate tendon capacity. However, calcsare incomplete with several sections missing.

5.7 Hinge Design

- Design goals not provided.
- Demand seismic displacements not provided.
- Hinge beam calculations by hand. Demand loading from other analysis.
- Conclusion not provided.

5.8 Bearings

- One sheet only. Small font size and poor quality copy make sheet illegible.

5.9 Expansion Joints

- Design goals not provided.
-
- Very limited information provided.

  Note:
  - The Special Provision for “joint seal assembly”, for the various sizes required, is not provided in Document 353, “Technical Specifications for Structures, 100% PS&E in Progress” as is noted in that documents index.
  - See “remarks” below for comments on Document xxx , Reference Special Provision for Modular Joint Seal Assemblies. This documentwas received by the COE on 10-11-00.

Answers what part of Question? Describe.

Question 3:
Shows thorough evaluation of pile behavior giving credence to the estimated construction cost.

Question 4:
Shows thorouh evaluation of pier behavior giving credence to seismic reliability. However, the calculations for expansion joints, which are critical links in evaluating seismic behavior and performance are incomplete.

Additional Remarks.

5.9 Expansion Joints

FEE event: The mechanical opening capacity of the joint seal assemblies are to accommodate the FEE displacement demand. Therefore, it is expected that no damage, including loss of the rubber compression seals, would occur under such an event.
However, pounding of the joint seal assemblies due to closing of the joint gap during the FEE has not been addressed.

SEE event: The support beams span across the joint between ends of the superstructure, which support the transverse joint seal beams. They are extended to provide opening capacity, without unseating, under the SEE event. This would be well beyond the mechanical opening operation limits of the joint. Therefore, failure could occur in several ways. The joint seal beams or their welds to the support beams could fail, an edge beam could pull away from the concrete, or the edge beam could be designed as a fuse to allow pullout away from the concrete.

Again, pounding of the joint seal asssemblies due to closing of the joint gap has not been addressed.

Concern: The joint seal may be the weak link in the concept of providing a lifeline route across this bridge. Several concerns arise:

1. The original development of this type of joint was for slow thermal movements. The behavior of the joint under the high velocity of seismic motion has not been proven to be satisfactory. Rapid cycles of movement could cause major damage to the transverse seal beams.
2. The impact force under joint closing cycles could cause major damage to the transverse seal beams.
3. As discussed elsewhere in this report, the maximum credible earthquake (MCE) event could result in displacements up to 30% greater than anticipated under the design SEE event. This is well beyond the 10% factor of safety applied to the extended length of the support beams.
4. Per Document xxx, “Reference Special Provision for Modular Joint Seal Assemblies”, the joint is to be designed to accommodate both SEE $_{opening}$ (maximum movement of adjoining frames during the SEE event, which closes the expansion joint gap) and SEE $_{closing}$ (maximum movement of adjoining frames during the SEE event, which opens the expansion joint gap). Therefore, the total joint movement capacity must be the sum of these two movements. That is, the initial joint setting would near the midpoint of it’s full travel range. It is assumed that the joint would be field adjusted from this point to accommodate thermal movement and the remaining portion of concrete shortening.
   The design movement as shown in the calculations range from 1,252mm (combined opening and closing)at joint E06E/E07E to 314mm at joint E14E/E15E. The maximum allowable opening at initial setting and after all shortening and thermal contraction is 80mm. This equates to about 12 seals for the largest joint based on a rough assumption of thermal and shortening movements.

The consequences of either 1. or 2. occurring could be that the joints are rendered impassible. Large gaps between transverse seal beams and / or severely bent bars protruding above the deck could prevent the quick evacuation of vehicles from the bridge as well as delaying the accessibility of open lanes to emergency vehicles. Also, even if such damage does not occur, the violent movement of the joint seal assemblies and likely, at least transient, occurrence of large gaps, could result in numerous accidents. This would further disrupt the use of this bridge as a lifeline route.

The consequence of concern 3. is self evident. The unseating of the support beams could result in the total loss of the joint seal assembly. The unseated assembly would become hung up in the hinge beams below. This could cause severe damage to those items also. However, the greatest concern is the resulting very wide gap that would result in the deck (Note: The 100% plans show dimension “a” for the gap between faces of concrete segments, but the actual width are not provided. The table is left blank.). This would cause severe multi-vehicle accidents and probable fatalities, as well as create a severe obstruction to egress of the bridge. A scenario of a large number of panicked travelers abandoning their vehicles in order to get off of the bridge could occur.

San Francisco-Oakland Bay Bridge Data Review
Replacement Evaluation

Reviewed by: Bob Fish **Review Date:** 10-04-00
Discipline: Structural **Document I.D.** # 378, V 14 of 17

Answers Question 1, 2, 3, or 4? 3, 4

Description of Data Reviewed: ***Piers***

6.1 Piers

- Section properties (using program V.2.3.1)

  6.1.2 Description of the moment curvature program ANDRIANNA and its use.
  - The moment vs. curvature relationships for an envelope of seismic event axial load cases are then input into the global non-linear ADINA model for the two orthogonal directions

  6.1.3 Section Characteristics
  - Confinement design using PCACOL with a minimum based on ACT32 and Caltrans' BDS requirements.
  - Axial load vs. moment Interaction curves using PCACOL.. Various %steel (0.66 to 1.42) for both f'c = 32.5MPa @ 0.004 strain (for seismic loads with phi = 0.7 & 0.9) and f'c = 25MPa @ 0.003 strain (for service loads with phi = 1.0), are calculated.
  - Moment vs. curvature relationships using ANDRIANNA. Use expected strength, 1.3f'c, 1.1fy

  6.1.4 Design Forces (for piers)
  - Service load forces from the global models SFRAME (time dependent analysis), see Volume 1
  - Seismic load forces from global models SAP2000 (linear elastic ARS analysis), see Volume 2
    - Note that the SAP model is linear-elastic and is used for force based comparisons (flexural ductility). therefore, only cross sectional properties for elements are used.
    - The above non-linear curves for the piers developed by ANDRIANNA, and non-linear curves for piles, found in Volume 15, are not used in the SAP global model and, therefore, are not a factor in the design forces of this section.
    - These non-linear characteristics are used in the global non-linear time history analysis using ADINA, found in Volume .3. The results of these non-linear analyses are used for displacement based comparisons (displacement ductility) found in Volume ?
    - A push-over analysis for individual pile caps (piers are not included) using ADINA, is found in Volume 3. The results of these non-linear analyses are used for displacement based comparisons (displacement ductility) found in that Volume. No push-over analysis, which includes the piers, was found in the documents provided.

  - Program Access is used to combine loads from above into combinations specified in the design criteria.

  6.1.5 Flexural Design

Answers what part of Question? Describe.

Question 3:

Shows significant analysis gives credence to the estimated construction cost.

Question 4:

Covers the seismic evaluation of pier behavior, which is a major component to seismic reliability.

Additional Remarks.

6.1: Piers: Description

States that top of end piers for each frame are detailed to allow temporary longitudinal sliding of superstructure (to accommodate creep, shrinkage, and PT shortening). These connections will not be made fixed until after 12 months.

It is further stated that all service and seismic load analysis are performed with these joints fixed. Does this statement refer only to the pier analysis, or for the superstructure also? A full service load and seismic (Some what above the construction duration seismic level?) analysis must be performed to cover the 12 month period until the temporary contraciton joints are closed and fixity is achieved.

San Francisco-Oakland Bay Bridge Data Review
Replacement Evaluation

Reviewed by: **Bob Fish** | **Review Date:** 10-09-00
Discipline: **Structural** | **Document I.D.** # 378, V 15 of 17 and # 389

Answers Question 1, 2, 3, or 4? 3, 4

Description of Data Reviewed: ***Piles and Pile Caps***

6.2 Pile Caps
- Design Brief (for prestressed concrete option, which is not used)
- Flow chart for cap, pier, and pile design
- Good description of relationship between SAP analysis and non-linear and pushover analysis
- At end of 6.2 is unnumbered section, “Alternate pile-cap – Steel pile-cap”. This section is incomplete and does not match the 65% plans (which are similar in the 85% and 100% plans)

Note: see Document # 389, dated Oct. 4, 2000, “Design Calculations for Skyway Substructure”, “Pile Cap Calculations” for analysis of selected steel cap as shown on the plans.

6.3 Piles

Note:
- This section contains only the cross section characteristics of the pile.
- For service load elastic analysis of entire bridge with piles modeled (as a pile group w/ cap) using SAP2000 see Volume 1.
- For elastic RSA load analysis of entire bridge with piles modeled (as a pile group w/ cap) using SAP2000 see Volume 2.
- For push-over analysis of piles only (as a pile group w/ cap) using ADINA see Volume 3. This analysis pushes the pile cap (pier is not included) over to the RSA SAP2000 displacements. See Volumn 3, section 4.6.2.1, note 1.
- For non-linear time history analysis of entire bridge, used as a check of design, with piles modeled (as a pile group w/ cap) using ADINA see Volume 4.
- Note: The service load time-dependent frame analysis program SFRAME has also been used. This program is esentially the segmental superstructure design. Although, the piers are modeled in, the piles are not explicitly included and, therefore, not considered in the pile analysis.

- Design Brief
- Additional Pile Design Process flow chart
- Axial-Moment diagrams for various pile sections (program used not indicated)
- Moment-Curvature and Strain-Curvature families at varying axial loads for various pile sections using computer program ADRIANNA
  The ADRIANNA output comprises a significant portion of this volume.

6.3.5 Site Soil Characterization
- Includes Preliminary Geotechnical Site Characterization
- Log of test borings
- p-y curve development using program PYCUR12A
- Load-deformation curves (load at pile head vs. displacement based on combination of skin friction and end bearing.
- t-z curve

6.3.6 Springs for linear and non-linear analysis.

See Volumn 17 for the following:
- 6.3.7. Seismic Demands, RSA, NLTH, and push-over analyses.
- 6.3.8 Design of Plastic Hinge

6.3.9. Design of Pile Section (Seismic)
6.3.10. Design of Shear Transfer Device
6.3.11. Shear Design Check at of Pile Shaft
6.3.12. Design of Pile – Bearing and Uplift (Seismic)
6.3.13. Pile Design for Ship Collision
6.3.14. Pile Design for Service Loads

See Volumn 16 for the following:
6.3.15. Pile Installation
6.3.16. of Pile Corrosion Design

Answers what part of Question? Describe.

Question 3:
Shows thorough evaluation of pile behavior giving credence to the estimated construction cost. However, pile caps are a concern regarding constructibility.

Question 4:
shows thorough evaluation of pile behavior giving credence to seismic reliability. However, pile caps area concern regarding seismic reliability.

Additional Remarks.

6.2 Pile Caps

- Document 378 comments:
  The calculations provided thus far are the "65% Design Calculations". They do not reflect the details shown in the 65% (* 3-31-2000) plans. Volume 15 of 17, section 6.2, "Pile Cap", contain calculations for a reinforced, prestressed, concrete cap. They where performed by RV Nutt and dated April 1999. The 65% plans show an intricate plate steel shell cap, with numerous steel plate stiffeners, filled with concrete.

  Following these calcs, at the end of the same section, is a set of calcs labeled "Alternative pile-cap - Steel pile-cap". These alternative calcs are incomplete and also do not reflect the details shown in the 65% plan set.

- Document 389 comments:
  - The bases of these supplemental calculations, which cover the "steel alternative" pile cap, is finite element analysis. Intricate, detailed models were developed using ADINA. The results are depicted with color coded stress distribution plots through various cross-sections. Both SEE displacement demands, from other analysis, and application of the plastic over-strength forces from the piers, from other analysis, were used as the input loads. In addition, a push-over displacement of 1.0 meters was applied to determine the probable failure mechanism. All analysis were performed with two separate structural conditions of steel shell and plates only and steel shell and plates with concrete in-fill.
  - Although, stresses, as depicted in the stress plots, seem reasonable, with only a few locations being just above yield, a written summary of results is not provided. Also, input is not provided, thus plate thicknesses etc. can not be verified.
  - The pile to pile cap connection analysis consists of separate hand calculations. See discussion of three alternatives considered, located at the front of that section in Document 389.
  - The pier to pile cap connection analysis consists of separate hand calculations. A design methodology is provided.
  - Perimeter wall. Plans label this as "precast". It is actually cast-in-place around 25mm thick steel plate perimeter shell.
  - Fabrication Review. This section is empty. Not coincidently, this is an area of great concern. The constructability of this item has not been addressed and we have the following concerns:

1. The possibility of a shaft placement not being within tolerance toallow the prefabricated steel shell to slip over the 4 and 6 shaft group pattern is very likely. This would require considerable field modifications, which could bring the original analysis into doubt.
2. Design of the welding of the individual plates used to fabricate the steel shell, though detailed in the plans, is not provided in the calcs.
3. Corrosion protection of the steel plate components have not been discussed. Particularly in regards to the exposed bottom plate.

- Plan comments:

  Note also, that the 85% (* 2-15-99, we assume that the year is in error and is 2000) and the 100% (* 8-2-2000) plans provided to the COE team are similar to (using the same plate steel, concrete filled, design) the 65% plans. Therefore, it is apparent that the current calcs have been superceded. This was confirmed by the submision to the COE review team of Document # 389, dated Oct. 4, 2000, “Design Calculations for Skyway Substructure”, “Pile Cap Calculations” for analysis of selected steel cap as shown on the plans

6.3.4: Pile Section Properties (Stiffness and P-M Diagrams)

The computer program ADRIANNA-M was used to develop Moment-Curvature relationships for the piles under various axial loads (-100MN to 180MN). This analysis was performed at three main x-sections of the pile (top-cased and heavily reinforced, center-cased and mildly reinforced, bottom-uncased and unreinforced), which were further divided based on degree of assumed casing corrosion.

These Moment-Curvature relationships are then used as input in the ADINA models as the “section properties” of the piles, along with soil springs. Based on calculated moment and axial force, ADINA uses this input to determine the deflected shape of a pile and, hence the pile cap displacement.

6.3.5.4 Dynamic Component of Axial Resistance

Per last paragraph on first page of March 9, 1999 memo.: Has further update in the input motion and / or p-y and t-z soil spring models been done?

San Francisco-Oakland Bay Bridge Data Review
Replacement Evaluation

Reviewed by: Cameron Chasten **Review Date:** 10/04/00
Discipline: Structural **Document I.D. #** 378, V15(a)

Answers Question 1, 2, 3, or 4? 4

Description of Data Reviewed: This document includes much information. The data that is reviewed and described by this data sheet pertains to criteria established for the analysis and design of the pile caps.

Answers what part of Question? Describe.
Is the currently proposed replacement alternative seismically safe? Yes
Does the currently proposed replacement alternative meet lifeline criteria? Yes

The criteria for analysis and design is specific and is appropriate for a lifeline performance goal if the pile caps experience an SEE event. If the pile caps are designed in conformance with the defined criteria, the caps will be seismically safe and meet a lifeline condition.

The following performance criteria specific for the pile cap is specified. This is a subset of the global performance criteria found in Volume 1 of [378].

Serviceability. Pile cap shall resist the SEE with minimal damage.

Strength. The design ultimate strength shall be sufficient to force a ductile failure of the piling at extreme displacements.

Specific design and analysis guidance is also provided. The design guidance by R. Nutt accounts for pile fixity requirements, design for flexure and shear due to pile and pier forces, design for joint shear, and design for local effects including prying and bursting forces. The general design and analysis guidance is summarized below.

Serviceability. The cap shall be designed to resist SEE pile and pier forces as determined from a pushover analysis of the cap and pile system (ADINA model). design forces will be those that correspond to the pile cap being pushed to the target displacements. Serviceability design principles control (reinforcing shall be elastic so all cracks close after SEE) (Page 3, Nutt). The target displacements will be determined as the demand required using a linear dynamic global SAP analysis. The target displacements as determined by the SAP analysis are to be checked with a nonlinear dynamic analysis using ADINA. A local 3D finite element analysis of the pile cap will be developed to verify the final pile cap design.

Strength. The pile cap shall be designed by strength design principles for maximum forces that occur as determined by pushover analyses carried out until a collapse mechanism is formed in the piles. This conforms to Criteria Section 8.6 found in Vol. 1 of [378].

Additional Remarks.

San Francisco-Oakland Bay Bridge Data Review
Replacement Evaluation

Reviewed by: Cameron Chasten **Review Date:** 10/04/00
Discipline: Structural **Document I.D. #** 378, V15 (b)

Answers Question 1, 2, 3, or 4? 4

Description of Data Reviewed: This document includes much information. The data that is reviewed and described by this data sheet pertains to the analysis and design calculations for the pile caps in Frame 2 (Pier E7 to Pier E10).

Answers what part of Question? Describe.

Is the currently proposed replacement alternative seismically safe? No
Does the currently proposed replacement alternative meet lifeline criteria? No

The answer to these questions can not be yes at this time. The data reviewed includes significant errors in the analysis calculations. Additionally, the pile cap configuration shown on the contract drawings [257] is not represented by that shown in the analysis.

Design calculations for a concrete pile cap and some incomplete calculations for a steel alternative are included for Frame 2 (pier E7 to Pier E10) of the skyway. Calculations are provided for the strength design case. It appears that the design calculation procedures as outlined for a concrete cap are appropriate for preliminary design. In accordance with the established criteria, the force demand was assumed to be the simultaneous application of pier and pile plastic moments. These were applied longitudinally and transversely. The design of the pile cap is to utilize the earthquake load case (Group VII loading) with a load reduction factor of 1.0 (assure that plastic hinge cannot for in the cap), and conform to Caltrans BDS. The calculations for a concrete cap, however, include significant errors, and the calculations for the steel alternative are incomplete. It is not clear what the steel design constitutes. Neither design matches the pile cap as shown on the contract drawings [257]. The following are specific comments on the design analysis.

1. For the maximum design condition, it is assumed that the pier and all piles develop their plastic moment concurrently. This is a valid assumption for design. As shown by the analysis calculations for both the longitudinal and transvderse direction, however, the force distribution that is used as a basis for determining pile cap design moments is not in equilibrium. The moments from the pier and each of the piles must be balanced by pile axial and shear forces. In the analysis, this effect is considered only for the pier moment. The resulting error is significant because first, the pile plastic moment is substantial and second, all of the pier cap flexure and shear design forces are affected by the estimated pile axial load. The calculations must be re-done.

2. Because the pier cap involves a complex geometry unlike a beam, the behavior is not represented well using beam design assumptions. Although a beam analogy is appropriate for preliminary design, a detailed stress analysis must be accomplished. It is stated that a finite element analysis will be conducted to verify the design. This has not yet been accomplished and is considered as a necessary step.

3. In the calculations for the concrete cap a value of 622 MN-m is used for the pier plastic moment. The table at the end of the section shows this value to be 800 MN-m for frame 2 piers.

4. The analysis for the steel pile cap alternative is incomplete and portions of the analysis are not identified. A finite element model of the cap and piles is shown, but results are not discussed and no information on modeling assumptions is provided.

5. A calculation of the shear capacity of the 1-1/2 inch thick webs shows that each web can carry 2756 kips. To match the maximum shear used in the concrete cap calculations, a total of eight webs across the cap would be necessary. This is not the case as shown by the contract drawings [257]. It is not clear what this calculation proves.

6. The moment diagram was calculated for the portion of pile that is within the pile cap. The calculation shows approximately 8 ft of length yields. This requires clarification. By the design criteria no yielding should occur.

Additional Remarks.

San Francisco-Oakland Bay Bridge Data Review
Replacement Evaluation

Reviewed by: **D. Gray/A. Pujol** **Review Date:** **9/13/00**
Discipline: **Geotechnical** **Document I.D.** **#378, vol 15**

Answers Question 1, 2, 3, or 4? 4

Description of Data Reviewed: Describes pile design criteria and procedures. Pile Design Bief memo summarizes loads (service, earthquake and collision) and performance criteria for service, earthquake, collision, corrosion, driving stresses and cyclic fatigue. Includes soil data and pile capacity (load) and deformation data by F-EM. Much of the F-EM data is also found in Document 332, 387 & a preliminary version of 336. Earthquake loading controls the pile design.

Answers what part of Question? Describe. Provides criteria to begin answering Question 4, parts a, b & c.

Additional Remarks. Performance criteria is listed as the Caltrans Bridge Design Manual except as modified by references 6, 7 & 8 at the end of the Pile Design Brief. Pile design by F-EM is based largely on API documents and offshore structure technology. The API reference is listed in the Pile Brief but is not specifically included as a source of performance criteria. We suspect this is an oversight as F-EM clearly states in Document 332 that current Caltrans pile design methods arenot appropriate for the large diameter piles being considered.

