Steel TIPS

STRUCTURAL STEEL EDUCATIONAL COUNCIL

TECHNICAL INFORMATION & PRODUCT SERVICE

August 2006

Alfred Zampa Memorial Steel Suspension Bridge, 2003, California, USA: A Fitting Legacy for Ironworkers

By

Alfred Mangus, PE and **Sarah Picker, PE**

Bridge Engineer — Consultant

Alfred Zampa Memorial Steel Suspension Bridge, 2003, California, USA: A Fitting Legacy for Ironworkers

By Alfred Mangus, PE and Sarah Picker, PE

Summary

This paper discusses the successful uses of fabricated steel on the Alfred Zampa Memorial Suspension Bridge. It also discusses others reasons the bridge is a success. This paper is an invitation to engineers, transportation agencies, planners, and others to consider steel for future bridges.

First Printing, September 2006

Authors:

Alfred Mangus P.E.
Phone: (916) 366-5736, e-mail: mangusalf@aol.com

Sarah Picker, P.E.
Oakland, CA Phone: (510) 597-0656 email: routec@yahoo.com

ACKNOWLEDGMENTS

Funding for the publication was provided in part by the California Iron Workers Administrative Trust. The publication of this report was made possible by the support of the Structural Steel Educational Council (SSEC).

The authors would like to thank SSEC for their interest in steel bridges. Particular members whose input encouraged, shaped, and improved this report are Jay Murphy, Abolhassan Astaneh-Asl, Ph. D., Jeffrey Eandi (current chair of the SSEC), Fred Boettler, SSEC Administrator, Brett Manning and James J. Putkey.

Mr. Jay Murphy, the owner of Murphy Pacific Corporation, has been a member of SSEC for many years. In addition, he has shared his knowledge about steel bridges at numerous meetings and conferences held by AISC/NSBA, FHWA, and ASCE, such as www.orthotropic-bridge.org. He inspired this report.

The authors wish to thank Greg Forbes from Nesco/XKT, Galen Jaeger of California Erectors, Bay Area, Inc., Frank Constantino of Sterling Lloyd Products and Timothy W. Klein of the Wire Rope Company of America for sharing their knowledge.

The opinions expressed in this report are solely those of the authors and do not necessarily reflect the views of the Structural Steel Educational Council.

Alfred Zampa Memorial Steel Suspension Bridge, 2003, California, USA: A fitting legacy for ironworkers

By:

ALFRED MANGUS, PE
Bridge Engineer

SARAH PICKER, PE
Consultant

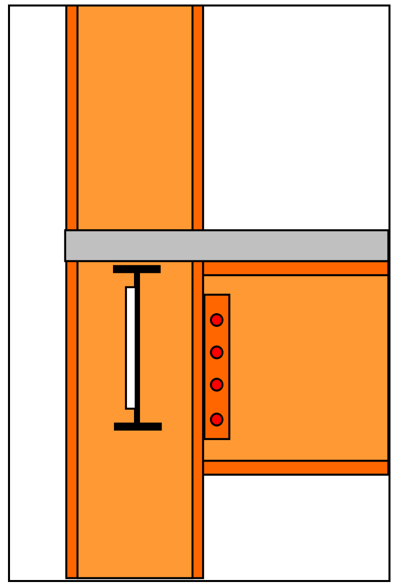


TABLE OF CONTENTS

1. Introduction and Alfred Zampa Biography

1.1. Purpose of Report

This report serves as an executive planning tool for owners, design engineers, fabricators, and contractors. It covers most of the basics of large steel suspension bridges. Our goal is to interest and educate the reader about the advantages of steel as a viable construction material for bridges. It is an executive summary of what has transpired and also a guide to the future. As Sir Isaac Newton so famously put it, “I can see further because I have stood on the shoulders of Giants.” We hope to accomplish our goal by describing the use of structural steel in bridge components for the Al Zampa Memorial Bridge (AZMB), which was built by an international consortium led by Caltrans (California Department of Transportation), the owner, and many California businesses, labor forces, and steel fabricators. The AZMB is an orthotropic deck design with structural steel components used in the bridge construction. A testament to the use of steel in bridge design and construction and a fitting tribute to San Francisco Bay Area ironworker Alfred P. Zampa, the AZMB is one of California’s and the 21st century’s steel success stories.

1.2. Introduction

High above the Carquinez Strait in California, connecting the northern bluff of Crockett to the southern tip of Vallejo, sits the monumental Alfred P. Zampa Memorial Bridge. It took an act of the California legislature to name this bridge after an individual. Normally bridges are named after their locations or a politician. The authors believe that this is the only bridge in North America named after a working man who built bridges. In addition a bronze monument representing Mr. Zampa in his ironworker tools was built in the scenic overview by the walkway on the north end of the bridge (see Figure A, Site Issues, in the Appendix).

1.3. Who Was Al Zampa?

Zampa was born in Selby, California, to immigrant parents in 1905. He helped build many large bridges in the Bay, including the 1927 Carquinez Bridge, the 1937 Golden Gate Bridge, the 1938 San Francisco–Oakland Bay Bridge, the 1955 Richmond–San Rafael Bridge and the 1958 Carquinez Bridge. The fact that Zampa was an ironworker is part of the great drama of his story. Zampa was a man almost always working in a physically demanding job, even up to the day he retired.

Zampa was offered work in the early 1920s on the construction of the first Carquinez Bridge which opened to traffic in 1927. His work involved constructing the foundation for the bridge. Still working on the bridge in 1926, he began learning the superstructure elements of the bridge-building trade. After Zampa's involvement on the first Carquinez steel cantilever truss bridge (see Figure A, Site Issues, in the Appendix), he worked on the piles of the San Francisco–Oakland Bay Bridge. In April 1936, he began work on the world-famous Golden Gate Bridge. By then he had become accustomed to working at great heights. He has been described as "always careful and always confident."

The Golden Gate Bridge was a project distinguished by artist Doug Minkler and historians as the first bridge in the Bay Area to be constructed employing all-union labor crews. During this time, Zampa joined Ironworkers Union 377. In October 1936, the tower and cable spinning had been completed, and Al worked as a connector in a raising gang that was erecting the steel for a road deck. Zampa experienced an accidental fall.

The fall happened on a very cold and densely foggy Monday morning. Al was proceeding out on a steel stringer onto an outcropping measuring only 25 inches in length and 18 inches in width. While stepping onto the wet, slippery steel, his right foot slipped, and he fell approximately 50 feet into a safety net. However, Zampa's fall was not completely broken by the net; the net dipped because it was not adequately fastened, and still in the net, he landed on the rocks below. Both the net and Zampa bounced off the rock, coming up and hitting one last time. Zampa later said that he remembered flipping three times as he descended to the net, but at the time he was not afraid because he assumed the net would catch and hold him. He also recalled that the first drop to the ground was not too painful, but the second hit caused him great pain. In his estimation, he fell about 43 feet.

Zampa was very badly injured. He was found barely conscious and in excruciating pain. Several vertebrae had been broken and his pelvis fractured, leaving him partially paralyzed at the time. Newspaper reports indicated that he was not expected to live. However, after three days, Zampa had fully regained his senses. Despite the pain and the patience and effort that would be involved in the recovery process, he was committed to getting well, walking again, resuming work, and living. Zampa later stated, "I've climbed halfway to heaven and fallen halfway to hell. Neither place wanted me, so I kept working."

He was not concerned about losing his nerve for heights, but he was concerned about the effect of his absence on his coworkers. Though they visited him in the hospital, as well as other workers hospitalized due to other accidents on construction of the Golden Gate Bridge, he did not want his coworkers to lose confidence in him. Still wearing a steel brace, he went out onto the Golden Gate Bridge the same day he was released from St. Luke's Hospital. He was known for saying, "This job is 10% know-how and 90% guts."

Zampa's return to the bridge from the hospital reflected his own "90% guts." It would take a total of six years for him be physically able to return to work as an ironworker in basically the same capacity he had worked before the fall. Zampa was interviewed initially by the print media and later on television. The TV interviews have been reissued as video clips that are shown in several bridge videos. In 1987, Doug Minkler created a poster to celebrate the "Halfway to Hell Club," referring to construction workers who fell during construction work on the Golden Gate

Bridge. One of the members of this club was Al Zampa. Even in his last days on the job, Zampa worked as a connector in a raising gang. Connecting is one of the most physically demanding jobs in the bridge-building trade.

Al Zampa participated in the ground breaking of AZMB. Six months later, Alfred Zampa passed away. The California legislature passed legislation to name the bridge after Al and to build a memorial monument. Zampa's son Richard plus his two grandsons are ironworkers.

The following is inscribed in the official State of California monument to Zampa at the bridge (see Figure A, Site Issues, in the appendix). "Built in his home town, by ironworkers from his home local, the AZMB is a fitting tribute to Alfred Zampa and the generation of men who built the early bridges. It is also a testament to the strength and courage of all bridge builders, past, present, and future."

1.4. Bridge Construction Safety

Bridge building is a dangerous occupation, and many people risk their lives to build bridges. The most dangerous phase of a bridge's life is the construction phase. During bridge construction, a dangerous time for workers in a structure's life is during the erection process. At certain times during the erection process, components are held in place by cranes, falsework, and temporary appurtenances. Sections are normally not erected during bad weather or poor wind conditions. During the construction process, weather conditions can affect concrete curing, welding quality performance, and applications of coatings and other materials.

There were catastrophic losses of life during the construction of bridges as recently as 1998. That year, 21 lives were lost during the construction of the Injaka Bridge in South Africa and the Kurushima Bridge in Japan. Twenty-five people died during the construction of the San Francisco–Oakland Bay Bridge, and another 11 died during the construction of the Golden Gate Bridge. Engineers in charge of construction for the Bay Bridge did not require hard hats, safety nets, or safety lines. A variety of accidents occurred, including objects hitting workers on the head and deaths due to falls.

On the Golden Gate, Chief Engineer Joseph Strauss instituted the use of leather "hard hats." Another safety requirement was the use of safety nets. It was the failure of the safety net that caused Alfred Zampa to fall 45 feet to the ground and sustain injuries.

There were no deaths during the construction of the first two bridges to cross the Carquinez Strait. The third crossing of the strait, the AZMB, upheld that remarkable record, much to the pride of those associated with the design and construction of the bridge. Engineers today, as in the past, manage bridge design and construction. They carefully consider the health and safety of the people who build bridges. They take that responsibility seriously and spend a great deal of effort to understand how to make bridge building safer. The process of bridge design is a human endeavor and not yet perfected. The consideration for construction safety that is incorporated into the orthotropic design of AZMB is worthy of attention. A key legacy was that no deaths occurred while building the AZMB, a legacy worth emphasizing.

2. Design Issues

2.1. Structural Design as an Art Form?

The well-known professor David Billington, PE, of Princeton has argued for decades that structural design should be considered as a structural art form, Caltrans has what it refers to as a design strategy meeting. The durability of materials, the ease of maintenance, and the aesthetic qualities of a large structure are at best intangibles. Numbers cannot necessarily prove right or wrong decisions. Classic designs become the reference for future successful projects. Architects and engineers, along with the general public, determine which massive sculpture or bridge is a winner. Thus case histories such as this report and photos become valuable tools to designers of bridges.

2.2. Issues Affecting Conceptual Design

The bridge owner, Caltrans (California Department of Transportation), is the transportation agency created in the mid-1970s by merging the State of California San Francisco Bay Toll Crossing (BTC) and the State of California Highway Department (CHD). CHD and BTC were agencies that built, operated, and maintained major bridges crossing water.

After the 1971 Sylmar earthquake, CHD began to develop an extensive program to retrofit highway bridges. Then, the 1989 Loma Prieta and 1994 Northridge earthquakes significantly increased Caltrans' body of knowledge regarding seismic effects on bridge details and retrofit methods. From the analysis of these seismic events and other seismic activity, engineers gained a greater understanding of the effects of an earthquake on the built world.

A massive bridge analysis program for new seismic standards was undertaken by Caltrans in the mid-1990s. Included in this program was an assessment of the 1927 and 1958 Carquinez Strait crossings. These two bridges are seen in Figure A, Site Issues, in the appendix.

These are two multi-span, cantilever steel truss bridges spanning the Carquinez Strait with a maximum span of 300 meters. The 1927 bridge carried traffic in the westbound direction, and the 1958 bridge carries traffic in the eastbound direction of Interstate 80.

These studies showed that the 1958 bridge would need substantial structural reinforcing and seismic upgrading. It further showed that a seismic retrofit for the 1927 bridge would have been significantly complex and, accordingly, would have had a high cost and would have required significant and costly future maintenance. Therefore, total replacement for the 1927 bridge was chosen as the seismic retrofit strategy.[1]

In the 1971 San Fernando, 1989 Loma Prieta, and 1984 Northridge California earthquakes, some concrete bridges totally collapsed. Although steel bridges have more desirable features in earthquake loading, they have also sustained damage but none collapsed during these earthquakes.

After the 1994 Northridge earthquake, all bridges in the state, regardless of ownership, were examined for their seismic characteristics and toughness.

An "importance factor" was assigned to larger bridges that were more essential to transportation. Caltrans has on-staff seismologists to help determine higher risk sites closer to major faults in the state.

2.3. Suspension Bridge Selection Process

Caltrans studied four bridge types to replace the 1927 Carquinez Bridge. Considerations of the four types included widening the existing bridge to accommodate vehicles as well as pedestrians and bicyclists. Caltrans considered:

1. A cantilever truss built with two similar main spans of 1,100 feet
2. A cable-stayed bridge built with two similar main spans of 1,100 feet
3. A steel bridge built with two similar arches with main spans of 1,100 feet
4. A steel orthotropic suspension bridge built with one main span of 2,000 feet

Bridge piers get in the way of ship traffic. The primary users of the deep waters that travel through the Carquinez Strait are oceangoing ships that carry cargo from San Francisco Bay to as far east as the Sacramento River and Stockton. These waters are difficult to navigate as the river is narrow at the strait and currents change direction. One of the main advantages of a suspension bridge is fewer piers, thereby increasing the width available for ship navigation.

The 1958 bridge reduces the width of the navigation channels to 1,100 feet. Construction of the new span presented an opportunity for the foundations to be on the sides of the strait, and they would be less costly to build. One disadvantage was that there would be conflict between

[1] The late 1920s were a time of great adventure and growth for the United States, and the 1927 bridge created much fanfare when it was built. The 1927 bridge opened on the same day that Charles Lindbergh made his famous transatlantic flight. President Coolidge phoned during the opening day celebration, which was well attended by the local community and its leaders. The 1927 bridge was the first span to connect the northern parts of the San Francisco Bay to the southern parts. After 80 years, the 1927 Carquinez Bridge was still serviceable when the decision to replace it was made. As recently as October 2005, the 1927 bridge was used as a detour to carry eastbound traffic while repairs were carried out on the 1958 bridge. In 2006, the 1927 bridge was demolished.

construction activities and the river traffic during foundation construction and other steps to build the bridge.

The issues considered while planning the replacement bridge design were ease of both construction and maintenance, aesthetics, width of vehicle lanes, safety shoulders, and the ability of ships and trains to pass below the bridge. Input was solicited from citizens residing on both sides of Carquinez Strait. This input included visual concerns. The community also desired the ability to walk and bicycle across the bridge.

The basic structural system of a suspension bridge consists of flexible cables and, suspended from them, stiffening girders or trusses that carry the deck framing. The vehicular traffic travels on a surface between the main supporting systems. Other major elements of the bridge are the superstructure, anchorages, cable systems, floor system, and towers.

Decking is used for stability as well as the driving surface for vehicles. The box girders or trusses that are used as stiffening supports for decking can be made of concrete or steel. Generally, box girders as stiffening supports are lighter than trusses, and since steel superstructure is lighter than concrete superstructure, the elements made of steel will result in less weight for the towers and cables to hold.

The characteristics of steel decrease the vulnerabilities of long-span bridges in earthquake regions. The suspension bridge is vulnerable to severe effects from the direction and force of wind and earthquakes. As bridges become lighter, wind increases vulnerability. Using steel in structures tends to make them lighter.

2.4. Suspension Bridge Wind-Loading Issues

Bridge wind-loading factors came to the attention of the world with the collapse of the Tacoma Narrows One Bridge in Washington State in 1940. It was a steel superstructure suspension bridge stiffened by plate girders. Wind caused self-increasing resonant vibrations on the bridge, which caused severe motions, and the bridge was given the name “Galloping Gertie” prior to its collapse because of excessive deflections that bothered drivers.

Although aeronautical engineers have used a great deal of wind engineering to design airplanes, wind engineering had never been applied to bridges. Civil engineers quickly adopted wind tunnel analysis for bridges and tall buildings after the Tacoma Narrows One Bridge collapsed. After its collapse, suspension bridges all over the world were analyzed for similar issues. Many older suspension bridges are still being modified for improved wind performance.

During the design of the Severn Bridge in 1965, the superstructure design team adopted a new concept utilizing a wing-shaped superstructure to allow the wind to flow around it, with minimal turbulence. This was an aerodynamic design that successfully dealt with wind loading.

The aerodynamic superstructure proved to be a brilliant and elegant engineering solution and was adopted by many bridge engineers around the world. Advantages of the wing design include a smaller exterior surface area to coat or paint; it also allows a safe area for maintenance employees and engineers to perform inspections. Openings allow access for maintenance and inspection.

A detailed wind-loading analysis for the proposed AZMB was performed by testing scale models in a wind tunnel. The adjacent 1958 truss bridge was also included in these model studies. These design numbers and effects were given to the structural designers to verify that sufficient structural capacity existed throughout the bridge. Interestingly, the test showed the box girder would remain stable in any wind event, but the 1.8-meter tall railings on the deck would cause problems. The railing spacing was altered and the problem eliminated.

