行政院及所屬各機關出國報告
（出國類別：其他）

出席第三屆橋梁新尺寸之國際會議報告

出國人姓名／職稱／服務機關：
薛讚添　課長　交通部公路總局第四區養護工程處
出國地點：馬來西亞／吉隆坡
出國期間：九十二年四月八日至十一日
報告日期：九十二年七月八日

系統識別號：C09202010

行政院所屬各機關出國報告提要

出國報告名稱：出席第三屆橋梁新尺寸之國際會議報告

頁數：27 頁 含附件：∨是□否

出國計畫主辦機關／連絡人電話

交通部公路總局 薛讚添 03-9973321
第四區養護工程處

出國人員姓名／服務機關／單位／職稱／電話

薛讚添 交通部公路總局 第四區養護工程處 課長 03-9973321

出國類別：□1.考察 □2.進修 □3.研究 □4.實習 ∨5.其他

出國期間：92 年 4 月 8 日至 92 年 4 月 11 日 出國地點：馬來西亞／吉隆坡

報告日期：92 年 7 月 11 日

分類號／目：G4/土木工程 G4/土木工程

關鍵詞：創新、結構設計、懸索橋、斜張橋、拱橋

內容摘要：

由於經濟成長與發展之需要，世界各國政府對於提供便捷且良好之鐵、公路相關之公共基礎，以應大眾及貨物運輸之需求，均視為重要之課題。而橋梁在鐵、公路跨越河川、溪谷及都市間交通常扮演重要之角色，亦是擔負大眾旅運時生命財產安全之重要結構物。本研討會乃基於研思改進橋梁方面問題而起，旨在藉由專家、工程師、學者、承包商及材料供應者之腦力激盪，冀能提供創新的解決辦法及有創造性的解答，使未來的橋梁工程在創新作法中能有較佳之結構設計，改良的分析方法及獨創的結構概念等，以提昇橋梁工程水準，並藉由新材料的使用與新施工法的誕生，在橋梁工程領域開啟重要且令人矚目的里程碑。

本文電子檔已上傳至出國報告資訊網

摘要

由於經濟成長與發展之需要，世界各國政府對於提供便捷且良好之鐵、公路相關之公共基礎，以應大眾及貨物運輸之需求，均視為重要之課題。而橋梁在鐵、公路跨越河川、溪谷及都市間交通常扮演重要之角色，亦是擔負大眾旅運時生命財產安全之重要結構物。本研討會乃基於研思改進橋梁方面問題而起，旨在藉由專家、工程師、學者、承包商及材料供應者之腦力激盪，冀能提供創新的解決辦法及有創造性的解答，使未來的橋梁工程在創新作法中能有較佳之結構設計，改良的分析方法及獨創的結構概念等，以提昇橋梁工程水準，並藉由新材料的使用與新施工法的誕生，在橋梁工程領域開啟重要且令人矚目的里程碑。

目　　錄

壹、 前言

本次國際會議主要論題係關於橋梁工程在設計及施工方面一些創新理念之介紹，其目的旨在讓世界各地的橋梁工程師研討目前橋梁工程發展趨勢、橋梁造型、設計載重、電腦輔助分析某些特殊地區橋梁施工特色、新材料使用與探討...等問題。期望透過參加本研討會之工程師、專家、學者、承包商之意見交換與討論，能喚起橋梁工程界對於橋梁〝創新〞理念的重視，摒棄以往反復使用類似的構造型式及追求低價與快速解決的作法，以免其高度單調無味的造型污染我們的環境。

由於橋梁工程師注重所負之公共責任，因此在設計上常傾向較保守，且因研發之經費較短絀，以致一些看似相似而廉價的標準橋梁充斥於各地道路，此種作法不僅不合時宜，也令人惋惜！甚者大大降低橋梁工程師在社會上的專業形象。因此，透過意見之相互交流與研討，期使工程師們能小心翼翼地逐步踏出創新的每一小步，在心態與觀念上漸進改變。當然也不能為創新而創新，或一味追求譁眾取寵的外型與過度昂貴的造價，而隱匿或扭曲橋梁結構的本質。

由本會議瞭解到創新是橋梁工程獲得進步的唯一途徑，也使得橋梁工程師在工作上更具有挑戰性。畢竟，能打造一座好而美且嶄新而不朽的橋梁，不僅敷應交通運輸之需要，更是工程師們在其有生之年體現自我潛能的一紙證明書。

貳、 行程

九十二年四月八日中午十二時三十分自桃園中正國際機場搭乘華航班機飛往香港(停留一小時)，再續飛馬來西亞，於下午七時十五分抵達馬來西亞吉隆坡國際機場,隨即驅車前往旅館住宿。四月九日上午八時三十分至會場(Hotel Istana）辦理報到註冊手續及參加至四月十日止之會議。四月十一日下午三時分自吉隆坡國際機場搭乘華航班機於七時四十五分返抵桃園中正國際機場。

參、 會議議程及特色

本會議係由新加坡 CI-PREMIER CONFERENCE ORGANISATION 負責籌劃於 Hotel Istana 舉行，計宣讀論文 25 篇。成員共來自二十三個國家，六十人參加，其議程如下：

April 09, 2003, Wednesday

0800-0900: 報到及註冊

0900-1015: 開幕式及受邀來賓致詞

1030-1100: 專題報告 1-新舊金山奧克蘭港自錨式懸索橋概述

1100-1330: 特殊報告 1-馬來西亞及鄰近國家之橋梁

1430-1545: 專題報告 2-亞、澳地區長跨徑橋梁之發展趨勢

1545-1700: 特殊報告 2-北印度高震區橋梁創新之設計與施工

1715-1830: 技術報告 1-宣讀論文

April 10, 2003, Thursday

0900-1110: 專題報告 3-市區橋梁之美學與工藝

1130-1245: 技術報告 2-宣讀論文

1400-1545: 專題報告 4-宣讀論文

1610-1730: 特殊報告 3-短跨徑橋梁自由振動分析概論

本會議發表之論文主要分為（一）專題報告、（二）特殊報告、（三）技術報告等三大類，研討內容著重在探討橋梁工程相關之理論與實務，重點涵蓋如：長跨徑橋梁之發展趨勢、自錨式懸索橋設計概述、橋梁美學與藝術、斜張橋吊索張力問題探討、各類橋梁施工概要、以電腦輔助分析橋梁應力行為及規範之探討等。

本會議期間適逢嚴重急性呼吸道症候群(SARS)肆虐期間，會議深受影響，現場發表之論文與原訂項目頗有出入，且參加人數不多，故論文發表之地區大半集中在亞洲地區、美國、匈牙利、墨西哥等國家，惟其內容豐富，所舉各地實例甚多，頗值從事橋梁工程者借鏡。