Pile batters of 1:6 were changed to 1:8 to avoid interference of the adjacent pile. It is not clear whether the analysis were reperformed using the steeper batter.

An allowance for pile degradation from corrosion is applied to the pile thickness based on theoretical and experimental corrosion rates in Bay Area.

Vessel collision loads are check against the SEE acceptance criteria assuming earthquake and collisions do not occur simultaneously. This appears to be a reasonable assumption.

San Francisco-Oakland Bay Bridge Data Review
Replacement Evaluation

Reviewed by: **Bob Fish** | **Review Date:** 10-09-00
Discipline: **Structural** | **Document I.D.** # 378, V 16 of 17

Answers Question 1, 2, 3, or 4? 3

Description of Data Reviewed: ***Piles***

6.3.15 Pile Installation

- Driving acceptanc flow chart
- Drivability Analysis
- Underdrive and Overdrive Contingency Plans
- Pile Handling and Installation

6.3.16 Pile Corrosion Desig

Answers what part of Question? Describe.

Question 3:
Shows thorough evaluation of pile installation giving credence to the estimated construction cost.

Question 4:
Seismic loads control the issues of "seismic safety and meeting lifeline criteria" and cannot be judged from data in this volume.

Additional Remarks.

<u>San Francisco-Oakland Bay Bridge Data Review</u>
<u>Replacement Evaluation</u>

Reviewed by: **Bob Fish** **Review Date:** 10-05-00
Discipline: **Structural** **Document I.D.** # 378, V 17 of 17

Answers Question 1, 2, 3, or 4? 3, 4

Description of Data Reviewed: *<u>Piles</u>*

6.3.7. Seismic Demands, RSA, NLTH, and push-over analyses.
- 6.3.7.1 Spectral Analysis: Copy of output from <u>elastic RSA load analysis of entire bridge</u> with piles modeled (as a pile group w/ cap) using SAP2000 see Volume 2.
- 6.3.7.2 NLTH Analysis: Copy of output from <u>non-linear time history analysis of entire bridge</u>, used as a check of design, with piles modeled (as a pile group w/ cap) using ADINA see Volume 4.
- 6.3.7.3 Push-over Analysis: Copy of output from <u>push-over analysis of piles only</u> (as a pile groups w/ cap) using ADINA see Volume 3. This analysis pushes the pile cap (pier is not included) over to the RSA SAP2000 displacements. See Volumn 3, section 4.6.2.1, note 1.

6.3.8. Design of Plastic Hinge
- 6.3.8.1 Design base on RSA demands
- 6.3.8.2 Design based on NLTH
- 6.3.8.3 Design based on push-over

6.3.9. Design of Pile Section (Seismic)

6.3.10. Design of Shear Transfer Device
6.3.11. Shear Design Check at of Pile Shaft
6.3.12. Design of Pile – Bearing and Uplift (Seismic)
6.3.13. Pile Design for Ship Collision
6.3.14. Pile Design for Service Loads
6.3.15. Pile Installation
6.3.16. of Pile Corrosion Design

Answers what part of Question? Describe.

Question 3:
Shows thorough evaluation of pile behavior giving credence to the estimated construction cost.

Question 4:
Shows thorough evaluation of pile behavior giving credence to seismic reliability.

Additional Remarks.

San Francisco-Oakland Bay Bridge Retrofit/Replacement Evaluation

Date Document Reviewed by COE: September 2000 Reviewed by: Chung Wong

Document I.D.#: 381

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(1) Structure

Suspension span and skyway structure

(2) Ground Motion Criteria

¥ A summary report on two-dimensional site response analysis.

¥ Site response analyses were not conducted for the vertical component motion. Instead, the vertical component rock motion was used directly for analysis with allowance for wave passage effects of the vertical motion along the embedded pile length

¥ The longitudinal ground motion at East Pier was investigated with the two-dimensional site response analysis using the computer program QUAD4M. The transverse ground motion was developed with the one-dimensional site response analysis using the program SHAKE.

¥ Note that input motions to the QUAD4M analyses were generated by first conducting the appropriate 1-D soil column deconvolution analysis using SHAKE.

¥ Figure 4 shows the finite element mesh for 2-D site response analysis. The finite element mesh closely models the existing geological conditions as shown in Figure 1.

¥ Note that 1-D site response analysis was performed for most of the skyway to the east using SHAKE.

(3) Remarks

This document provides a general description of the procedure for site response analysis at East Pier. Although the document does not provide all analytical details, it indicates that the site response analysis follows the state-of-the-art procedure using the equivalent linear two-dimensional computer program QUAD4M, which can develop a reasonable estimate of ground motions ampllified through deep soft soils. Also included in this document are QUAD4M input and ouput files for six sets of ground motions. It was indicated in Document 384 that nonlinear site response analyses were conducted in addition to the conventional equivalent linear site response analyses and that free-field displacement time histories from the nonlinear analyses wee used as seismic input to evaluate the effects of permanent ground displacement. It cannot be determined what the nonlinear site response analyses involve because no data was provided.

(4) Conclusions

This document provides good background information for the QUAD4M analyses. However, Document 384 implies that more sophisticated nonlinear site response analyses were performed to generate seismic input for dynamic response analysis of the bridge. It cannot be determined what the role of the QUAD4M analysis is. Therefore, information in this document will not be considered in answering Questions 3 and 4.

San Francisco-Oakland Bay Bridge Retrofit/Replacement Evaluation

Date Document Reviewed by COE: October 2000 Reviewed by: Chung Wong

Document I.D.#: 383, 384

(1) Structure

Suspension span

(2) Performance Criteria

The executive summary of Document 384 provides the following performance criteria:

¥ Under the SEE event, the structure will sustain repairable damage confined to pre-determined structural components that can be readily inspected and repaired, without significant disruption to general traffic and any disruption to emergency vehicles. The damage will be minor spalling in designated plastic hinge regions of concrete piers, and controlled yielding of steel reinforcement in the piers and main tower shear link beams. The bridge permanent set displacement has been limited to 0.20 meters maximum at the deck level, in order to provide immediate service to emergency traffic.

¥ Under the FEE event, the structure will sustain minimal damage limited to minor concrete cracking and zero permanent set displacement.

(3) Design Criteria

¥ See Volume 1 of Document 367. In addition to the design criteria specified in Volume 1, the following criteria were adopted for specific structural components:

¥ All structural components except the East and West piers and the main tower shear links, are designed to remain essentially elastic under seismic demands due to SEE. Plastic hinges that may occur during the SEE event are limited to the main tower shear links, and to the top and bottom of the East and West Piers. The deck box girders, crossbeam plates, and the main tower shafts, which are designed to remain elastic under the SEE demands, are also designed to be ductile according to the Caltrans Bridge Design Specifications (BDS) and ATC-32.

¥ The main tower shear links are designed to have a plastic rotation capacity exceeding 0.06 radians, up to 0.09 radians.

¥ The main tower shafts are designed to remain essentially elastic under SEE, and to be compact per ATC-32 to avoid local buckling before any yielding occurs.

The East and West piers are designed to have a displacement ductility of 4, up to 6.

¥ The tower grillage is designed to remain elastic under a bending moment corresponding to 1.25 times the plastic moment capacity of the tower shaft. The grillage plates are designed to carry the entire shear load as unstiffened webs.

¥ For the uncased pile sections at Pier W2 foundation, the concrete strains are limited to 0.004 under SEE. See Volume 1 of Document 367 for other allowable strains.

(4) Analysis Methodology

Refer to Document 384, Section 3 **METHOD STATEMENT ..** for more details. Also, Appendix O of Document 384 provides a brief description of the procedure used to calculate the Demand/Capacity ratios for load combinations.

(4) Modeling Assumptions

ADINA ANALYSIS

¥ The ADINA time-history analysis accounts for nonlinear geometry, nonlinear material, and multiple support excitation. Nonlinear geometry is important because the geometric stiffness is a function of displacements (not small displacements as were assumed in conventional analysis) and P-delta effects are significant for slender structures. Nonlinear material was modeled to account for the change in stiffness in the structure with increasing deformation, and the actual capacity of the elements (piers, main tower shear links, and the soil surrounding the piles) which are expected to yield. Time-history analyses with multiple support excitation were performed to properly capture the variability of the seismic input motions at the locations of each of the bridge foundations with different soil characteristics. The ground motions at Pier W2 and main tower foundations are essentially rock outcrop motions, whereas the motions at Pier E2 have site amplification effects.

¥ The suspenders were modeled with non-linear tension-only truss elements, which will allow the suspenders to go slack, if necessary, under dynamic loadings.

¥ The cable was modeled with elastic truss element and was idealized as passing through the cable PI s (intersection points of tangents to the cable profile where cable changes direction through the deviation saddles) at Pier W2, main tower, and Pier E2.

¥ The bridge deck was modeled with two parallel spines of elastic beam elements representing the axial, bending, and torsional properties of the suspended structure. Stiff beam elements were also used from the spine to the edge of the bridge deck to provide nodes for connection of the suspenders. These elements modeled the deck stiffness for vertical deformations and were rigid for transverse deformations. The crossbeams were also modeled as elastic beam elements representing their axial, bending, and torsional properties.

¥ The global ADINA model was constructed to represent the dead load state of the bridge. Initial strains associated with this condition were applied to the deck, cables, and suspenders.

¥ The main tower shafts were designed to remain essentially elastic under SEE. However, they were modeled with nonlinear beam-column elements so that other types of analyses such as pushover analyses can be performed to determine the collapse mechanism. The shear links connecting the main tower shafts were modeled with inelastic moment-curvature beam elements.

¥ Pier E2 and Pier W2 were modeled with nonlinear beam-column elements using moment-curvature relationships calculated for a wide range of axial forces, which are expected during an earthquake. Pier W2 was monolithically connected to the cap beam while pier E2 and the deck were connected through spherical uplift bearings and shear keys (pinned connection between deck and Pier E2). The nodes representing the bearing and shear key location are connected to the top of the pier through two rigid beam elements. Additional nodes at the same location are rigidly connected to the geometric center of the spine at the pier. These coincident nodes (a pair of nodes for each bearing) are constrained to move together in the longitudinal, transverse, and vertical directions (allowing relative rotation).

¥ Each pile in the main tower foundation was discretized as several nonlinear beam elements, extending from the bottom of the pile cap to the pile tip. Nonlinear p-y springs and t-z springs were used for each pile locations. The mass of the foundation, including rotational mass inertia and hydrodynamic added masses, was lumped at a node at the center of gravity of the pile cap. The ground motions were applied to each spring.

¥ Each pile in the Pier E2 foundation was modeled with non-linear beam elements supported with nonlinear p-y and t-z springs with varying properties along the height. t-z dampers were also included in the model to account for viscous damping. The mass of the foundation, including rotational mass inertia and hydrodynamic added masses, was lumped at a node at the center of gravity of the pile cap. Depth varying ground motions were applied to the p-y and t-z springs.

¥ Pier W2 is founded on rock. The boundary condition was assumed to be fixed at the base of the pier and the ground motions were applied directly at that level.

¥ The global ADINA model includes boundary frames representing the transition structure on Yerba Buena Island and the first frame of the skyway in order to ensure accurate boundary conditions. A simple beam element model was used to idealize the transition structure. Nonlinear behavior of the skyway piers and a hybrid model of the skyway foundations were considered. The hybrid model was composed of beam elements to model each pile from the bottom of the pile cap to the mud-line. Below the mud-line, each pile was modeled with a 12 degree of freedom stiffness and damping matrices. These impedance matrices were used in a local coordinate system at each pile, oriented along the pile axis, so that battering of the piles could be rigorously modeled. The ground motions were applied at the bottom nodes of the pile springs. The ground motions for the hybrid model are not the mud-line motions, but the motions at a firm soil layer below the Young Bay mud. This assumption was confirmed by kinematic motion studies of the pier foundations and soil profile.

¥ The hinge connections (expansion joints) of the main span structure to both the Skyway and the Yerba Buena Island Structure are in the transverse and vertical directions only. These two structures are free to move independently, relative to the main span, in the longitudinal direction.

The expansion joints were modeled using rigid beam elements and elastic beam elements. Basically, four beams constitute each expansion joint. Two beams restrain the relative transverse movement, and the other two restrain the vertical motion. The East expansion joints are fixed to the Skyway Structure and the West expansion joints are fixed to the Yerba Buena Island Structure.

SAP2000 ANALYSIS

SAP2000 was used to evaluate service load conditions. The features of the SAP2000 model are similar to the ADINA model. Features that are different from the ADINA model are noted as follows:

¥ The SAP2000 model does not include the YBI and skyway structures. Instead, they were idealized as equivalent stiffness springs.

¥ All material properties are elastic since inelastic behavior is not expected under service load.

¥ The foundations are simplified using equivalent impedance matrices. The piles at Pier E2 foundation were explicitly modeled down to the level of the mud-line. The portion of each pile below the mud-line was replaced with a stiffness impedance matrix. For the main tower foundation, the piles were modeled down to the rock surface and fixed at that elevation. The impedance matrices were selected on the basis of the expected level of displacement for service conditions.

¥ The hinge connections (expansion joints) of the main span structure to both the Skyway and the Yerba Buena Island Structure were modeled using equivalent stiffness matrices at the end of each expansion joint. At the expansion joints, the connection between the main span and the stiffness matrix is such that the nodes are constrained for movements in the vertical and transverse directions, but free for all relative rotations and longitudinal movements.

ANSYS ANALYSIS

¥ In the ANSYS models, two element types were predominantly used: the three dimensional plate element with both membrane and bending stiffness and three dimensional beam element with tension, compression, torsional, and biaxial bending capabilities.

(6) Structural Analysis Results and Design Calculations

Appendices A through O of Document 384 provide analysis results which are intended to respond to our questions for the meeting on 13 September 2000. Analysis results presented in the appendices include the following:

¥ Plots showing the sequence of yield mechanism for main tower and piers.

¥ Demand/Capacity (D/C) ratios for the improved tower shaft design since the 65% Submittal (increase in tower shaft cross section between EL. 50 and EL. 120). The D/C ratios indicate that the tower shafts remain essentially elastic during SEE.

¥ ANSYS finite element analysis results (displacement and Von Mises stress contour plots) for the improved tower grillage design since the 65% Submittal.

¥ Analysis results based on detailed local ADINA finite element analyses of the shear links (modeled as shell elements). The shear-displacement relationship obtained from these analyses was used to calibrate the inelastic moment-curvature beam elements for modeling the shear link beams in the global model.

¥ Analysis results based on several types of deck stability (buckling) analysis for different load patterns. Analyses include ADINA models and closed-form analytical solutions. It was concluded from these analyses that the deck spine system has a very large factor of safety against instability due to elastic buckling.

¥ Analysis results based on tower stability (buckling) analysis for different structural configurations to evaluate the effects of the shear links. It was concluded from these analyses that the tower buckling capacity is larger than its corresponding yield capacity.

¥ A summary of results comparing the performance of the bridge under SEE and FEE.

(7) Remarks

Documents 383 and 384 provide performance and design criteria, which are basically consistent with the provisions specified in Volume 1 of Document 367. In addition, design criteria to be used in conjunction with those in Volume 1 of Document 367, were provided for specific structural elements. For instance, the concrete strains are limited to 0.004 for uncased pile sections at Pier W2 foundation.

These documents also give a more comprehensive description of the SAP2000 and ADINA global analytical models, as compared to the very brief description provided in Volume 5 of Document 367. Based on this description and a cursory review of the sample SAP2000 and ADINA input files (Volumes 3 and 5 of Document 367), it is fair to say that the finite element modeling techniques used are very sophisticated and represent the state-of-the-art analytical procedures. However, the description is not detailed enough for us to verify all the critical features of the analytical models which include but are not limited to the following:

¥ Expansion joint modeling

In the case of the ADINA global model, it is not clear how the expansion joints were modeled using rigid beam elements and elastic beam elements and what properties were used.

In the case of the SAP2000 global model, it cannot be determined what degrees of freedom were considered for the equivalent stiffness matrix for modeling the expansion joints.

¥ Boundary conditions

In the case of the ADINA global model, it cannot be determined what boundary conditions were applied at the end of the boundary frames representing the YBI transition structure and the skyway structure.

In the case of the SAP2000 global model, it cannot be determined what degrees of freedom were considered for the equivalent stiffness springs used to model the YBI transition structure and the skyway structure.

¥ Damping

There is no discussion of damping used for the ADINA global model, except for a plot showing Rayleigh damping with $\alpha = 0.94248$, $\beta = 0.002387$ in Volume 5, Section 4.4.2.12 of Document 367. No basis was provided for selecting these values of α and β.

Documents 383, 384 provide descriptions for the ANSYS Level 1, Level 2, Level 3 models and explain how the analysis results were used to supplement the ADINA models. Appendices L and M of Document 384 present some plots of finite element meshes and stress results. However, the information is not detailed enough for us to determine the adequacy of the ANSYS models.

Section 8 **SEISMIC GROUND MOTION** of Document 384 notes that multiple-support seismic excitation was generated at each of the bridge pier locations, based on the latest advances in earthquake engineering. It also points out that “nonlinear site response analyses” were conducted in addition to the conventional equivalent linear site response analyses and that free-field displacement time histories from the nonlinear analyses were used as seismic input to evaluate the effects of permanent ground displacement. The conventional equivalent site response analyses presumably correspond to the QUAD4M analyses presented in Document 381. Data for the “nonlinear site response analyses” cannot be found. It cannot be determined what analytical technique or computer program was used to implement the nonlinear site response analyses and how they are different from the QUAD4M analyses.

The Executive Summary of Document 384 notes that laboratory testing of critical bridge components such as the piers, pile to pile cap connections, precast girder joints, main tower shear links, and expansion joints are currently in progress at various university testing laboratories. These tests are being conducted to confirm design assumptions made and to calibrate the design It is therefore logical to assume that the bridge design cannot be finalized until the testing program is completed and the design effort will certainly continue beyond the 65% submittal.

(8) Conclusions

The information in this document cannot answer the following aspects of Question 4: (1) seismic safety, (2) performance in a maximum credible earthquake, and (3) meeting lifeline criteria.

San Francisco-Oakland Bay Bridge Data Review
Replacement Evaluation

Reviewed by: Michael G. Mills **Review Date:** 10/05/00
Discipline: Structural **Document I.D. #** 384

Answers Question 1, 2, 3, or 4? 4

Description of Data Reviewed: Comments, SFOBB Descr. of Design and Analysis Approach.... TYLI 9/27/00 Draft

Page 2. Self-Anchored Suspension Span Description and Structural System. The bridge deck is counterweighted to balance the short back span with the long main span. This creates a balance independent from the anchorage at the west pier (*Load Path and Internal Redundancy*). The tower is decoupled from the "floating" deck to isolate seismic demands from the deck on the tower. And the bike path is balanced by a counterweight on the opposite side of the adjoining deck.

Par. 3. Method Statement for Design of Various Components of the Self-Anchored Suspension Span.
3.1 General Seismic Design Philosophy. The design references Design Criteria, Section 8.2.3 and all components, except as noted, remain elastic under demands for the SEE[ii] event. Components are designed for ductility per BDS[iii] and ATC[iv]. Elasticity is ensured by limiting yield exceedance to a small cross section. This exceedance is exemplified as 5% in the narrative and is reasoned by considering such factors as material overstrength.

The SAS design is primarily governed by the demands from a SEE event. To meet lifeline criteria, the SAS bridge is designed for immediate service following a SEE event.

3.2 Plastic Hinge Formation. During a SEE event, plastic hinges may occur at the tower shear links, and the top and bottom of both the east and west piers. The shear links are designed to attract seismic forces away from the tower so that *the tower components remain elastic.* The links become plastic under peak seismic demands. The plastic capacity of the links ranges from 0.06 to 0.09 radians of rotation whereas the peak SEE demand is on the order of 0.04, or less than 70% of capacity. *Since the tower remains elastic, any damaged links can be replaced, and the tower will returned to its original position.*

The piers are designed in accordance with the SEE design criteria. Non structural or minor damage in the form of minor spalling of the concrete-cover has been described. No loss in structural capacity has been stated. Supporting numbers are given showing that displacement ductility ranges from 4 up to 6, but with SEE demands less than 2, or less than 50% of capacity.

The sequence of plastic hinge formation has been predicted and diagramed.

3.3 Suspension System, Suspenders. Sizes are based on service load, allowable stresses and design calcs are referred to in volume 13. Demand, from the seismic event, is stated at less than 50% of capacity. (*event has not been identified.*)

3.4 Superstructure, Deck Structural System. The deck system has been modeled both globally and locally. Design has been correlated with suspender and cable tensions as they affect the box girder and to achieve the design cable profile.