To further minimize the amount of steel needed to build the bridge, an orthotropic framing system was selected. The bridge also featured placement of the suspenders at a 30-degree incline from vertical rather than the traditional vertical suspender cables. This allowed the suspender cables to take longitudinal loading from vehicles, wind, or seismic forces.[2]

2.5. Earthquake Loading Issues for Suspension Bridges

Extensive bridge construction began in the 1920s with the growing popularity of automobiles. At the time, designers did not consider movement of the structures due to earthquakes. Bridge design has evolved greatly over the past 80 years and has come to encompass the concept of earthquake loading. An essential characteristic that ensures a bridge can successfully resist seismic loading is ductility. The metals and other materials used in airplanes are highly ductile. Unless the rears are arranged in patterns to create ductility in concrete, which is a brittle material, then problems will arise during an earthquake. Another key issue is the behavior of the bridge foundations when soils move due to earthquake activity. Therefore, major bridges built in active seismic zones must have the lowest mass possible and be ductile in order to be cost-effective. Most bridges of significant size are built of reinforced concrete or steel or combinations of these two primary materials. A fundamental principal of seismic loading is that the lower the total mass of the structure, the lower the seismic loading. A large concrete bridge can have three times the dead-load mass of a same-sized steel bridge. For large-span bridges, the concrete weight can easily be 60% to 70% of the total weight. The weight of vehicles, people, and/or trains makes up the smaller portion. Although a solid cubic foot of steel weighs 490 pounds compared to the 150-pound weight of a solid cubic foot of reinforced concrete, large steel bridges have a much lower dead-load mass.

2.6. Earthquake Loading Issues for AZMB

The acceleration response spectra (ARS) curve used for the Al Zampa Memorial Bridge is shown in Figure A, Site Issues, in the Appendix. Geomatrix Consultants created the following table, which summarizes their study of maximum credible earthquakes (MCE) for the site.

---

2 The Golden Gate Bridge was retrofitted in 1952 to prevent its collapse during heavy winds by connecting the lower chords of the steel trusses with additional horizontal steel sway bracing.

The new AZMB was designed with earthquake forces in mind. Studies were done using state-of-the-art analysis methods. Rocking motions at each support were developed to correspond with Seismic Evaluation Earthquake (SEE) incorporating the near-source effects. The three-dimensional model of the bridge incorporated geometric nonlinearities in the cables and material nonlinearities in the concrete elements, which show inelastic behavior.

Table 2.6.1 The Larger Bridges Normally Have Site-Specific Loadings.

Earthquake Fault Source	Magnitude Mw	Distance from Bridge to Fault (km)	Peak Rock Accelerations (g) Horizontal	Peak Rock Accelerations (g) Vertical
San Andreas	8	41	0.26	0.19
Hayward	7 1/4	13	0.55	0.47
Franklin	61/2	1	1.00	0.96

The design engineers used a global inelastic dynamic computer model of the entire bridge for analysis. Seismic loadings obtained from the structural analysis were compared with the capacities of the respective bridge components to ensure that the bridge would perform as desired. A global model of the bridge from the north to the south anchorage was developed. The model included the viaduct for the Crockett approach on the south side of the bridge, and thus captured the interaction between the approach viaduct and the suspension bridge. This analysis was supported by detailed component analysis of the steel box girder deck, tower legs, and tower piles.

The global model accurately represented the bridge geometry, stiffness, and mass and considered all the important structural components including pile foundations, anchorages, main towers, transition pier, cables, suspenders, and orthotropic deck. To start with, a linear elastic model was used, but this was refined during the analysis as geometric nonlinearity was introduced in the cables, and material nonlinearity was introduced in concrete elements found to experience inelastic behavior.

In the time-history analysis, which was based on the Nemark Implicit Integration method, Rayleigh damping was used for the structural elements with the damping matrix.

2.7. The Orthotropic Deck System

The first major orthotropic bridges opened to traffic in North America were the San Mateo–Hayward and the San Diego–Coronado. The bridges, which are still in service, were fabricated and erected by California iron workers employed by Murphy Pacific Corporation. Without the durability of these legacy structures, newer orthotropic bridges, including the Alfred Zampa Memorial Bridge, would not be selected by owners.

The evolution of a 100% steel superstructure, or orthotropic superstructure, took decades. In the 1920s, American engineers began creating designs that would entail riveting steel plates to steel beams in order to obtain and facilitate large, movable bridges. The purpose was to minimize the dead mass load of lift spans.

This type of bridge system and shipbuilding are similar. The American Institute of Steel Construction (AISC) referred to the orthotropic system as a "battle deck" in 1938. Low weight, or mass, is a desirable and significant design characteristic of orthotropic bridges. One of the considerations in the development of orthotropic steel bridges was to save weight or tonnage of steel.

The word *orthotropic* is actually a contraction of "orthogonally" and "anisotropic." *Orthogonal* means "at right angles"; *anisotropic* means "having equal different properties." Therefore *orthotropic* is defined when used for bridges as an all-steel superstructure. German scientists patented components called "orthogonally anisotropic," or orthotropic, in their writings, which were translated into English.

The Germans invented the use of orthotropic steel components with the primary goal of absolutely minimizing the steel weight. The variation of steel weight for various span types is shown on Figure D, Reduce Entire Self-Weight of Superstructure by Using an Orthotropic Steel Deck, in the Appendix. This change in design occurred after World War II, when steel was very expensive and in short supply.

The orthotropic design was first used after World War II. The Germans utilized this system, which minimizes the total steel weight or mass of a bridge. The Germans began to use the 100% steel-deck bridges as grade-separation bridges for their "autobahn" in 1934. After World War II, the Germans patented this system, naming it an orthotropic system, and one German company, MAN, created a manual for the system.

Table 2.7.1 The Millau Viaduct Deck Area Compared With Other Orthotropic Bridges.

Bridge Name	Rib Type	Deck Area (Sq. meters)	Deck Area (Sq. feet)
Dublin 580/680 Test 1965	closed	1011	10,880
Ulatis Creek Test 1966	open	411	4,420
San Mateo – Hayward 1967	open	43,476	468,875
San Diego – Coronado 1969	closed	10,266	122,220
Queensway Twin 1971	closed	10,256	110,400
Four—BART Rail 1972	closed	449	4,840
Colusa 1972	closed	372	4,006
Miller – Sweeney 1973	closed	722	7777
Braille Trail Pedestrian 1977	open	33	360
Golden Gate redecking 1985	closed	35,934	387,000
Maritime Offramp 1997	closed	7,921	85,287
AZMB, Carquinez 2003	closed	30,586	339,133
TOTAL Completed CA bridges in service		**143,554**	**1,545,198**
Akashi-Kaikyo, Japan 1998	closed	84,086	905,093
Millau Viaduct, France 2005	closed	184,800	1,989,168

In 1963, the American Institute of Steel Construction (AISC) funded and published the only design aid available in English, *Design Manual for Orthotropic Steel Plate Deck Bridges*, by Roman Wolchuk. This prompted the states of California, Illinois, Michigan, Missouri, and Oregon to create prototype bridge systems and the construction of one small bridge per state. Orthotropic steel decks in North America are very rare. They number about 51 out of 650,000 inventoried bridges. More than 99% of bridges have concrete decks. Fewer than 100 orthotropic bridges exist in North America. A few have solid steel-plate decks, and a larger number use various types of steel grating. Out of the 51 orthotropic bridges in the United States, 17, or more than 25% of the orthotropic bridges, are in California. These few completed orthotropic bridges are a vital part of the California infrastructure (see Table 2.7.1). About 25,000 bridges exist in California.

2.8. Why Use the Orthotropic Deck System for AZMB

The overall goal of the Federal Highway Administration (FHWA) is to achieve a 100-year life span for bridges. The oldest orthotropic bridges in the United States and California are about 40 years old. In Germany, some orthotropic bridges are about 50 years old. The evolution of orthotropic steel suspension bridges is shown in Figure B, Orthotropic Decks Dominate Big Bridges, in the appendix. The evolution of the aerodynamic orthotropic box girder for suspension bridges is shown in Figure C, Aerodynamic Orthotropic Superstructures, in the appendix. The figure also shows the box girder used on AZMB. So the designers and owners both knew it was a durable solution.

An orthotropic steel bridge (a 100% steel superstructure) involves welding the steel deck plate and all steel members, such as beams or girders, which creates a more efficient system. Decking is used for stability as well as the driving surface for vehicles. Box girders or stiffening trusses are placed and used as stiffening supports for the superstructure. They can be made of materials such as concrete or steel. Generally, box girders as stiffening supports are lighter than trusses, and steel is lighter than concrete, so the elements being made of steel results in less total superstructure weight for the towers and cables to support. Steel box girders result in the lowest overall bridge weight, which reduces the required strength of towers and cables and is most desirable in earthquake loading. Caltrans adopted this system for major bridges in the 1960s because a lower-weight bridge receives less stress and impact from an earthquake. Hence, in high seismic areas, large bridges have an orthotropic or “steel deck” system.

Table 2.8.1 Deck Weight Comparison of Various Shapes and Materials for a 453-ft by 55-ft Movable Span Bridge

Deck Type	Total Deck Weight (Kips)
Orthotropic plate	1520
Lightweight concrete deck – 8 inches	3001
Partial-filled steel grid with monolithic overfill	2456
Exodermic deck	2198
This table was originally created and published by Dr. Thomas A. Fisher, HNTB Corporation	

The foundation, and in many situations the amount of steel necessary in the foundation, depends on the superstructure weight. If a bridge is designed with a higher-mass concrete superstructure, the need for a larger foundation capable of holding that great weight also increases costs.

The financial advantage of steel orthotropic deck bridges over concrete decks results from the weight savings, which reduce costs of all other components. Steel deck itself costs more per square foot than reinforced concrete. But savings in all other components is very significant. After the Golden Gate Bridge was redecked with orthotropic steel, the midspan of the superstructure was about 7 feet higher! With a 4200-ft span, the 7-feet increase is not noticeable to a driver. The lower mass probably helped this bridge a great deal when the 1989 Loma Prieta Earthquake struck.

2.9. Are Orthotropic Steel Components Durable from Metal Fatigue?

Metal fatigue in orthotropic bridge decks occurs from heavy truck traffic. Fatigue is increased by minimizing materials and improper welding details, although millions of alternating truck cycles are needed to damage a bridge with fatigue cracks. Generally bridge beams and welded components are much larger than building components. They are also exposed to outside environmental conditions. Building members are protected from the weather by building cladding. Building members are not subject to metal fatigue. Fatigue can damage improperly designed or poorly fabricated welds that are important to the function of the composite behavior of the steel girder box and its “wearing surface.” How the component is manufactured or fabricated is vital to consideration of the longevity of the bridge and is discussed in Chapter 3.

Many earlier orthotropic bridges have had a shorter useful or fatigue life due to a primary goal of absolute minimization of materials. Fatigue of the rib system is an area of much interest to engineers and academics. One part of the debate is open ribs versus closed ribs for orthotropic bridges. Engineers and research professors continue to study the myriad of rib shapes to achieve proper welded or bolted splice details.

Three simultaneous uses of deck plates create an orthotropic bridge. This is achieved by designing and welding the steel deck to perform in three simultaneous systems:

- First, the steel deck plate acts as a deck element—the steel deck acts as the top portion of the rib. Ribs are divided into two categories, open and closed. An open rib is welded in only one location to the deck. A closed rib is welded in two locations to the deck. An open rib is a torsional soft deck, which deflects more with the same weight of steel as a closed rib. A closed rib is a torsional rigid deck, and has the lowest self-weight of any orthotropic deck. The efficiency of this rib is shown graphically in Figure F, Rib Geometry and Detail, in the appendix, by the solid black area. A closed rib is a box beam, while an open rib is a T beam.

- Second, the steel deck plate is used as the top flange of a transverse floor beam, or as in the AZMB, deck plates are used as the top and bottom flanges of the full-depth diaphragms and partial-depth diaphragms.

- Third, the steel deck plate acts as the top flange of the main longitudinal girders of the superstructure. In the case of the AZMB, a gigantic steel tube or wing was designed as a gigantic beam spanning between the hangers.

Dr. John W. Fisher of Lehigh University is considered the world’s leading authority of bridge steel metal fatigue and developed the American Association of State Highway and Transportation Officials (AASHTO) standards. Metal fatigue problems had occurred in earlier European bridges. John Fisher spent his career researching methods of how and where fatigue cracks start. One error on some bridges was to fill up abandoned bolt holes with plug welding. Today we know that this should never have been done. Dr. Fisher has several books on details to avoid for steel bridges. Normally steel bridges with fatigue cracks are saved, but major retrofits or replacements of key components are needed. Caltrans has retained Dr. Fisher to investigate concrete deck bridges with steel superstructures with fatigue issues. Some of these projects are written for the public domain.

2.10. Dr Fisher’s Rib Detail to Increase Orthotropic Steel Deck Fatigue Life

The AZMB was the first entirely new orthotropic steel deck bridge to use a construction detail developed by Dr. Fisher to increase fatigue life to 100 years. The evolution of this detail for orthotropic ribs is described in this section.

The rib fabrication detail Dr. Fisher developed for the Williamsburg Bridge orthotropic redecking was selected by the designers. Caltrans approved its use for AZMB, so that stresses transverse to the ribs occur in the deck. No specific fatigue studies were performed on the AZMB. Information analyzed from tests on other bridges was applied to this bridge. One bridge test on the Williamsburg Bridge in New York City was used in the design of this bridge. It was partly from this information that the detail was created by Dr. Fisher.

This detail is used where the ribs go through a floor beam or diaphragm plate. Fewer welds result in a better bridge by diminishing the possibility of fatigue cracks occurring in the welds. Trisecting welds are not allowed. Reducing welds reduces costs and self-weight. Cope holes occur to prevent trisecting welds. After welding occurs, the steel shrinks; thus additional stresses result in the weld. Engineers and research professors continue to study the myriad of details to achieve proper weld details plus coping holes and cut-hole shape and dimensions.

Despite the fact that closed ribs are more difficult to splice and inspect, the designers of the AZMB chose primarily closed ribs. The reason designers use some open ribs on the corners of the superstructure is to allow faster alignment for bolting of the superstructure's fabricated segments. Closed ribs produce a lower-weight superstructure compared to open ribs. Two types of closed ribs (top deck and soffit) and two types of open ribs were detailed. A tab plate for attaching the open rib is welded to the flat plate rib and bolted to the diaphragm. Thus, fatigue cracks cannot transfer across this bolted plate.

The practical economical solution to achieve a 100-year life for the AZMB was to use Dr. Fisher's detail. The most expensive part and key feature of Dr. Fisher's detail is the use of a run-off tab on both of the lower outsides of the trapezoidal ribs. The process of welding a trapezoidal rib to a transverse steel plate is shown in Figure F, Rib Geometry and Details, in the appendix. In other bridges, it also could be used for any transverse floor beam or element. The run-off tab for rib welds have been used for a variety of welded components. This is because welding fatigue cracks are much more likely at the beginnings and ends of welds. The technique of throwing away a sacrificial tab is also used on welded steel pipe piles. Thus a weld starts and ends on a tab plate. Then this tab is cut or ground away because fatigue cracks are more likely to start at beginnings and ends of welds.

Run-off tabs are a standard welding technique. Welds start and stop on sacrificial tabs. Starts and stops are common sources for cracks and slag inclusions. Dr. Fisher applied the following procedure to the welding of ribs to diaphragms.

Step 1: The "cut out" for the rib is made with a run-off tab. An internal diaphragm plate is welded inside of the trapezoidal rib. The thickness of this baffle plate matches the alignment and plate thickness of the diaphragm. The rib is welded to the deck plate using a pair of automatic welding machines.

Step 2: The deck plate and rib are welded to the diaphragm using fillet welds.

Step 3: The final four inches consist of a Complete Penetration Groove Weld (CPGW).

Step 4: The removal, primarily by grinding, of the run-off tab and weld. This eliminates any stress risers. (The procedure is shown in the lower half of Figure F, Rib Geometry and Details, in the appendix.) There is a higher cost to fabricate this detail.

Figure 2.10.1: Deck weight comparison of various shapes and materials for 453-ft by 55-ft movable span bridge.

The small ferry terminal was built during the writing of this report. It has no coping, nor Dr. Fisher's detail. Apparently it is a temporary ramp and subjected to salt air and possible sea waves.

Figure 2.11.2: Two photos by a passenger from inside a car driving across the AZMB heading south

3. Fabrication and Construction

3.1. Building the Bridge—The Marriage of Fabrication and Construction

The towers, anchorages, and cables are major features to building a suspension bridge such as the AZMB. The contractor decides where to create each portion of the bridge. The basic two choices are shop fabrication and site construction. So it's a marriage of inside work and outside work. It's difficult to separate the process, and the contractor was required to state which part was the most critical, or to produce a CPM drawing for Caltrans. Caltrans provided staff to inspect fabrication in the various and numerous shops. See Figure H, Fabrication of Bridge Components for AZMB in Shipyards, in the appendix. Figure I, Key Stages or Phases of Bridge Erection for AZMB, in the Appendix, shows six stages of construction, which is a simplification of reality. The towers and anchorages are constructed prior to aerial spinning of the main cable. Next, superstructure sections are hung from the cable. The follow sections will detail this complex process from foundations to protective coatings.

3.2. Foundation Cofferdams

The original design called for reinforced concrete casting offsite. Ben C. Gerwick, Inc., recommended switching to floating steel caissons to the bridge and landing them on preinstalled erection frames, or falsework. The intended concept was to use the footing shells as temporary cofferdams and as templates for the installation of drill shaft casings.

The original design was modified to include a temporary support system that allowed the float-in cofferdams to be landed directly on the preinstalled drill shaft casings rather than on a separate falsework system. This strategy to modify the design to further reduce the field work was developed by Gerwick.

There are six drilled shafts 3 meters (m) in diameter that support each main tower leg. The top of the drilled shaft was to be topped with 5.14 m deep footings, 18 m by 20 m in the plan. The bottom elevation of the footing is 2.34 m below mean sea level. Cofferdams were used to create a dry working area at these locations.

Benefits of the modification resulted in:

1. Early start for the drilled shaft installation (an activity on the critical path) by allowing casing installation to commence before completion of the float-in caisson or cofferdam.
2. Shorter construction schedule by allowing casing installation to run independently of and concurrently with cofferdam fabrication.

3. Lower construction cost by eliminating the need for an erection frame or any other temporary falsework support for the float-in cofferdam.

3.3. Foundation Casings Were Shop-Fabricated

The two tower foundations of the AZMB were made of reinforced concrete. However, each tower is supported by 12 rock-socketed drilled shaft piles. The steel piles are 3 m in diameter and 90 m long. Reinforced concrete pile caps transfer the vertical and lateral loads between the piles and towers. The piles for the northern tower were owner-supplied and built by Ameron/TransBay Steel.