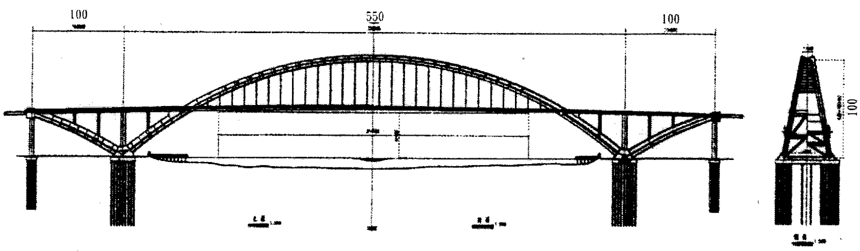
肆、 心得

由本會議研討內容可略知近年來世界各國在橋梁工程之發展趨勢及研究方向，本報告謹就所得資訊粗略提出心得。

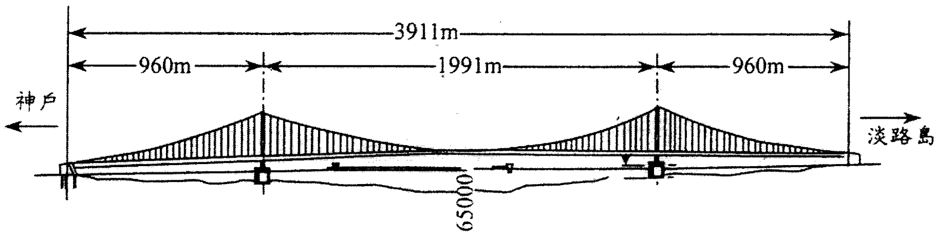
跨徑桂冠之競逐

如同高層建築以高度衡量一樣，跨徑始終是衡量橋梁設計、施工技術水準的第一要素，跨徑越大，設計和施工的難度就越大。但橋梁工程師們並不畏懼向高難度的設計與施工挑戰，全世界各地的工程師無不竭盡心思，甘冒風險地競逐跨徑的鰲首。甫於二〇〇二年十月主拱合攏之中國大陸上海盧浦大橋，其主跨徑長達 550 公尺，已成為世界第一的全鋼結構中承式繫桿拱橋(圖一)；於一九九八年四月完工通車，跨越日本明石海峽之懸索橋，其破紀錄之橋梁尺寸，引起橋梁工程界極大震撼。直至目前為止，明石大橋仍保持世界上最長懸索橋之地位，其總長 3911m，中央主跨為 1991m，兩個邊跨各為 960m(圖二)。

世界上著名的大橋都是橋梁工程師集體的創作，以科學創新為動力，盡心盡力，兢兢業業地注入心力所增添的里程碑。大橋的建造是一項高難度，高風險的系統工程，需要各方面的配合，以確保品質與安全，其過程凝聚眾多橋梁工程師的心血，在傳統中注入創新的理念，而成就一座座令人驚探的曠世鉅作。這種永遠競逐的精神，在橋梁工程師的歲月中是永無止境的。



(圖一)



(圖二)

造型美學與藝術

橋梁雖為克服自然障礙，越過河流、峽谷等通濟利涉之工具，但當我們看到它橫渡海峽，跨越長江大河、峽谷深淵時，其規模之宏大，令人胸襟開闊；其優雅之美姿，令人心曠神怡；可以感受到它的律動與張力，令人不禁讚詠它富有韻律變化的線條，以及肅穆的挺拔之姿；它們各自展現不同的美感與生命力，是各具不同風格的藝術品。

由於橋梁係築造在人們的生活空間中，其成為橋址處的環境和景觀的一部份，是大眾的視覺對象，因此，橋梁設計者必須根據使用要求的條件，以客觀的條件為出發點，在力學計算確定方案的同時，從藝術的角度漸漸成熟地產生美學上較完整的建築想像。坊間有關橋梁美學、藝術方面的專著雖不普遍，但仍可供設計者參考，能擷取多少精髓則視各人而定。惟術業有專攻，仍以委由具有造型藝術或美學之專業人才參與規劃較易收效。

但在此要強調的是，橋梁工程師對橋梁應有呼應時代潮流與大眾期望的體認，橋梁工程師不應在設計一座橋梁時，僅單純地考量其功能性而忽略其所帶給人們的視覺感受；橋梁工程師不應再築造出破壞景觀，污染環境與令人視之無味且毫無特色的橋梁。橋梁工程師應充實自身的美學素養並發揮創造力，以表現橋梁豐富美的形象，讓橋梁成為人們生活中賞心悅目的藝術品。

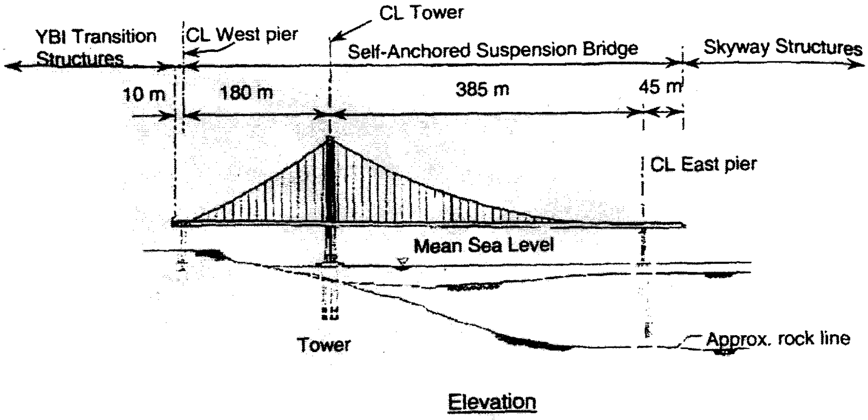
跳脫傳統的構思

本會議所發表之專題論文，其中介紹一座極具獨創性的橋梁—美國新舊金山奧克蘭港灣橋(圖三、四)，是一座深富巧思的自錨式懸索橋(吊橋)，茲附列其原文(附件一)提供參考。

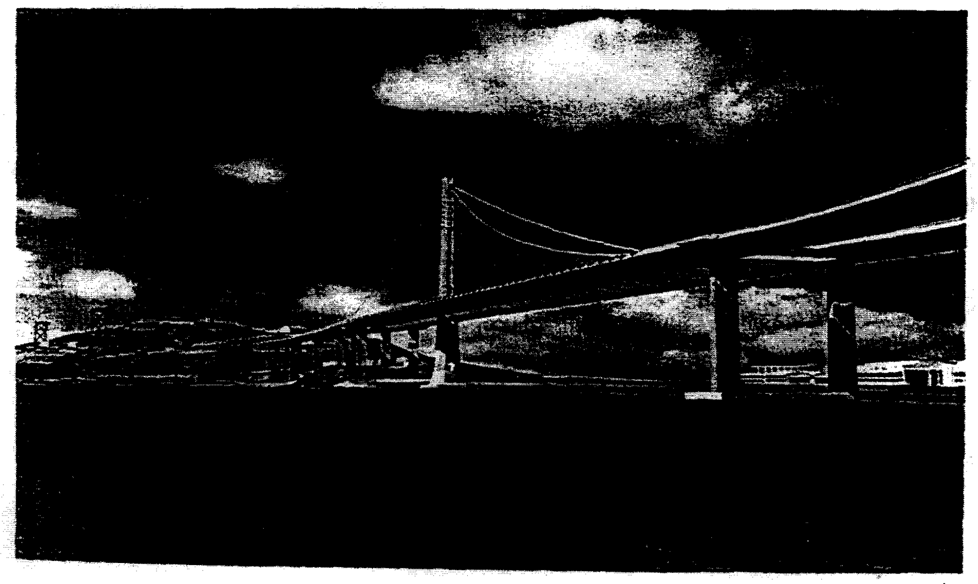
本橋因受橋址地質、地形條件限制，而且舊金山港的水道內有很多地標建築，包括金門大橋，故需特別考量橋與週遭環境融合問題。設計團隊在單柱斜張橋與自錨式懸索橋間，選擇了具有創新的結構組合，以及極具設計挑戰的後者，其在耐震的考量、橋梁的型態(包括橋塔)、懸索系統及箱梁的設計等，都有獨到之處，故能受到評選委員的青睞。