Design is progressing through the various stages and methods, depending on the degree of needed refinement. As stated, design criteria is based on BDS and ATC 32. Analysis has used spread sheets. ANSYS[v], *and in conjunction with ADINA*[vi]. Each element is designed to satisfy factored service loads and the SEE demands. Elements include the box girders, orthotropic deck, floor beams, cross beams, and anchorages. (*Reference to design loads has not included that due to maximum cable deflection which should result in maximum bending in box girder.*)

3.5 East and West Piers. Design criteria for the FEE and SEE events is referenced and its application discussed for damage, structural strength and stiffness, and connections between pier and cap beam

Main Tower Shafts. The shafts have minor inelastic behavior during a SEE event and are compact per ATC-32 to ensure ductile behavior. It is noted that the method of computing demand overestimates by 5%. Shear links become plastic during a SEE event and are replaceable, with the tower remaining elastic and returning to a plumb position. The location for plasticized struts is identified and the tower stability is noted when struts are absent.

Tower Grillage. The grill remains elastic. Since the 65% submittal, design details have been revised to improve constructibility and accessibility. Point is noted that design continues and is not complete.

3.6 Foundations. These are still in design process and will meet the SEE event. Strains are stated to address damage limits. A design procedure is presented to consider corrosion, elastic and inelastic dynamic analysis, pile capacity, pile ductility, permanent set, and extreme event response. (*Per TYLI-RM, this is SEE.*)

Expansion Joints. Alternative #3 has been evaluated and selected. (*Per TYLI-RM, the 60m node is the cantilever from the skyway.*) The design for the joint is described and will be completed at a future submittal. (*Expansion Joints are proprietary.*)

Bearings and Shear Keys. The east pier is keyed to the deck after construction and the pier flexes without cracking under service load movements. Design criteria for bearings has not been stated in the description.

Par. 4. ADINA Analysis. The bridge is analyzed for SEE using non-linear geometry and material properties, and using multiple support excitation. (*Stated otherwise, these program-features model variables to provide a near-exact solution.*) The non-linearities have also been used to perform other analyses and design checks, such as collapse mechanisms.

4.1 The various bridge elements, suspension system, superstructure, tower, piers, foundations, bearings, boundary conditions, and expansion joints are discussed as they are modeled. Pile locations considered the natural slope of the rock foundations. Soil structure Interaction[vii] has been modeled to account for the bay soil properties. Mass has been considered using lump properties for both soil and water. (*The bearing descriptino is unfinished.*) The ends of the suspension bridge have been modeled using stiffness matrices to captrue the influence of the adjoining structures. In summary, the modeling has been thorough and consistent with the Design Criteria, 8.3.5.

4.2 Foundation Model Parameters. Soil and rock parameters were developed by Fugro-EMI based on results from tests on borehole samples, downhole testing and geophysical surveys. The pile lateral and axial capacities are established by means of p-y, t-z and Q-d curves, which in turn are based on the soil and rock parameters. Soil-structure interaction effects are captured by using explicit pile elements in the global ADINA model and by modeling the geologic medium (soil or rock) using depth-variable soil springs. Spring stiffnesses are fitted from the p-y, t-z and Q-d curves.

4.3 Dead Load State of the Model. Initial strains have been applied to the model components to replicate the as-constructed loads. These strains are based on final equilibrium conditions due to the dead load state. This model uses a restart procedure for time history analyses and under loading from the six ground motions.

4.4 Modal Analysis-Fundamental Mode Shapes.

4.5 & 4.6 Deck Stability Analysis. AND Tower Stability Analysis. Plots are presented to demonstrate stability and buckling capacities. For the tower, buckling capacity is larger than yield capacity.

4.7 SEE and FEE events. The six ground motions correspond to SEE. Results for the FEE event are summarized. No results are similarly summarized for the SEE event, but a comparison table is presented in APPENDIX J.

Par. 5. SAP 2000 Analysis. SAP is used to evaluate service load conditions and to make use of the program's live load generator. As a check, comparisons are made to the ADINA model output, ADINA being the more refined model. Adjoining structures are modeled as equivalent stiffness springs. Foundations are simply modeled as impedance matrices.

Par. 6. ANSYS Analysis. ANSYS is used to evaluate submodels, principally the box girder of the superstructure. The program offers automated procedures which provide an interface between the coarse (global) ADINA model and the refined ANSYS submodels. This is a linear static analysis and has also been used to complete fatigue studies. Stiffnesses have been calibrated and verified for use in ADINA and have resulted in a reduction of the apparent seismic demands.

Par. 7. Testing Program. Testing includes the tower shear struts, skyway pier, the connection for pile to pile cap, and the expansion joints.

Two shear struts are currently undergoing testing to confirm shear force / displacement relationship, deformation capacity, material overstrength, energy dissipation, and bolted connection performance.

Answers what part of Question? Describe.

Additional Remarks. Recommend that the maximum cable deflection be determined to verify whether bending is at a maximum in the box girder.

At paragraph 7, the testing program does not mention fatigue, which requires actual thickness to capture the thru-thickness variations.

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[i] SAS Self-Anchored Suspension
[ii] SEE Safety Evaluation Earthquake
[iii] BDS Caltrans Bridge Design Specifications
[iv] ATC Applied Technology Conference
[v] ANSYS a proprietary program for structural analysis
[vi] ADINA a proprietary program for structural analysis
[vii] SSI Soil Structure Interaction

San Francisco-Oakland Bay Bridge Data Review
Replacement Evaluation

Reviewed by: R. Turton **Review Date:** 10/12/00
Discipline: Structural **Document I.D.** #384

Answers Question 1, 2, 3, or 4? 3 & 4

Description of Data Reviewed:

Description of Design and Analysis Approach of the Self-Anchored Suspension Span of the New SFOBB (with Appendices) by TYLI – Draft dated 9/29/00. This document was prepared in response to inquiries and discussions that took place in a meeting with TYLI on 9/20/00. (A final version of Doc. 384 dated 9/29/00 was received on 10/03/00.)

Answers what part of Question? Describe. *Question 3 - Seismic Reliability*

There are concerns regarding the serviceability of the design of the replacement structure given some of the discussion presented in Doc. 384. Design, detailing, and anticipated performance suggests that degradation of the structure is anticipated in time and as a result of an SEE. Given this degradation, it remains to be seen that lifeline performance criteria can be ensured for the life of the structure.

Additional Remarks.

Specifically, the following items were identified:

- On page 7, the pile cap is referred to as “a steel moment resisting frame encased in concrete”. Given the marine environment and the proposed design life of 150 years, documentation was not observed whereby it was demonstrated to be reasonable and prudent to rely on the integrity of a submerged steel system. Consideration for corrosion (provisions for sacrificial section) was not observed, nor does it appear reasonable to assume that it could be adequately monitored.
- Basis for an upper bound on ductility of 4 to preclude closure for bridge repair subsequent to the SEE was not observed.
- On page 28, it was stated that pile corrosion was assumed at 0.2mm per year for 20 years (4mm). This is in contrast to the design calculations that indicate 20mm of sacrificial thickness on the steel casings.
- On page 29, it is stated that the deck joints are expected to suffer only minimal damage during the SEE. This seems contrary to what was implied in Doc. 344, where it was stated that the deck joints would have to be replaced.
- On page 30, it is stated that “large uplift restraint bearings” are needed at the east pier (E2). (Is there a precedent for bearings of this nature and size?)
- On page 31, it indicates that the suspender cables may go slack. Detailed information on how many cables and where this phenomena is expected to occur, and an analytical evaluation of this condition with respect to buckling of the deck (due to cable force) were not observed.
- The Final Version of Doc. 384, received from Caltrans on 10/3/00, indicates in the Executive Summary that “minor damage to expansion joints, at their extreme edges, may occur” as a result of the SEE. This is contrary to the post-SEE scenario presented in Doc. 344, where the deck joints are damaged to the extent that they need to be replaced.
- The details of the design and failure mode of the modular deck joints have not been observed.
- The MCE may require a greater displacement capacity than the SEE. (See Fig. 8.1 in the Final Version.)
- The potential for degradation of the structure due to environmental effects may impact the seismically reliability of the bridge.

Note: All page references are to the Draft Version unless noted otherwise.

San Francisco-Oakland Bay Bridge Data Review
Replacement Evaluation

Reviewed by: **D. Gray/A. Pujol** **Review Date:** 10/11/00

Discipline: **Geotechnical** **Document I.D.** # 367, V. 33

Answers Question 1, 2, 3, or 4? 4

Description of Data Reviewed: Draft Lateral Pile Design for Main span Pier E2 and Skyway Structure San Francisco-Oakland Bay Bridge East Span Seismic Safety Project

This report develops lateral pile capacity parameters for use in structural evaluations of the bridge. The report only addresses friction piles, i.e., those for pier E2 and the skyway foundations. The lateral pile capacity is defined in terms of p-y curves, which describe the lateral load-displacement characteristics of the soil material surrounding the pile. The p-y curves are developed following the practices recommended by the American Petroleum Institute for use in the design of off-shore structures. The effects of pile grouping and cyclic degradation on soil resistance are addressed. Additional modeling refinements for dynamic loading are discussed.

Answers what part of Question? Describe. Does not directly answer Question 4. However, the report presents reasonable lateral capacity parameters for the pile foundations. These parameters, if properly incorporated in the global ADINA model of the bridge, should enable the design team to perform a reasonable analysis of the bridge performance under seismic loading.

Additional Remarks.

San Francisco-Oakland Bay Bridge Data Review
Replacement Evaluation

Reviewed by: Cameron Chasten **Review Date:** 10/12/00
Discipline: Structural **Document I.D. #** 389

Answers Question 1, 2, 3, or 4? 4

Description of Data Reviewed:

This document includes preliminary hand design calculations for various elements of a steel pile cap for the skyway. Discussion and results for a finite element model of the cap and piles is also included.

Pile Cap Alternatives.

A discussion on four steel cap alternatives is included in [389]. Primary variations that were considered included whether to use the cap as a driving template or not and whether to use steel keyed shear plates or concrete grout to make the shear connection. It was considered preferable to use a separate template to minimize damage to the cap. A steel shear plate was preferred because of concerns of possible group cracking due to pile flexure.

Pile-to-Pile Cap Connection Design Calculations.

1. Pile head shear plates are designed to resist ultimate pile axial load. Shear plates are to be 17 mm minimum.
2. Pile moment is assumed to be resisted in shear by stiffener plates outside of the pile shell. Concrete strength between the pile and pile sleeve is checked for strength to resist the moment shear. LRFD and ACI criteria are followed.
3. The top plate is designed to have the yielding or fracture strength to resist the ultimate shear at the pile tip. Thickness of the plate must be 40mm. The plate is also checked to resist a 150 mm out of tolerance shear load developed by a pile.
4. Shear studs are designed to transfer shear from the pile to the surrounding concrete. This analysis ignores the contribution of the pile head shear plates and is, therefore, conservative.

Pier-to-Pile Cap Connection Design Calculations.

Distribution of forces in the pile cap are based on simplified assumptions such as the shear plate takes only vertical shear and the pier moment is resisted only by the couple involving horizontal shear at the top and bottom of the cap.

1. Area of steel to concrete interface around interface of pier socket casing is checked for resisting maximum axial load of the pier. The strength of the connection is calculated assuming a concrete-steel bond stress allowed for development of reinforcing bars.
2. For longitudinal pier bending, the top concrete slab is checked for strength to resist the horizontal component of shear due to pier plastic moment and is shown to be adequate. The slab thickness is 1.13 meters in the calculations and is shown as 1 m in the contract drawings. A bottom concrete slab of 0.45 m is assumed, but no basis is given.
3. For transverse bending, the concrete is checked for the shear component at the top and no check is made for the concrete or steel at the bottom of the cap.
4. Stiffeners are sized to support the top of the steel casing in bearing. No stiffeners are sized for the bottom shear component.

Perimeter wall design calculations.

Analysis of the composite wall section including a 25 mm inch thick steel plate and 300mm thick concrete slab is included. The section is analyzed for hydrostatic loading during construction. The results show that the concrete skin should be continuous to the bottom of the cap. The contract drawings, however, show the bottom 1 m of wall without concrete. The bottom plate is analyzed as resisting a fixed end moment from the perimeter wall. The calculation assumes an 80 mm thick plate and the stress is shown as 5% above the allowable stress. The design drawings [257] show the bottom plate to be 66 mm.

Finite Element Analysis.

Finite element (FE) push analysis of the steel pile cap was conducted using ADINA for longitudinal and transverse displacements. In the FE model, all steel plates and pile casing were modeled using elastic shell elements, and in-fill concrete was modeled with 3-D solid elements. Pier loading was applied to the pile cap elements through a frame of truss and beam elements, and piles are modeled explicitly. As in the global SAP models, piles are represented by nonlinear beam elements with p-y, t-z, and q-u springs incorporated to simulate soil effects.

To check the design of the pile cap, the model was pushed to longitudinal and transverse displacements that correspond to SEE demands, and the model was pushed to a longitudinal displacement that results in the formation of a mechanism in the piles. A transverse mechanism was not considered. For each analysis, loading that represents the dead load and plastic moment of the pier was applied at the top of the pile cap. Analyses were conducted considering the steel shell of the cap without concrete in-fill and with concrete in-fill.

The presented results indicate that for the SEE displacement demands, all critical elements of the pile cap remain elastic. For the ultimate displacement (pile mechanism), it was indicated that pile head shear plates (located between the pile and pile sleeve) and vertical shear plates are significantly overstressed if concrete in-fill is not considered. With concrete in-fill, the pile cap was shown to remain essentially elastic.

Answers what part of Question? Describe.
This information helps to answer the following questions. Analysis and design calculations are described, and these steps are necessary in developing a structure with adequate seismic performance.

Is the currently proposed replacement alternative seismically safe? Yes, provided the concerns listed below are addressed.

Additional Remarks.

Concerns regarding the FE analysis.

1. The pier will not maintain its over strength plastic moment simultaneously with the SEE displacement. With a lower moment applied, the stress distribution in the cap would change with certain elements having a higher stress. This may or may not affect the overall result.
2. The pile cap displacements assumed for the analyses were 0.275 m and 0.2 m for longitudinal and transverse SEE demand, respectively. The SAP 2000 linear elastic response spectrum analysis indicated longitudinal and transverse demands of between 0.5m and 0.6m for the pile caps at piers 13 through 16. These results are indicated for the Frame 3, 4, and 5 - revision 6 analysis (dated 6/30/99), and the full skyway model analysis (dated 7/16/99) [378, Volume 2].
3. As shown by the FE model results, the concrete in-fill is necessary when considering the ultimate design condition. The concrete will include shear and tensile forces that should warrant some reinforcement. No reinforcement is shown in the drawings [257].
4. Although a brief written summary of finite element results [389] indicates that critical members in the cap remain essentially elastic, a complete set of design calculations was not provided. Given the stated performance criteria, a factor of safety (strength reduction factor) regarding steel yielding must be provided. The FE results do not necessarily show conformance to this criteria.

Concerns regarding pile-to-pile cap connection design.

1. The design forces (pile moment and axial force) are not consistent through the calculations. The shear plate design and shear stud design include different forces.
2. The contract drawings [257] call for 480 studs per pile, while the design calculations show a minimum of 1478 studs.
3. There are no calculations regarding the concrete reinforcement between the pile shell and the pile sleeve. Significant reinforcing is shown on the contract drawings.

Concerns regarding the pier-to-pile cap connection design.

1. The slab thickness is 1.13 meters in the calculations and is shown as 1 m in the contract drawings.
2. Stiffeners are not sized for the bottom shear component, and no check is made for resisting the bottom shear component given transverse bending of the pier.

Concerns regarding the perimeter wall design.

1. No calculations are provided to show the affect of filling the interior of the footing unit with wet concrete. The concrete must be placed in specified lifts to ensure that the lateral load from the wet concrete is not too high.
2. The outside concrete should extend to the bottom of the plate per calculations.
3. the dimensions used in the calculations and shown on the contract drawings are not consistent (i.e. bottom plate).
4. The contract drawings show the concrete skin to be a precast concrete wall. The calculations assume that the steel edge plate and concrete skin are a composite section. It is assumed that the concrete skin will be precast onto the unit after the steel unit is assembled.

Appendix 7 - Review of Cost Estimates and Economic Analyses

Appendix 7. Review of Cost Estimates and Economic Analyses

1. Purpose of Review

Review of the cost and economic data assists in answering Questions 1, 2 and 3 from the Scope of Work. The COE Team reviewed the cost and economic data for reasonableness, accuracy, and completeness. The questions stated in the scope are as follows:

1. Was Caltrans' selection of the proposed retrofit alternative reasonable -- i.e., was it based on appropriate criteria and sound analysis, including consideration of **realistic, accurate and complete cost figures**?

    a. Did Caltrans adequately consider/evaluate other retrofit alternatives, including a West Span-type retrofit and other steel retrofits, and did this evaluation include consideration of **realistic, accurate and complete cost figures**?

    b. Did Caltrans adequately consider/evaluate the ability of other retrofit alternatives, including a West Span-type retrofit and other steel retrofits, to meet lifeline criteria? Which (if any) retrofit alternatives meet lifeline criteria?

    c. Did Caltrans adequately consider/evaluate the **costs of retrofitting** the span to meet lifeline criteria?

2. Was Caltrans' **cost-benefit analysis** comparing the originally proposed replacement alternative vs. the proposed retrofit alternative reasonable -- i.e., was it based on appropriate criteria and sound analysis, including consideration of **realistic, accurate and complete cost figures**?

3. How does the currently proposed replacement alternative, including as well any work in progress, compare to various retrofit alternatives in terms of a) **cost** and b) seismic reliability (including ability to meet lifeline criteria)?

2. Documents Reviewed

The documents reviewed in depth by the COE Team relative to cost issues are listed in Table 7-6. Of the 74 documents reviewed, 36 were further evaluated as they contained pertinent data to answer the Scope of Work questions pertaining to costs. Eighteen documents are identified as key documents in Table 7-6 because they enumerated the most useful cost information. Additional emphasis was placed on reviewing these 18 documents. In addition to the documents reviewed, the COE Team met with Caltrans cost estimators and design engineers numerous times for clarification and to view the project s numerous cost estimating backup files located in Caltrans offices.

3. Proposed Retrofit Alternative — Construction and Design Costs

The retrofit design consists of 11 (12 including the completed Contract 1) individual design contracts and corresponding cost estimates. The COE team reviewed the cost estimates for the eleven individual contracts that make up the retrofit design [Document 253]. In Document 253, titled San Francisco-Oakland Bay Bridge East Bay Spans Seismic Retrofit Cost Summary, Caltrans submitted one retrofit cost estimate summarizing the 12 contracts for review and evaluation. Table 7-1 provides an overview of this summary. The estimated total construction cost for SFOBB retrofit as proposed by Caltrans is **$733,464,000**. This cost does not include support or roadway costs. The dates of these estimates are from May 21, 1996 thru November 24, 1996.

Table 7-1. Caltrans Retrofit Strategy — Contracts/Costs Review

Contract No.	Project Name/Description	Design Level	Const. Cost ($=k)	% Contingency.
1	East Approach Piers E23 — E39	Bid	$25,130	
2	YB2 — YB4 Tower	GP/PS&E	$22,564	20
3	E2 — E5 Caissons	GP	$91,000	20
4	E6 — E9 Foundations	PS&E	$44,641	10
5	E10 — E16 Foundations	PS&E	$77,920	10
6	E17 — E23 Foundation	PS&E	$70,558	10
7	E5 — E16 Towers (w/o Tower E9)	PS&E	$19,932	10
8A&B	Cantilever Towers — E2A/2B	AP	$91,000	25
8C	Truss Separation	AP	$8,000	25
8C	New Cant. Superstructure	AP	$175,000	25
9	E9 Tower and 504 Trusses from E4-E9	GP	$41,319	20
10	288 Trusses	GP	$66,400	20
	Total from Document No. 253		**$733,464**	
	Roadway Costs (1)		$62,000	
	Support Costs (2)		$126,700	
	Total Cost		**$922,164**	

AP	Advanced Planning	0 — 35% Complete
GP	General Planning	35 — 75% Complete
PS&E	Plans, Specifications, and Estimates	75 — 95% Complete
Bid	Contract Bids Received and Awarded	100% Complete

(1) Document 370 provides roadway costs in the amount of $62,000,000, based on a preestablished Caltrans percentage. Roadway costs are costs associated with the interface of the existing interchange with the retrofit design.

(2) Document 370 provides support costs in the amount of $126,000,000, based on a preestablished Caltrans percentage. The support costs includes, but are not limited to design, site investigation, right-of-way, architectural/engineering (A/E) services, support during construction, and material testing costs. ***The retrofit support costs appear to be low when compared to support costs provided for the currently proposed replacement [Document 370].***

Neither support or roadway costs can be substantiated because backup cost data was not provided.