Nesco/XKT, of Mare Island, California, built the piles for the south tower and supplied some of the steel that went into the south anchorage. During construction, the constructors found that the specified pier length was not adequate, and they had to build extensions, or “add-ons,” for the piles. They extended the length twice. See Figure H, Fabrication of Bridge Components for AZMB at Shipyards, in the Appendix.

The difficulty with creating the extensions was that the extensions were not milled at the same time as the piles. The thickness of the flat steel produced from different mill runs is not precisely equal.

3.4. Cast-in-Place Reinforced Concrete Towers Were Poured On Site

The height of a tower is based on necessary sag or curvature for the cables. The anchorages hold the ends of the cables and transfer the cable tension loadings into the ground. Cables are an important part of a bridge because they support the roadway and give the bridge its shape and character.

3.5. Cast Steel Components Were Shop-Poured

Cast steel is used for the saddles and cable bands and suspenders on AZMB. Casting for suspension bridge elements is a specialized topic. Evolution of suspension bridge casting technology is summarized in Table 3.5.1. Other uses of cast steel are oil drilling and roofs that are supported by cables or wire ropes.

The AZMB has four saddles, which change the direction of the main cable placed at the tower. The saddles for AZMB were cast in three pieces each for transportation concerns. The saddles, shown in Figure K, Suspension Cable Details for AZMB, in the appendix, were a sole-source bid item and were purchased in England. Figure K shows the strand layout with a photo of a tower. There is a saddle at each tower and a pair of splay saddles at each anchorage. Other components of the anchorage were made of fabricated steel.

The largest saddle piece weighed 14,500 kilograms, which could be lifted by the tower cranes, one crane placed at each tower. See Figure I, Key Stages or Phases of Bridge Erection for AZMB, in the appendix. One tower crane was used to build each tower and lift supplies as needed. Once placed, the saddles were field-bolted together and had a field-bolted weather cap bolted to the saddle after aerial spinning was completed. The separator vanes are newer saddle technology.

Table 3.5.1 Evolution of Steel Castings for Suspension Bridges.

Bridge, Country, Date	Cast Steel Cable Saddle	Cast Steel Cable Band
Williamsburg, USA, 1904	Cast Steel – semicircle	Groove USA type
Golden Gate, USA, 1937	Cast as three pieces, then machined for cable. Semicircles for each strand	Groove USA type
Forth of Firth, Scotland, 1964	Grooved in hexagonal pattern with vanes	Groove USA Type
Severn, UK, 1966	Unknown	Single tab European-style created inclined suspenders
Little Belt, Denmark, 1971	Only groove in single hexagon shape was cast; remainder is welded steel	Single tab European-style vertical suspenders
Akashi-Kaikyo, Japan, 1998	Unknown	Double-tab European-style created
AZMB, Carquinez USA, 2003	Cast as three pieces then machine-grooved in hexagonal pattern with vanes	Both types used

3.6. Cable Spinning

The AZMB cables were made of one continuous strand of galvanized wire, pieced together by "air spinning" the wire off of bundles, over the towers, and through the anchorages.

The evolution of suspension bridge air-spinning technology is summarized in Table 3.6.1 The table shows bridges, their deck area (which gives an idea of the weight that is being held by the cable), and how the bridge was created (by using air spinning or a bundled parallel wire system [PWS]). Air spin means that each individual wire was arranged in the air and bundled. The individual wires were pre-bundled on the ground in a shop. PWS doesn't take as much field construction time as air spinning. The AZMB construction project used the best technological innovations available.

The goal in both methods was to provide equivalent tension in all wires, so that every wire shared an equivalent amount of stress as much as was practical. Larger equipment was needed to erect bundles in PWS, but it was a time saver when air spinning was on the critical path of construction. Installing air spinning steel wires for suspension bridges is a specialized topic.

Figure J, Air-Spinning Method, in the appendix, shows the overall air-spinning schematic and how each 5-mile reel is attached to create the 13,000 miles of No. 6 wire gauge needed for the AZMB.

Some bridge air-spinning operations require 24-hour work, but AZMB did not. The constructors were able to utilize two 8-hour shifts because spinning was not on critical path to complete construction. The galvanized wires for the main cables were manufactured and shipped to California in approximately 5-mile bundles. About 13,000 miles of cable were required, or 2,600 bundles. There are 8,584 wires in the main cable, which is about 512 millimeters (mm) in diameter. The net area of steel is 167,000 square mm because there are also air voids, even after the compaction process. The wire has an ultimate strength equal to 1,570 n/sq. mm; proportional limit equals 1,570 n/sq. mm; and service load stress is less than 100 ksi, or 698.7 n/sq. mm. Ferrules are used to splice each wire; there are about 2,600 ferrules, or splices, on the AZMB. These were randomly located throughout the length of the cable. The direct control tension keeps about 80% of the dead load on the wire at all times during the spinning process.

Table 3.6.1 Evolution of Air Spinning of Suspension Bridges.

Bridge, Location, Date	Technique	Clear Span (feet)	Summary
Golden Gate Bridge, USA, 1937	2-loop spinning wheel	4,200 ft 1,280 m	17% voids in compaction of wires vs. 22% of George Washington
Forth of Firth, Scotland, 1964	Grooved cast steel saddle	3,300 ft 1,006 m	First bridge to use it
First Bosporus, Turkey, 1973	Coils brought directly from plant to site	3,575 ft 1,090 m	Also had included suspenders for longitudinal forces
Vincent Thomas, Long Beach 1965	Idea for direct tension-control method	1,500 ft 457 m	Idea for direct tension-control method developed with this bridge
Shimotsui-Seto Bridge, Kojima-Sakaide, Japan, 1988	Air spinning, used the tension-control method	3,136 ft 956 m	First bridge to use it
Akashi-Kaikyo, Japan, 1998	PWS	6,527 ft 1,990 m	Jackson Durkee advised them
AZMB, Carquinez, 2003	Air spinning, used the tension-control method	2389 ft 728 m	Air spinning was deemed to be the best

Dorman Long Hordaland was awarded the contract for the cable erection on the new AZMB. The air spinning of the cables was performed using the two-loop system that was successfully used on the Triangle Link project in Norway. Dorman Long Hordaland was also responsible for engineering on the project, together with cable compacting, cable wrapping, and required staff.

3.7. Fabrication Techniques for Orthotropic Bridges

As with bridge design, construction techniques have evolved over time. The fabrication of orthotropic steel components for bridges is an evolving process (see Table 3.6-2). The AZMB superstructure is a continuous member, thus resulting in a continuous superstructure of 1,056 meters without expansion joints; it has expansion joints only at the ends of the orthotropic box. , Generally speaking, shop fabrication is more productive than field fabrication. The fabricated orthotropic members are normally built in facilities that previously built ships. Building an all-welded steel ship is very similar in many ways to an all-steel orthotropic bridge. The bridge is fabricated in pieces and then moved to the construction site and erected, and the final pieces are welded together. The advantage for the contractor is that erecting the largest practicable pieces in the field minimizes the more difficult and higher-cost field connections. Maximizing piece size minimizes field connections.

Table 3.6.2 California Suspension Bridges vs. World's Largest.

Bridge, Location, Date	Cable Type	Bridge Owner	Comment
SF–OBB. San Francisco Bay, California, 1936	Air	Caltrans	Two decks; originally built for trains on lower deck; world's deepest bridge pier at time of construction
Golden Gate Bridge, San Francisco Bay, CA, 1937	Air	Golden Gate Bridge District	World's longest bridge span at time of construction
Klamath River, Orleans,1965	Bundled	Caltrans	Multicell steel box girder with reinforced concrete deck
Vincent Thomas, Long Beach. 1965	Air	Caltrans	Reinforced concrete deck failing; safety net installed by Caltrans 2005
Guy A. West Bridge, UCSC Pedestrian, Sacramento, 1968	Bundled	City of Sacramento	Pedestrian bridge and emergency vehicle use
Bidwell Bar, State Route 62, Feather River, CA, 1964	Parallel structural strand cables	California Department of Water Resources	New covering system developed by Bethlehem Steel and DuPont
AZMB, Carquinez, 2003	Air	Caltrans	State-of-the-art design, fabrication, and construction
Akashi-Kaikyo, Japan, 1998	PWS	HSBA	World's longest bridge span

An orthotropic box girder is fabricated of thin steel plate joined together by extensive use of asymmetric welding and therefore subject to significant and complex distortions during fabrication. Should any fatigue damage happen in the future, repair of these welds is very costly. Because of this, codes have been enacted just for bridges around the world. The code also permits the use of limited localized heat to correct distortions. Heat distortion is a major consideration when attempting to achieve the tight tolerances for the final geometry when thin steel members are welded.

In North America, The AWS D1.5 was specifically created to discuss welding issues unique to bridges. The global issue of metal fatigue is different for bridges than buildings. There are very few specific requirements unique to orthotropic bridges in this code, because fewer than 100 orthotropic bridges exist in North America. For AZMB, appropriate special provisions were written to require fabrication and assembly of prototype orthotropic panels.

The closed rib-to-deck, soffit, and side-panel plate welds are about 90 miles in total length, and constitute 90% of the welding in the orthotropic superstructure. These welds are completed from only one side, and the interior is not accessible for visual inspection. The ribs are normally welded in an upside-down position for both the top and soffit decks of bridges. The reason for this is that steel is molten, and downward welding can be accomplished by automatic welding machines, which welders set up at the beginning position.

It has been established that 80% penetration for closed ribs to the deck plate is the ideal joint penetration to provide the necessary fatigue performance while providing sufficient unfused base metal to serve as a backing and prevent the weld from melting through the back side of the joint.

The goal is to eliminate weld melt-through and overlap on the back side of the joint. Experts debate whether overlapping the notch from the melt-through is any more severe than the inherent notch at the root of the PJP.

A fully automatic process was used to ensure consistency. PJP weld conditions were inspected by nondestructive testing and destructive sampling. A minimum of 15% of the length of these welds was subject to the ultrasonic testing procedures of the AWS D1.5 code.

3.8. Fabrication of Orthotropic Bridge in Shipyards

The fabricator's first set of trial-fabricated members showed significant distortion. Modifications were made, and bridge fabrication proceeded after new trial fabrication succeeded. The flat steel panels were cambered prior to welding in order to counter these distortions caused by ribs that are welded on only one side of the deck plate.

The fabricator employed several techniques, such as mechanically precambering the member in the elastic range of the material properties prior to welding, hold-down of free ends of the members, sequencing the welding operation, and finally applying minimized local heat straightening on some specific members.

The full deck was fabricated in 24 sections. Each deck section was made up of smaller pieces and fabricated together. Because of a decision to switch to using a shipyard rather than a bridge fabricator, 24 full-width sections were fabricated. Each of the 24 full-width sections was subdivided into six units. Thus, 140 somewhat identical units were fabricated inside the shop buildings.

Since the shipyard had bigger rollers and cranes, the steel plates were butt-welded to three plate components—a top deck plate, an edge plate, and a bottom plate for every unit. Next, the shipyard chose to weld both exterior sides of a trapezoidal rib with a pair of automatic welding machines to eliminate distortion.

The orthotropic ribs were placed on top of the flat steel plate, then 50% of the diaphragm plates were added to form one-sixth of the section. Next, these units were transported to locations in the dry dock using the massive overhead gantry crane. The units were welded together inside of portable welding enclosures to form a section. The sections were loaded onto the transport ships using the gantry crane. Figure H, Fabrication of Bridge Components for AZMB at Shipyards, in the appendix, includes a schematic of the shipyard and how it was converted to a bridge superstructure fabrication facility.

A shipyard can build larger components that a normal class 3 AISC/NSBA fabrication shop, since welded steel ship components are normally much larger. Thus it was possible to fabricate in only 24 sections. See Figure H, Fabrication of Bridge Components for AZMB at Shipyards, in the appendix.

Table 3.8.1 Evolution of Orthotropic Steel Decks for Suspension Bridges.

Bridge, Location, Date	Innovation	Fabrication Location	Comment
Germany	Patents on rib shapes and development of the first manual	Inside buildings	
Cologne–Muelheim, Germany, 1951	Rebuilding of existing bridge resulted in large dead-weight savings	Inside buildings	A practicable example used to encourage worldwide use of orthotropic decks
Severn, UK, 1966	Aerodynamic shape of superstructure	Fabricated outside near bridge	The actual sections were floated, not transported on barge or ship
Humber Bridge, UK, 1981	The largest sections that were fabricated and lifted using main cables	Fabricated outside near bridge	Record held for 17 years
Akashi-Kaikyo, Japan, 1998	Longest span in world	Inside buildings	Simultaneous welding of four ribs
AZMB, Carquinez, 2003	First new bridge to use Dr. Fisher's detailing	In a shipyard	The largest practicable sections were fabricated, shipped, and lifted into place

3.9. Cast Steel Cable Bands Were Shop-Poured

The structural connector between the main cable and the suspender ropes is the cable band, which is clamped around the main cable and shaped to allow the wire rope suspenders to properly carry their loads. The cable band consists of two semicylindrical halves connected with high-tensile steel bolts to develop the necessary friction. A cast steel thin wall thickness and good ductility are required to develop adequate clamping or frictional forces. See Figure L. Suspender Cable Details for AZMB, in the Appendix.

The cable bands have traditionally been cast with grooves on the upper surface so that the suspenders' wire ropes can simply be looped around the band. These are called "grooved" cable bands because an integral cast groove is provided for each wire rope. Grooved cable bands have been used on all American suspension bridges built in the last 100 years. For longer suspender ropes on the AZMB, the grooved cable bands were utilized.

As the inclination of the main air-spun cable varies along the span and as the grooves for the wire rope suspenders have to be arranged in a vertical plane, a larger number of molds for the steel casting process in the foundry are required. There were 15 types of castings required for this bridge. There is a higher foundry cost of casting U.S.-type, or “grooved,” cable band due to a more complicated mold. But cost savings were achieved because no special fittings, such as clevis, were required at the top of the suspender ropes.

The looping of the suspender wire ropes around the band has not been used on recent European suspension bridges. A clevis, or pin connection, is required between the cable band and an open socket at the top of the suspender wire rope. In this case, the lower part of each cable band half was shaped as a vertical gusset plate with the relevant pinholes. The cost advantage was that a larger number of cable bands could be made identical and eccentricities could be avoided by drilling the pinholes after casting the lower part of the cable band. However, because the number of clamping bolts had to be increased with the slope of the main cable, a number of different cable band sizes were required.

For the cable bands near the abutments, the U.S. tradition of looping the wire rope suspenders around the cable band has been substituted by the European system of having hangers with open sockets. The positioning of the gusset plates closer to the vertical tangent plane of the band cylinder also created a favorable effect in reducing the out-of-plane bending. Both styles of cast-steel cable bands were used on the AZMB.

3.10. Steel Wire Rope Suspenders Were Shop-Fabricated

A suspender clamp is a pair of plates that hold the four locations of wire rope in a pattern (see the right side of Figure L, Suspender Cable Details for AZMB, in the Appendix). The suspender clamp is positioned below the cable band to reduce the distance between the hanger ropes. Whether such a suspender clamp has to be used depends on the requirements of suspender rope spacing at the stiffening orthotropic box girder section. The AZMB used suspender clamps to reduce the suspender wire rope spacing. The Golden Gate Bridge also utilizes suspender clamps (see Figure L, Suspender Cable Details for AZMB, and the lower right part of Figure M, Steel Details—Tub Girders and Bridge Maintenance Access for AZMB, in the Appendix).

3.11. Clamping the Cable Bands in the Field

Waisting of a bolt means the shank has a smaller diameter than the threaded portions at each end. Normally, the bolts are highly stressed during the tightening of the nuts. To minimize the probability of bolt breakage in tension, it can be advantageous to make a “waisting” of the bolts so that the highest tensile stresses occur on the smooth central part of the bolt rather than the threaded portion. With such a “waisting,” the bolts may actually be tightened into the plastic range to provide a maximum margin against having the tension relaxed. The required friction at the cable band depends on the inclination of the cable, and the largest friction is consequently required near the towers. For this reason, the number of bolts in each cable band is generally increased from midspan toward the towers.

The details of cable-band castings for suspension bridges are shown in Figure K, Suspension Cable Details for AZMB, in the appendix. Also shown is the location of castings on a suspension bridge and typical details of the different types. For AZMB, the casting types are list casting from specials, drawings, and change order.

As the bolts are tightened, the cable bands will also be subjected to a bending force that will reduce the diameter until sufficient frictional forces are established around the main cable. The traditional practice is to leave a small gap between the two cable band halves to avoid the bolt tension transferring directly from one half to the other by contact pressure. The gap should be sufficient to allow retightening of the bolts, if needed at a later date. (Bolts stretch during the tensioning process). The design engineers required a 22 mm gap.

The constructor engineered additional cable bands based on the European style, which were made just to lift the sections from the oceangoing ship. These temporary cable bands were painted red and used to hold the strand jacks. See Section 3.6 for further discussion.

3.12. Transporting the Fabricated Sections

The deck section was fabricated in 24 pieces and then transported by ship across the Pacific Ocean. The bridge's orthotropic box girder section was fabricated into 24 sections carried by three special semisubmersible ships. The bridge sections, or units, were stacked two high in four rows, or eight per ship, with the fabricator's gantry crane.

Special care was taken in stacking the orthotropic steel components for ocean transport. The Pacific Ocean storm waves can rack or twist a transport ship, which is not infinitely rigid. Any movement of the ship could cause movement of the fabricated sections, and any change to the fabricated sections during transport would have been devastating. Three-dimensional finite element calculations demonstrated that any fatigue cracking caused by Pacific Ocean storm waves or improper handling could reduce the life of the bridge. A wave height of 10.5 m was assumed in the calculations.

Special steel "racks," or frames, were welded to the ship. Special, very large cantilevering "tab brackets" were welded to the sides of every bridge segment. Bearing pads were located at the bases, and the 24 bridge segments were bolted in the shipyard to the weld steel tops of welded steel "racks," or frames. Lateral stability parallel to the length of the ship and transverse direction was required. Diagonal bracing and details used by building or falsework engineers provided lateral stability under the forces generated by the waves. The geometry allowed double stacking in the shipyard and very quick unbolting during direct lift from the ship at Carquinez Strait.

The special, very large cantilevering "tab brackets" were removed once the superstructure had been properly erected. The criteria for grinding away a weld is discussed in Section 3.18,

3.13. Using the Suspension Cable as a Lifting Platform

The Coast Guard and tides determined when a ship could arrive to unload a section. The ship was moored nearby downstream, and everyone was notified when the ship was to arrive to unload a section.