本懸索橋在橋塔前後採不等長跨徑，外形貌似不對稱之斜張橋(圖五)，主懸索錨錠在上部結構兩端，而不使用傳統懸索橋大而笨重的地錨(圖六)；此外，在懸索系統上的精心設計與獨創的方式，更是世所罕見。不但具有耐震的特性

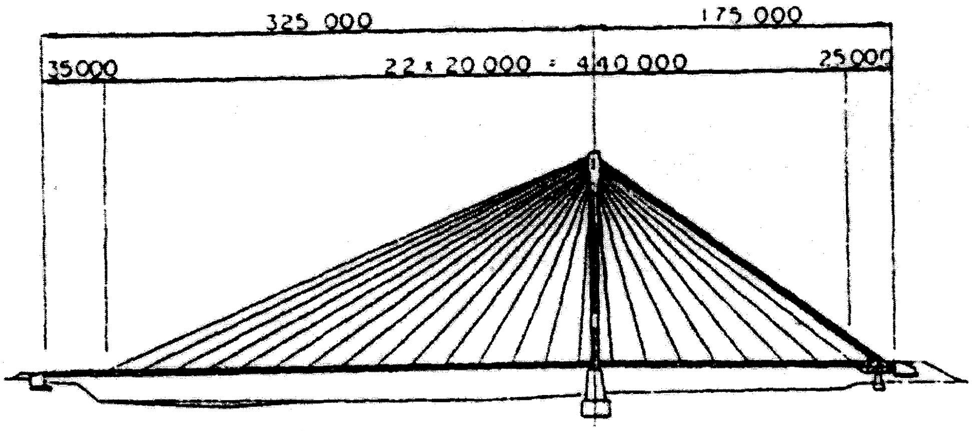
更有引人的外表，實為一座在傳統懸索橋中成功蛻變的稀世佳作。它不但跳脫懸索橋的傳統思維，創造懸索橋的新觀念，此一構思當值橋梁工程師們學習。也唯有在橋梁工程師們有跳脫傳統的新構思，始能再創造出讓世人驚嘆的橋梁新風貌，為橋梁史憑添一頁頁的佳話。



(圖三)



(圖四)



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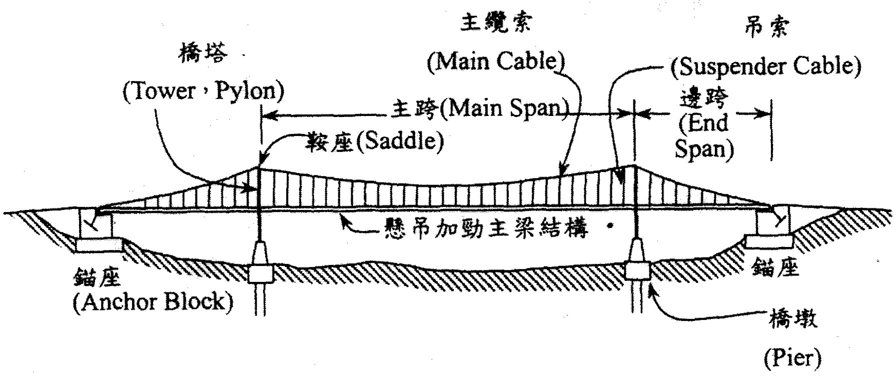

(圖六)

施工法繽紛多樣

由於世界各地公共基礎建設蓬勃發展，在橋梁工程方面為克服地形限制及配合橋梁本身特性等因素，因此，在施工法上不斷推陳出新，以因應在施工上之需求。

在鋼拱橋架設方面大致可區分為臨時支撐工法(Staging method)、吊索懸吊架設工法(Cable erection method)、推進工法(Launching method)、單側懸臂架設工法(Cantilever erection)、大組件架設工法(Large block erection)等。至於混凝土拱橋則有支撐工法(又可細分為接地式支撐工法、拱肋鋼骨架支撐工法、鋼拱先行工法)、懸臂架設工法(可分為支柱斜吊工法、鋼骨架工法、支柱斜吊與眉

攬(Melan)併用工法、鋼骨架與眉攬併用工法)、拱肋迴降工法(Arch lowering method)等。

在連續變斷面混凝土橋梁則有五種方法：1.就地支撐法。2.預鑄節塊懸臂施工法。3.場鑄懸臂施工法。4.推進機架工法(Launching gantry method)。5.吊索輔助工法等。

現代斜張橋施工法，在主桁方面已發展有單側架設工法、平衡架設工法、臨時支撐併用千斤頂工法等(橋塔、吊索部份不再贅述)。

以上各類橋梁依性質不同，各自發展出不同工法，除單一工法外，亦可視實際需要併同使用，其工法均經各該類橋梁施工之實際驗證，皆可作為橋梁施工之援引參考，為日後創新之橋梁在施工上奠定穩固的基礎，加以施工機具日益改良，機具能量大增，附屬設施、材料不斷研發，材質精進，將有助於在既有的工法上精進、創新，提昇橋梁之建築技術。

伍、 建議

由於橋梁建築技術突飛猛進，工程材料日益精良，設計規範逐漸完備，輔助設計之相關電腦程式普遍，因此，在橋梁工程上時有突破性的展現，尤其大型橋梁施工案例紛紛出籠，實值本國橋梁工程界參考借鏡。

他山之石，可以攻錯，參考其他橋梁相關之規劃、設計、施工資料，可節省甚多盲目嘗試的時間，因此，在相關資料蒐集方面建議應有專責技術單位為之，對於世界各國橋梁先進國家既有之案例廣泛收集，提供本國工程師開拓寬廣視野，以提昇國內技術水準。

附　　件

附件一

3rd International Conference on New Dimensions in Bridges, Flyovers, Overpasses & Elevated Structures
9 – 10 April 2003, Malaysia

New San Francisco Oakland Bay Bridge
– overview of self-anchored suspension bridge design

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M. Nader, T Y Lin International, USA

Abstract

A design review of the new self-anchored suspension bridge is presented. A single tower supports a unique suspension system. Dual steel box girders span 585 meters. A single cable will span the crossing twice, being the first suspension bridge cable to loop around an anchorage and return.

Keywords: suspension bridge, self-anchored, box girder, orthotropic deck, prefabricated wire strand, cable, anchorage

1. Introduction

The San Francisco Oakland Bay Bridge is the main route between San Francisco and Oakland, California, carrying more than 280,000 vehicles daily. The 15km long structure contains dual suspension bridges, a tunnel, truss spans, and a cantilever bridge. In 1989 an earthquake of magnitude 7.1 occurred at the city of Loma Prieta, 95 km from San Francisco. Among the damaged structures was the fatal collapse of one span of the Bay Bridge (Fig. 1 and 2). The bridge remained closed for one month at great economic cost to the region.

Investigations over the following years examined the causes of the failure, and effective solutions. Criteria for the design and retrofit of truss framing for local buckling were established. It was determined that the western spans could be retrofitted to meet current seismic requirements, but similar improvement of the eastern spans would be prohibitively expensive, disruptive to traffic, and unreliable. The significant difference between the two structures lay in the foundations. While the western spans rest on caissons extending to bedrock, most of the eastern spans are supported on timber piles in deep sediment. [1]

It was decided that retrofits would commence on all parts of the Bay Bridge, but that those to the eastern spans would be interim measures, until a replacement bridge could be built. The eastern spans will be replaced with a new more robust crossing, consisting of a self-anchored suspension main span, post-tensioned concrete box girders to transition from the main span and the east portals of the Yerba Buena Island tunnel, a 2.4 km skyway of segmental concrete box girder spans to the Oakland shore (Fig. 3).