Document 374 summarizes the level of design for the 12 individual contracts that makeup the retrofit alternative. The level of design ranged from 0-100 percent, with nine of the 12 contracts being 75-100 percent complete.

Contingency costs, as a percentage of the construction costs, varied as the design level progressed from Advanced Planning, to General Planning, to Plan/Specification/and Estimate, to Bid Estimate. ***The contingency percentages used for cost estimates at the different levels of design appear to be appropriate.***

The uniqueness of the retrofit (construction above and below the water, minimizing traffic impacts, lead paint removal and abatement, etc.,) required an increased level of judgment on Caltrans part to develop accurate and reasonable costs.

The cost estimates, excluding Contract 8, were reviewed and found to have been adequately prepared with material takeoff, contractors quotes for materials, equipment costs, and labor costs. The COE Team could not verify the construction cost for Contract 8 because Caltrans had not developed enough cost detail. Since, according to Caltrans, lifeline design criterion for Contract 8 was not achieved, it could be expected that the construction costs for Contract 8 would increase due to cost associated with completing the design to meet lifeline criteria.

Comments on Contracts

Contract 1 - Eastbound Approach, Piers E-23 to E-29. Not evaluated concerning cost. This contract was bid and awarded. Design at 100 percent complete.

Contract 2 - YB2-YB4 Towers, E-1 Tower and Foundations. There is adequate pricing data with backup. Level of design and cost estimate is General Plan. Design at 95 percent complete.

Contract 3 - E2-E5 Caissons. There is adequate pricing data with backup. Level of design and cost estimate for contract is General Plan. Design at 95 percent complete.

Contract 4 - E6-E9 Foundations. There is adequate pricing data with backup. Level of design and cost estimate is Plan/Specification/and Estimate. Design at 75 percent complete.

Contract 5 — E10-E16 Foundations. There is adequate pricing data with backup. Level of design and cost estimate is Plan/Specification/and Estimate. Design at 75 percent complete.

Contract 6 - E17-E23 Foundations. There is adequate pricing data with backup. Level of design and cost estimate is Plan/Specification/and Estimate. Design at 80 percent complete.

Contract 7 - E-5 to E-16 Towers (without Tower E-9). There is adequate pricing data with backup. Level of design and cost estimate is Plan/Specification/and Estimate. Design at 80 percent complete.

Contract 8 - Cantilever Trusses, Truss Separation, and New Towers E-24 and E-2B. Level of cost estimate is Advanced Planning. Design at 0-35 percent complete.

Contract 9 consisted of E4-E9 504 Trusses (including E-9). There is adequate pricing data with backup. Level of design and cost estimate is General Plan. Design 75 percent complete.

Contract 10 - E9-E23 288 Trusses, and YB-1 to YB-4 288 Trusses. There is adequate pricing data with backup. Level of design and cost estimate is General Plan. Design 50 percent complete.

Assessment of Cost with Design Considerations

As stated in Appendix 4, Caltrans approach of base isolation was not reasonable for all segments of the bridge. Additionally, only a conceptual design for retrofitting the cantilever section had been completed. Caltrans proposed retrofit alternative apparently stops short of meeting lifeline criteria for several reasons, including unknown performance of the cantilever structure, excessive displacement of caissons E3, E4 and E5, and excessive displacements between the superstructure segments. However, although the proposed retrofit did not meet lifeline criteria, the design (and cost) data suggests that the no-collapse criteria would have been greatly exceeded.

To assist in verifying the reasonableness of the cost estimate for the proposed retrofit, the COE Team can suggest the following adjustments with corresponding rationale for each Caltrans contract to assist in approximating a lower bound cost estimate. This lower bound estimate is assumed to be for less than lifeline design. Note the following:

- This assessment does not rule out the possibility of costs actually exceeding Caltrans cost estimate, especially for a lifeline or near lifeline design.

- It is the COE Team s opinion that if a no-collapse retrofit alternative (for the entire bridge) were completed the construction costs would be less than Caltrans $733 million estimate.

- The ground motions used for the SFOBB site have increased significantly since the retrofit design.

Documents 253 and 117 would be useful to assist in adjusting the contract cost estimates for this type of assessment.

Contract 1 — No change.

Contract 2 — No change as towers require strengthening regardless of base isolation.

Contract 3 — No change as caissons E2 and E3 are not part of base isolated structure and it is unlikely that caissons E4 and E5 caissons would significantly impacted by base isolation. **This number would likely rise as Caltrans had not solved all problems (displacements).** No-collapse criteria would likely result in less substantial foundation work and costs.

Contract 4 — No change, unknown impact on foundation requirements if base isolation not used, probably minor. No-collapse criteria would likely result in less substantial foundation work and costs.

Contract 5 — No change, unknown impact on foundation requirements if base isolation was not used, probably minor decrease for designing to lifeline criteria. No-collapse criteria would likely result in less substantial foundation work and costs.

Contract 6 — No change as base isolation of superstructure is reasonable for E17-E23.

Contract 7 — No change as towers require strengthening regardless of base isolation, cost could go down somewhat if steel alternative used.

Contract 8A and B — This work may not be necessary if the performance criteria are to no-collapse and may be conservative for lifeline criteria, as the design is conceptual. For near lifeline design, costs could be similar to work proposed for piers and foundations of E6 and E7.

Contract 8C (Truss Separation) — No change to the work on the truss separation for a near lifeline design. Probably not necessary for a no-collapse design.

Contract 8C (New Cantilever Superstructure) — It is likely that much of the new exterior truss system is unnecessary for no-collapse criteria and a less extensive strengthening of the existing superstructure could be used. For this assessment it is assumed that the new exterior truss system is overly conservative and a much less extensive retrofit would suffice to meet lifeline criteria. A reasonable assumption would be that the retrofit cost for this contract would be equivalent to the combined retrofit cost for the 288 and 504 trusses.

Contract 9 — This contract could be reduced by $4.3 million as work on Tower E9 is primarily due to isolation bearings.

Contract 10 — No change as base isolation would not significantly change required superstructure work.

Estimates for the retrofit included cost for structural changes that would increase the structural stiffness of the existing bridge. This was to be accomplished by adding trusses to the superstructure and increasing the capacity of the existing foundation. Removing the cost for the materials, associated with the trusses and leaving the foundation as designed, will reduce the total retrofit cost. The extent of reduction must be based on a correct retrofit design. The reduction in cost becomes very subjective since the retrofit strategy was inappropriate and no other retrofit alternative was submitted for review. It would be inappropriate for the COE team to place a numerical number on how much of a cost reduction would occur since a completed design is not available. A cost number may be determined by taking cost estimates and simply subtracting out the cost associated with the inappropriate features of the proposed retrofit.

4. Originally Proposed Replacement Alternative (Skyway) Construction and Design Costs

The originally proposed replacement alternative (Skyway) is generally described as a haunched girder skyway structure that follows the same alignment as any other replacement bridge. This type of structure makes up a significant portion of the self-anchored suspension (SAS) and cable stayed alternatives with the difference being in the main span. Document 276 describes the skyway design as a structure constructed of either concrete or steel, supported by piers. With this structure type, each bridge span would be constructed as a separate, independent structure. Under the Skyway design alternative, spans over the navigation channel could be a maximum of 490-550 feet in length which would require 3 spans for the Skyway alternative as compared to 2 spans for the self anchored suspension or cable-stayed alternatives.

Over time, several variations of the skyway design and subsequent cost estimates were developed, as shown in Table 7-2. Cost estimates used in two of the primary reports [Documents 249 and 250] demonstrate the lifecycle cost superiority of the replacement approach over the retrofit, where a composite of several different replacement alternatives, including cable stayed, were used to represent the cost of the proposed replacement. The originally proposed replacement alternative, also described by Caltrans as Associated Structure Alternative #4, has cost of $531.62 per square foot [Document 252]. When multiplied out, this yields a total cost for replacement between $888,629,411 and $923,822,655 based on the length range given (these numbers are extrapolated and not stated in Document 252.) It appears support and roadway costs ($126 and $62 million respectively for the retrofit alternative) when added to the replacement alternative would raise the construction cost to between $1,077,329,411 and $1,112,522,655. These types of figures, extrapolated from Document 252, do not appear to be used in the other reports.

Appendix 5, Originally Proposed Replacement Support Documents, addresses the design efforts for the originally proposed replacement. Table 7-7 gives a summary of the replacement alternatives considered by Caltrans and documented for this review.

According to Caltrans, the originally proposed replacement alternative is essentially the least-cost replacement alternative that provides the required seismic performance (lifeline). This replacement alternative was used by Caltrans in late 1996 and early 1997 to compare the cost of retrofit to a replacement and was the primary basis for recommending replacement over retrofit. Replacement was recommended by Caltrans and the State of California in early 1997.

Although there were variations in the alternative over time and different reported costs, it does appear that the first cost figures developed for the originally proposed replacement alternative were reasonable and **the cost figures used in the primary decision reports [Documents 250, 249, and 23] may have actually been overly conservative**.

Table 7-2. Reported Costs for Skyway/Viaduct Alternative

Doc No.	Provided by	Description or Title	Date	Reported Cost (1)
169	Ventry Engr	Value Analysis Summary of SFOBB Replacement	08/96	605
252	Caltrans	The Gray Report Cost Estimate Investigation for the East Spans Replacement	09/96	(2)
170	Ventry Engr	Value Analysis Summary of SFOBB Replacement Bridge Retrofit Project — Structural Report	09/96	660
23	Ventry Engr	San Francisco-Oakland Bay Bridge East Bay Crossing Replacement Value Analysis Findings	12/96	797
249	Caltrans	The Yellow Report Replacement Study for the East Spans of the SFOBB Seismic Safety Project	12/96	987 (3)
329	Caltrans	Governor s Action Request (GAR)	02/97	1,075 (4)
250	Caltrans	RETROFIT VS. NEW BRIDGE	04/97	990 (3)
263	Caltrans	30% Type Selection	05/98	1,100 (5)
276	Caltrans	Draft Environmental Impact Statement (DEIS)	09/98	1,200 (5)
267	Caltrans	Replacement vs. Retrofit	04/00	1,170

(1) Millions of $, includes construction and design costs, including approaches, demo, interim retrofit, temporary structures.
(2) Report did not include original replacement alternative, however a similar viaduct structure was presented with a $531/SF cost.
(3) Average of 4 different replacement alternatives including 2 cable-stayed and 2 viaduct types taken from Documents 23 and 252. Includes interim retrofit and bridge demolition.
(4) Origin unknown, but may be either from use of Document 252 or simply rounding up. $1,000 million in design and construction and $75 million for interim retrofit and bridge demolition.
(5) 1998 dollars, DEIS included rounding otherwise the same as 30 percent Type Selection.

5. Economic Analyses/Lifecycle Cost

An economic analysis was completed by Caltrans, and summarized in Document 250, titled Retrofit vs. New Bridge, An Economic Analysis For The East Span of The San Francisco-Oakland Bay Bridge, dated April 1997.

In addition to Document 250, Documents 23 and 249 also address lifecycle costs of the retrofit alternative and the originally proposed replacement alternative. These documents have the same conclusion, i.e., the replacement approach is more desirable from a lifecycle cost standpoint. Based upon the data reviewed it is difficult to evaluate the actual numbers (or methodologies) used in the economic or lifecycle analyses as the backup data was incomplete and at times scattered over several documents; however the analyses appear to address the significant costs.

Document 250 presents the most comprehensive list of lifecycle cost items and divides the economic analysis into three primary categories as follows:

1. Basic Facility/Caltrans Costs
2. Additional Facility/Caltrans Costs
3. User Costs

This assessment follows the format of Document 250. Table 7-3, on the following page, summarizes the cost items for the three documents and is formatted into the three primary categories of cost items listed above. The following provides a description and summary of intent for the economic or lifecycle analyses presented in each key document.

Document 23, San Francisco-Oakland Bay Bridge East Bay Crossing Replacement Value Analysis Findings, presents a value analysis study that primarily identifies replacement alternatives and their costs. An attempt was made to evaluate and compare lifecycle costs of the report s recommended replacement alternatives and the Caltrans proposed retrofit. This document excludes costs for probable earthquake damage and probable earthquake human loss. Inclusion of these items would tend to make the replacement more attractive. Information on the retrofit was taken directly from Caltrans, as this report did not investigate alternative retrofit designs.

Document 249, Replacement Study For The East Spans Of The San Francisco-Oakland Bay Bridge Seismic Safety Project, presents a lifecycle cost analysis that emphasizes costs for probable earthquake damage and probable earthquake human loss. The differences associated with maintenance and operation costs are minimized. This report also includes a $150 million item on the Skyway alternative (but not the retrofit alternative) to bring the West Span retrofit up to lifeline criteria.

Document 250, Retrofit vs. New Bridge An Economic Analysis For the East Span of the San Francisco-Oakland Bay Bridge, presents an economic analysis/comparison of the proposed retrofit and the originally proposed replacement. This document includes such items as sunk costs, residual values, salvage value, interest, operation and maintenance costs, future deck rehabilitation costs, earthquake damage costs, traffic accident costs, and traffic delay costs. This document is the most comprehensive of the three documents with regards to an economic or lifecycle analysis.

Appendix 7

Table 7-3

SFOBB Project - Corps of Engineers Review

Economic Analysis of Retrofit vs. Replacement

	Items from Doc 250	Retrofit Costs			Skyway Costs		
	Basic Facility/Caltrans Costs	Doc 23 [1]	Doc 249 [2]	Doc 250 [3]	Doc 23 [1]	Doc 249 [2]	Doc 250 [3]
1	Construction and Design	852.00 [5]	909.0	909.3	797.11	987.0	915.0
2	Interim retrofit		6.0			(25.0) [4]	29.0
3	West Span Retrofit			391.0		150.0	391.0
4	Past East Span Expenditures	(-50) [4]				30.0	58.0
5	Bridge Demolition				(38.0) [4]	(40.0) [4]	46.0
6	Inflation (on New Bridge construction only)						85.0
	SUBTOTAL	**852.00**	**915.0**	**1300.3**	**797.11**	**1,167.0**	**1524.0**
	Additional Facility/Caltrans Costs						
7	Maintenance and Operations	21.70	17.0	283.6	7.30	0.0	192.5
8	Deck Rehab/Replacement/Paint Removal	69.72	27.0	175.9		0.0	0.0
9	Probable EQ Damage Repair		1203.0	97.8		517.0	66.9
10	Salvage Value			***-53.0***		-53.0	-53.0
11	Residual Value						-97.0
12	Past Expenditures (Applied to all options)			58.0			
13	Inflation Adjustment						***-85.0***
	TOTAL FACILITY COSTS	**943.40**	**1,841.0**	**1862.6**	**804.40**	**1,661.0**	**1548.4**
	User Costs						
14	Traffic Accidents	264.00	32.0	638.1	196.10	0.0	385.6
15	Probable EQ Human Loss		110.0	241.4		14.0	32.9
16	Potential Traffic Delays	19.40		156.4	0.40		39.1
	TOTAL USER COSTS	**283.40**	**463.0**	**1035.9**	**196.50**	**14.0**	**457.6**
17	Residual Value	-171.70			-199.90		
	TOTAL FACILITY AND USER COSTS	**1,055.10**	**2,304.0**	**2898.5**	**801.00**	**1,675.0**	**2006.0**
	Net Difference				-254.0	-629.0	-892.5

1 25 year lifecycle cost, 70 year retrofit life, 125 year replacement life
2 50 yr retrofit life
3 50 year lifecycle cost, 50 year retrofit life, greater than 50 year replacement life
4 Included in construction and design
5 Sunk design costs excluded (Approximately $60 million)

5. A. Basic Facility/Caltrans Costs

1) Construction and Design. This item includes estimated construction costs, roadway costs and support costs. In some cases, as noted on Table 7-3, this item also included interim retrofit costs, past expenditures and bridge demolition costs.

Retrofit

- Document 23: $810 million construction costs (Caltrans, 7/96), $42 million support during construction **= $852 million** ($50 million design [expended] = $900 million total)
- Document 249: $721 million construction costs (Caltrans, 12/96), $62 million District or roadway costs, $126 million support costs = $**909 million**
- Document 250: $**909 million** taken from Document 249

Skyway

- Document 23: $715 million construction costs, $82 million support/design costs = $797 million
- Document 249: $832 million construction costs, $25 million interim retrofit, $40 million bridge demolition, $90 million support/design costs = $987 million
- Document 250: $915 million construction and support/design costs. This number appears to be in line with Document 249 when bridge demolition and interim retrofit are added in, as this total is $990 million.

2) Interim retrofit. The interim retrofit reduces the seismic risk of the existing bridge during construction of the replacement. The actual bid price cost for the Interim retrofit (East Span) was $12,878,088 (no cost data for field mods available).

Document 249 reports a cost of $6 million for the retrofit without an explanation.

3) West Span Retrofit. The cost to retrofit the SFOBB West Span is the actual bid award cost of $146,641,203 (no cost data for field mods available). Document 249 includes an additional $150 million for the replacement alternative to bring the West Span up to lifeline standards. By not including the West Span retrofit in the retrofit alternative this would appear to favor the retrofit costs.

4) Past East Span Expenditures. This is essentially design, research, and support costs already expended. Document 250 includes $58 million for the retrofit under item 12 and $58 million for the proposed replacement bridge under item 4. Document 249 adds $30 million for the replacement only. Document 23 excludes about $50 million from the retrofit costs, assuming design is complete. There was no backup data provided for these estimates.

5) Bridge Demolition. This is the demolition of the existing bridge. Document 23 provides an estimate and methodology. The $40 million dollar figure used in Document 249 is also shown in document 252 and is likely based on the $38 million from Document

23. As a reference, Document 263, 30 percent Type Selection, 1998, gives a bridge demolition cost estimate of $54 million.

6) Inflation (on New Bridge Construction Only). Inflation in the amount of $85,000.000, as shown in Document 250, without backup data or an explanation, was only applied to the cost of the proposed replacement bridge. **Exclusion of inflation on the retrofit may be an error that favors the retrofit.**

5.B. Additional Facility/Caltrans Costs

7) Maintenance and Operations. Maintenance and operations (M&O) include the cost of all overhead, personnel, engineering, equipment, materials, traffic controls, and outside contracts. For the retrofit alternative, a major cost included in the M&O is the cost of regular painting of the bridge.

Document 250 includes East and West Span M&O costs for both the alternatives. The West Span M&O costs are $154 million. For the retrofit it is estimated that M&O will be $2 million per year and escalated at one percent per year to account for aging of the structure. For the replacement this number is $0.7 million per year.

Document 249 includes a present value of $8.5 million for painting (based on $37 million in the next 50 years) and $8.5 million for other maintenance activities for the retrofit. M&O is assumed to be negligible for the replacement.

M&O costs included in Document 23 appear to be based on annual costs of $.75 and $3.3 million dollars for the retrofit and replacement respectively.

8) Deck Rehabilitation/Replacement/Paint Removal. This item is essentially major costs of rehabilitating the existing bridge. The cost for deck rehabilitation, replacement, and lead paint removal for the short- or long-term for the proposed replacement bridge is $0 in all three documents.

Document 250 includes $118 million for major paint removal in 25 to 30 years (this has a present value of $37 million). Additionally, deck rehabilitation is included with $12.4 million required immediately and another $45 million for deck rehabilitation in 20 to 30 years.

Document 249 includes $12 million immediately and a present value of $15 million based on $45 million in 20 to 30 years. Painting is included in item 7.

Document 23 includes $11 million for deck replacement, $21 million for deck treatment and $37 million for lead paint removal.

Among the documents, the basis for these numbers is the same, however they have been treated differently with regards to present value and future costs.

9) Probable Earthquake (EQ) Damage Repair. The probability of a major and destructive earthquake(s) inflicting some economic damage was incorporated into the analyses.

During the construction period of both the retrofit and a new bridge, and before either alternative is fully completed, there is a chance for either or both of the two structures to be damaged by an earthquake. The earthquake assumption used in Document 250 is based on the likelihood of a magnitude 7.0 to 7.5 earthquake, even though there are higher probabilities for lesser magnitude earthquakes. The assumption made is that the retrofit will suffer more damage after construction is complete than the replacement. This accounts for most of the $30 million difference. This appears to be a reasonable approach though the backup for the probability/cost calculations was not provided.

Document 249 includes the costs and probabilities of two ranges of earthquakes, before construction is complete and after construction is complete for the retrofit and the replacement. A major assumption of this analysis is that the retrofit will suffer significantly more damage than the replacement after construction is complete as the retrofit is designed to a lower level of performance. A significant percentage of costs related to damage are associated with traffic delays.