During construction, the 24 orthotropic steel bridge sections were safely lifted from each ship. If a section was lifted unequally, sections could have racked or twisted, since each one was not infinitely rigid. Also, to maintain balanced loading onto the new cable and towers for the AZMB, sections were located in balanced patterns along the length of the bridge. That way, no component was overloaded or overstressed. The bridge sections, or units, were lifted using four "strand jacks," one per corner of the section. Each section had eight permanent suspender wire ropes, or double the size assumed necessary to practicably erect the sections.

The construction engineer devised temporary cable bands that clamped to the permanent suspension cables. All temporary components were painted red by the contractor to distinguish them from any final components. Permanent suspender wire ropes were later attached with final cable bands and hardware.

Some sections were placed on the bridge in order to be hoisted into places where the oceangoing ships could not be located directly below. Other techniques, such as trapezing on the north end over land and/or shallow water and using a horizontal movement on the south end, were necessary to build the bridge economically.

The contractor's practicable solution based on erecting and lifting many orthotropic steel bridge sections was a unique combination of existing techniques for the AZMB project. To manage the erection process of the superstructure, the constructors used concepts from a previously used technique of lifting full-width aerodynamic orthotropic sections for suspension bridges. This technique goes back to the 1966 British Severn Bridge, where sections were floated directly below the cables and lifted into place.

Safety was an important criterion during the orthotropic box girder lifting stage. First, the direction of current in the straits changes with the tide. The bridge is located in a narrow spot in the river that carries oceangoing ships transporting cargo upstream and downstream. There are high-power lines on the north end of the site and active intercontinental railroad lines to the south. The new bridge was very close to the active 1927 bridge carrying south-bound highway traffic. The general contractor, in conjunction with the fabricator, developed practicable-sized components for properly handling and erecting the bridge.

3.14. Horizontal Movement of Bridge Sections: Trapezing and Launching

The erection methodology to lift the deck sections into place is a procedure called trapezing, or launching, and is similar to how Tarzan or circus performers travel forward by hanging from a suspended rope. The switching of load from the rope behind the direction of travel must be properly synchronized. The trapezing process has been completed on other bridge projects, such as the 1988 Bosporus Bridge built by Mitsubishi Heavy Industries.

The contractor for AZMB had to trapeze sections in three locations. The first location was the center section of the bridge. The next location was the north end of the bridge, where there was shallow water. The final location required the ship to moor at the north face of the trestle built by Caltrans. The three sections (for the back span) further south of the south tower were trapezed through the tower base and set onto the launching equipment.

The level of construction engineering effort may be underestimated by those unfamiliar with this type of engineering. The contractor used extra temporary cable bands to perform the trapezing. Continuous movement of sections was achieved by wire rope or strand jacks supported by these extra bands. A series of planned locations, calculations, and erection drawings were required. Sections on the north and south back span had to be lifted in symmetrical positions to keep the overall bridge in balance (see Figure I, Key Stages or Phases of Bridge Erection for AZMB, in the appendix).

Launching, or horizontal movement of bridge sections, is much more common in Europe, with more than 1,000 bridges launched by contractors after World War II. In North America, fewer than 100 bridges have been installed by launch. Experts from Holland were retained by the contractor to engineer and launch the deck sections.

Since the constructor, in conjunction with the fabricator, had to develop practicable-sized components for properly handling and erecting the bridge, they utilized the trestle to maximum effect in the erection process.

Next, “Johnny Walkers” (using ironworkers’ slang), or devices composed of horizontal jacks and vertical jacks, were used to pull the sections to their final horizontal locations. Each section was moved on top of two steel beams that were the length of the trestle and four walking skates. A walking skate consists of a U-shaped steel plate frame about 7 feet long, 2 feet wide, and 2 feet high. Inside the steel frame were three hydraulic rams, or jacks, that were used for vertical lifting. Additional horizontal rams or jacks were used to push the section horizontally. The ram’s base slid onto the U plate, which kept it from toppling over. The coefficient of friction between the beams and skates kept them in position. To prevent human operator error due to boredom, computers controlled the cyclic behavior of the alternating lifting and pushing of synchronized rams. (A computer does not get bored with this slow-moving process.) All three sections had to be positioned horizontally on the trestle prior to vertical lifting off of the two steel beams by the strand jacks. This was necessary to achieve overall load balancing of the entire bridge.

3.15. Field-Bolting the 24 Orthotropic Steel Superstructure Sections

After each 600-ton section was lifted, it was bolted in place to the adjoining section. The critical issue in field-bolting the sections was how to achieve the precise root opening for the deck plate welding. The erectors had to bring two swinging objects close together enough so as to achieve a root opening of a few millimeters. The advantage was that the very large deck sections allowed a very large work area.

Ironworkers used oval-shaped access portals fabricated to provide passage inside the superstructure. During construction, access hatches and doors to the bridges were locked to prevent unauthorized people from entering the structure. Three construction alignment devices were used, in addition to the permanent bolted splices.

The bolting process can be described as follows:

A. Long bolts or rods were nested inside steel frames. A frame shape series of plates were welded to the top deck plate. A divided frame consisted of three long plates and two end plates, with bolt holes through the end plates. A frame was welded to each adjoining section to provide the appropriate alignment. The long rods had extra length with extra thread and allowed a mechanical method to slowly pull the sections together.

B. Tab plates were flat plates about 12 inches high and more than several feet long. These were shop-welded to the edge of the deck plate of the section with bolt holes. A shim plate was required to make up the gap from the thickness of the welds at the tab-plate base plus the root opening for the deck's permanent weld. Drift pins were used to line up the plates, and then ironworkers installed the temporary erection bolts.

C. T-plates were bolted to the exterior faces of the vertical-edge longitudinal bulkhead plate faces to allow quick alignment. The top part of the T was bolted to one section's bulkhead plate, and when the adjoining section was swung into place, the two sections were hooked together. Drift pins were used to line up the plates, and then ironworkers installed the temporary erection bolts. Access holes for employees are in the bulkhead plates.

Two different types of rib splices were used on the AZMB. All of the ribs were field-bolted. The bolting of open ribs was much easier than closed ribs. Hand holes were required for closed ribs. There are 20 trapezoidal ribs on the bottom side of the superstructure and 28 on the top deck. There are 36 bolts per rib. Ironworkers installed 36 bolts x 48 ribs = 1,728 bolts, providing precise alignment prior to welding. There are seven open ribs on the inside edges of the superstructure. These splices were made quicker and aligned both sides of the superstructure quickly. Ironworkers installed 18 bolts x 7 ribs = 126 bolts providing precise alignment prior to welding. Caltrans required unpainted surfaces at splice plates because paint in the joint increases friction.

The bolting process for the orthotropic steel bridge sections was a concern since the pulling of sections could rack or twist them since they were not infinitely rigid.

Welding of sections did not start until all 24 sections were bolted. Welding of adjoining sections was next after connections were completed between all 24 sections. Caltrans determined that the initial bolts were damaged, and final replacement bolts were required.

3.16. Field-Welding the Sections at the Top Deck Plate

To field-weld 24 orthotropic steel bridge sections, the designers recommended field-welding the top steel deck plate so as to achieve the thinner wearing surface. The field-bolting of the sides and bottom of the superstructure's ribs helped to field-align the top deck plate. However, to achieve the proper root opening for deck welds, vertical tab plates were required for erection purposes. Vertical plates were welded to allow adjustment bolts to create the correct root opening for the weld. (See Figure G, Orthotropic Steel Superstructure Details of AZMB, in the appendix.)

Preheating the steel deck plate was required by code for the welding process of the orthotropic steel bridge sections. Subcontractor California Erectors made the welds after bolting 23 erection splices across the top deck, sloping sides, and tub section. The welds were made in a sequence but were not done continuously to spread the heat input to control distortion.

A flux-core self-shielded electrode process was used for submerged arc welding. Welding was done on the underside of the superstructure including the sloping sections.

The deck welds were different and presented some trouble. There were also porosity problems, and the welders could not pinpoint the cause. One theory put forth was that there was contamination in the joint from the foam used to protect the joints in the transit across the ocean. Also, contamination could have come from the construction job site, as constructors were working on the bridge surface in the open air. There may have also been some paint or rust in the joints, as these contaminants are made up of very, very small particles.

Because of the contamination, the welders made an extra effort to thoroughly clean and seal all joints with stick electrode SMAW, abbreviation of shielded metal arc welding. The joints were sealed before the submerged arc weld was made.

3.17. Wrapping the Cables for Corrosion Protection

Corrosion protection is paramount to protecting the steel cables of suspension bridges. Many older suspension bridges have had to be retrofitted to protect cables and anchor bars from corrosion. The designers for AZMB selected a traditional proven system of protections that have been used on a large number of suspension bridges. Figure 3.11 shows the arrangements of the strands before they were compacted into the cable. It also shows the finished protected cable.

The evolution of suspension bridge cable protection technology is briefly summarized in Table 3.17.1.

Table 3.17.1 Evolution of Corrosion Protection for Suspension Bridges.

Bridge, Location, Date	Main Cable Wires	Wrapping System	Comments
Brooklyn, NY, USA, 1888	Galvanized	Annealed wires, wrapped lead paint	John Roebling's patent of 1841; original wires still in use
Williamsburg Bridge, NY, USA, 1903	No. 6 gauge uncoated bright wires	Cotton duck soaked in oxidized linseed oil and varnish with a sheet iron shell sheet iron covering	7 years later in 1910 corrosion of wires was discovered; more repairs and inspections have occurred since that time
Golden Gate Bridge, CA, 1937	Galvanized	Galvanized wire wrapping with red lead paste	Original wires still in use; now lead-based paint is very limited
Bidwell Bar, Orville Dam, CA, 1965	Helical-type galvanized bridge strands	Acrylic-resin and glass fabric covering with nylon film and polyethylene cable fillers	Patent by Bethlehem Steel, DuPont 20-year warranty; protection system remains unchanged
Humber, UK, 1981	Galvanized wire liberally coated with red lead paste	3.5 mm diameter wire with 5-coat paint treatment	Lead-based paints have been the most durable, but are toxic
AZMB, Carquinez USA, 2003	Galvanized wire with paste smeared all over	3.5 mm wrapped galvanized wire with 3 coats of paint	Deemed to be most practicable for the environmental conditions of this site

3.18. Grinding the Sections at the Top Deck Plate Welds

After the sections were connected, the cantilevering tab brackets used for the ocean transport ships were cut away. The remaining plate ends and welds were ground in accordance with the Caltrans special provisions.

Tab plates used for aligning two sections were cut away. The remaining plate ends and welds were then ground.

The grinding process for the orthotropic steel bridge sections was necessary as there was field welding on the top steel deck plate. A steel deck is used to achieve a thinner wearing surface. Before the wearing surface was placed, there was field-grinding of the 23 welded joints for the 24 orthotropic steel bridge sections.

The field vertical plates were cut and field-ground to eliminate the occurrence of fatigue cracking and to ensure the proper installation of the bonding agent between the wearing surface and top deck. Any poor welding was rejected by Caltrans. Unacceptable field welds were ground and rewelded or repaired.

3.19. Details about the Wearing Surface on Top of the Orthotropic Steel Deck

A wearing surface is placed on top of a steel deck to protect it from tire abrasion. The deck wearing surface is included in the structural properties of steel orthotropic bridge deck systems. The deck system optimizes:

- Light weight
- Strength and rigidity
- Structural redundancy
- Capability of acting as an integrated component
- Low structural depth
- Elimination of deck joints or radical reduction of their number
- Durability and long-term economy

Surfacing on a steel orthotropic deck acts compositely with the steel deck plate and must be regarded as an integral part of the structural deck system.

The local deck flexibility can be a factor in satisfactory performance of the wearing (driving) surface. Another location underneath a coating where the coating surface can be overstressed by flexure is where two pieces of steel meet. For this reason, engineers try to limit the number of pieces of the whole deck.

The wearing surface is a basic requirement as load and resistance factors in a bridge steel girder design. The integrated structural elements of the deck include the wearing surface.

Bridges last longer than wearing surfaces, and the wearing surface may eventually need to be replaced. The projected life span for the AZMB is 100 years. However, wearing surfaces can last for decades. For example, the San Mateo–Hayward Bridge's original wearing surface is in good condition after approximately 40 years of heavy traffic.

Since the coating can never match the life of structural steel, its attractiveness relates to how long it lasts and how easy the coating is to apply—both in the original fabrication and then *in situ* on the structure over the life of the bridge. Due to its importance, there has been a significant investment by FHWA and other governmental agencies to sponsor studies that substantiate the predicted failure of the orthotropic deck wearing surface coatings, study their endurance, and validate analytical models.

One way to reduce dead load on an orthotropic deck is to use thin mastic and/or a polymer-modified asphalt surface. These tend to require higher application and compaction temperatures than conventional surfacing. This in turn necessitates a waterproofing system of the highest quality and ability. Waterproofing material should be durable and resilient enough to withstand the conditions and other strains caused by orthotropic deck features. The waterproofing material can develop high bond strengths to the steel deck and asphalt pavement.

A steel deck is a stern test for any waterproofing system, but orthotropic deck behavior is livelier than that of conventional truss-stiffened decks. The deck is subject to a great level of expansion and contraction since the heating and cooling cycle of a steel deck is much quicker than that of a concrete deck.

The design mix for the AZMB wearing surface is:

1/8-inch Epoxy seal, shop-applied

1-1/2 inches of asphalt, field-applied using Trinidad Lake asphalt to protect the 16 mm steel deck plate

3.20. Placing Wearing Surface

The bridge deck membrane was applied over an area of 285,240 square feet, or 26,500 square meters. The waterproofing system has the following characteristics:

- Rapid curing methacrylate (MMA) system
- Spray-applied
- Bonds to itself, even after curing
- Produces a seamless membrane with no vulnerable joints
- Coats the contours of the deck evenly
- Primer is MR-6 metal primer, with superior adhesion to the deck
- Liner membrane also has superior adhesion to the primer
- Adhesion properties eliminate shoving of the membrane while applying the asphalt paving and while subject to construction traffic before paving
- There is a resistance to penetration or indentation during paving and compacting

The application of the deck wearing surface system took place in the months of August and September 2003 in a location where air temperatures could fluctuate. Sea fog played havoc with the dew point, so application decisions had to be on a day-to-day or night-to-night basis. To prevent condensation, the material had to be applied when the steel deck temperature was at least 3 degrees Fahrenheit above the dew point to prevent condensation forming. There was constant monitoring of air and deck temperature.

The deck coating took place in three phases. The largest phase, which began in early September 2003, was waterproofing the main road. Complications for this work included strong crosswinds and temperature and dew-point limitations. But the contractor was also required to keep a lane open for all the construction traffic that continued for other phases of the work. Despite the complications, the work was completed in 18 days. The second phase included waterproofing and paving, which occurred in October 2003. The third and final phase—the bicycle and pedestrian lane—was completed in just 2 days.

This staging allowed for completion of one phase of the system, per given area, before going on to the next. This expedited the installation of a complete membrane system and opened up large areas to construction traffic.

For quality assurance purposes, the membrane was applied in two color coats. The first coat was bright yellow and a pigment concentration that produced an opaque coating when installed at the specific thickness of 50 mils (1.25 mm). The second coat, also applied at 50 mils, was light gray with the same pigment characteristics.

Other quality assurance methodologies included wet film gauges during the entire application. They confirmed that the minimum film thickness per coat was met and the total membrane thickness was 100 mils (2.5 mm).

3.21. Steel Tub Girder Placement over Railroad Tracks

Caltrans engineers have designed steel tub girders throughout California since the mid 1960s. The structural connection or transition section between the prestressed concrete approach bridge spans and the AZMB were steel tub girders. The steel tub girder can quickly be installed and not impact railroad traffic or its safety. These tub girders span the main rail lines that go east to west. A typical section at the tub girders is shown on Figure M, Steel Details—Tub Girders and Bridge Maintenance Access for AZMB, in the appendix. Caltrans bridge engineers designed these tub girders. Welds were designed to match plate strength.

The Adams and Smith Company began constructing the tub girder placement in late August 2003. This consisted of setting four 100-ton tub girders across the 160-foot gap between the suspension span and the freeway approach. This work was done at night. The placement of the girder was also constrained because it occurred over vital active railroad tracks.

3.22. Maintenance Traveler

The fabricated steel maintenance traveler is used to provide access for bridge maintenance painting and inspections and utilities maintenance by Caltrans employees. It is a moving platform on which materials, people, and equipment can be transported the length of the bridge. Since the AZMB superstructure is one continuous box girder for the entire length, only three travelers or one maintenance traveler system was built for each span. Access from the traveler is to the outside of the orthotropic-shaped box girder.

On the AZMB, one can inspect inside the orthotropic-shaped box girder. One must be able to inspect for fatigue cracks inside the box that may not be seen on the outside. Interior inspection carts were not included for the Carquinez Bridge due to its short length. A traveler suspended from rails attached to the bottom of the superstructure as seen in Figure M, Steel Details—Tub Girders and Bridge Maintenance Access for AZMB, in the appendix, allows for the required exterior inspections. Interior inspections are allowed through openings in diaphragms.

Another AZMB maintenance feature is shown in Figure M. That feature is the reinforced concrete tower "doughnut"-shaped stairs or elevator shaft to the main towers and main cables. Hand ropes allow workers to walk on main cables with safety harnesses with safety tie downs.

3.23. Steel Protective Coatings

Steel protective coatings are designed to provide long-term corrosion protection, a light stable aesthetically pleasing color coat, and ease of application. Properly applied zinc-rich coating systems are considered permanent, if maintained by repairing damaged areas that compromise the integrity of the zinc-rich primer. After 25 to 50 years, re-topcoating the system may be required. However, a zinc-rich system will never required complete removal to bare metal.

The coating system limits the weather or corrosion effects on structural steel bridge systems. The coatings reduce inspection efforts and maintenance cost and increase the structural effectiveness and redundancy of the bridge. Material choices come about from a collection of influences: (1) past experience, (2) expertise, (3) new technology, and (4) competitive marketplace.

New bridge construction painting takes advantage of high-performance coatings. In 1983, the expected performance of an organic or inorganic zinc-rich primer coating system was considered permanent.

If constructors and/or painters take special care when applying the coating, a single coat can perform well in a neutral environment with a pH of 5–10. For conditions outside this range, a single topcoat of either vinyl or epoxy is recommended. Coatings are formulated to flow, level, and cure efficiently in high and low temperatures.