The principal criterion given to the designers was that the bridge must be operational within days of an earthquake with a return period of 1500 years. The San Francisco Oakland Bay Bridge lies 12km from the Hayward and 25km from the San Andreas faults, which can generate earthquakes of magnitude 7.5 and 8.1, respectively, at this return period. [2,3]

A further design condition related to the surroundings of the bridge. The waterways of San Francisco Bay are graced with landmarks, including the Golden Gate Bridge, the Richmond-San Rafael Bridge, and the Bay Bridge itself. The bridge selection committee charged the design team to provide a main span structure harmonious with the site and the other bridges. This paper will discuss that bridge, the innovative structural components, and the challenges met in their design.

2. Bridge Type Selection

The bridge is designed to resist two levels of earthquake, with return periods of 450 and 1500 years. After a 450 year earthquake, the bridge will provide full service almost immediately and there should be only minimal damage to the structure. After a 1500 year earthquake the bridge should provide full service within days and should only sustain repairable damage, with minimal risk of a loss of functionality. The bridge will have dual roadways on a single level. While in principle two-level structures can be designed with adequate safety, they can be complex and expensive. The California Department of Transportation (Caltrans) policy is to avoid them in new structures for the safety and peace of mind of the traveling public.

The site geology varies dramatically along the length of the bridge. At the western end of the replacement bridge lies bedrock at Yerba Buena Island. The bedrock drops abruptly east of the island and is overlain by deep Bay muds. The eastern end has interlayered clays and sands. [3] This geology determined bridge type. The main bridge will extend from a rocky bluff on the island to the deep mud of the bay (Fig. 4). It will cross a shipping channel with a span of 385 m and have a 180m back span anchored on the island. The bedrock extends underwater to the west side of the channel, and provides suitable support for the full weight of the bridge. By cantilevering the span out over the deep channel, the pier on the east end is lightly loaded. Several bridge types conform to this configuration, especially the single-tower cable-stayed, and self-anchored suspension bridges.

Twelve variations of these two types were studied in the preliminary design phase. The single tower self anchored suspension bridge was a solution well adapted to its site (Fig. 5). The bridge will consist of two separate box girders that provide two roadways designed to carry six lanes each way. Transverse diaphragms; or floorbeams, at 5m spacings inside the girders support the orthotropic decks and distribute the suspender loads though the section. Crossbeams, spaced at 30 meters and integrally connected with the girders, frame the girders together. The suspender ropes will lie in two planes, each sloped 20 degrees from the vertical. Within each plane all suspenders are parallel and slope transversely to the roadway alignment. Each pier bent has two reinforced concrete shafts and a prestressed concrete cap beam. The deck is monolithically anchored to west pier by prestressing tendons. Beyond the main span the deck is supported by bearings on top of the east pier, and cantilevers 45m to meet the adjacent skyway bridge. By means of a transition structure on the west end the two roadways on the main bridge merge with the existing two-level tunnel and western spans.

3. Seismic Design

The asymmetrical self-anchored suspension bridge was chosen for its seismic performance and attractive appearance. The main suspension cable is anchored at both ends to the superstructure, without the massive ground anchorages of traditional suspension bridges. This has two favorable aspects: the mass of the bridge is less, and the cable is isolated from the ground. Both features reduce the seismic excitation.

The bridge decks and tower are also isolated from one another. The box girders are not supported by the tower directly, but only by the suspenders. During earthquake the box girders have no contact with the tower (Fig. 6). This protects both the tower and the girders from large concentrated loads. Dynamic horizontal loads are resisted by the end piers, which are of reinforced concrete. Longitudinal forces are resisted mainly by the west pier, while transverse loads are divided between both piers.

The asymmetry of the bridge and the angle of the cable subjects the west pier to vertical uplift, which is fully offset by the weight of the post-tensioned cap beam. A tie-down system within each shaft of the pier connects the cap beam to the foundation, providing support against seismic uplift. Isolation and redundancy is provided, because all service load cable reactions are resisted at the roadway level. Thus, extensive damage to the piers does not affect the carrying capacity of the cable system. Similarly, the tower, which carries the weight of the bridge, is not needed for horizontal support. [4,8]

4. Tower Design

A single tower shaft has no redundancy in the event of a major earthquake. Several multi-legged tower types and configurations were studied. The prototypes fell into three groups: Dual Portal Towers, three-legged tower (three single shafts joined by a crossbeam below the roadway), and Single Tower consisting of four shafts. Evaluation of the alternatives was based on seismic performance, aesthetics, and cost. While the single four-shaft tower was the most appealing aesthetically, (Fig. 5,6) its seismic design presented difficult problems. To improve the seismic performance and reliability of the single tower the following measures were incorporated into the design:

- For redundancy in supporting the weight of the bridge, four shafts were used to form the tower.

- Cross-braces and shear links tie the shafts together and provide redundant horizontal framing. The links are designed to yield plastically, while the crossbeams have hinged connections.

Each tower shaft is a stiffened, tapered pentagonal box section. Transverse internal diaphragms are spaced at 4m. Although the tower shafts were designed to remain elastic throughout the 1500 year earthquake response, they were stiffened for compactness, according to the provisions of the Caltrans Report ATC 32 [5]. This ensures large inelastic compressive strains without local buckling.

The tower shear links were designed:

- to provide the necessary bracing and stiffness for the service loads,
- to remain substantially elastic during a 450 year earthquake,
- to plastify without local buckling during a 1500 year earthquake, dissipating energy, and limiting damage to the tower shafts.

The shear links will be bolted to the tower with high strength bolts, so that they may be replaced (Fig. 7). The webs are designed to yield as a tension field, and are stiffened accordingly. Yielding is directed to a weaker center zone, where the plates are thinner and fabricated of a lower yield steel. The portion of the link at the connection to the tower is composed of thicker plates of high-performance steel of higher yield strength to ensure that it remains elastic.

The four tower shafts connected with shear links function like a ductile frame, which has been successful in tall buildings. The vertical, weight-carrying components of the tower remain elastic for very large displacements. The desired ductility of the tower and the individual links was provided by the design. Full-scale ductility testing of the shear links was conducted at the University of California at San Diego. The link yielded as expected through 0.07 radians of rotation. The 1500 year earthquake rotational demand is 0.04 radians. [6]

5. Design of the Suspension System

The configuration of the main suspension cable arose from key aesthetic and structural considerations. The ratio of the span lengths led to a high cable angle at west anchorage. Static equilibrium of the cable anchorage atop the western pier dictated that the cable anchors be located below the roadway girder. Separate anchorages of the two cables would cause large eccentric forces on the west pier and the anchorage chamber. This dictated the unusual solution of looping a single cable under the roadway, producing symmetrically opposite reactions on the pier. (Fig. 8). An isometric of the main suspension cable in its final geometry is shown in Figure 9.

Strands of the single main cable are anchored inside the two box girders at east pier. Starting at one anchorage, the cable passes through a saddle above the pier, and emerges from the housing tangent to the deck. The cable then passes over the tower top, through one side of the twin-trough tower saddle. The cable loops below the deck at the western pier, and returns over the tower to the east anchorage.

The **Looping Cable Anchorage** system consists of a prestressed concrete frame bent, a looping anchor cable, two deviation saddles, a jacking saddle, and independent tie-down systems. The cable looping is achieved using a pair of deviation saddles at the outer edges of both roadways, and a jacking saddle located at the rear center of the cap beam. The massive tension forces in the main cable are balanced by the distributed compression stress behind the deviation saddles. The three-dimensional compression thrusts behind the deviation saddles are resisted by the bridge box girders in the longitudinal direction, by the cap beam in the transverse direction, and by the cap beam weight in the vertical direction (Fig. 8). The independent tie-downs serve as redundant elements to resist the unbalanced vertical loading in the unexpected event that both east and west piers are severely damaged under a seismic action beyond the specified 1500 year earthquake.