Document 23 neglected costs associated with probable earthquake damage. **Most likely, this favors the retrofit in the analysis.**

10) Salvage Value. This is the salvage value of the existing bridge. The demolition cost of the existing bridge is included in the cost of a new bridge. However, after the demolition, there will be some economic value to the steel and other construction materials salvaged from the existing bridge.

The salvage value for the retrofit is a credit of $53 million in Document 250; neither Documents 23 or 249 contain a cost or credit for this item. Apparently this relates to the fact that the retrofit bridge s life span is assumed to be 50 years, however there is not a corresponding demolition cost. **This appears to be an error that favors the retrofit.**

The salvage value for the proposed replacement is a credit of $53 million in Documents 249 and 250. Document 23 does not include a cost for this item.

11) Residual Value. This is the value of the bridge beyond the economic life.

Document 250 states that the new bridge is going to provide more than 50 years of service, and a residual value of $97 million has been deducted from the total costs of this alternative. No residual value was assumed for the retrofit alternative, as the bridge would have to be replaced after 50 years.

Document 23 includes a residual value of $171.7 million for the retrofit and $199.9 million for the replacement based on 25-year lifecycle and remaining life (included in item 17). This document assumes a 125-year bridge life for the replacement and a remaining life of 70 years for the existing retrofitted bridge, which equates to a cumulative 125-year bridge life for the retrofitted bridge.

Document 249 does not include this factor for either alternative.

12) Past Expenditures (Applied to All Options). This is the same as Item 4 listed above.

13) Inflation Adjustment. The inflation adjustment cost for the proposed replacement is a credit of $85 million as shown in Document 250 without backup data or explanation. The inflation adjustment cost for the retrofit is $0. **This appears to be an error that favors the replacement.**

Neither Document 23 nor 249 include a cost for this item.

5.C. User Costs

14) Traffic Accidents. This item represents accident cost associated with normal use of the bridge.

The costs in Document 250 are based on the current average daily traffic (ADT) of 283,000, and an average cost per fatal, injury and property damage accidents. The total cost of traffic accidents, over a 50-year period for the retrofit option, is estimated to be about $638 million, and for the new bridge about $386 million.

A traffic accident cost is not included in Document 249, however a cost item identified as Safety in the amount of $32 million is included for the retrofit without backup or explanation. For the purposes of this review it is included here.

Document 23 takes the same approach as Document 250 but utilizes a 25-year period.

15) Probable EQ Human Loss. The usual daily traffic using the bridge is subject to the potential of experiencing an earthquake sometime in the future.

In Document 250 the annual probability of the occurrence of such an event was applied to the daily traffic count, under four scenarios representing the extent of potential bridge damage. The four scenarios consider the possibility of four different lengths of bridge collapse —approximately 1,000 feet., 2,000 feet., 4,000 feet., and collapse of the entire bridge. To estimate the potential number of vehicles that might be involved in such a bridge collapse, additional stopping distances were added to the damage distances. The estimate is based on an average fatality count of about 350, resulting from collapse of 4,100 feet. of the bridge.

For Document 249, probable earthquake human loss was actually added into the overall probable earthquake costs. The numbers presented here were pro-rated from the calculations and are approximate.

Document 23 neglected costs associated with probable earthquake damage. **This most likely favors the retrofit in the analysis.**

16) Potential Traffic Delays. These costs are defined differently among the three documents and refer to either earthquake damage or construction related delays.

Document 250 defines traffic delays as follows: when the retrofit is completed, there will still be some risk of short-term bridge closure due to an earthquake. It is expected that there will be some regulated traffic delays during construction for both alternatives. Under the new bridge alternative, the existing bridge will undergo some interim earthquake retrofitting, while in full use.

Document 249 addresses traffic delays associated with probable earthquakes. These costs are included in Item 9 listed above.

Document 23 considers traffic delays during construction for the replacement and during construction, deck repair and treatment for the retrofit.

17) Residual Value. This item is the same as Item 11 listed above, except for Document 23 that uses significant residual value for both alternatives based on remaining life beyond the 25-year analysis period.

5.D. Summary of Economic Analysis / Lifecycle Cost Evaluation

- The lifecycle cost analysis presented in Document 23 is the most straight forward of the three documents and appears to use reasonable numbers and provides backup for most of the items included. The significant costs associated with probable earthquake damage, probable earthquake human loss and probable earthquake traffic delays are neglected. Neglecting these items would tend to favor the retrofit, especially if a near lifeline design is not possible. This analysis suggests that if a retrofit cost were reduced by $200 to $300 million, or the replacement cost increased by this same amount, that the decision to go with a replacement was reasonable.

- The primary economic analysis documents have the same conclusion, i.e., the replacement approach is more desirable from a lifecycle cost standpoint.

- Caltrans compared a feasibility cost for the replacement alternative with limited design against a retrofit design completed to a level between advanced planning and PS&E. This type of comparison should be viewed with caution. As the design of the replacement alternative advanced, another economic analysis would result in a more viable comparison. However it is noted that the 1998 cost estimate [Document 263] for the originally proposed replacement alternative is based on a significant design effort and the estimated costs are in line with the feasibility costs [Documents 250, 249, and 253].

- Caltrans economic analysis [Document 250] position for replacement over retrofit is largely based on probable/potential cost formulated prior to March 1997. These costs include maintenance, salvage, post earthquake damage repairs, and traffic impact costs. Construction costs for the economic analysis comparison represent 31.4 percent of the total cost for the retrofit alternative, and 45.6 percent of the total cost for the original replacement alternative.

- Simply based upon Caltrans initial costs of the retrofit vs. the originally proposed replacement, from a qualitative point of view, it appears, even without performing the actual lifecycle/economic analysis that the replacement would be preferred. Additionally, seismic reliability, operations and maintenance, traffic issues, etc., further support the replacement alternative.

- The economic analyses indicate that a replacement alternative is preferable to a retrofit alternative even if a significantly less-cost retrofit were available.

6. Currently Proposed Replacement Alternative (SAS)

The documents reviewed that are directly related to the currently proposed replacement are included in Table 7-4. Documents 263, 351, and 370 provide the most comprehensive cost estimates for the currently proposed replacement alternative. The cost estimates in these documents are detailed unit cost estimates based on appropriate levels of design and contingencies. These cost estimates appear to be reasonable, comprehensive, and of good quality.

Many of the elements that are now escalating the currently proposed replacement cost would similarly affect the costs of retrofit. These elements that effect costs (usually in an upward trend) include: buy American requirements recently enacted; availability of labor, materials, and equipment; accelerated schedule; and economic conditions.

Table 7-4. Reported Costs for Currently Proposed Alternative

Doc No.	Provided by	Description or Title	Date	Reported Bridge Costs (1)	Reported Amenity Costs (1)
263	Caltrans	30% Design Concepts of Single Steel Tower with Haunched Concrete Skyway	05/98	1,497(2)(3)	51
264	Caltrans	Supplemental and Final 30% Design Report	06/98	1,514 (2)(3)	65
267	Caltrans	Replacement vs. Retrofit Cost Summary	04/00	1,500 (2)	Not Reported
272	Caltrans	Engineering and Design Advisory Panel Recommendations	06/98	1,285	310
315	Caltrans	Demolition of Existing East Bay Span — Considered Part of Replacement Costs	05/98	54	
351	Caltrans	Summary Retrofit and 30% Design Replacement	08/00	1,231(4)	
370	Caltrans	Summary of Costs at 65%	08/00	1,464(5)	

(1) Cost in millions of dollars
(2) Escalated 2002 dollars, at 3%
(3) Includes construction and design, ground motion contingencies, earthquake repair, and M&O lifecycle costs
(4) Construction and design, 1998 dollars
(5) Construction and design, 2000 dollars

7. Comparison of Costs, Proposed Retrofit vs. Currently Proposed Replacement

Table 7-5 summarizes and compares the construction and design cost estimates for the proposed retrofit alternative, the originally proposed replacement alternative, and various levels and configurations of the currently proposed replacement alternative. Document 370 provides a summary of costs that provides definitions and explanations for differences in structures costs, roadway costs, and support costs for the retrofit, 30 percent (design complete) replacement, and 65 percent (design complete) replacement. The retrofit design is incomplete and possibly based on an unreasonable design approach. Additionally, the ground motions for the site increased substantially since the retrofit design was accomplished.

Cost effectiveness should, as Caltrans points out in the conclusion in Document 267, remain a function of base case replacement structure (originally proposed replacement) vs. retrofit as the currently proposed replacement includes significant aesthetic costs that were requested by the MTC. Special financing is provided for these additional costs.

Table 7-5

ITEM	Proposed Retrofit 1996 [Document 253]	Originally Proposed Skyway 30% 1998 [Document 263]	Current Proposed 30% Base 1998 [Document 263]	Current Proposed 65% Base 2000 [Document 370]	Current Proposed 65% W/ Amenities 2000 [Document 370]
Mainspan	NA	149.10	300.77	348.98	390.79
Skyway	NA	526.60	515.30	505.07	565.59
YB Trans	NA	50.50	50.50	84.59	88.58
OTD	NA	29.00	29.00	96.76	101.43
YB Detours	NA	49.00	49.00	46.45	46.45
Demo	NA	54.10	54.10	54.10	54.10
Struc. Total	**733.50**	**858.30**	**998.68**	**1,135.95**	**1,246.94**
Roadway Cost	62.00	89.50	77.20	83.56	83.56
Support Cost	126.70	155.40	155.40	244.25	244.25
TOTAL	**922.20**	**1,103.20**	**1,231.28**	**1,463.77**	**1,574.75**
Escalated to 2000	**1,038**	**1,170**	**1,306**	**1,463.77**	**1,574.75**

Notes:
- Retrofit costs based on incomplete, unreasonable design.
- Percentages indicate completeness of design.
- Amenities include bikeway, aesthetic lighting, and light rail loading capacity.
- 3 percent per year escalation taken from Document 263, 30 percent Type Selection, 1998.
- Ground motion contingency costs, lifecycle costs, operations and maintenance costs, and post earthquake repair costs are not included.

Table 7-6 SFOBB Table of Document Evaluation - Cost Team Study

Date: 13Sep00

The Document Evaluation - COE Cost Team Study lists documents reviewed in depth by the team relative to cost matters. Of the 74 documents reviewed, 36 were further evaluateded as they contained pertinent data to help in answering the Scope of Work questions as they relate to costs. Additional emphasis was placed on reviewing those documents labeled below as "key document". These 18 key documents enumerated the most useful information on costs relative to the team answers to cost related questions.

Doc ##	Usefulness to Answer Questions (Yes/No)	Description	Cross Reference * - Primary	Date	Key Document (Yes/No)
9	N	Seismic Retrofit Concepts for the Bay Bridge - Astaneh		24Aug92	N
22	Y	Value Analysis Summary of the SFOBB East Bay Spans Foundation, Contract No. 53Y286 --(VE cost proposal)	169, 170	16Aug96	N
23	Y	SFOBB East Bay Crossing Replacement Value Analysis Findings --(life cycle cost matrix)	169, 170	00Dec96	Y
27	Y	East Span of the SFOBB Log of Test Borings (Preliminary Geologic Report to the MTC EDAP. Projects 3, 4, 5 & 6. EQ Retrofit.	*253	18Apr97	N
39	N	Cantilever superstructure, retrofit project No. 8, E1, E4 anchorage. Retrofit release analysis, retrofit concept development, quantity & cost summary and detail generation		22Oct96	N
44	N	Connection Team: various plans, meeting notes, memos, details, alt. Concept, retrofit. -- (Details of 'drop span' joint mod. @ added piers & steel Jt design criteria)			N
59	N	SFOBB Isolation - [504' Truss Frame / Heel Stiffener Install SEQ] (Friction Pendulum Bearing w/ Tension Load Capacity) -- (vendor cost est / Exp. Joints; includes FPS isolation)		00Jun96	N
60	Y	SFOBB 504' Spans Notes + Various (Scheduling, Estimates, Tasks, etc.)		yr1996	Y
71	N	SFOBB Retrofit Strategy for the Foundations of Piers E-17 to E-22			N
82	N	SFOBB Project Planning Descriptions		13Apr95	N
85	Y	Predicted Large Earthquake Response Scenario for the East Spans of the SFOBB in its Current (12-1996) State and Condition Seismic Life Safety Evaluation		06Jan96	Y
105	N	Analysis & Design Concrete Encased Steel Towers -- (details & x-section runs for pushover)		28Aug95	N
117	N	SFOBB - Cantilever Project Engineer Binder 04-0434GI	*253	00Jan97	N
123	N	SFOBB Towers E13-16 Quantities --(Steel retrofit alternative; Quantity backup w/ no cost data.)		00Dec95	N
132	Y	Quantity Estimates --(East approach beyond pier 23; Quantity / Cost summary)	*322, 136	1994	N
134	N	Engineering Criteria Review Board - Retrofit Presentation to BCDC		27Jul94	N
136	Y	Estimating File (3 folders) --(East Approach beyond pier 23; EQ retrofit; Bid cost seismic retrofit;)	*322, 132	94-95	N
142	N	Contract 04-043001 Interim Retrofit - East Bay Yerba Buena Island Viaduct --(for West Bay)	136	1997	N
143	N	Retrofit Options (various) --(struct analysis for West Bay, detail calcs for YBI approach, retrofit)		00Jun96	N
151	Y	Bay Bridge PE File SFOBB Seismic Retrofit Cost Estimate (Piers E17 - E 22, mtg mins, environmental, etc.)		00Nov96	N
162	Y	SFOBB E5 - E16 Memos & Estimates -- (Includes - Interim Retrofit Strategy; encasement vs. steel, FPS bearings, etc.; Cost summary)		00Mar00	N
169	Y	Value Analysis Summary of SFOBB East Bay Replacement Contract No. 53Y 286, Oakland, CA	*224, 23	7/8-8/23/96	N
170	Y	Value Analysis Summary of SFOBB East Bay Replacement Bridge Retrofit Project "Structural Report"	*22, 23	00Sep96	N

Table 7-6 SFOBB Table of Document Evaluation - Cost Team Study

Date: 13Sep00

Doc ##	Usefulness to Answer Questions (Yes/No)	Description	Cross Reference * - Primary	Date	Key Document (Yes/No)
200	N	Replacement Alternative No. 1 binder -- (FEA of original Alternative)			N
201	N	Replacement Alternative No. 2 binder -- (FEA of original Alternative)		00Aug96	N
202	Y	Replacement Alternative No. 2 cost estimate --(Quantity takeoff & cost summary)	202, *216	00Mar96	N
203	Y	Replacement Alternative Presentation Binder --(cable stayed alternative, double deck steel; Quantity takeoff & cost summary)	*216, 202		N
204	N	Replacement Alternative Planning Estimate Binder --(cable stayed alternative)		26Aug96	N
208	N	E1 Anchorage Retrofit Estimating Binder -- (Stringer Seats) -- (other retrofit)		00Nov96	N
216	Y	Replace Alt. 2 Presentation -- (Items for Cost Savings) --(cable stayed alternative)	203, 202	00Aug96	N
219	N	Retrofit Project No. 8 Wrap-up Binder		00Feb97	N
230	N	E1 Anchorage Retrofit Conclusions Binder -- (Peer Review; Cant. Truss Alt Development)		00Oct95	N
245	N	Retrofit construction schedule		05May00	N
246	N	Replacement construction schedule -- Governor’s Schedule		Yr2000	N
247	N	Competing Against Time		31May90	N
249	Y	"The Yellow Report" Replacement Study for the East Spans of the Bay Bridge Seismic Safety Project		00Dec96	Y
250	Y	Retrofit vs. New Bridge, An Economic Analysis for the East Span of the Bay Bridge		00Apr97	Y
252	Y	"The Gray Report" Cost Estimate Investigation for the East Spans Replacement		00Sep96	Y
253	Y	Seismic Retrofit Cost Summaries	27, 117	30Dec96	Y
254	Y	Senate Bill 60 (SB 60) Transportation Funding for Toll Bridges		06Dec96	Y
257	N	85% Skyway structure plans -- (super and sub - structures)		15Feb00	N
263	Y	30% Type Selection Report -- (Summary for replcmnt, w/ costs)		01May98	Y
264	Y	Supplement to 30% Design Report -- (Cost summary, EDAP directives)		22Jun98	Y
265	N	Contractors Information Session -- (East bay; Estimates, Geology, Environmental, Description, Demolition)		23Jun98	N
267	Y	Replacement vs. Retrofit for East Span -- (Summary of history)		01Apr00	Y
270	N	Preliminary contract quantities. Structure: SAS Oakland Approach (CIP), YBI Transition, YBI Temporary Detours, Skyway (segmental box girder). Roadway: Skyway, YBI/SAS.		Var	N
271	N	MTC Planning & Design recommendations for replacement bridge		01Jul97	N
272	Y	Additional replacement recommendations from EDAP for Single Tower Design		04Jun98	Y
277	N	65% Main Span Suspension Bridge -- (Revised to Mar 2000)		02Aug99	N
295	N	Transmittal Letter w/ copies of slides, articles, and references to information.		03Jul00	N
301	N	Power Point Slides for Presentation, 6/28 by Steve Hulsebus & Maroney		28Jun00	N
305	N	30% Design Definition for Type Selection of the Bay Bridge East spans Replacement Project		14Aug97	N
312	Y	San Francisco-Oakland East Bay Bridge Replacement EA04-10200K, Contract 53A0005		00Jun98	N
313	Y	SFOBB East Span Seismic Safety Project-Demolition Technical Memorandum-Bridge Removal		24Apr98	N
314	N	SFOBB East Span Seismic Safety Project-Demolition Technical Memorandum-Bridge Removal-Supplement to Section 5, 290’ Span Steel Truss		24Jun98	N
315	Y	SFOBB East Span Seismic Safety Project-Cost Report-Bridge Removal		11May98	N
322	Y	Bid Analysis and Summaries on various contracts from 03/20/97 to 09/08/99, Add 8/08/00		11Aug00	Y
323	N	Retrofit Strategy for the SFOBB Foundations E17 - E23 - summary --(strategy discussion and brief analysis summary)		18Jul00	N

Table 7-6 SFOBB Table of Document Evaluation - Cost Team Study

Date: 13Sep00

Doc ##	Usefulness to Answer Questions (Yes/No)	Description	Cross Reference * - Primary	Date	Key Document (Yes/No)
326	N	SFOBB East Bay Cantilever Retrofit, EQ Retrofit Strategy Summary Report --(detailed summary of cantilever.)		11Jul00	N
328	N	SFOBB East Bay Cantilever Retrofit, Seismic Summary Report at Cease Work		22Nov96	N
329	Y	Governor's Action Request (GAR) Decision to Replace Existing with New		07Feb97	Y
343	N	Deputy Directive - Transportation Management Plans - Traffic Delays & contingencies.		15Jun00	N
346	N	SFOBB East Bay - Skyway - **Index** for 65% Design Cacs & Analysis		??	N
351	Y	SFOBB East Bay Summary of Cost Estimates, Comparison of Retrofit to 30% Replacement	370,	20Jul00	N
354	Y	SFOBB East Spans Retrofit - Project Description 7/96 & Performance Criteria Statement 1/97 - by Zelinski		21Jan97	Y
356	N	Work Product by Maroney - for Astaneh Document 1992		19Jul00	N
358	Y	SFOBB Cantilever Segment -Seismic retrofit, strategy selection.		yr96	Y
360	N	SFOBB Retrofit Strategy 288' Trusses, E9-E23, Exit Report for Peer Review		30Oct95	N
361	Y	SFOBB Cable Stay, Alternatives #1-#3, Cost Backup for "GAR" Alternatives, Structure cost only.		00Aug96	N
362	Y	SFOBB Viaducts, Alternatives #1-#3, Cost Backup for "GAR" Alternatives, Structure cost only.		00Aug96	N
370	Y	SFOBB Cost-Estimate Comparison. Compares between Retrofit, 30% Replacement, and 65% Replacement.	*351	11Aug00	Y
371	N	SFOBB GP/ Planning study - Alternative #1 Viaduct - Seismic analysis by Mario Velado, lead design engineer (see gray report Doc 252) (Vols. A - C)	263	12Mar96	N
372	Y	CALTRANS Seismic Advisory Board Meeting Minutes, 1/03/93 thru 10/10/95 and 12/05/95 thru 5/30/00.		17Aug00	N
374	Y	SFOBB Retrofit for Cantilever & Towers E3, E2 - Design Calcs & Exit Notes		28Jan97	Y

Table 7-7
Documentation of Replacement Alternative

Doc #	Alignment	Alignment Alternative	Length	Comments	Structural Alternative	Description	Material
	Existing		Existing Length		Existing	5 Lanes & 0 Shoulders on each of 2 Decks;	Steel Truss & grate.
252 (Sept96)	South	Adjacent	140' Shorter	(Gray Report) Utility Conflicts at Oakland shore. Lower navigation clearance.	#3	5 Lanes & 2 Shoulders on each of 2 Decks; with Cable Stayed (650/650) and Viaduct (500) spans.	Steel & steel deck on stayed spans and concrete on viaduct.
					#4	Combinations of #1 - #3. 10 Lanes on 1 Deck; Viaduct spans.	Concrete Viaduct
					#5	Combinations of #1 - #3. 12 Lanes on 2 Decks; Viaduct spans.	Steel Truss Viaduct
	North	Adjacent	160 to 490' Longer	Alignment is non-compliant.	#6	Combinations of #1 - #3. 6 Lanes on each of 2 Decks; with Cable Stayed and Viaduct truss spans.	Steel Truss Viaduct
					#7	Combinations of #1 - #3. 10 Lanes on 1 Deck; Cable stayed & concrete box girders.	Concrete Box Girders
					Advanced Planning Studies	These studies offer substandard geometry and conclude as non-viable options.	
	North	Extended	1400 to 1700' Longer	Extended tangents for cable supported spans. Proximity to historic buildings.	#1	10 Lanes & 4 Shoulders on 1 Deck; with Cable Stayed (600/1400/600) and Viaduct (500) spans.	Concrete
					#2	6 Lanes & 2 Shoulders on each of 2 Decks; with Cable Stayed (700/1600/700) and Viaduct (700) spans.	Steel & steel deck
252	Various	Various			Aesthetics & Models	Aesthetic Schemes, "A" thru "L" to be used with various Alignments.	
23 (16Dec96)	Various			Value Analysis by Ventry; begin 7/96, finish 16Dec96	14 alternatives considered.		