The paint system for AZMB included general coatings, volatile organic compound (VOC) content, product data sheets, application equipment, and surface preparation. The use of protective coatings included shop-painting of bare steel, grinding, and field-painting. For the orthotropic box girder, surface grinding in the field was kept to a minimum.

After the fabricated pieces were lifted into position, welded, and bolted into place, painters then thoroughly cleaned the prepared steel. The AZMB Caltrans contract provided special specifications regarding the composition of each coating—what binders, solvents, and additives were part of the composition—that affected the ability to control corrosion.

Major sections of the steel superstructure that required coatings or painting included the deck, cable, railings, traveler, rebar, light poles, box girder, verticals stringers, exterior surfaces of the box girder, wind tongue, rocker links, traveler rails and saddle, ornamental steel rail, tub girders over the railroad tracks, and the bicycle/pedestrian lane.

AZMB is painted on the inside. The coating systems for AZMB varied with each major structural element. For example, the cables are "coated" or painted as part of the cable installation, whereas painting of the orthotropic box girder and deck was done at fabrication. The components have been treated separately or, as in the example of the big bolts or saddle, may have been coated at fabrication.

The color topcoat adds the aesthetic appeal to the bridge and is a very important part of the public interaction in the bridge type selection process. The Caltrans Bridge Architecture and Aesthetics branch was responsible for choosing the colors. The architects used a color selection process that involved studying the environment and obtaining local public input.

The community of Crockett was involved in choosing the colors of the bridge. There was considerable debate regarding the color of the concrete and steel. The selection of the colors considered seasons. Native flora color was evaluated and a palette of acceptable color combinations was created. Simulations of the bridge in various colors were presented to community members for feedback.

Figure 3.23.1: Photo by passenger from inside car driving across the AZMB heading north (the black object is the rearview mirror)

Figure 3.23.2: Photo by passenger from inside car driving towards the AZMB heading south (see Figure A in the appendix).

Figure 3.23.3: Photo by passenger from inside car driving across the AZMB heading south (1927 and 1958 bridges)

4. Conclusions and Further Reading

4.1. Costs

. The AZMB was built at a cost of $813 per square foot of deck area.[3]

4.2. Other Key Facts

Following are additional statistics for the AZMB:

- Main span – 728 meters, 2388 feet
- North side span – 183 meters, 600 feet
- South side span – 148 meters, 486 feet
- Center clearance over water – 47 meters, 54 feet
- Tower height from water – 131 meters, 430 feet
- Structural concrete – 44,380 cubic meters, 1,568,000 cubic feet
- Structural steel – 12,722,000 kilograms, 28,000,000 pounds
- Steel pile – 6,030 meters, 19,783 feet
- Start of construction – January 28, 2000
- Bridge opened to traffic – November 2003

4.3. Credits to Primary Organizations

The AZMB is owned by the California Department of Transportation (Caltrans). However, as in any great infrastructure project, the participation to bring this bridge to fruition reached around the world. The suspension bridge was designed by a joint venture called Parsons–OPAC-JV. The design oversight was conducted by the Caltrans Engineer Service Center. The bridge was constructed as a joint venture by FCI/Cleveland Bridge. Tub girders and concrete bridges were built by Caltrans engineers.

The design of the bridge was completed in June 1999. Caltrans was responsible for the public bidding of the contract. The bids were opened in January 2000, and the lowest bid was submitted by a 65–35% joint venture of FCI Constructors Inc. and UK-based Cleveland Bridge. The construction contract used a bidding specification called “A+B” to allow the public agency to consider time when determining who the lowest bidder is. FCI/Cleveland had a bid of $188 million and 1,000 days.

---

[3] Based on information on FCI website.

The Lincoln Electric Co. provided equipment and consumables for AZMB and also trained welding operators working on the bridge. Lincoln Electric supplied welding supplies and published the *Orthotropic Bridge Handbook* through the James F. Lincoln Arc Welding Foundation.

Suspender wire rope was made in the United States by the Wire Rope Corporation of America.

Fabrication of the bridge deck was carried out by the Japanese firm Ishikawajima–Harima Heavy Industries Co. Ltd. (IHI).

Fabrication of the cables was carried out by Changqing Wonderbridge Cable Co. Ltd. in China.

In addition to these organizations, many other organizations and individuals contributed in various capacities and various ways to the construction of the AZMB.

4.4. A Successful Conclusion to a Complex Steel Bridge, the AZMB

The FHWA has been promoting the ABC (accelerated bridge construction) process. Its mantra "Get in, get out, and stay out," was achieved. The general driving public was not affected by utilizing all three bridges. The contractor built the AZMB without affecting the vital oceangoing ship traffic to the various ports along the Sacramento River. The owner chose a structure that fits the site and that the general public is pleased with in all aspects. The AZMB is a fitting memorial to Alfred P. Zampa and the California union ironworkers and a tribute to the dedication of all who worked to design and construct this bridge. It is also a testament to the technological innovations and possibilities that evolved from building successful and durable steel bridges.

4.5. Future Orthotropic Suspension Bridges in the USA

The AZMB is not the last large orthotropic bridge to be built in the United States. In the summer of 2006 orthotropic steel deck sections will be lifted for the Tacoma Narrows Bridge owned by the Washington Department of Transportation.

Figure 4.5.1: Two photos of the Tacoma Narrows Three Bridge lifting device built by a Korean firm

Caltrans has installed two similar orthotropic sections of the east span's replacement project. Both of these steel box sections were fabricated by two different Oregon-based firms. The SAS (self-anchoring suspension) bridge will be awarded in the summer of 2006.

In New York City the orthotropic redecking of the Bronx Whitestone Bridge is under way. It's a complex conversion to remove the retrofit wind-stiffening trusses added to the bridge, which was similar to Tacoma Narrows One or "Galloping Gertie."

4.6. Further Readings

- **Al Zampa Memorial Bridge 2003**

Bayol, Greg, Ney, Bart and Boal, Brian. *Spanning the Carquinez Strait: The Alfred Zampa Memorial Bridge*. Sacramento, CA: California Department of Transportation, 2003.

Marquez, M., Orsolini, G., Ketchum, M and Spoth, T "Suspension Solution," *ASCE Civil Engineering Magazine*, June 2000.

Mathur, Ravi, Ketchum, Mark A., Orsolini, Greg and Chang, C.Y. "Seismic Design of the New Carquinez Bridge." *International Bridge Conference 2001*, Pittsburgh, PA Engineers Society of Western Pennsylvania (paper number IBC-Ol-54).

Robinson, John V. *Spanning the Strait*. Crockett, CA: Carquinez Press, 2004.

Schatzmann, Richard, and Boal, Brian. Video: *Spanning the Carquinez Strait—The Alfred Zampa Memorial Bridge*. Sacramento, CA: California Department of Transportation, 2004, 35 minutes. This video can be purchased in either VHS or DVD format by e-mailing library@dot.ca.gov)

- **Wind and Earthquake Design Issues**

Caltrans. Bridge Design Aids. Available for downloading at www.dot.ca.gov/hq/esc/techpubs.

- **Other Useful Case Histories on Other Suspension Bridges**

Gimsing, Neils. *Cable Supported Bridges*, 2nd ed. New York: John Wiley & Sons, 1997.

- **Al Zampa**

Adams, Charles F. *Heroes of the Golden Gate*. Palo Alto, CA: Pacific Books, 1987.

Bayol, Greg, Ney, Bart and Boal, Brian. *Spanning the Carquinez Strait: The Alfred Zampa Memorial Bridge.* Sacramento, CA: California Department of Transportation, 2003.

Robinson, John V. *Spanning the Strait*. Crockett, CA: Carquinez Press, 2004.

- **1927 Bridge and 1958 Bridge Readings**

Caltrans. *Historic Bridges of California*. Sacramento, CA: California Department of Transportation, 1990.

Derleth, C., Jr. "Cantilever Highway Bridge Across Carquinez Strait," *Engineering News Record*, Vol. 95, No. 13, Sept 24, 1925.

Hollister, Leonard. *Now Read This—Carquinez Bridge Project Tests Ingenuity of Engineers*. Sacramento, CA: California Highways and Public Works.

Steinman, D. B. "Designing the Carquinez Cantilever Bridge," *Engineering News Record*, May 12, 1927.

- **Bridge Background**

AASHTO. *Load and Resistance Factor Design Specification for Bridges*. Washington, DC: American Association of State Highway and Transportation Officials, 1998.

Caltrans. *Competing against Time*. Sacramento, CA: California Department of Transportation, 1990.

Caltrans. *Continuing Challenge*. Sacramento, CA: California Department of Transportation, 1994.

Petroski, H. *Engineers of Dreams: Great Bridge Builders and the Spanning of America*. New York: Alfred A. Knopf, 1995.

- **Orthotropic Bridges**

AASHTO. *Load and Resistance Factor Design Specification for Bridges*. Washington, DC: American Association of State Highway and Transportation Officials, 1998.

AISC. *Seismic Provisions for Structural Steel Buildings,* 2005, Chicago, IL: American Institute of Steel Construction, 2005.

ASCE. *Proceeding from the Orthotropic Bridge Conference*. Sacramento, CA: ASCE Capital Branch of Sacramento, 2004.

Ketchum, Mark. Carquinez Strait Bridge Page. Available at http://www.ketchum.org/carquinez.html.

Orsolini, G., et al. "Design of Box Girders," in *Proceedings of the Orthotropic Bridge Conference*. Sacramento, CA: ASCE.

- **Cable Protection**

The National Association of Corrosion Engineers (NACE) and the American Galvanizing Association (AGA) are other organizations that can provide useful information.

References

Adams, Charles F. *Heroes of the Golden Gate*. Palo Alto, CA: Pacific Books, 1987.

Angeloff, Carl, and Eric S. Kline. “Protective Coatings for Steel Construction,” a presentation sponsored by the National Steel Bridge Alliance (NSBA) and the California Department of Transportation (Caltrans), Sacramento, CA, 1998.

Ashton, N. L. “Arc Welding in Design, Manufacturing, and Construction,” Chapter VII in *Arc Welded Steel Plate Floors Applied to Bridges and Viaducts*. Cleveland, OH: The James F. Lincoln Arc Welding Foundation, 1939.

Bayol, Greg, Ney, Bart and Boal, Brian. *Spanning the Carquinez Strait: The Alfred Zampa Memorial Bridge.* Sacramento, CA: California Department of Transportation, 2003.

Bennett, David. *The Creation of Bridges*. Chartwell Books, Inc. Edison, New Jersey, 1999.

Caltrans. *Competing against Time*. Sacramento, CA: California Department of Transportation, 1990.

Caltrans. *Continuing Challenge*. Sacramento, CA: California Department of Transportation, 1994.

Caltrans. *Historic Bridges of California*. Sacramento, CA: California Department of Transportation, 1990.

Connor, John W., and Robert J. Fisher. “Results of Field Measurements Williamsburg Bridge Replacement Orthotropic Deck—Final Report on Phase III.” ATLSS Report No. 01-01, Lehigh University, 2001.

Constantino, Frank. “Wearing Surface on the New Alfred Zampa Suspension Bridge across the Carquinez Strait” in *Proceedings of Orthotropic Bridge Conference*. Sacramento, CA: ASCE Capital Branch of Sacramento, August 2004.

Fisher, John. “Existing and Future Steel Bridge Infrastructure.” in *Proceedings of Orthotropic Bridge Conference*. Sacramento, CA: ASCE Capital Branch of Sacramento, August 2004.

Fisher, John. *New Deck Williamsburg Suspension Bridge*. Lehigh University Bethlehem, PA, 1993.

Gimsing, Niels J. *Cable Supported Bridges—Concept & Design*, 2nd ed. New York: John Wiley & Son, 1997, 471 pages.

Gotchy, Joe. *Bridging the Narrows*. Puyallup, WA: Valley Press, 1990.

Hetzendorfer, Robert. “Retrofitting San Francisco’s Bay Bridges to Withstand Earthquakes,” *Welding Journal*, February 2005, Volume 84, Number 2.

Khazem, Serzan. “New Deck Williamsburg Suspension Bridge”. in *Proceedings of Orthotropic Bridge*

Conference. Sacramento, CA: ASCE Capital Branch of Sacramento, August 2004.

Marquez, M., Orsolini, G Ketchum, M and Spoth, T "Suspension Solution," *ASCE Civil Engineering Magazine*, June 2000.

Marquez, Michael, Thimmhardy, Eugene and. Wolfe, Raymond W. "New Carquinez Strait Suspension Bridge—Fabrication and Sea Transportation of Orthotropic Steel Box Girder Segments" in *Proceedings of Orthotropic Bridge Conference*. Sacramento, CA: ASCE Capital Branch of Sacramento, August 2004.

Mathur, Ravi, Ketchum, Mark A Orsolini, Greg and Chang, CY. "Seismic Design of the New Carquinez Bridge." International Bridge Conference, 2001 (paper number IBC-Ol-54).

Orsolini, G., et al. "Design of Box Girders," in *Proceedings of Orthotropic Bridge Conference*. Sacramento, CA: ASCE Capital Branch of Sacramento, August 2004.

Petroski, H. *Engineers of Dreams: Great Bridge Builders and the Spanning of America*. New York: Alfred A. Knopf, 1995.

Robinson, John V. *Spanning the Strait*. Crockett, CA: Carquinez Press, 2004, 115 pages.

Schatzmann, Richard, and Boal, Brian. Video: *Spanning the Carquinez Strait—The Alfred Zampa Memorial Bridge*. Sacramento, CA: California Department of Transportation, 2004, 35 minutes.

Strauss, Joseph B. *Golden Gate Bridge—Report of the Chief Engineer to the Board of Directors of the Golden Gate Bridge and Highway District, California*. San Francisco, CA: Golden Gate Bridge and Highway District, September 1937, 246 pages (5000 COPIES); reprinted on the 50th anniversary, 1987 (Anniversary Edition).

Thimmhardy, Eugene, Marquez, Michael and Wolfe, Raymond W. Report presented at Steelbridge 2004: Steel Bridges Extend Structural Limits at Millau on June 23, 2004.

Troitsky, M. S. *Orthotropic Bridges—Theory and Design,* 2nd ed., Cleveland, OH: The James F. Lincoln Arc Welding Foundation, 1987.

Wahbeth, Mazen, Boal, Brian and Merrill, Jim. "Technical Fabrication Issues with Orthotropic Steel Box Girder Bridges" in *Proceedings of Orthotropic Bridge Conference*. Sacramento, CA: ASCE Capital Branch of Sacramento, August 2004.

Wolchuk, Roman. "Seminar on Orthotropic Deck Bridges" part of in *Proceedings of Orthotropic Bridge Conference*. Sacramento, CA: ASCE Capital Branch of Sacramento, August 23 and 24, 2004.

Wolchuk, Roman. *Design Manual for Orthotropic Steel Plate Deck Bridges*. Chicago, IL: American Institute of Steel Construction (AISC), 1963.

Wolchuk, Roman. "Old Bridges Give Clues to Steel Deck Performance," *AISC Journal*, October 1964, pp. 137–140.