A single **Tower Saddle** with twin vertical troughs was selected (Fig. 10). This decision was based on the complex geometry of the cable, the narrow dimensions of the tower top, the cable erection method, cable twist, and stress analysis. Note that four segments of the cable diverge at the tower top. The degree of transverse tension which is generated makes the single piece saddle the more reliable and compact than twin saddles. Tie rods not shown retain the outer trough walls to reduce stresses.

The **East Splay Saddles** turn the cable enters through an angle of about 11.5°. The saddle axis is itself tilted some 65° from the vertical (Fig. 11). As the saddles will rest upon rocker bearings, the equilibrium of the cable in the saddle was carefully determined, using vector geometry. In order further to facilitate fabrication, the girder and cable saddle geometries have been defined as independently as possible. Alignment between the box girder and the saddle is accomplished by a separately fabricated saddle grillage, which is set into a simple cutout in the girder plates.

At the **East Anchorage** the cable is anchored into a grillage formed by nine vertical shear plates aligned to fit between the splayed strands. The bottoms of the shear plates are welded to the bottom plate of the box girder, while their tops are welded to a top anchor plate which lies 1 meter below and parallel to the deck plate. The east edges of the shear plates are welded to a rear anchor plate which is vertical and normal to the cable axis in plan. The top anchor plate is connected to the roadway deck plate by seven top shear plates which are parallel to the deck ribs. The function of this framing is to decompose the cable tension into three orthogonal components that are resisted by independent structures. The deck plate resists only longitudinal compression due to the cable, and is protected from direct tensile stress, which could cause fatigue damage. (Fig. 12 through 14).

The **Main Cable** has a diameter of 0.79m. It consists of 17399 - 5.4 mm wires, which have a breaking strength of 1760 Mpa. The service load cable tension is 255 MN. The wires are arranged in 137 strands of 127 wires each. Both Aerial-Spun Method (AS) and Prefabricated Parallel Wire method (PWS) have been studied for the construction of SFOBB – SAS main suspension cable. While the main cable can be erected using either method, the PWS strands are preferred.

This recommendation is based on the better manufactured and erected quality of PWS, given the unique cable geometry. The PWS cable construction produces fewer crossed wires, a lower void ratio, lower wire stress at the strand anchors and eliminated splicing ferrules, as compared to a cable made up of individual wires and erected by the aerial spun (AS) method. This reduces stress corrosion and water-induced corrosion.

To provide a 150 year useful life, the recommendation for the **Cable Corrosion Protection System** of the main cable consists of the following components:

- Zinc galvanization of the steel wires that compose the main cable
- Grease application to each individual cable wire during PWS manufacturing
- A paste composed of a blend of zinc oxide with zinc dust and a non-drying thermoplastic polymer base to protect the outer perimeter of the main cable
- S-wire, an interlocking wrapping wire
- Elastic Noxide primer and paint
- Dehumidification of the cable sections at locations of East Anchorage, West Loop Cable Anchorage, and the Tower Saddle

The **Hangers** consist of two suspender ropes looped over a cable band, and anchored into a bracket on the box girder by threaded rods into the rope sockets. The hangers are spaced at 10m and lie in two sloping planes. Within each plane all suspender ropes are parallel and exert no longitudinal force with respect to the deck girders or cable. The parallel arrangement was selected to avoid the transfer of horizontal shear between the cable and the deck by the suspenders during service loading. Such a phenomenon occurs in zigzag rope arrangements, and reduces service life. Because sloping suspenders tend to sag, they need flexible end connections. On the Bay Bridge the suspender ropes will be looped over the cable bands rather than socketed into clevises, which provides flexibility in both longitudinal and transverse directions. In addition, at the lower ends the suspenders will be clamped above the sockets, using elastomeric sleeves to provide an adequate bending radius. (Fig. 15).

The suspender ropes are typically 75mm diameter. They have safety factors of 4.0 for service loads, and 2.0 for seismic loads. Where the girders pass around the towers there is a 20m length of deck which has no structural hanger, because the cable is confined in the tower saddle. The ropes for 20m on either side of the tower are increased to 90mm diameter to support the weight of the segment without hangers.

At the tower suspender ropes are not structurally required. However, for architectural reasons, elastic **Aramid-Polyester Ropes** were provided. Installed at the end of construction, these ropes will not be subjected to the permanent weight of the structure, but will be tightened only enough to match the sag and aerodynamic behavior of the other ropes. The ropes will have a maximum tension only 10% of those of the typical steel ropes. Such ropes are currently used as antenna guys. This may be their first use in a suspension bridge.

6. Box Girder Design

The roadway is supported by two **Box Girders**. The east approaches of the girders also house the main cable anchorages. The following challenges were faced:

- Each girder is supported by suspenders on one side only. This required crossbeams, spanning 71 meters between the suspenders, to support the girders in the transverse direction (Fig. 16 and 17).

The two boxes are framed as a vierendiel truss for lateral wind and seismic loads. The crossbeams needed to be designed for strength, but also for flexibility, to avoid force concentrations at the connections to the girders.

The girders are compression members that resist the tension of the main cable. Any added weight increases both the cable stress, and the girder axial stress. Every kilogram of structural weight in the girder required an additional three quarters of a kilogram of crossbeam, suspender, cable, and anchorage.

The box girder cross-section consists of ten or more stiffened plates, designed to satisfy the compactness requirements of Caltrans Bridge Design Specifications [9], with the further requirement that wall stability conditions of Caltrans report ATC 32 be met. The girders are designed to remain elastic under all combinations of the design loads. In the dynamic response there is no time when a significant portion of the section will be subjected to yield stress. The compactness of the design lends added safety both against catastrophic failure and localized seismic damage. [10]

The shape of the box girder was selected for its aesthetic merits, and for weight-saving. It was determined that the optimal weight of the perimeter plates of the girder was relatively insensitive to the cross section of the box. However, tapering the box on the inboard side reduced the total weight, including the transverse floorbeams, though it increased the complexity of the crossbeam connections. In addition, the taper improved the flexibility and redundancy of the crossbeams to seismic loads, as described below.

The interaction of the box girders and the **Crossbeams** results in complex stress combinations. The plates were designed for the Von Mises Limit State. All walls were stiffened for the compactness requirements. It is to be noted that the crossbeam top flange spans 14m while the bottom spans 33m, due to the taper of the girder. As a result, the bottom segments of the crossbeam have far more lateral flexibility than the top, and the stresses due to lateral flexure occur primarily in the upper two meters. This portion was reinforced and stiffened. The configuration has the advantage of reducing seismic global demands through flexibility, which diverts stress from the critical tension components of the crossbeams.

The bridge orientation is determined by the highway alignment needed to approach the existing tunnel on Yerba Buena Island with adequate sight distances. In order to meet the existing alignment in the minimum length of the bridge, highway horizontal and vertical curvature, as well as a transition in superelevation, occurs in the east approach. The box girder and the cable anchorages within them follow this geometry.