Table 7-7
Documentation of Replacement Alternative

Doc #	Alignment	Alignment Alternative	Length	Comments	Structural Alternative	Description	Material
	Northern	approx. 1000 ft north of existing	12010' Total		Value Anal. #3; Highest rank of 14	10 Lanes & 4 Shoulders on 1 Deck level; with Cable Stayed (700'/1100') and Viaduct (550') spans. Suspended is Concrete Box & Steel Box. Via-duct is Cast in place & Pre-cast Concrete Segmental Box.	
					Value Anal. #10; 2nd high-est rank of 14	10 Lanes & 4 Shoulders on 1 Deck level; Viaduct w/ 550' spans. Viaduct is Cast in place & Pre-cast Concrete Segmental Box.	
249 (Dec96)				17Dec96, "Yellow Report"	#1, #2, #3	Replacement Alternatives per Doc #252.	See Doc #252
	Not Available			Figures are taken from Ventry report, Doc 23, -Sep96	Fig 22	Concrete Box Viaduct by Value Analysis Team.	Concrete
	Not Available				Fig 23	Cable Stayed Concrete Box by Value Analysis Team.	Concrete & Steel
263 29May98	Northern	N6 (Extended)	11800	Per 30%Type Selection by EDAP to MTC Task Force.	Dual Tower	Suspension Bridge (215/275m) and skyway viaduct.	
					Single Tower	Suspension Bridge (180/385m) and skyway viaduct.	
					Cable Stayed	5 Alternatives considered.	
	Northern	N1	No Info	Dropped for Geologic reasons.			
	Northern	N3, N4, N5	No Info	Refinements leading to N6.			
	Southern	S1	No Info	Conflicts w/EBMUD Outfall.			
264 22Jun98	Northern	N6(revised)		Supplemental and revisions to Doc. 263	Single Tower	Self-Anchored Suspension Bridge and Skyway with steel or concrete superstructure.	

Table 7-7
Documentation of Replacement Alternative

Doc #	Alignment	Alignment Alternative	Length	Comments	Structural Alternative	Description	Material
276 (Sep98)	Northern	N2 (Adjacent)	11759 ft.	Envirn. Impact Statement Study.	Viaduct	10 Lanes on 1 Deck level; Viaduct.	
		N6 (Extended)	11877		Viaduct	Similar to N-2, 10 Lanes on 1 Deck level; Viaduct.	
	Sourthern	S4 (Extended)	11644		Viaduct	Two parallel structures, assumed similar to N-2.	
261 15Jan99	Northern			45% Submittal for Skyway	Skyway portion	Two parallel decks, 1 level.	Conc. - Segmental box, Deck & Piers.
259 31Mar00				65% Submittal for Skyway, Add Light Rail			
257 15Feb00				85% Submittal for Skyway, Add Light Rail			
260 15Jan99	Northern			45% Submittal for Main Span	Self-Anchored Susp. Bridge	Two parallel decks, 1 level, single tower.	Steel Box, Deck & Tower.
256 15May99				65% Submittal for Main Span, Add Light Rail			
277 2Aug99				65% Submittal for Main Span, Add Light Rail			
258 23Aug99	Northern			65% Approaches			

Appendix 8. Lifeline Criteria

The Interim Letter Report identified a design criteria summary as a major data gap (Data Gap 1). The criteria available were assessed as significant to answer Questions 1, 2, and 3 which contained reference to the term lifeline criteria . Although several documents contained some definitions of the seismic performance criteria, no single document established a clear engineering definition of the lifeline criteria.

Documents relative to the seismic performance criteria have been cataloged and provided in Appendix 1 of this report. This catalog of documents is a subset of the documents provided in the Data Catalog (Appendix 3). The documents referenced include, but are not limited to, meeting minutes, letters, designers notes and memorandum. The catalog contains brief discussions of the documents relevance to seismic performance and lifeline criteria. The discussion is intended to clarify the differences in the criteria as it developed.

Seismic performance criteria were established in the early stages of planning for the SFOBB retrofit to guide research and design. The research and design was to eventually provide a retrofit solution that would increase the safety level of the bridge thereby preventing loss of life as well as providing full serviceability following a major earthquake.

In 1992, as defined by Bruce Bolt, a decision was made to use the SEE earthquake scenario and not the MCE scenario. This set the stage for the type of seismic analysis that would be used in the retrofit study.

> " *Due to the importance of the Bay Bridge, a decision was made not to use the concept of "Maximum Credible Earthquake." It is believed that this concept is not adequate for the scope of the studies and the importance of the Bay Bridge. Instead, a decision was made to use the concept of "Safety Evaluation Earthquake" (SEE). According to Bruce Bolt in Reference (18)*, "The SEE is defined as an earthquake whose occurrence is judged to be sufficiently unlikely so that it can represent the maximum earthquake that should be considered in evaluating the safety of the structure." The major difference between Maximum Credible Earthquake (MCE) and the Safety Evaluation Earthquake (SEE) are: (a)* ***the SEE is more forceful*** *and has widespread frequency content to excite structures modes; and (b) the SEE is generated to match a certain spectra as well as include characters of fault and the site.* [Document 9]

Despite this decision, the existing bridge and its retrofit design were evaluated for the two MCE events on the Hayward and San Andreas Faults, as referenced in the Draft Environmental Impact Statement. In other words, for the existing bridge the SEE was equated to the deterministic MCE. The probabilistic SEE was developed by Fugro-Earth Mechanics Inc., (F-EMI) [Document 335] and adopted for the replacement alternative. The SEE for the replacement alternative is more forceful than the MCE for short periods up to 2 seconds, but less forceful for longer period ranges (i.e., 2 to 5 seconds), which are significant to this bridge (see Sections 1.D. and 1.E. of Appendix 6).

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The Seismic Performance Criteria for Design of Bridges, as developed, categorized bridges into two major groups, Minimum Bridges and Important Bridges. There is no distinction for lifeline bridges or routes . At a Seismic Advisory Board meeting on July 17, 1995 an update on lifeline routes was presented. Twenty-eight routes throughout the state were placed in the lifeline category, as of July 1995. Following a major earthquake, these lifeline routes would allow for immediate access for emergency equipment and movement of supplies in or through a region. The SFOBB (Interstate 80) was not identified as a lifeline route at that time but was identified as an Important bridge [Document 397]. It was not until December 1997 that its category was upgraded to a lifeline designation.

" *Mr. Roberts then explained to the panel that the SFOBB was designated as an important bridge, and as such, would be retrofitted to a higher performance level allowing the main structural members to perform within the elastic range. This means the structure would not sustain damage preventing it from being immediately opened to traffic following a seismic event. Mr. Roberts then asked Jim Gates to review the Seismic Performance Criteria handout, which is one of the main items the Panel, is expected to review.* [Document 303]

At a meeting on March 15, 1996 only the Benicia-Martinez Bridge was identified as part of a designated lifeline route and thus would be retrofitted to higher standards; the standards being full serviceability, to ensure full public access, following inspection after a safety evaluation seismic event. The other toll bridges were to be retrofitted to satisfy a no collapse and minimum performance criteria but with higher emphasis placed on serviceability due to their regional economic importance. The SFOBB, as presented in this document, was to be retrofitted to no collapse with higher standards [Document 372].

The requirement for the bridge to be retrofitted to a higher performance level required the main structural members to perform within the elastic range. Additionally the structure was not to sustain any damage that would prevent it from being opened to traffic immediately following a seismic event. Certain documents suggested that the traffic refers to only emergency vehicles or vehicles used to transport large equipment and supplies. Additionally, other documents have defined a functional requirement as establishment of normal traffic within 72 hours after the seismic event.

As the design effort for the retrofit continued, the Peer Review Panel remained focused on complying with the design performance objectives stated in the June 2, 1990, Executive Order D-86-90 issued by the governor [Document 396]. Two objectives were identified in this order: 1) the bridge was to be safe from collapse and 2) the bridge would maintain its required function following an earthquake.

During a meeting on September 14, 1993, there was discussion as to whether the retrofit design would be elastic, or some plastic deformation would be permitted. The response in Document 81 is that the actual design would be elastic. It was explained that no yielding for gravity-carrying members would occur. Further ductility would come from those areas that are not essential in carrying gravity. The idea was to keep the main elements, which carry gravity, elastic and come up with a way of letting the bridge move at certain locations followed by repairs of any damage.

On January 4 and 5, 1994, elastic performance in terms of the overall approach to retrofitting the bridge, was defined as follows [Document 303]:

1. *Minor damage to main gravity load carrying members can be tolerated*
2. *Some structural damage in non-gravity load carrying members can be tolerated*
3. *Structural damage that cannot be easily observed shall be avoided*
4. *Underwater damage should be avoided*

Additionally, the seismic performance criteria for retrofitting a major bridge was presented [Document 303] as follows:

1. *Full access to traffic immediately following the event*
2. *Repair with minimum risk of losing functionality*
3. *Structure must stay elastic (functional motion)*
4. *Deck and immediate support system must also remain elastic*
5. *Limit damage to secondary members such as wind bracing*
6. *Minor buckling if no loss in load carrying capacity*
7. *No permanent deflections exceeding a few inches*
8. *Movement at joints will be designed at 1.25 the maximum calculated displacement*
9. *Damage below the water will not be permitted*

In the Project Scope Summary Report dated January 30, 1996 it was stated that the SFOBB qualified as an Important Bridge as defined in the Seismic Criteria for the Design and Evaluation of Bridges. The seismic performance criteria for this important bridge was based on a functional evaluation-ground motion which specifies performance levels equal to immediate service level and minimal damage. The safety evaluation ground motion specifies performance levels equal to immediate service level and repairable damage.

On September 25, 1996, a letter to Caltrans Director from the Seismic Advisory Board stated the following:

> *The Board was particularly interested in the expected performance on the bridge under different earthquake scenarios and the overall reliability of the retrofitted structure. For low seismic excitation no damage and full serviceability is expected; for moderate seismic events damage is expected in the movement joints (fracturing of bolts and finger-joint damage) which will require repair with lane by lane traffic interruptions; in the case of a major seismic event, the structure should not collapse but is expected to show permanent offsets and inelastic deformations in the trusses which will require jacking and centering following the event (provisions for these operations are made in the current retrofit designs). Emergency vehicles should be able to use the bridge after steel plate placement over the damaged movement joints, and re-striping of traffic lanes should allow for a few lanes of traffic in each direction*

> *shortly after the event. Due to the large number of substandard members and connections in the existing bridge structure any isolation and/or retrofit concept will result in a systems reliability which is less than that of a new bridge. On the other hand a staged retrofit program will progressively improve the seismic safety of the bridge, a benefit which will not be derived during the construction period of a replacement bridge."* [Document 372]

The descriptions contained in Documents 372 and 344 appear to describe the Important Bridge criteria.

The criteria for design was not well established or defined even while design and anticipated construction activities for retrofitting were in progress. Variations of the criteria were being introduced as the retrofit design was in progress. On numerous occasions there were request from the Engineering Criteria Review Board to the design team asking for the design criteria that would meet the established performance criteria. The designers for the retrofit were provided few if any numerical parameters that would establish definitive criteria in terms of allowable stress and strain levels and displacement limits. The closest thing to a design procedure was a set of objectives that all elements would remain in the elastic range.

In summary the Seismic Performance Criteria has been discussed and reviewed since 1992 and as such has been open to interpretation and continuous explanation. Several documents are referenced in Appendix 6 that demonstrates the numerous versions of the Seismic Performance Criteria. Examination of these documents demonstrates that the descriptive criteria applied during the retrofit have essentially remained the same. The seismic criteria were presented in terms of performance objectives rather than well defined and concise engineering terms.

What can be extracted from these documents is that to meet lifeline criteria the bridge must be designed and constructed to be elastic and to provide full serviceability immediately following a major earthquake. Minimal damage, defined as damage that will not impede the flow of traffic any time after an earthquake, may be tolerated. Examples could be damage to lampposts, handrails, guardrails, and minor spalling of concrete. When this criterion is applied to the proposed retrofit design it is not difficult to conclude that the selected retrofit design does not meet lifeline criteria. This was basically stated in Document 354 when a lesser criterion was invoked for the retrofit design. However, it appears in Documents 344 and 367 that the current replacement design is describing the performance requirements of an Important bridge. When the previously stated criteria is compared to the current replacement design criteria, a similar conclusion can be made that no lifeline criteria other than a set of objectives has been formulated.

Appendix 8

The following timeline summaries key events in the SFOBB project regarding lifeline criteria. The timeline is organized by date and references the applicable document number. Caltrans provided all referenced documents. Passages taken directly from the referenced documents are provided in italics.

Date	Doc. No.	Comments
1992 06/23/92 Meeting: 06/18/92	372	Seismic Advisory Board - Caltrans Meeting Agenda and Summary Notes *The presentation on the proposed seismic performance criteria for design of new bridges lead to extensive discussion, partly because of different views of what the performance criteria should involve and partly because verification was needed as to implications for design procedures. Finally, there was a meeting of the minds and recast seismic performance criteria will be prepared by James Gates and copies will be circulated to members of the Board.*
1992 08/00/92	9 pg.7	Document 9 contains the beginning strategy of the seismic retrofit for the SFOBB. The document contains various strategies that were explored and in some cases implemented by Caltrans design team. Determining displacement of structural components after an earthquake is the bottom line for determining how close the structural elements of the bridge come to not deforming or deflecting such that the remaining condition is susceptible to failing when additional force is placed on the element. Determining the correct displacement to meet such conditions requires input of ground motions from earthquake analysis. Determining which ground motion is selected will obviously influence what the displacement will be after the analysis. In Document 9 the ground motions selected for the bridge analysis were developed from the concept of "Safety Evaluation Earthquake". Document 9 states: *Due to the importance of the Bay Bridge, a decision was made not to use the concept of "Maximum Credible Earthquake." It is believed that this concept is not adequate for the scope of the studies and the importance of the Bay Bridge. Instead, a decision was made to use the concept of "Safety Evaluation Earthquake" (SEE). According to Bruce Bolt in Reference (18)*, "The SEE is defined as an earthquake whose occurrence is judged to be sufficiently unlikely so that it can represent the maximum earthquake that should be considered in evaluating the safety of the structure." The major difference between Maximum Credible Earthquake (MCE) and the Safety Evaluation Earthquake (SEE) are: (a) the SEE is more forceful and has widespread frequency content to excite structures modes; and (b) the SEE is generated to match a certain spectra as well as include characters of fault and the site.* **Astaneh, A., Bolt, B., Fenvas, G., Fillippou, F., Lysmer, J., Monteiro, P. and*

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Date	Doc. No.	Comments
		Powell, G., (1992) "Seismic Condition Assessment of the Bay Bridge", Research Report in Editing, Earthquake Engineering Research Center, to be released in 1992, University of California at Berkeley.
1993 06/29/93 Meeting: 06/10/93	372	Seismic Advisory Board - Caltrans Meeting Agenda and Summary Notes *6. Caltrans had mailed copies of the "Seismic Performance Criteria" and the Board requested that at the next meeting James Gates should give a technical explanation of this document.*
1993 09/14/93	81 pg. 30 - 31	Meeting of the Engineering Criteria Review Board *Chairman Nicoletti inquired whether there would be a summary of the recommended design criteria with respect to the ground motion and the other parameters.* *Professor Astaneh explained that one volume is basically a summary of the whole package as far as starting from what are the functional criteria that Caltrans established how this bridge should perform, what level of damage is acceptable, which areas can be damaged, but the group will stop short of providing design criteria, which is not its job. It stops at applied design studies.* *The group provides information to the Caltrans team so they can use it to design and for engineering functions.* *Chairman Nicoletti commented that ECRB would be interested in a summary of the hazards that are going to be used in the design, and what the performance objective is with respect to the hazards, and how Caltrans is going to assure itself of having attained this performance objective.* *Mr. Bridwell said he hoped that the teams effort will be continued, and through the research efforts and the Sacramento people, who have been doing quite a lot of work on the analysis of ground motions, it will be a collective effort of the people involved. Hopefully there will be well-established clean criteria how this bridge was retrofitted, what the essential criteria for damage are, tolerance, what will be the expected damage in future earthquakes, etc.* *Mr. Bridwell did not think this information would come from Caltrans alone but through working as a team. Caltrans does the actual engineering design.*
1993 09/14/93	81 pg. 33	Meeting of the Engineering Criteria Review Board *A Caltrans representative observed that the performance is no damage of the structure; it will be open to traffic immediately after an earthquake; basically, virtually elastic performance of the structure.*

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1993 09/14/93	81 pg. 35	Meeting of the Engineering Criteria Review Board *Chairman Nicoletti said...But ECRB would like to see at least a summary of the design criteria. What are the design criteria? What are the performance objectives? What are the acceptance criteria? How do you satisfy yourself that you have attained the performance objective?*
1993 09/14/93	81 pg. 58 - 59	Meeting of the Engineering Criteria Review Board *Mr. Arnold believed that Mr. Bridwell had expressed a general strategy, saying, "we could do this; we could do that; perhaps this might happen," but a little more information about that is needed.* *It seems to be desirable that at some point there should be a general set of criteria, from the very basic one which says that the bridge should remain functional throughout an earthquake, on to what that really means in terms of what happens to the members, what the displacements are, what has to be taken care of, leading to numbers and to a design strategy which says, "if we have displacement of 8 inches or 30 inches, here in conceptually how we propose to take care of it.*
1993 09/14/93	81	Meeting of the Engineering Criteria Review Board *Chairman Nicoletti inquired whether Caltrans was going to use the same ground motion in both directions. Mr. Bridwell said this was correct, but the orthogonal axes are going to be somewhat arbitrary, because one doesn't have a straight line. There is a curve there, and Caltrans is looking at the curve and the forces in both directions, which is a problem.* *Tomorrow Caltrans will meet with the finite elements people on the towers and see if one can't come up with how many towers have to be done. All this costs money and time. Is this apropos to do, or does one have to go over to a tall tower? Then, how does one develop the details?* *All of this can be developed before Caltrans finishes it. Then it is sized to what the forces are. But as to details, one can't remove a bolt and put it in unless one knows exactly where it is, so one does the details of the shop plans, and a lot of times one goes out and measures. If you don't you risk having five holes where you want one.* *Professor Astaneh pointed out that Professor Bolt had undertaken extensive studies of the issue, and he is one who developed the ground motion. He developed what he calls seismic evaluation earthquake, different maximum credible earthquake, and this is quite elaborate, basically starting the spectra*