Appendix: Figures A to M

Figure A. Site Issues

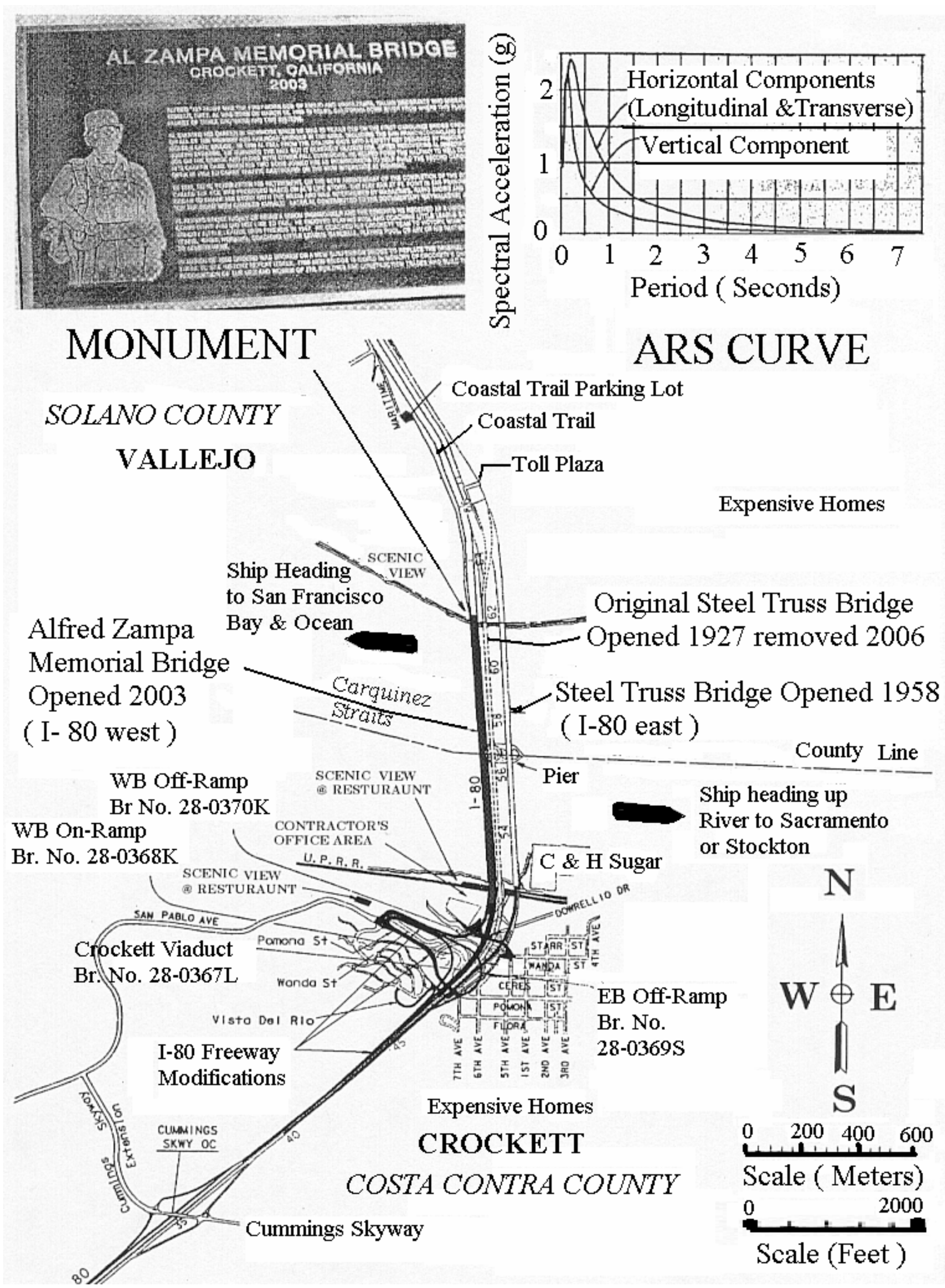


Figure B. Orthotropic Decks Dominate Big Bridges

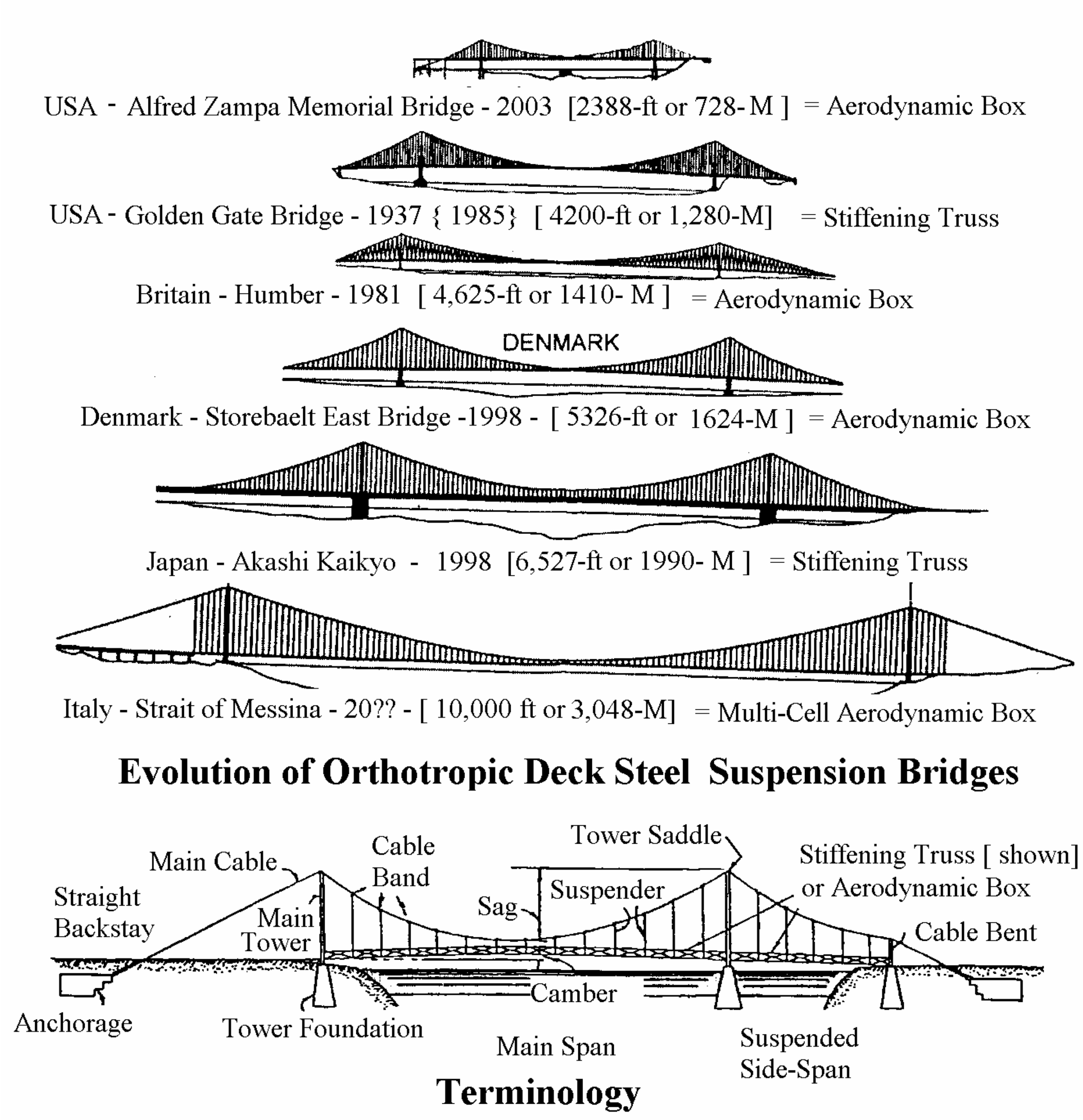
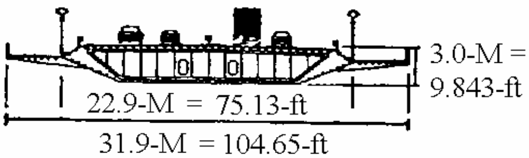
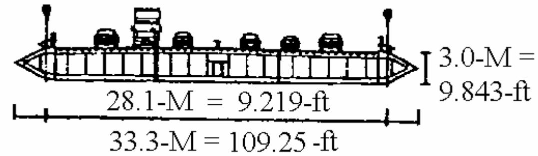


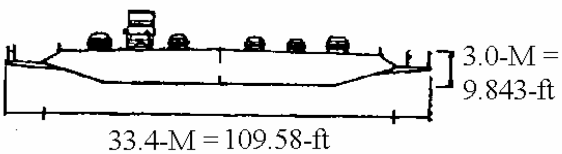
Figure C. Aerodynamic Orthotropic Superstructures



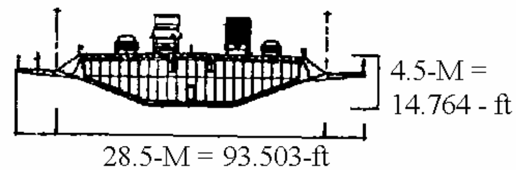
Severn @ Transverse Diaphragm
(1966) Britain



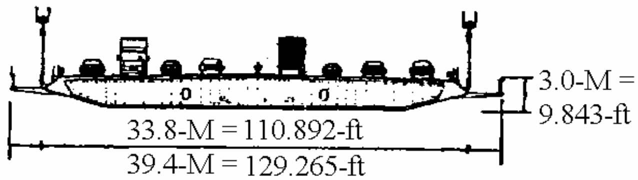
Littlebaelt @ Transverse Diaphragm
(1970) Denmark



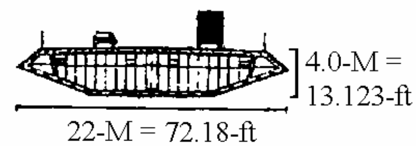
1st Bosphorus @ Transverse Diaphragm
(1973) Turkey



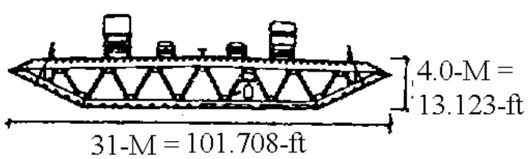
Humber @ Transverse Diaphragm
(1981) Britain



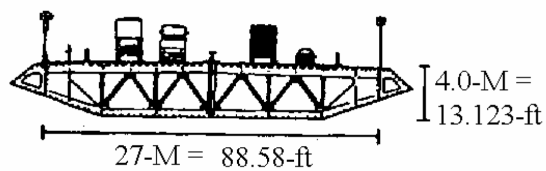
2nd Bosphorus @ Transverse Diaphragm
(1988) Turkey



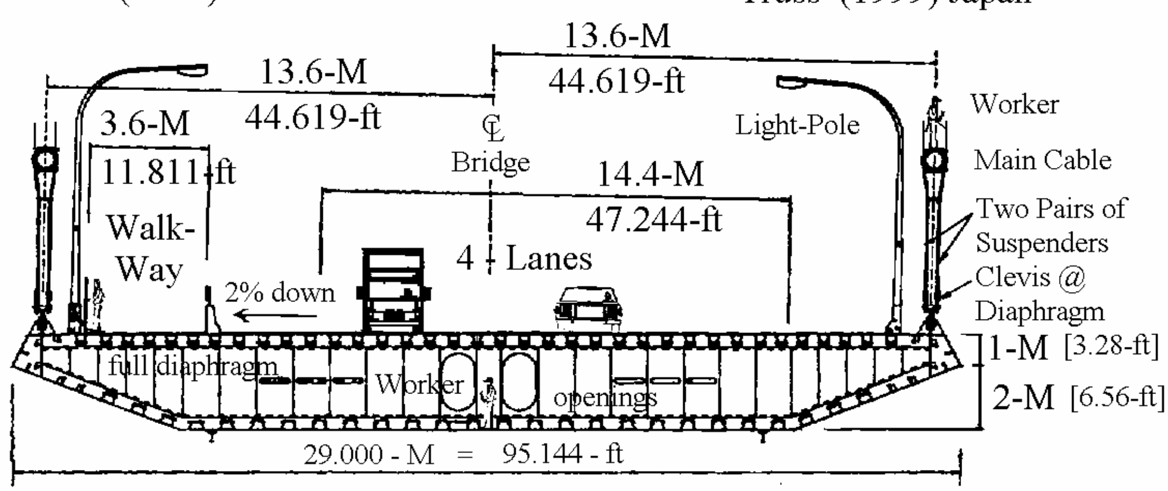
Hoga Kusten @ Transverse Diaph.
(1997) Sweden



Storebaelt, East Bridge @ Transverse Truss
(1998) Denmark



Kurushima "2" & "3" @ Tranverse
Truss (1999) Japan



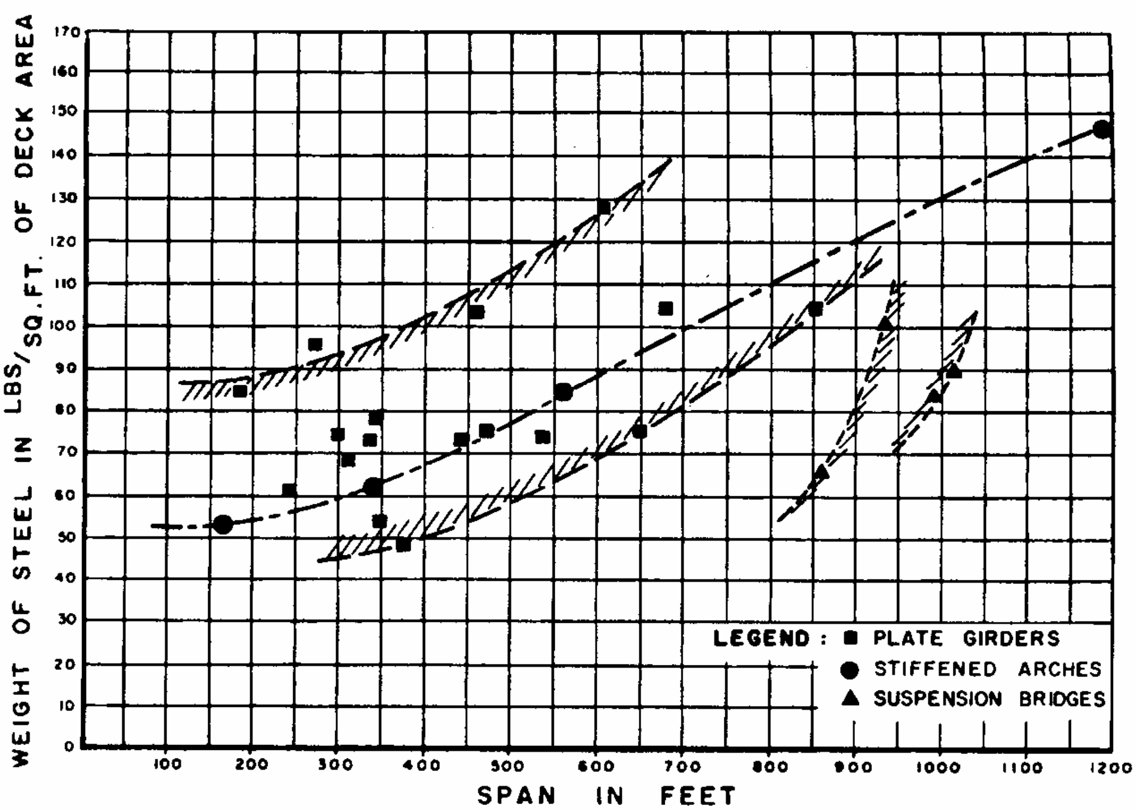
Alfred Zampa Memorial Bridge @ Transverse Diaphragm - (2003) USA

Figure D. Reduce Entire Self-Weight of Superstructure by Using an Orthotropic Steel Deck

Selected Completed Orthotropic Redecking Projects in the U.S. and in Canada
Table created by Roman Wolchuk, PE - Consulting Engineer

SUSPENSION BRIDGE	Lions Gate Vancouver [a]	G. Washington N.Y.C.	Golden Gate S. Francisco	Throgs Neck V. N.Y.C.	Ben Franklin Philadelphia
YEAR OF REDECKING	1975 / 2001	1978	1985	1986	1987
Old deck weight (psf)	100	106	104	107	123
New deck weight (psf)	61	60	79	83	89
Redecking cost ($/sf)[b]	47	40	70	72	79
Deck integration with main members	Yes	No	No	No	Yes
Redecking work	Night	Night	Night	Night	Day

[a] The 1975 orthotropic roadway was replaced by a new widened orthotropic deck.
[b] Excluding bridge repairs, inspection walkways, utilities relocation and other items not related to roadway redecking



- Variation Of Steel Weight With Span For Existing Orthotropic Bridges.
COURTESY OF JAMES F. LINCOLN ARC WELDING FOUNDATION