The **Orthotropic Deck** consists of 14mm top plate and 12mm triangular closed ribs with rounded bottoms. While most of the girder design is governed by global effects, the orthotropic deck design was governed by traffic loads, which subject the bottom fibers of the orthotropic deck ribs to a compressive stress near their support diaphragms. In a typical deck this effect would be insignificant, but in the Bay Bridge this local compression combines with the global compression stress of 145 Mpa due to the cable force. Direct reinforcement of the ribs and deck would have added considerable weight to the bridge. Instead, the deck has been designed such that a permanent hogging moment will exist throughout the suspended span in the dead load state. Such a moment reduces the compression of the top deck plate and ribs by 20 Mpa. The Design Moment Diagram is shown in Figure 18. A range of moment is given to permit variation due to erection tolerances.

7. Erection

In classic suspension bridge design, the entire design dead load is assumed to be carried by the suspension system, while the stiffening trusses or girders only serve to distribute live loads and limit local deflections. The stiffening system typically has very small bending moments under the design dead load. The suspenders are typically vertical, and the longitudinal component of cable tension is a constant. At each cable band a simple equilibrium equation relates the downward force from the suspenders with the tension in the main cable and the slope change in the cable. A recursion formula based on this equilibrium allows one to project the cable profile.

During erection of the deck, the suspension cable and suspenders are erected first. The deck segments are hung from the suspenders, and little falsework is needed. Erection is facilitated by the following factors:

1. the suspension system supports the deck during erection,
2. the dead load cable profile is statically determinate when the suspenders are vertical,
3. the deck segments have no dead load moments built into the construction.

The first simplification is forfeited in self-anchored suspension bridges. As the box girder maintains the tension in the cable, it or some temporary tieback system must be in place to hold the cable during its erection.

The second simplification is lost for cable-stayed bridges, and some self-anchored bridges which have sloping suspenders. It is a non-linear problem to find the profile of a suspension system with sloping hangers, even when the hanger supports don't move. In cable-stayed bridge tower the hanger supports move a small horizontal distance. The strands are very stiff and small extensions of the strands can greatly affect their tension. Thus, in the erection of cable-stayed bridges control of strand tension is important.

In contrast to the cable-stayed bridge, the hangers in a self-anchored bridge connect to the main cable, which moves large horizontal and vertical distances during erection. The hanger support system is thus very flexible, and insensitive to errors in length. In a self-anchored bridge, as in a traditional suspension bridge, control of the suspender lengths is sufficient for successful erection, even when the ropes are inclined. The overall geometry of the bridge is sufficient to produce the correct tensions.

Non-linear analysis can solve the indeterminate suspension system geometry once the criteria are defined. The important issue is to understand the consequences of the indeterminacy, and to select the proper solution from among the many potential cable profiles.

The third simplification, that of zero girder moments, is also absent in the new Bay Bridge. At the West end the girders are rigidly connected to the cap beam of the West pier, permitting no rotation. As the cap beam is concrete, the potential for creep is present, and the designers chose to minimize the dead load moments there. Over the East pier a large moment must be present to support the cantilever eastern approach. Throughout the suspended portions of the girders the design moment must be built in. The permanent box moments require that the girders be constructed with camber if they are to meet the desired Profile Grade Line. In addition, field splicing must be performed in such a way that the desired cambers and moments are achieved.

A suspension cable must be erected in a free-hanging state, in order to have good compaction. The Bay Bridge cable will have to move as much as 10 meters from its initial to its final position, when the inclined hangers are attached and fully loaded. This complicates the attachment of the suspenders. Most of the main span hangers will be too short to reach between the cable and their anchor points on the girder. In addition, the angle of the hangers, if attached to the free cable, would change as the cable is moved to the final position. The suspender brackets (figure 15) cannot accommodate rotation.

A **proposed erection method**, which addresses these issues for the Bay Bridge has been determined. The box will be constructed first on temporary falsework. It is contemplated that box segments as long as 80 meters may be transported to the site on barges and lifted onto the permanent and temporary piers (Fig. 19). Field splicing of these segments will consist of combined bolted and welded connections. Shop and yard splices will be fully welded.

Because the western end of the box will be connected to the cap beam, the girder will be aligned at its final profile grade at erection. When the weight of the girder is fully supported, camber for the permanent negative moments will curve the box upward, starting at the west pier. Falsework under the side span is to be placed at elevations higher than the final Profile Grade Line at positions calculated to match the girder camber. In the side span the girder will lie less than 1 meter above the profile grade.

In order to develop the erection plan, the girder is treated as a continuous member with no supports between the east and west piers. Self-weight and falsework support forces must be applied. The unknown falsework forces must be determined to meet the following requirements:

1. zero moment inflection points must occur at most of the field splice locations,
2. maximum erection moments must not exceed girder capacities,
3. girder deflection in the main span, including the desired camber, must be on average at least 3m above the final profile.

The resulting shape of the girder defines the elevations of the falsework supports which must be used during the erection. The support points must be adjustable to align the girders for splices not at inflection points.

The **Erection of the Suspension System** will be controlled by the fabrication of all suspenders to the computed strained length under the specified tensions. The cable will be brought into a position which allows free attachment of the hangers by the following means:

- the deck profile will be raised,
- several haul ropes will bring the cable outward towards its final position.

The permanent suspenders will not carry any of the box girder weight during erection, and will be installed by draping them over the cable band and attaching them to the their brackets. Some jacking will be needed mainly to pull against the self-weight sag of the hangers. Once all suspenders are in place and the cable cleared of temporary rigging, the entire box may be slowly lowered into its permanent position.

Shims are included in the design to allow for minor adjustment of the deck to the desired profile. Shimming will change the suspender tension by a small amount, adjusting for possible inaccuracies in the estimation of deck weights and fabrication tolerances.

The proposed self-anchored scheme will shorten the construction time compared with a traditional suspension or cable-stayed bridge, because the tower and box construction may be performed simultaneously.

Summary and Conclusions

The eastern spans of the San Francisco Oakland Bay bridge will be replaced with a new more robust crossing, consisting of a self-anchored suspension main span, and a segmental concrete box girder skyway. The crossing has been designed for a useful life of 150 years. The bridge is expected to carry to emergency vehicles within hours of an earthquake with a return period of 1500 years and a magnitude of up to 8.1. Full operation would be expected within days.

Self-anchored suspension bridges are currently finding applications around the world. The San Francisco Oakland Bay Bridge will have the longest span of this class of bridge. The single tower can support the full weight of the bridge, and has a ductility far in excess of the displacement demands. A single cable will span the crossing twice, being the first suspension bridge cable to loop around an anchorage and return.

The waterways of San Francisco Bay are graced with landmarks, including the Golden Gate Bridge, the Richmond-San Rafael Bridge, and the western span of the Bay Bridge itself. The self-anchored suspension bridge will be harmonious with the site and to the other bridges. The remarkable appearance of the bridge makes it worthy of the city of San Francisco.

The geometry, analysis, and the design of the practical details of the San Francisco Oakland Bay Bridge suspended structure posed a series of unique problems. These challenges were met by the joint effort of all those who participated in the design.

Acknowledgements

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This project has been designed by the joint venture of TY Lin International and Moffatt & Nichol Engineers. Weidlinger Associates Inc. has performed the designs of the box girders, crossbeams, and anchorages, and independent review of the tower. The engineering firms of Modjeski and Master, Roman Wolchuk, Paul H. Mueller, and Jackson Durkee have consulted on, and performed independent reviews of the girders, anchorages, and suspension system. For the Self-anchored Suspension Bridge design the authors served as project engineers and managers for Weidlinger Associates and TY Lin respectively.