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		that was chosen to be the spectra for the design, which was the initial starting point of the process.
1993 11/05/93	372	Seismic Advisory Board - Caltrans Meeting Summary Notes *4. Importance Criteria. James Gates gave an update on definition and implementation of Importance Criteria and presented a rough draft. The Board felt that this was heading in the right direction but the rough draft needed a thorough rewriting. This should include a description of the Design Process and could be presented at the next meeting of the Board. The final write-up could include photos and diagrams as examples. The Board feels that a good presentation of Performance Criteria would be great value to Caltrans engineers and to engineering consultants.* *5. ATC -32. It was reported that a first draft of the ACT-32 report on seismic design code was now available. The Board requests that members receive copies of the first draft prior to the next meting.* *6. The Chairman of the Caltrans Research Committee (I.M. Idriss) said that a committee report will be completed by the end of 1993. Copies of the report should be sent to the members of the Board prior to the next meeting.*
1993 11/17/93	81	Letter from Joseph Nicoletti, San Francisco Bay Conservation and Development Commission, to John Amaral, Caltrans requesting information regarding proposed seismic retrofit. *Documentation which demonstrates that the proposed seismic retrofit will comply with the performance objectives stated in the June 2, 1990 Executive Order D-86-90 issued by the governor of California. One of these objectives states that all essential transportation facilities (such as the San Francisco-Oakland Bay Bridge) be functional after the occurrence of a major earthquake.*
1993 11/30/93	81 p.5-6	Summary of the BCDC-ECRB Meeting of September 14, 1993 *Bridwell stated that Caltrans was asking the BCDC for a permit for Seismic Retrofit Project No. 1. Mr. McAdam of Caltrans District 04 stated that Caltrans is poised to submit an application to the BCDC for a permit for Projects Nos. 1 and 2, and will be submitting applications for the other projects over the next year. Chariman Nicoletti stated that the charge of the Board is primarily to review criteria, and implementation of the criteria. The detail plan check, the design check, calculations and specifications should not be done by BCDC or the Board. But the Board would like to see at least a summary of the design criteria, so the performance objectives and acceptance criteria can be reviewed. Mr. Lucia stated that the Board won't be prepared*

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		to talk about a permit until these and other documents have been reviewed. *Mr. McAdam observed that traditionally the Board reviews projects after the BCDC has issued the permits, with the condition in the permit that the permitee convince the Board that the criteria used in designing the project were acceptable. However, the BCDC staff indicated that they would defer to the ECRB for these projects. Chairman Nicoletti indicated that the ECRB wants to review the criteria before Caltrans gets too far into the design, and in advance of any permits being issued. Mr. Wilson indicated that the Peer Review Committee should also be acting on this now.* *Responding to a statement from Mr. Wilson that there was a inconsistency in Professor Astaneh's report, as to whether the retrofit design would be wholly elastic, or would allow some plastic deformation. Mr. Bridwell replied that actual design will be elastic. Professor Astaneh replied that the report envisions no yielding for gravity-carrying members, so ductility should come from those areas that are not essential in carrying gravity. The idea in the report is to keep the main elements that carry gravity elastic and come up with a way of letting the bridge move at certain locations; if one gets damage at these locations, you can come back and change these elements, replacing the damaged details.*
1993 12/03/93	303	Initial Meeting Minutes for Peer Review Panel *Mr. Roberts then explained to the panel that the SFOBB was designated as an important bridge, and as such, would be retrofitted to a higher performance level allowing the main structural members to perform within the elastic range. This means the structure would not sustain damage preventing it from being immediately opened to traffic following a seismic event. Mr. Roberts then asked Jim Gates to review the Seismic Performance Criteria handout which is on of the main items the Panel is expected to review.* *Gates reviewed background material from "Competing Against Time" making particular note of the statement on page 81 that sites a 20 year time frame for designing and retrofitting the major bridges. He then reviewed the Seismic Performance Criteria noting the following requirements for retrofitting a major bridge:* • *Full access to traffic immediately following the event* • *Repair with minimum risk of losing functionality* • *Structure must stay elastic (functional motion)* • *Deck and immediate support system must also remain elastic* • *Limit damage to secondary members such as wind bracing* • *Minor buckling if no loss in load carrying capacity*

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		• *No permanent deflections exceeding a few inches* • *Movement at joints will be designed at 1.25 the maximum calculated displacement* • *Damage below the water will not be permitted* *Gates discussed letter to consultants doing vulnerability studies noting the following points:* • *The Safety Evaluation Earthquake shall be based on the 84th percentile spectra from the deterministic event* • *The functional Evaluation Earthquake shall be based on the spectra for the 300 year return equal hazard event* • *40% chance of occurrence over a projected bridge life of 150 years.* *He also noted that Geomatrix has been given the task of performing hazard assessments for the toll bridges, and University of California at Berkeley (UCB) and United Energy Service were reviewing the Caltrans computer model of SFOBB as well as performing a separate analysis.* *Cooper then discussed the Seismic Design Criteria that was distributed to the Panel. Idriss noted that we would have to address what ends up in the bay when liquefaction occurs.* Noted the difference between performance and design criteria.
1994 01/06/94	303	Peer Review for the Seismic Retrofit of the SFOBB East Bay Spans - Minutes of the January 4 and 5, 1994 Meeting *Professor Hassan Astaneh of U.C. Berkeley, made his presentation: BCDC needed to be told what "elastic" performance means to Caltrans. His definitions were:* *1. Minor damage to main gravity load carrying members can be tolerated* *2. Some structural damage in non-gravity load carrying members can be tolerated* *3. Structural damage that cannot be easily observed shall be avoided* *4. Underwater damage should be avoided*
1994 06/23/94	81	Memorandum to Mickey Horn, Ray Zelinski, Tom Cooper, Brian Maroney from Caltrans Office of Structures Maintenance and Investigations, Subject: BCDC Concerns to be Resolved. *A primary concern raised by the Board members as a result of that presentation was the failure of Caltrans to present concise structural and*

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		seismic design criteria. *Documentation which demonstrates that the proposed seismic retrofit will comply with the performance objectives stated in the June 2, 1990 Executive Order D-86-90.* *This document should include the acceptance criteria used to achieve this performance objective as well as specific criteria used for the basis of the design.* *Mr. Arnold said he understood Caltrans' desire to move ahead with the contracts and said he believed Mr. Bridwell had expressed a general strategy but more information is needed; there should be a general set of criteria, from the very basic one which says that the bridge should remain functional throughout an earthquake, on to what really means in terms of what happens to the members, what the displacements are, what has to be taken care of, leading to numbers and to a design strategy which says, "if we have displacement of 8 inches or 30 inches, here in conceptually how we propose to take care of it.* *Mr. Wilson was concerned about an inconsistency, namely that it was presented that there would be an elastic design, basically no damage, and yet in figure 27 there is a curve, showing some permanent deformation in the structure that goes from -10 inches to +10 inches, with semi-rigid plates.*
1994 07/14/94	81	Engineering Criteria Review Board Meeting Notice - Attached Presentation Titled " Caltrans Agenda for the ECRB of the BCDC Meeting on 7/27/94. *San Francisco-Oakland Bay Bridge - East Approaches* ***Function Requirements*** ***Immediate Service Level - Safety Evaluation*** *- Full access to normal traffic within 72 hours (repairable damage)* *- Emergency access immediately* ***Immediate Service Level - Functional Evaluation*** *- Full access immediately* *- Essentially elastic* ***Performance Objectives*** *- Essentially elastic under Safety Evaluation Earthquake (1000yr. return)* *- Minimal architectural changes* *- Minimal interruption to traffic* ***Seismic Design Criteria*** *Bridge Category: Important* *Service Level: Immediate* *- Structure to remain serviceable following the "Safety" level earthquake (maximum credible seismic event) with "repairable" damage.* *- Structure to sustain "Minimal" damage following the "Functional" level*

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		earthquake. *Analysis Methods* *- Equivalent Static Analysis* *- Elastic Dynamic Analysis, assuming effective moments of inertia* *- Inelastic Static Analysis (for components)* *- Elastic Dynamic Analysis - does not apply to a structure designed in the elastic range*
1994 09/12/94 Meeting: 07/27/94	81	BCDC ECRB Meeting Minutes July 27, 1994 *James Roberts opened the Caltrans presentation by offering an introductory statement on the purpose of Caltrans' presentation and some introductions.* *Brian Maroney presented the ECRB with an overview of the complete structure and the analysis strategy Caltrans will use to develop the appropriate retrofit strategy. He also provided the board with a tentative schedule for the bridge:* *PS&E - June 1996* *Complete Construction - December 1997* *Ken Jackura followed Brian's presentation with a detailed review of the foundation analysis techniques used by Caltrans to design the foundations for the project under review for the permit, the East Approach to SFOBB.* *Finally, Tom Cooper presented the Board with the design criteria was developed and implemented for the East Approach Seismic retrofit. The retrofit strategy, along with details and computer enhanced photos of the proposed retrofit were also presented to the Board at this time.* *Following the Caltrans presentations, a short question and answer period ensued.* *The Board then brought the permit to a vote and passed the resolution (5-0) to recommend to the Commission that the permit be approved.*
1994 10/27/94 Meeting: 10/21/94	303	SFOBB Peer Review Meeting Summary *There was a short discussion concerning the life span of the SFOBB and its seismic design life span. Chuck stated that the life span of the SFOBB is multi-hundred years, however, the seismic design life used on the Golden Gate Bridge is considered to be forty years. This reduction is due to the changing technology. It is not desirable to have a retrofit that will be obsolete in five years. It is necessary to give the bridge retrofit a reasonable, durable life. This duration does not imply the next review to be forty years from now, the review should follow the changing technology. Professor Seible indicated*

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		that it is important to insert this kind of statement into the final report as toll bridges are special structures with special rules. He also suggested that the Seismic Advisory Board address this issue. *The performance objectives of the SFOBB were also reviewed. The panel had several suggestions for this document. Brian volunteered to revise the performance objectives in accordance with the panels' suggestions.*
1995 05/30/95 Meeting: 05/23/95	303	SFOBB Peer Review Meeting Summary, 5/23/95 *Ray Zelinski provided information regarding the progress of the East Bay spans contracts and the alternatives currently being investigated. Our strategy has been altered and, consequently, the performance criteria has been reduced. A staged retrofit of the superstructure is the new plan. As a result of the revised performance criteria, a few PS&E dates may slip. Currently, there is no staff assigned to the cantilever truss span contract (contract #8).* *Dr. Seible suggested possibly using a higher damping value as the damping helps the rocking models. He suggested using 7% or 8% as opposed to 5%.*
1995 06/00/95	117	*To: All designers evaluating the seismic vulnerability of the San Mateo-Hayward Bridge, the Richmond San Rafael Bridge, the Carquinez Bridges, the Benicia-Martinez Bridge, the San Diego Coronado Bridge and the Vincent Thomas Bridge.* *The CALTRANS Seismic Performance Criteria (attached), requires two separate analysis:* *Safety Evaluation* *Functional Evaluation* *The exact performance level required for each of the bridges has not yet been defined. The final performance level required will be determined after an evaluation of the analysis for the safety and functional ground motions for the minimum performance level. In all probability, the performance level selected will be higher than the minimum, but below that required for an important bridge. The following earthquakes shall be used for the seismic evaluation of these Toll Bridges:* *1. The Safety Evaluation earthquake shall be based on the "Target Response Spectra".* *2. The Functional Evaluation earthquake shall be based on the spectra for a 285-300 year return equal hazard (probabilistic) event.*

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		The spectra and corresponding time histories for all of these events shall be those previously specified in the site specific hazard studies prepared for each bridge or as modified by subsequent evaluation as part of this project. *Discussion:* *For the San Francisco Bay area, the 84th percentile rock motion spectra for the maximum credible event on the San Andreas fault corresponds approximately to the 1000-2000 year return period equal hazard spectra and were selected as target spectra. This appears to be a reasonable choice for large bridge in the San Francisco Bay Area and is in agreement with the current Performance Criteria.* *For the San Diego and Long Beach areas, motions a little below the 84th percentile deterministic rock motion spectra were selected for the target spectra. This corresponds approximately to the 1000-2000 year return period equal hazard spectra. This appears to be a reasonable choice for large bridge in the Long Beach and San Diego areas and is in agreement with the current Performance Criteria.* *The useful life for average bridges specified by AASHTO is 75 years. Major bridges are not mentioned. A reasonable choice for useful life for these facilities would be about two times that of an average bridge or 150 years. The Performance Criteria for a Functional event requires that there must be a 60% Probability of the ground motion not being exceeded during the useful life of the structure. This means that the return period of the Functional Evaluation earthquake computes to be.* *-150/ln(0.6) or -150/-0.511 = 293 years (Say.. 285 to 300 years).* *SEISMIC PERFORMANCE CRITERIA FOR THE DESIGN AND EVALUATION OF BRIDGES* *Ground Motion at Site* — *Minimum Performance Level* — *Important Bridge Performance Level* *Functional Evaluation* — *Immediate Service Level, Repairable Damage* — *Immediate Service Level, Minimal Damage* *Safety Evaluation* — *Limited Service Level, Significant Damage* — *Immediate Service Level, Repairable Damage* *DEFINITIONS* ***Immediate Service Level:*** *Full access to normal traffic available almost immediately.* ***Limited Service Level:*** *Limited access, (reduced lanes, light emergency*

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		traffic) possible within days. Full service restorable within months. ***Minimal Damage:*** *Essentially elastic performance.* ***Repairable Damage:*** *Damage that can be repaired with a minimum risk of losing functionality.* ***Significant Damage:*** *A minimum risk of collapse, but damage that would require closure for repair.* ***Important Bridge*** *(one or more of the following items present):* *Bridge required to provide secondary life safety.* *(example: access to an emergency facility).* *Time for restoration of functionality after closure creates a major economic impact.* *Bridge formally designed as critical by a local emergency plan.* ***Safety Evaluation Ground Motion*** *(Up to two methods of defining ground motions may be used):* *Deterministically assessed ground motions from the maximum earthquake as defined by the Division of Mines and Geology Open-File report 92-1 (1992).* *Probabilistically assessed ground motions with a long return period (approx. 1000-2000 years).* *For bridges above the Minimum Performance Level, both methods shall be given consideration, however the probabilistic evaluation shall be reviewed by a CALTRANS approved consensus group. For Minimum Performance level bridges, the motions shall be based only on the deterministic evaluation. In the future, the role of the two methods for these bridges shall be reviewed by a CALTRANS approved consensus group.* ***Functional Evaluation Ground Motion:*** *Probabilistically assessed ground motions, which have a 60% probability of not being exceeded during the useful life of the bridge. The determination of this event shall be reviewed by a CALTRANS approved consensus group. A separate Functional Evaluation is required only for Bridges above Minimum Performance Level. Minimum Performance Level bridges are only required to meet specified design requirements to assure Minimum Functional Performance Level compliance.*
1995 07/05/95 Meeting: 06/20/95	303	Peer Review for Seismic Retrofit of Toll Bridges Meeting Minutes for the June 20, 1995 Meeting. *Brian Maroney* • *Described the San Francisco Oakland Bay Bridge (SFOBB) performance criteria*

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		• *SFOBB original performance criteria resulted in a costly retrofit strategy* • *Adjustments will have to be made but in no case will the performance level be allowed to drop below the minimum performance level* *Maury Power - Geomatrix* • *Presented scope of Seismic Motion studies for all Bay Area toll bridges* • *84th percentile rock motion spectra was selected as a target spectra for Maximum Credible Earth Quake* • *Discussed the impact that local faults had on the Bay Area toll bridges* • *Discussed some possible supplemental seismic ground motion studies for SFOBB seismic retrofit design* • *Discussed the possibility of using more than one time history*
1995 07/21/95 Meeting: 07/13/95	303	SFOBB Peer Review Meeting Summary, 7/13/95 - Room 104 *Ray Zelinski introduced a new strategy to the Panel for the East Bay spans. His is redirecting efforts from the rocking tower strategy because the forces could not be reduced, modifications required to the towers would be very costly, and the various complexities that existed in capturing an accurate model. We will now pursue a strategy of isolating the superstructure from the tops of the towers.*
1995 08/17/95 Meeting: 08/17/95	303	Peer Review - Toll Bridge Design Criteria *2. SFOBB: 27 psf is the ave. car/truck mix on the bridge at any one time on each deck.* *3. "No Collapse" criteria unless ramps affect mainline service ramps at each bridge must be reviewed and ID'd for FEE.* *15. Damping — ABBAS used sensitivity studies between 2 and 5% to verify choices.*
1995 09/05/95 Meeting: 07/17/95	372	Seismic Advisory Board - Caltrans Meeting Agenda and Summary Notes <u>*Lifeline Routes*</u> *Jim Gates presented an update on Life Line Routes as required by the Caltrans Strategic Plan dated October 1994. He identified 28 routes in the State as of July 1995 as being in the Life Line category. Since the focus of this effort is to identify critical routes that will allow for the immediate movement of emergency equipment and supplies into a region or through a region and*

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		to keep them open following an earthquake. The Board strongly supports this requirement of the Strategic Plan.
1996 03/16/96 Meeting: 03/15/96	372	Seismic Advisory Board - Caltrans Meeting Agenda and Summary Notes *Jim Roberts gave the general introduction to all toll bridge retrofit projects emphasizing that only the Benicia-Martinez Bridge is part of a designated lifeline route and will therefore be retrofitted to higher standards, namely a full service ability criterion to ensure full public access following inspection after a safety evaluation seismic event. The other toll bridges will be retrofitted to satisfy no collapse and minimum performance criteria but with higher emphasis placed on serviceability for the San Francisco-Oakland Bay Bridge due to its regional economic importance.*
1996 06/06/96 Meeting: 05/21/96	372	Caltrans Seismic Advisory Board *This meeting is now judged by those in attendance as having been extremely informative and helpful in providing a consistent analysis approach to all retrofit designs based on a common understanding of the need for nonlinear analysis in predicting expected seismic response of these structures. Based on this results on this meeting, Caltrans will issue a memorandum to the consultants and their in-house designers on the need, level, and application of nonlinear analysis models in support of the toll bridge retrofit designs.*
1996 08/05/96	372	Letter to James Roberts, Director, Caltrans from Joseph Penzien, Chair, Seismic Advisory Board *The value engineering report, however, points out extremely large differences in cost for the evaluated design concepts and it is difficult to see and understand the differences of this magnitude (factor of 4) can result from the same criteria, assumptions, and guidelines.*
1996 09/25/96	372	Letter to James Van Loben Sels, Director Caltrans from Seismic Advisory Board *An update on the status of the retrofit design was given by Mr. Ray Zelinski. The Board was particularly interested in the expected performance on the bridge under different earthquake scenarios and the overall reliability of the retrofitted structure. For low seismic excitation no damage and full serviceability is expected; for moderate seismic events damage is expected in the movement joints (fracturing of bolts and finger-joint damage) which will require repair with lane by lane traffic interruptions; in the case of a major seismic event, the structure should not collapse but is expected to show permanent offsets and inelastic deformations in the trusses which will require jacking and centering following the event (provisions for these operations are*

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		made in the current retrofit designs). Emergency vehicles should be able to use the bridge after steel plate placement over the damaged movement joints, and re-striping of traffic lanes should allow for a few lanes of traffic in each direction shortly after the event. Due to the large number of substandard members and connections in the existing bridge structure any isolation and/or retrofit concept will result in a systems reliability which is less than that of a new bridge. On the other hand a staged retrofit program will progressively improve the seismic safety of the bridge, a benefit which will not be derived during the construction period of a replacement bridge. *The Seismic Advisory board re-emphasized the need for additional time histories to be used in the final evaluation of the toll bridge retrofit designs due to the expected noninearities in the structural response. Now that the ad hoc soil-foundation-structure-interaction committee has provided guidelines on a consistent rock and ground motion development these guidelines should be applied to at least one of the toll bridge retrofit projects as soon as possible to assess the impact and importance of additional time history analyses. Benicia-Martinez was proposed as a good candidate for such a study since the design is almost complete and the designation as a bridge on one of the identified lifeline routes necessitates this added design scrutiny. Caltrans will explore this proposed strategy.*
1996 10/09/96	117	Memorandum from Mike Van De Pol Team Leader, No-Drop Retrofit Strategy Development Longitudinal Superstructure Response Investigation - Subject: ADINA Model Output Data Request *The exact extent of data needs is not fully known at this time, since all of my efforts, have been directed to the no-drop investigation, to this date. The following list may be to broad for our ultimate needs, however for now, I am proposing it.*
1996 11/11/96	117	Memorandum from Jason Lynch to Transverse Portal Group *Ductility. The current strategy depends on the portal frames to resist transverse forces in the superstructure. To relieve high elastic forces, some portals may have to be designed to go plastic. This is a change from the original SFOBB retrofit philosophy, which required full serviceability after an earthquake. By targeting specific areas for inelasticity and by providing repair details, we will relax the criteria. Displacement ductiles should be kept to below 2-3. We should anticipate a permanent set in the damaged members equal to about half of their maximum excursion.*
1996 12/10/96	372	Letter to James van Loben Sels, Director Caltrans from Seismic Advisory Board