Figure E. Bridge Geometry of AZMB

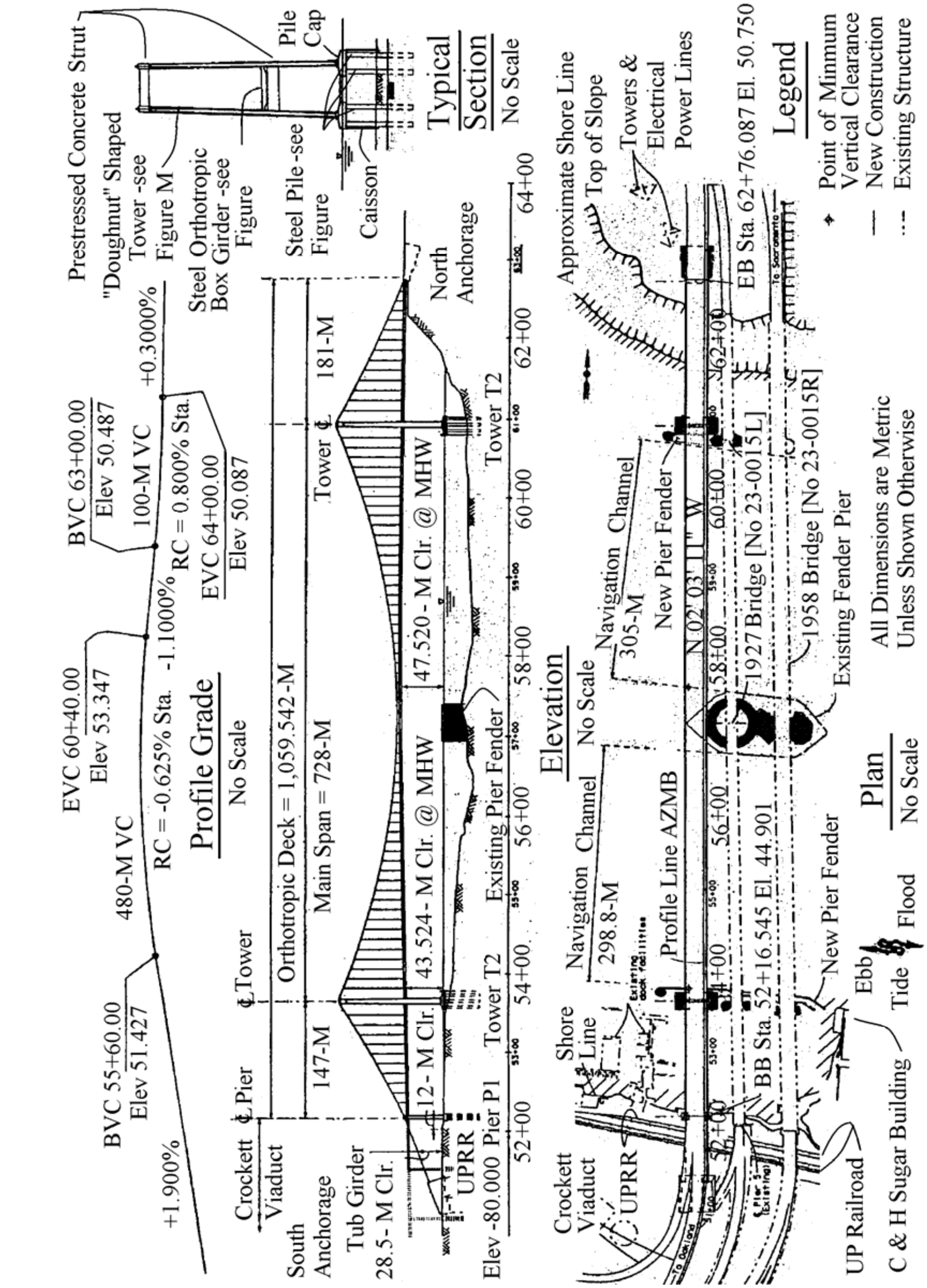


Figure F. Rib Geometry and Details

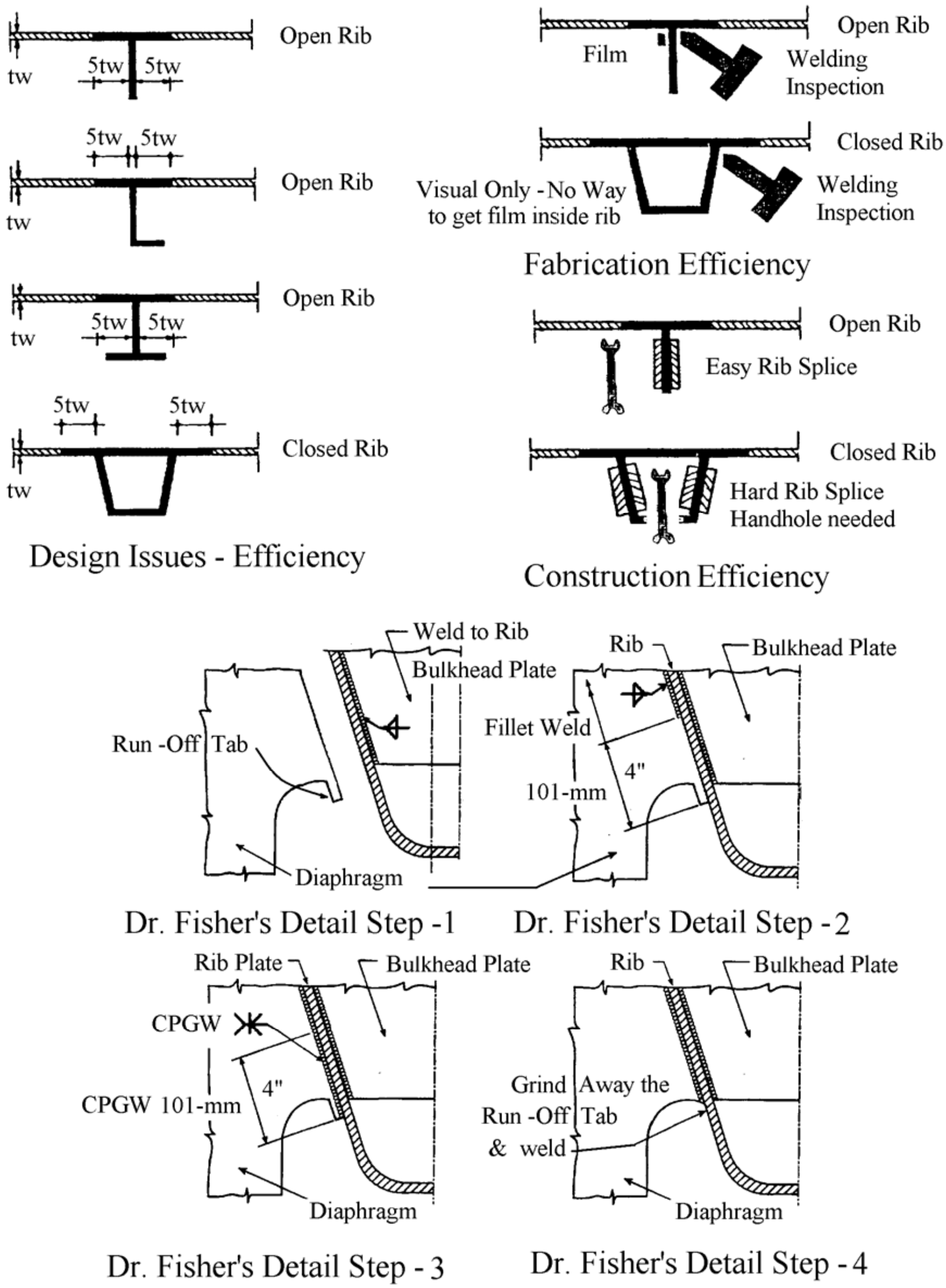


Figure G. Orthotropic Steel Superstructure Details of AZMB

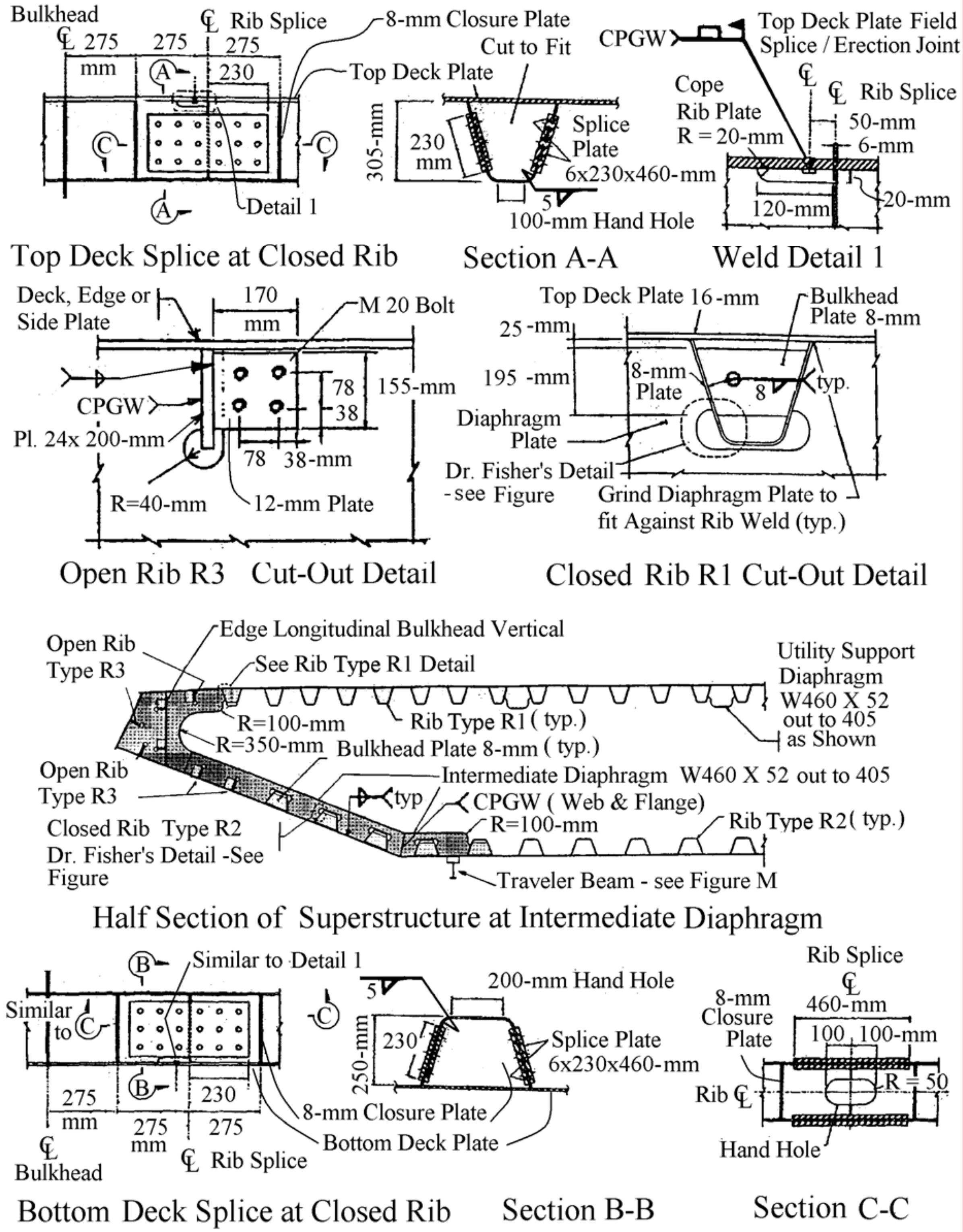


Figure H. Fabrication of Bridge Components for AZMB at Shipyards

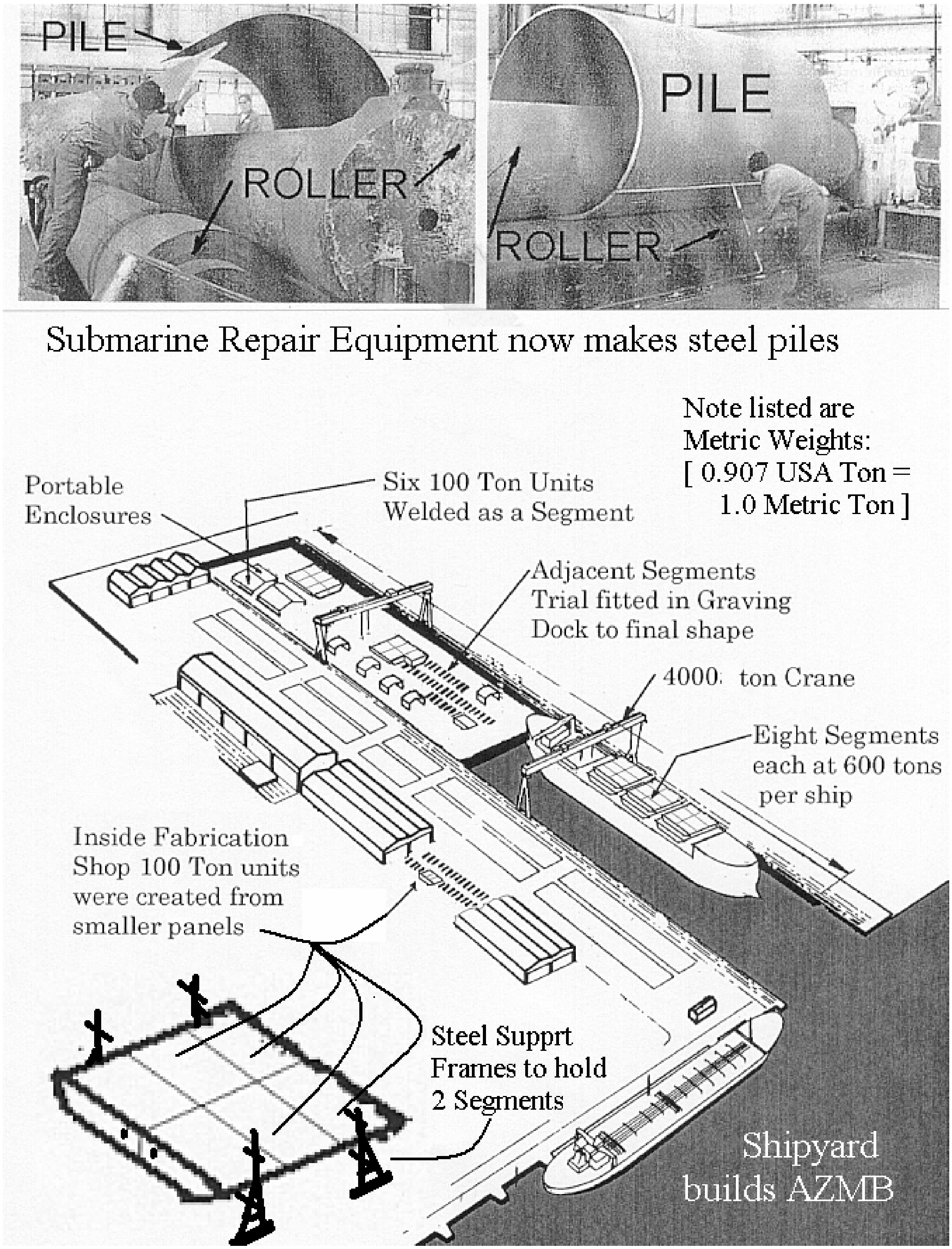


Figure I. Key Stages or Phases of Bridge Erection for AZMB

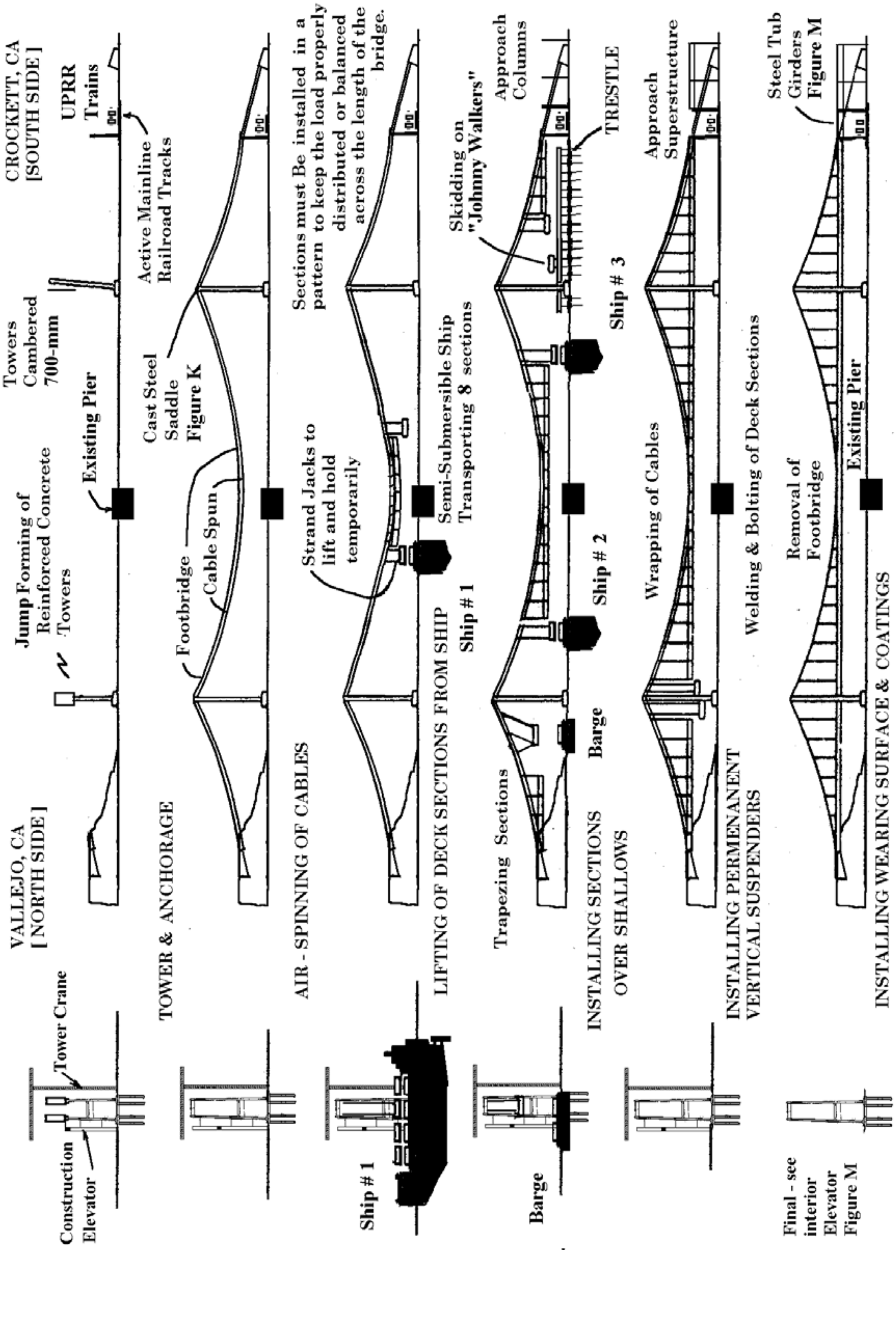


Figure J. Air-Spinning Method

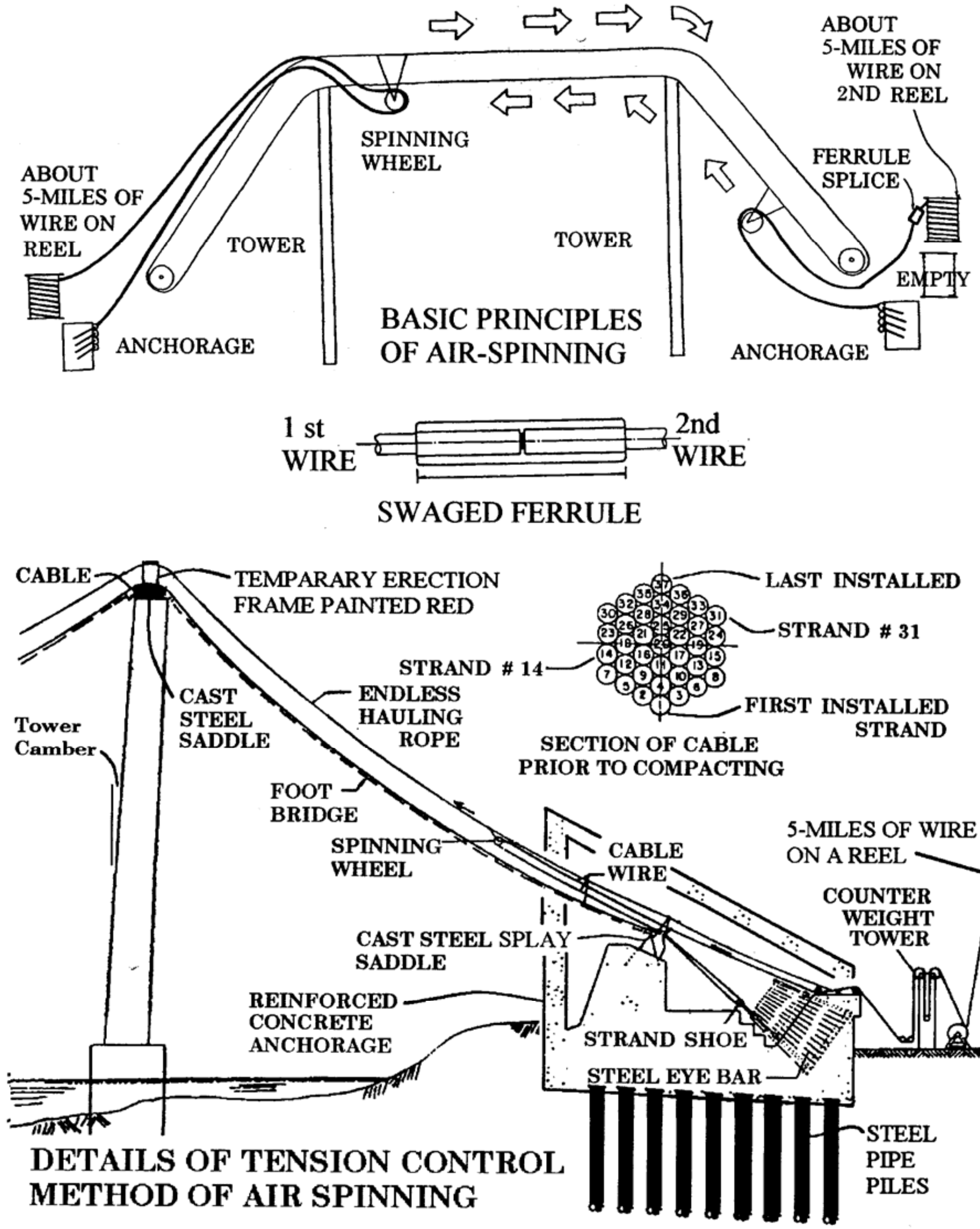


Figure K. Suspension Cable Details for AZMB

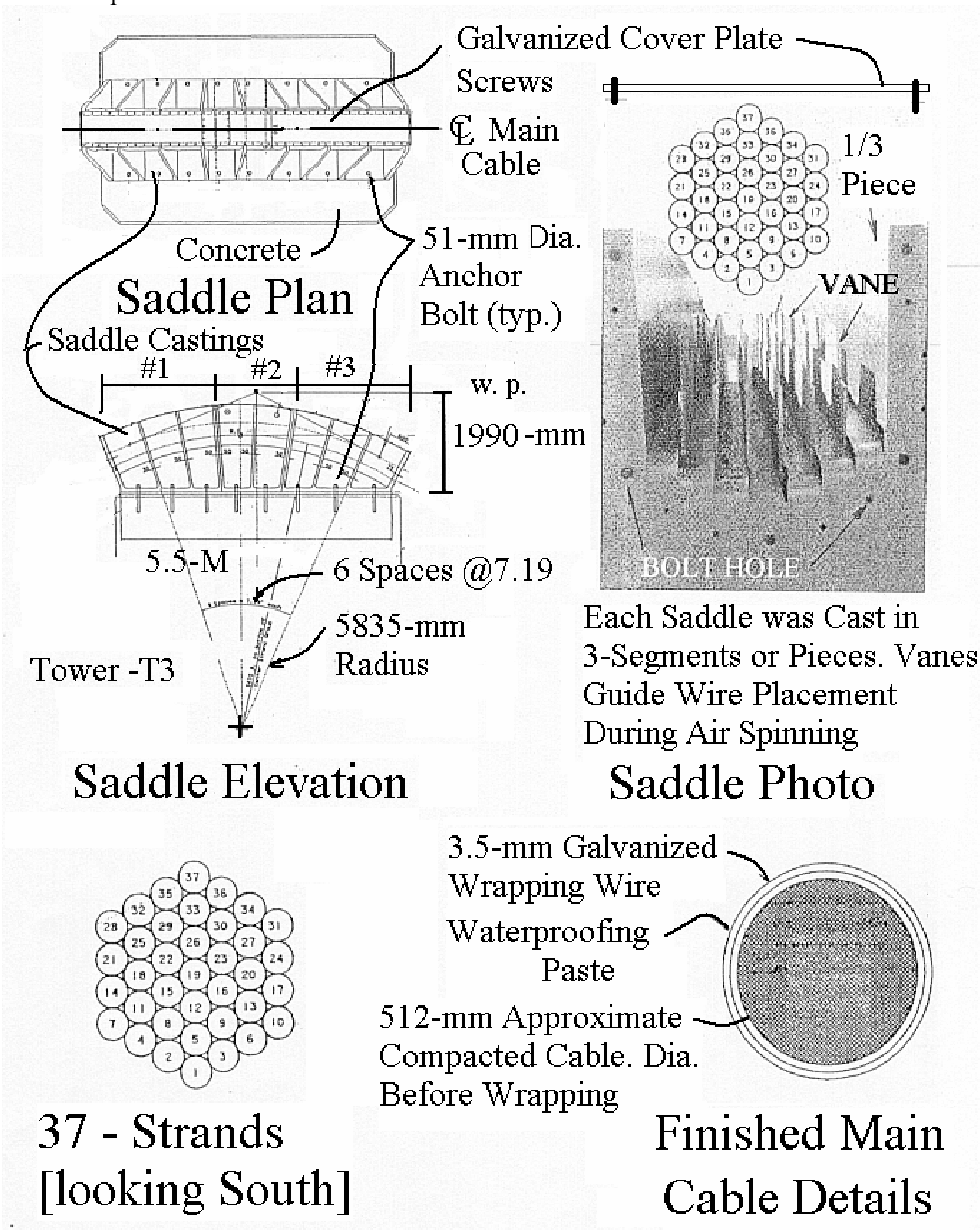


Figure L. Suspender Cable Details for AZMB

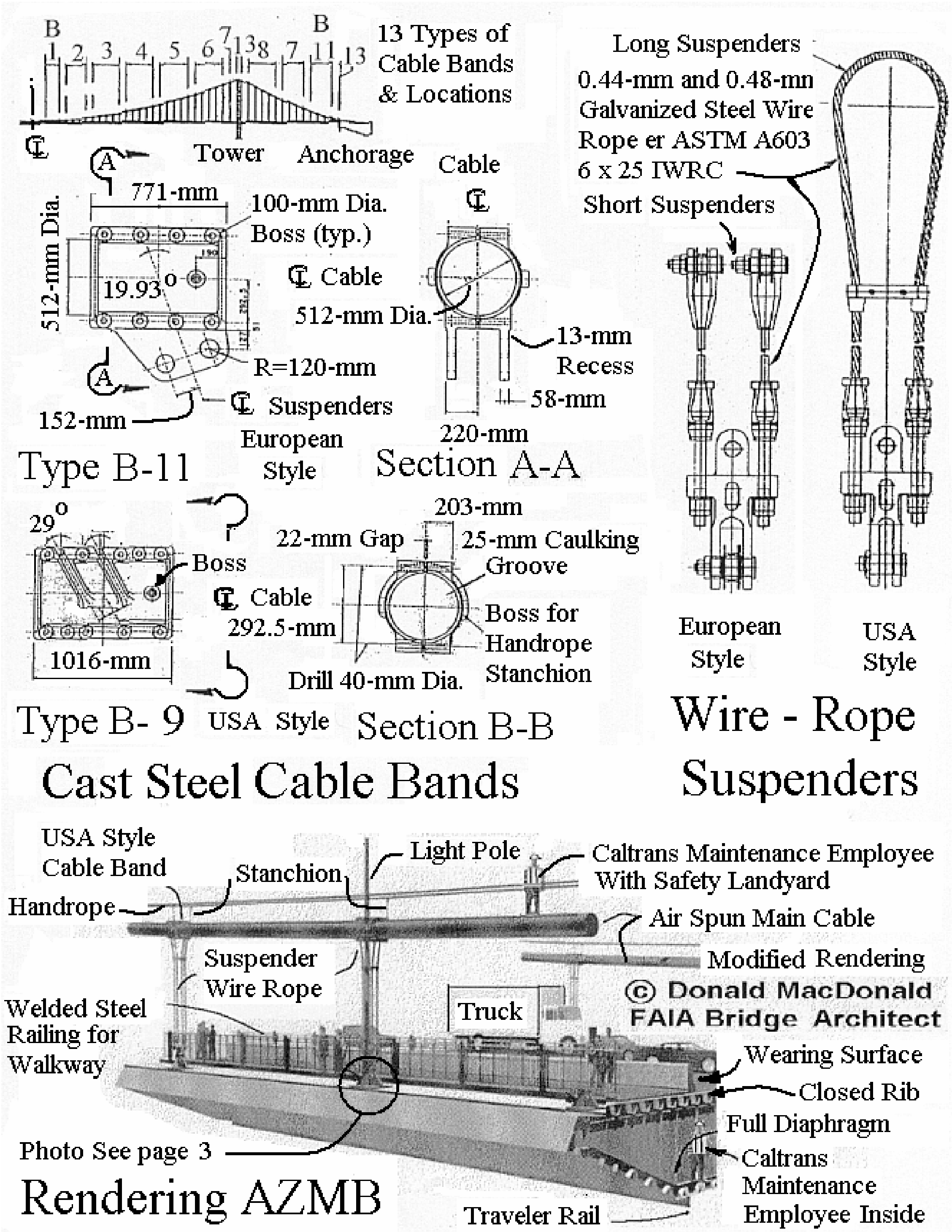
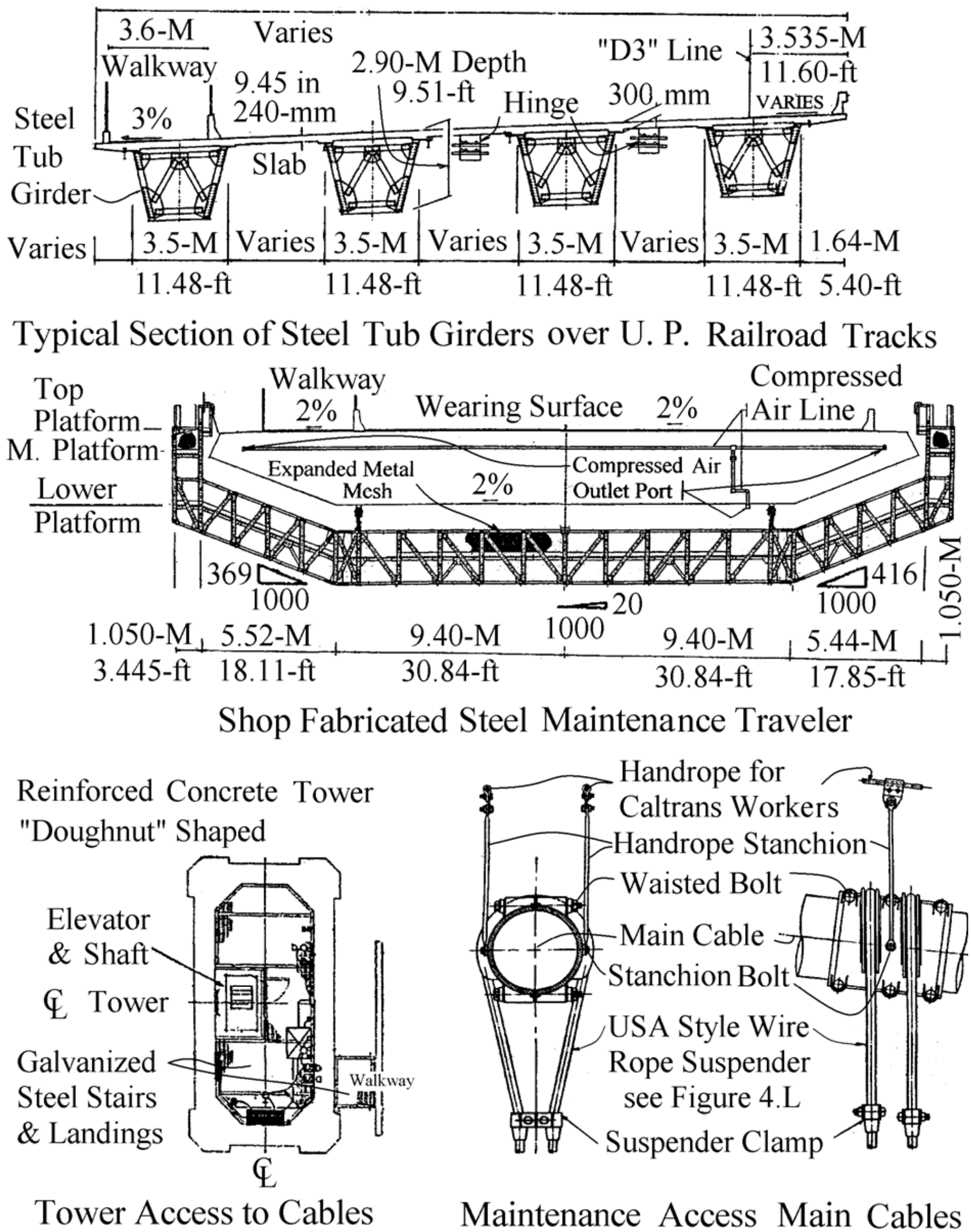


Figure M. Steel Details—Tub Girders and Bridge Maintenance Access for AZMB



About the Authors…

Sarah Picker, PE

Sarah Picker, PE is a professional civil engineer whose field of interest is implementing policy through legal and engineering instruments such as permits, environmental documents, and construction contracts. She is a member of the American Society of Civil Engineers' Construction Institute Constructability Committee.

She has been involved in researching and writing about existing orthotropic bridges and was a member of the organizing team of www.orthotropic-bridge.org, an ASCE conference.

She earned her Master of Science in Transportation Management from San Jose State University in 2003. She graduated from California State University Chico with a degree in Civil Engineering in 1984.

She can be reached at:

Sarah Picker, PE
5245 College Avenue
Suite 312
Oakland, CA 94618
Home Office Phone: (510) 597-0656
E-mail: routec@yahoo.com

Alfred Mangus, PE

Alfred R. Mangus, PE, is currently employed as a bridge engineer. He is the winner of the 1987 James F. Lincoln Arc Welding Award and co-winner of the 1999 James F. Lincoln Arc Welding Award.

In 1976, he received a Bachelor's of Architectural Engineering from Penn State University and in 1977 an MSCE degree from the University of California, Berkeley. He was a consulting structural engineer from 1978 to 1986 in Anchorage, Alaska, designing and constructing buildings and other steel structures. He was a bridge engineer with the Arizona Department of Transportation; and was a consulting engineer from 1990 to 1992 in the Puget Sound area of Washington designing bridges, buildings, and other steel structures. He is a licensed civil engineer in Alaska, Arizona, California, Oregon, and Washington.

Since 1996, he has been heavily involved in researching and writing about existing orthotropic bridges. He envisioned and assembled a team of volunteers creating this ASCE conference solely on the topic of orthotropic bridges in August 2004. He coauthored a textbook chapter and has written magazine and conference papers about existing orthotropic bridges since 1996.

He can be reached at:

Alfred R. Mangus, PE
P. O Box 163210
Sacramento, CA 95816
Home Office Phone: (916) 366-5736
Home Office Fax: (916) 366-5736
E-mail: MangusAlf@aol.com
www.orthotropic-bridge.org

List of Published Steel TIPS Reports*

August 05: Steel Angle & Tee Connections for Gravity and Seismic Loads, by Abolhassan Astaneh-Asl.
May 05: Design of Shear Tab Connections for Gravity and Seismic Loads, by Abolhassan Astaneh-Asl.
April 05: Limiting Net Section Fracture in Slotted Tube Braces, by Frances Yang and Stephen Mahin.
July 04: Buckling Restrained Braced Frames, by Walterio A. Lopez and Rafael Sabelli.
May 04: Special Concentric Braced Frames, by Michael Cochran and William Honeck.
Dec. 03: Steel Construction in the New Millenium, by Patrick M. Hassett.