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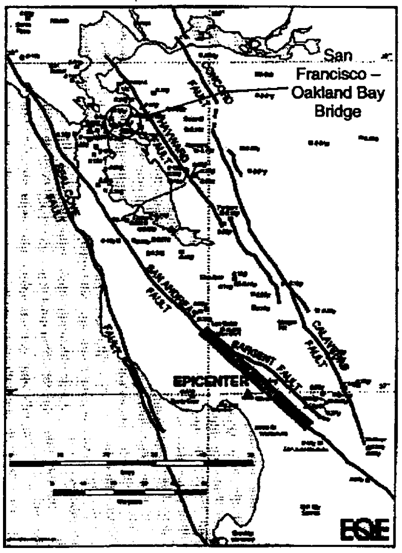

Figure 1. Site of Loma Prieta Earthquake (October 17, 1989)

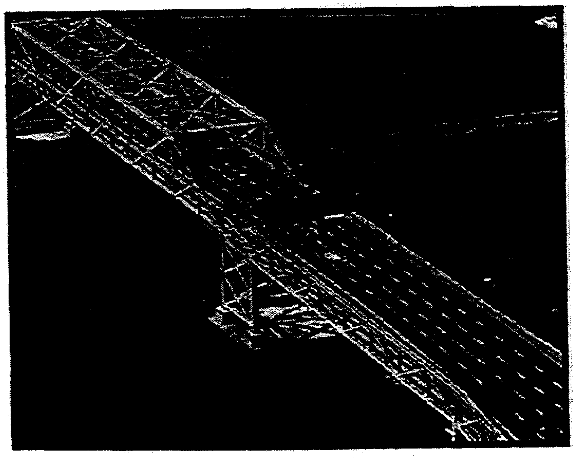

Figure 2. San Francisco – Oakland Bay Bridge East Span.
Collapse of upper roadway in Loma Prieta Earthquake in 1989.

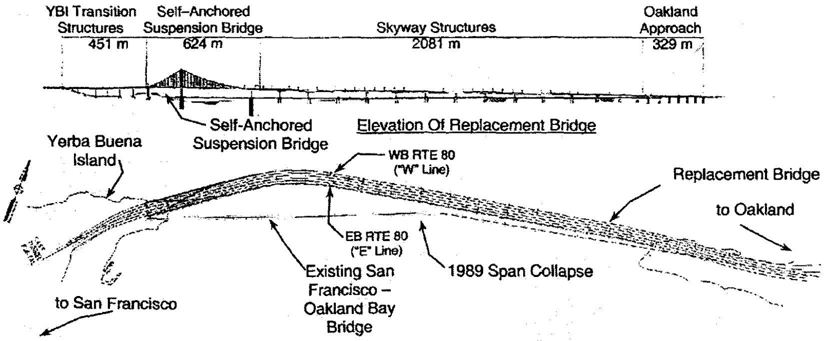

Figure 3. Plan of East Bay Bridge replacement

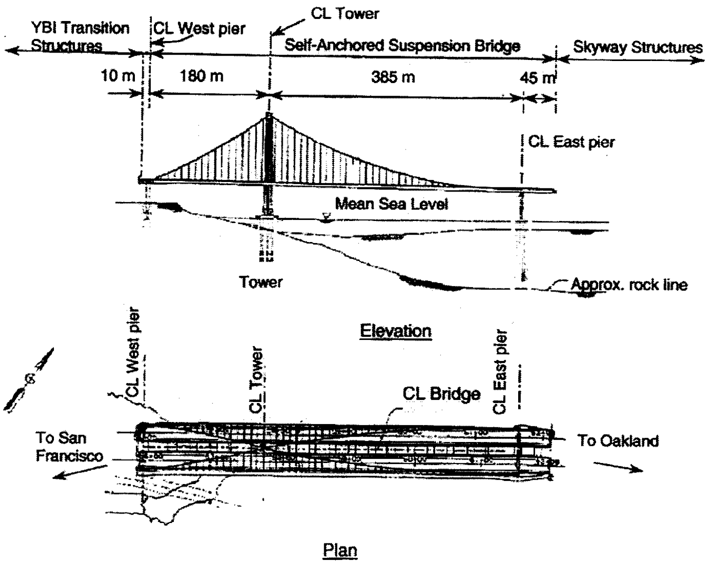

Figure 4. Single Tower Self-Anchored Suspension Bridge

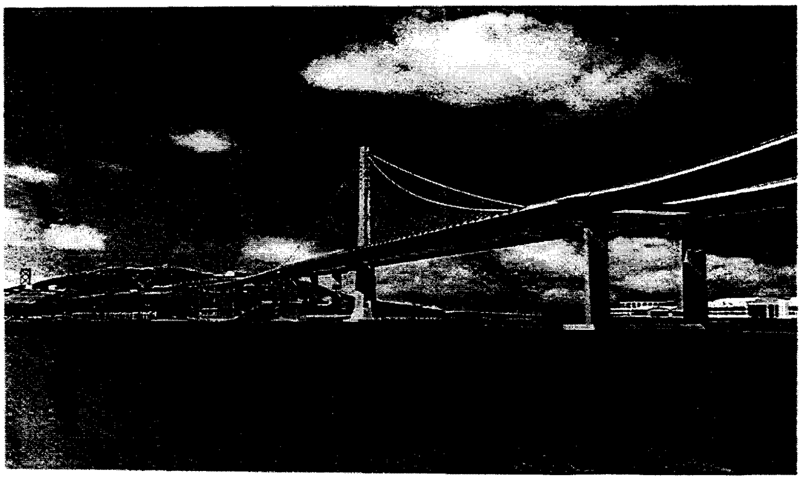

Figure 5. Architectural rendering of Single Tower Self–Anchored Suspension Bridge

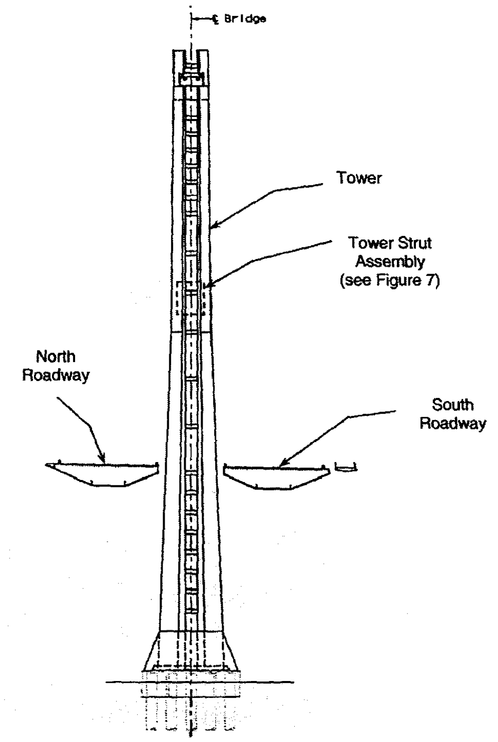

Figure 6. Cross–section of bridge at tower
(suspension system not shown)
(See also Figures 7 and 16)

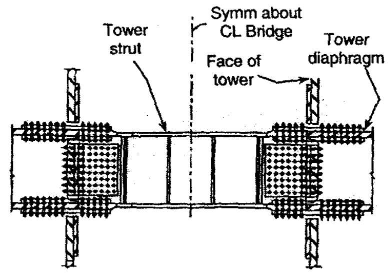

Figure 7. Tower strut assembly frames
between the tower legs (See Figure 6).
Architectural housing around strut not shown.