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		1. Seismic Safety and Reliability - it is our opinion that Caltrans' engineers are employing state-of-the-art technology and recent research results in the retrofit design for the existing bridge to ensure safety (no collapse) during a maximum expected earthquake for this location, but the expected performance envisions the occurrence of a major traffic interruption following the earthquake. A new bridge could however be designed with much greater reliability to respond favorably to a major earthquake of this type.
1996 12/10/96 Meeting: 11/26/96	372	Letter to James van Loben Sels, Director California Dept of Transportation *For a replacement structure, the decision process starts with the type selection, the alignment, and the desired performance criteria, but must consider environmental and permit issues as well as legal issues, both in terms of fiscal impact and potential construction delays. In addition to the initial construction costs, one of the key evaluation issues is the potential for earthquake exposure in case construction of the replacement bridge is delayed. Discussions focused on the fact that in the retrofit option for the existing bridge so many uncertainties exist regarding the actual state and remaining service life of many of the existing members and the performance of the new seismic protection devices, that current cost and construction time estimates for retrofitting the existing bridge contain a significantly higher degree of uncertainty than does the same estimates for a replacement structure. The Board also commented on the probabilistic nature and uncertainties in all other decision tree parameters and indicated that, because of these uncertainties, the current time and initial-cost differences for the retrofit versus replacement options are well within the margins of error. On the other hand, life cycle cost evaluations based on the remaining service life of the existing bridge, the necessary maintenance and upkeep for the old structure, the need for repositioning and repair of the retrofitted structure following moderate or large earthquake, and the indirect costs from lane closures in terms of socio-economic impact to the Bay area community, all clearly favor the replacement bridge option.*
1996 12/20/96	117	Peer Review - 504's and 288's Trusses *J Avila - 504s (Project #9)* *Ray's Performance Criteria — Full serviceability except for expansion joints.* *Freider questions why we'll have different performance criteria on adjacent bridge spans we're designing the isolation system for the <u>big</u> EQ for moderate EQ the bridge will be all over the place...dampers could really help here* *FS suggests that we consider allowing some damage in 504s/288s* *Ray says - good points. Regardless, we're improving the whole bridge - it gets closer to repair that way. We may revisit if we get cash flow problems.* *Chuck Seim wants dampers (especially to save the carbels)*

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		Ray says we haven't ruled it out. We anticipate them at E1, E4"
1997 01/21/97	354	*Performance Criteria Statement — SFOBB East Span Retrofit, Ray Zelinski* *The original performance goal was to provide full serviceability immediately following the safety evaluation earthquake (SEE). This goal was in accordance with the recommendation from the Governor's Inquiry Board following the 1989 Loma Prieta Earthquake. However, a series of analyses for varying strategies and cost-benefit studies caused Caltrans management to retreat from this idealist goal. The current goals will allow access to emergency vehicles within hours of the event; limited public traffic with one month; 3 lanes of public traffic each direction after 6 months; and full traffic after one year. The goal is designed to be consistent with expected damage on both sides of the bay in an SEE event, and the limited need for public traffic to cross the bridge under those circumstances.*
1997 01/30/97	372	Letter to James Van Loben Sels, Director *First, we wish to compliment Caltrans, under your guidance, on its outstanding overall effort in performing an unprecedented engineering feat that has never been done before in the history of bridge engineering—the seismic retrofit of very large, important, long-span bridges. Engineers from around the world are literally watching the good performance of Caltrans and its consultants in this exceedingly difficult undertaking.* *Our letter of January 8 was intended to advise you and your staff that to insure reliable performance evaluations of highly non-linear bridge structures, the use of at least three independent sets of spectrum-compatible ground motions are required. The seismic retrofits for all seven toll bridges use non-linear seismic devices and other non-linear structural elements.* *We now understand that Caltrans is presently planning to develop and apply additional ground motions and we certainly endorse that effort. Our main concern is that the two additional sets of ground motions be used to check the retrofit designs on a timely basis so as to avoid issuing costly change orders during construction.*
1997 10/08/97 Meeting: 10/07/97	372	Letter to Mr. James Van Loben Sels, Director *Consistent with the Caltrans two level Safety Evaluation Earthquake (SEE) and Functional evaluation Earthquake (FEE) assessment and design approach, Caltrans is also proceeding with the development of seismic hazard maps for the FEE event which are to be based on a 40% occurrence during the expected service-life of the bridge. The approach outlined to establish these FEE hazard maps is of a similar deterministic nature to that*

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		used for establishing the current SEE contour maps. This issue was again further discussed by the SAB in executive session. Additional information items on seismic hazard activities concerned ongoing correlation studies between observed damage, repair costs, and measured maximum ground shaking intensities during recent earthquakes, as well as collaborative hazard definition studies with Japan.
1997 12/26/97 Meeting 12/15/97	372	*Letter to James Van Loben Sels, Director* *<u>Effect of New Motions on the Response of the SFOBB West Bay Spans</u>* *However, due the to the conservative assumptions used in the original design motion development, only a few changes need to be made in the retrofit design for the SFOBB west bay spans as a result of using the new three sets of time-history inputs."* ***SAB Executive Session*** *The SAB recommends that Caltrans adopt a seismic design policy which states that for major bridge projects where site specific ground motions and time-history analyses are used, three sets of uncorrelated ground motions should be employed, based on the guidelines provided in an earlier report by the SAB Soil-Foundation-Structure Interaction Ad Hoc Committee. These three sets of input motions should be applied to a global model representing the final design to check that all critical design quantities show sufficient available capacities to accommodate the maximum response values obtained using the three sets of time-history inputs. This policy should apply to retrofit and new bridge designs alike. For new bridge design, even though in some cases essentially elastic material performance may be specified, the combination of the two horizontal and the vertical components of input motion provides sufficient variability for each of the three sets of input motion to reveal any special dynamic response effects resulting from bridge geometry, expansion joints, etc., which may not be apparent using a single set of time-history inputs.*
1998 09/24/98	276 P. S-2	*The Retrofit Existing Structure Alternative would retrofit the existing bridge to withstand an MCE, but the bridge would most likely experience substantial damage. The Retrofit Existing Structure Alternative would not permit changes to the existing bridge: therefore, current design standards could not be attained.*
1998 09/24/98	276 P. 1-2	*1.2.1. "Lifeline" connection—The existing SFOBB East Span does not provide a "lifeline" connection that is usable after an MCE.* *Improvements to the existing East Span are needed to address seismic safety deficiencies and provide a bridge crossing that is usable soon after a major*

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		seismic event. It is likely that the existing SFOBB East Span would develop multi-span failures leading to collapse and loss of life in the event of an MCE. The East Span does not provide for public safety during an MCE.
1998 09/24/98	276 P. 1-7	*On the existing SFOBB East Span, an MCE could cause catastrophic bridge failure, potentially resulting in numerous immediate casualties and requiring many months to reopen the bridge or years to build a replacement. Immediate emergency response and more long-term economic recovery would be delayed.*
1998 09/24/98	276 P. 1-8	*The Caltrans formula takes into account both construction costs and life-cycle costs[8]. Cost comparisons of retrofit and replacement alternative indicate that seismic retrofit of the existing span could be accomplished at a lower cost than the cost to replace the structure.*
1998 09/24/98	276 P. 1-9	*On completion of the West Span Seismic Retrofit Project, lifeline access will be provided between San Francisco, the San Francisco Peninsula, and YBI and ensure that bridge damage during an MCE would not affect navigation in the ship channel underneath the structure.*
1998 09/25/98 meeting 09/10/98	372	Letter to James Van Loben Sels, Director *<u>Caltrans Minimum Seismic Design Criteria</u>* *In an effort to update, unify, and consolidate bridge seismic design practice at Caltrans, a new seismic bridge design document has been drafted and is under review in various ESC offices. Mr. Tom Ostrom, the lead engineer in this effort presented to the Board the latest version of the Caltrans Seismic Design Policy and the Minimum Seismic Design Criteria (MSDC). The document reflects a shift in the Caltrans seismic design philosophy away from force based design concepts to a displacement and deformation based design approach, reflecting recent developments in research and design practice. The Board fully concurs with this shift in design approach since a displacement based design is less sensitive to variations in input motions and it allows better conformance with specified performance levels. The SAB also concurs with the development of MSDC for ordinary standard bridges as presented by Mr. Ostrum, but would like to see a more detailed road map for bridge seismic design in general which also addresses non-standard and important bridge structures such as long span bridges. To this effect the SAB suggests dividing the presented document into a general document or road map addressing policy, design philosophy, performance requirements, design criteria, and design procedures and a separate document for the individual bridge categories and classifications such as the MSDC for ordinary standard bridges.*
1999 10/14/99 Meeting 09/07/99	372	Letter to Mr. Jose Medina, Director *(e) <u>Signature Bridge</u>: Mr. Rafael Manzanarez from T.Y. Lin International presented the 65% design of the 617 m long signature bridge with 180 m and*

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		385 m self-anchored suspension spans. The superstructure consists of two parallel orthotropic steel boxes connected every 20 m by 10 m wide and 5.5 m deep cross beams. Vertical hangers are spaced 10 m apart and supported by a continuous main cable which has a loop anchor at the west pier (W-2) and splayed individual strand anchors inside the steel box past the east pier (E-2). The SAB questioned the dead load and earthquake load stability of the free spanning superstructure under the high internal compression forces generated by the self-anchored suspension bridge concept and requests a separate technical briefing on this important structural behavior issue in one important structural behavior issue in one of the next SAB meetings.
2000 06/27/00	344	San Francisco—Oakland Bay Bridge East Spans Seismic Safety Project Seismic Design Criteria ***1. Facility Classification — Lifeline Route*** *A need is recognized for a route 80 across the San Francisco Bay as a lifeline transportation facility.* - *No convenient alternative* - *Economic consequences of failure are large* - *Provide secondary life safety* - *Designed as important by local emergency officials* *All above criteria are likely satisfied.* ***2. System Performance*** *The most fundamental measure of post-earthquake performance for a bridge system is allowable traffic flow. The second level of measure is cost (which incorporates difficulty) of repair to the system. Both of these measures will be addressed in this design.* *Table 1. Proposed Seismic Performance Criteria* ***Seismic Event (attenuation)****: Safety Event (mainline)* ***Base Rock Motions at Site Reference Point****: 1500 year return period motions at site reference points.* ***Important Bridge Performance Level*** *for unusually important routes and bridges: minor to moderate damage**; slowed speeds, but operational within hours; fully operational in months.* ***Minimum Performance Level*** *for standard routes and bridges: NA* ***Seismic Event (attenuation)****: Safety Event (temp. detour structures)* ***Base Rock Motions at Site Reference Point****: Motions which have a 10% probability of occurring in about 10 times design life of 1 year* ***Important Bridge Performance Level*** *for unusually important routes and bridges: NA*

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		Minimum Performance Level *for standard routes and bridges: major damage; no collapse/no loss life* ***Seismic Event (attenuation)****: Safety Event (YBI ramps)* ***Base Rock Motions at Site Reference Point:*** *300-500 year return period motions at site ref. points* ***Important Bridge Performance Level*** *for unusually important routes and bridges: NA* ***Minimum Performance Level*** *for standard routes and bridges: major damage; no collapse/no loss of life* ***Seismic Event (attenuation)****: Functional Event (mainline)* ***Base Rock Motions at Site Reference Point:*** *Motions which have a 50% probability of occurring 2 or more times in 150 years.* ***Important Bridge Performance Level*** *for unusually important routes and bridges; minor damage**; slowed speeds,but operational within hours; fully operational in months* ***Minimum Performance Level*** *for standard routes and bridges: NA* *** See Table 2 for Categorization of Bridge Earthquake Damage Levels* *The bridge system shall be designed in such a way as to provide for the traveled way post-earthquake geometry to allow for use of all traffic lanes at reduced speeds in a reasonably short amount of time.* *Developing Post Earthquake Scenario For the East Spans* *Checking compatibility and acceptance* ***1. End of First Day*** *A.) Steel plates down (if necessary) and transportation link operational at reduced speeds.* *B.) Emergency contract engaged* *C.) Immediate and focused damage inspection phases have been completed* *D.) Smart bridge diagnostics evaluated and incorporated into field review* ***2. End of First Week*** *A.) Detailed damage inspection completed across bridge* *B.) Joint geometric survey complete* *C.) Original deck joint designs modified to be compatible with (2-A)* *D.) Fabricator identified and emergency delivery schedule negotiated* *E.) Deck joint replacements ordered* *F.) Contractor fully mobilized on site* *G.) Repair of damage throughout bridge designed repair plans part of as-builts* ***3. End of Second Week*** *A.) As traffic allows steel plates lifted and preparation of deck for joint*

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		replacements begun; plates put back into place until delivery *B.) Repair to the damage on the island transition has begun* *C.) Repair of the damage along skyway has begun* *D.) repair of damage in main tower has begun* ***4. End of Second Month*** *A.) Transport of replacement deck joint assembles to site has begun* *5. **End of Third Mont**h* *A.) Routine construction sequence of deck joint replacement is designed and underway on the bridge.*

Appendix 9. Supplemental Questions

Supplemental Question from City of San Francisco and Responses

Supplemental Question 1. Can the existing East Span be retrofitted to a comparable level as the West Span permanent retrofit to withstand the same intensity earthquakes in the Bay Area?

***Conclusion*:** A conclusive affirmative answer to this question cannot be proven at this time because a viable, substantiated retrofit design and related cost estimates have not been completed by Caltrans or any other party. A retrofit scheme that would provide the same level of safety as that of the retrofitted West Span is certainly possible given enough time and money to develop a solution. A separate detailed study would be required to develop a scheme that does not focus on isolation of the superstructure and to produce a workable alternative.

In the answer to the Scope of Work s Question 1, it is stated that Caltrans documents indicate that the retrofit design (proposed retrofit alternative) did not meet lifeline criteria. Likewise, the first conclusions to the Scope of Work s Questions 1 and 2 states that the documents provided did not demonstrate that any retrofit alternative met lifeline criteria. These statements are conclusions based on the documentation provided by Caltrans, and should not be interpreted as a statement that a workable lifeline retrofit is impossible.

Supplemental Question 2. Is Caltrans retrofit design as described in the DEIS for the East Span the most reliable system to be used at this location and for this steel structure?

***Conclusion*:** The selected strategy of isolating the superstructure in the truss spans is not reasonable, the retrofit design for the cantilever portion of the bridge is far from complete, and the reliability of any component of the proposed retrofit has not been adequately demonstrated. See Main Report and appendices.

Supplemental Question 3. Is Caltrans retrofit design for the East Span the most time and cost-effective retrofit?

***Conclusion*:** The COE Team reviewed cost data for only one retrofit design, and that design was incomplete. Without further data, it is not possible to make comparisons to answer this question. See Main Report and Appendix 7.

Supplemental Question 4. Is Caltrans estimate of $900 million for seismic retrofit of the East Span based on realistic cost figures?

Conclusion: See Appendix 7 for discussion of costs figures (items) used in the cost estimates.

Supplemental Question 5. Has there been any steel bridge, comparable to the East Span, anywhere in the world that is seismically retrofitted using concrete as Caltrans is suggesting for this bridge?

Conclusion**:** This question cannot be answered without completing a detailed literature search that is outside the scope of work.

Supplemental Question 6. Can the existing East Span be retrofitted with steel?

Conclusion**:** A complete alternative retrofit design using only steel has not been completed at this time. A steel retrofit, however, is certainly possible.

Supplemental Question 7. Is the partial concrete encasement retrofit for the East Span, and addition of concrete piers under the Cantilever truss, the most cost-effective, time efficient and seismically reliable solution? Is this approach standard in the industry? Has this approach been used in any other steel bridge in seismic or non-seismic regions?

Conclusion**:** Given the information provided to and reviewed by the COE team, this question cannot be answered, because there are no complete alternatives for comparison. The following comments, however, can be made.

The decision to avoid a typical strengthening approach was not adequately documented. A seismically reliability analysis of the design was not adequately addressed in the documents provided. It is the COE Team s opinion that this scheme is not the most seismically reliable solution.

There are few bridges in the world that can compare in size and complexity to the East Span of the SFOBB. Although standard practice and procedures should be utilized where applicable, retrofitting a structure of this scale is by no definition standard. Adding large concrete piers under cantilever spans is not industry standard.

Use of concrete encasement has been used in various situations involving retrofit of steel bridges. It is unlikely, however, that the concept of introducing towers to support the suspended span has ever been used.

Supplemental Question 8. With regard to the soon-to-be completed interim retrofit by Caltrans, what work, if any, is needed to further strengthen the current interim retrofit on the East Span to the level of seismic safety of the West Span? What would be the cost of further strengthening?

Conclusion**:** This question cannot be fully answered without completing a detailed study that is outside the scope of work. Information on the interim retrofit and West Span retrofit were not provided.

Supplemental Question 9. With regard to seismic safety and stability during maximum credible earthquakes, is the proposed self-anchored, asymmetric, single-tower suspension bridge as reliable or more reliable than other standard long span bridges (e.g., standard suspension bridges like the Golden Gate Bridge and West Bay Bridge)?

Conclusion: A reliability / stability analysis for the proposed bridge and other standard long span bridges for a MCE event is out of the current Scope of Work.

Supplemental Question 10. Is there any data on the actual seismic performance of a self-anchored single tower bridge resembling the proposed Caltrans self-anchored bridge? If not, what are the reasons for using such an unknown system for the most trafficked and perhaps most critical U.S. Bridge located just a few miles from two of the most active faults (Hayward and San Andreas)?

***Conclusion*:** This question is outside the current scope of work. Reasons for selecting this system cannot be answered at this time since no correspondence has been provided to the team relative to this issue.

Supplemental Question 11. To what magnitude of earthquake is the proposed self-anchored East Span bridge being designed?

***Conclusion*:** The answer to this question is addressed in Appendix 6, Section 1.D.

Supplemental Question 12. Why are permanent ground displacements, expected to occur at the site, not considered in the design of the proposed self-anchored East Span bridge?

***Conclusion*:** The possible effects of permanent ground displacements that occur in addition to tectonic differential movements have not been addressed. This should be addressed as stated in the recommendations of the main report and also the appendices.

Supplemental Question 13. The proposed self-anchored single tower bridge is designed to experience significant yielding of steel damage in the main tower as well as damage to reinforced concrete support columns. The damage, according to designers' published papers, will be significant enough to require replacement of the members that connect the four legs of the main tower. Is there any long span bridge designed anywhere in the world (seismic or non-seismic regions) that is designed to sustain yielding in its main tower to the extent that some pieces of the tower will need to be replaced?

***Conclusion*:** The main tower is designed to remain essentially elastic and not to yield. The question is in error. The shear links will yield and are designed to move possible damage to repairable zones. To answer this question regarding other bridges in the world, requires a detailed literature search that is outside the scope of work. Discussion relative to the single tower bridge and reinforced concrete support columns is presented in Appendix 6.

Supplemental Question 14. On a scale of A to F weighing seismic safety reliability and cost efficiency, where A is the best system and F is unacceptable, how would you rate the standard anchored suspension bridge and the proposed self-anchored, asymmetric single tower, pile-supported East Span replacement.

Conclusion: The rating requested in this question is outside the current Scope of Work. In general terms, there are advantages and disadvantages regarding cost and seismic reliability associated with both bridge types. A weighing of seismic reliability cannot be made since no analysis has been provided for evaluation.

Supplemental Question 15. In the event of an MCE earthquake, Caltrans states that the damage expected to occur on the self-anchored suspension bridge will not be significant. Is that acceptable if the East Span is designated as a lifeline bridge? Will the new span be passable and safe immediately following an MCE earthquake as Caltrans has promised the public? Please catalogue the total extent of damages to the new span after an MCE earthquake?

Conclusion: The bridge has not been designed for an MCE event. The performance criteria requires full service almost immediately and only repairable damage . Cataloguing the total extent of damages is outside the current scope of work.

Supplemental Question 16. Please prepare a comparative seismic safety assessment of the stability of the proposed self-anchoring design vs. a retrofitted East Span. This comparison should take into consideration impacts and changes to: 1) Bay Mud's ancient Temescal Creek in future years; and 2) possible permanent ground displacements caused by Bay area earthquake along the Hayward and San Andreas faults.

Conclusion: The design for the SAS is not complete, and the retrofit alternative design is incomplete. Therefore, a comparative study regarding seismic safety is not possible at this time. Such a study is outside the Scope of Work.

Supplemental Question 17. In accordance with federal environmental laws, how can the replacement of the existing Bay Bridge, a historic landmark, be justified, when a retrofit would preserve the existing East Span?

Conclusion: The historic landmark status of the East Span is outside the current Scope of Work.