August 2002: Cost Consideration for Steel Moment Frame Connections, by Patrick M. Hassett and James J. Putkey.
June 02: Use of Deep Columns in Special Steel Moment Frames, by Jay Shen, Abolhassan Astaneh-Asl and David McCallen.
May '02: Seismic Behavior and Design of Composite Steel Plate Shear Walls, by Abolhassan Astaneh-Asl.
Sept. '01: Notes on Design of Steel Parking Structures Including Seismic Effects, by Lanny J. Flynn, and Abolhassan Astaneh-Asl.
Jun '01: Metal Roof Construction on Large Warehouses or Distribution Centers, by John L. Mayo.
Mar. '01: Large Seismic Steel Beam-to-Column Connections, by Egor P. Popov and Shakhzod M. Takhirov.
Jan '01: Seismic Behavior and Design of Steel Shear Walls, by Abolhassan Astaneh-Asl.
Oct. '99: Welded Moment Frame Connections with Minimal Residual Stress, by Alvaro L. Collin and James J.Putkey.
Aug. '99: Design of Reduced Beam Section (RBS) Moment Frame Connections, by Kevin S. Moore, James O. Malley and Michael D. Engelhardt.
Jul. '99: Practical Design and Detailing of Steel Column Base Plates, by William C. Honeck & Derek Westphal.
Dec. '98: Seismic Behavior and Design of Gusset Plates, by Abolhassan Astaneh-Asl.
Mar. '98: Compatibility of Mixed Weld Metal, by Alvaro L. Collin & James J. Putkey.
Aug. '97: Dynamic Tension Tests of Simulated Moment Resisting Frame Weld Joints, by Eric J. Kaufmann.
Apr. '97: Seismic Design of Steel Column-Tree Moment-Resisting Frames, by Abolhassan Astaneh-Asl.
Jan. '97: Reference Guide for Structural Steel Welding Practices.
Dec. '96: Seismic Design Practice for Eccentrically Braced Frames (Based on the 1994 UBC), by Roy Becker & Michael Ishler.
Nov. '95: Seismic Design of Special Concentrically Braced Steel Frames, by Roy Becker.
Jul. '95: Seismic Design of Bolted Steel Moment-Resisting Frames, by Abolhassan Astaneh-Asl.
Apr. '95: Structural Details to Increase Ductility of Connections, by Omer W. Blodgett.
Dec. '94: Use of Steel in the Seismic Retrofit of Historic Oakland City Hall, by William Honeck & Mason Walters.
Dec '93: Common Steel Erection Problems and Suggested Solutions, by James J. Putkey.
Oct. '93: Heavy Structural Shapes in Tension Applications.
Mar. '93: Structural Steel Construction in the '90s, by F. Robert Preece & Alvaro L. Collin.
Aug. '92: Value Engineering and Steel Economy, by David T. Ricker.
Oct. '92: Economical Use of Cambered Steel Beams.
Jul. '92: Slotted Bolted Connection Energy Dissipaters, by Carl E. Grigorian, Tzong-Shuoh Yang & Egor P. Popov.
Jun. '92: What Design Engineers Can Do to Reduce Fabrication Costs, by Bill Dyker & John D. Smith.
Apr. '92: Designing for Cost Efficient Fabrication, by W.A. Thornton.
Jan. '92: Steel Deck Construction.
Sep. '91: Design Practice to Prevent Floor Vibrations, by Farzad Naeim.
Mar. '91: LRFD-Composite Beam Design with Metal Deck, by Ron Vogel.
Dec. '90: Design of Single Plate Shear Connections, by Abolhassan Astaneh-Asl, Steven M. Call and Kurt M. McMullin.
Nov. '90: Design of Small Base Plates for Wide Flange Columns, by W.A. Thornton.
May '89: The Economies of LRFD in Composite Floor Beams, by Mark C. Zahn.
Jan. '87: Composite Beam Design with Metal Deck.
Feb. '86: UN Fire Protected Exposed Steel Parking Structures.
Sep. '85: Fireproofing Open-Web Joists & Girders.
Nov. '76: Steel High-Rise Building Fire.

STRUCTURAL STEEL EDUCATIONAL COUNCIL

Funding for this publication provided by the California Field Iron Workers Administrative Trust.
A Union Trust Fund

P.O. Box 6190
Moraga, CA 94570
Tel. (925) 631-1313
Fax. (925) 631-1112
Fred Boettler, Administrator

Steel TIPS may be viewed and downloaded for a nominal fee at
www.steeltips.org

Participating Members of SSEC