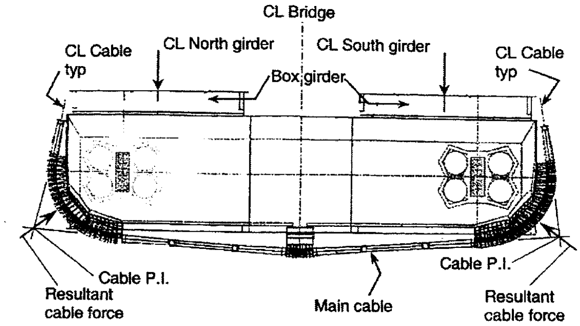

Figure 8. Plan West cable loop anchorage

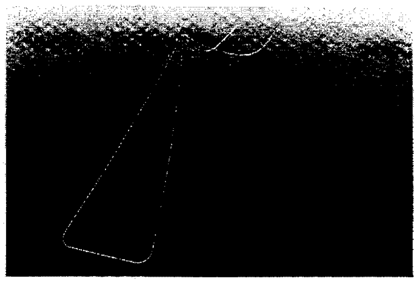

Figure 9. Isometric view of main cable geometry

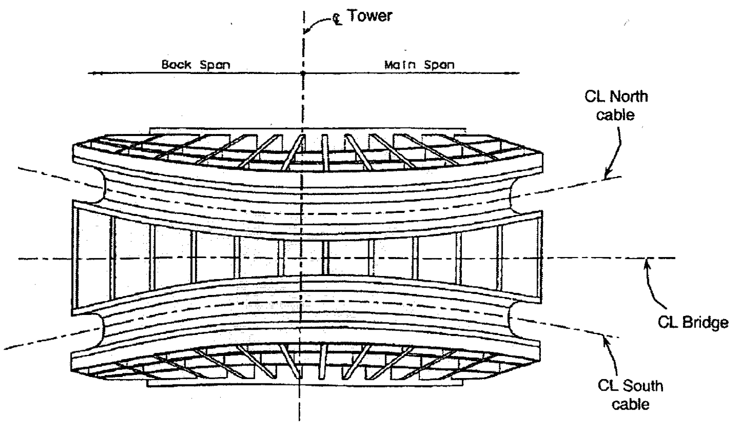

Figure 10. Plan of Saddle Tower

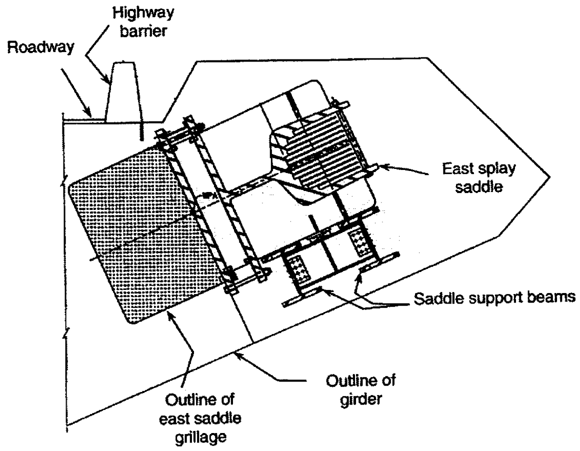

Figure 11. Section of east splay saddle, looking East.
For location see Figures 12 and 13.

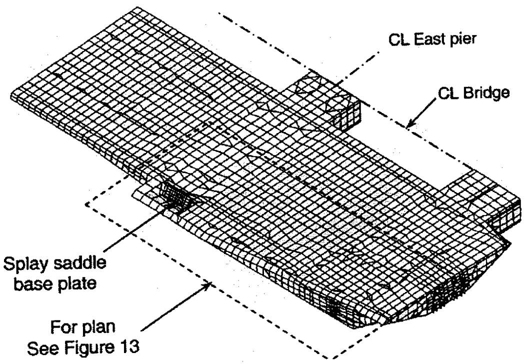

Figure 12. Rendering of computer model of South girder
at East Anchorage (North girder not shown)

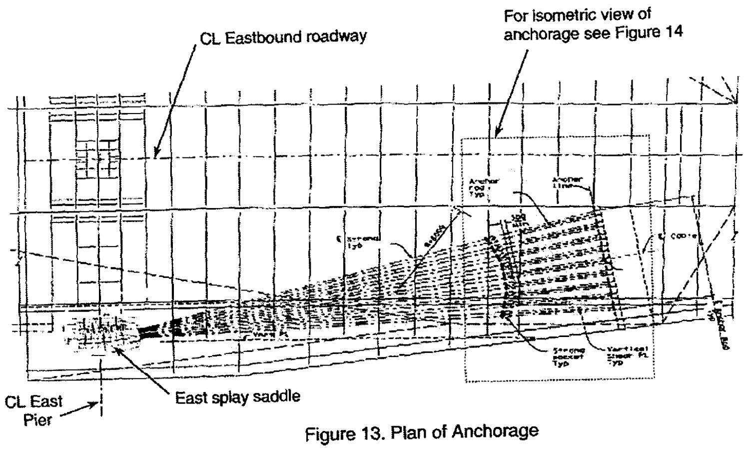

Figure 13. Plan of Anchorage

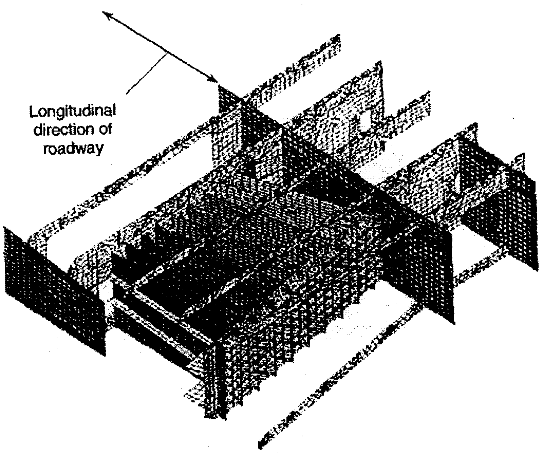

Figure 14. Isometric view of east cable anchorage. Box girder, top anchor plate and top shear plates removed for clarity. For location see Figure 13.

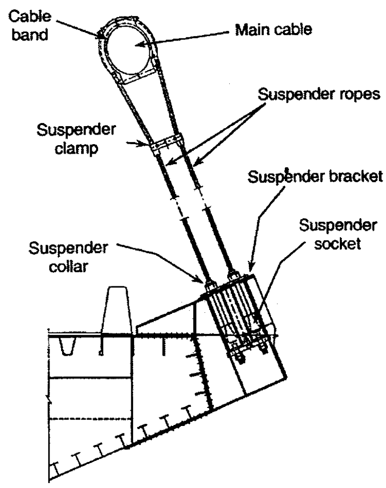

Figure 15. Typical hanger assembly

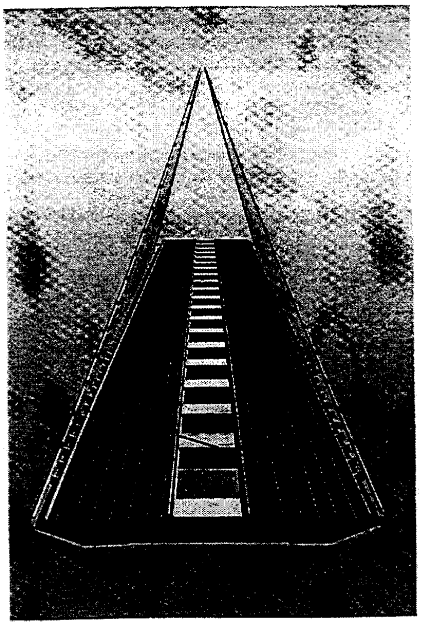

Figure 16. Longitudinal isometric view of girders and crossbeams (Tower and East anchorages removed)

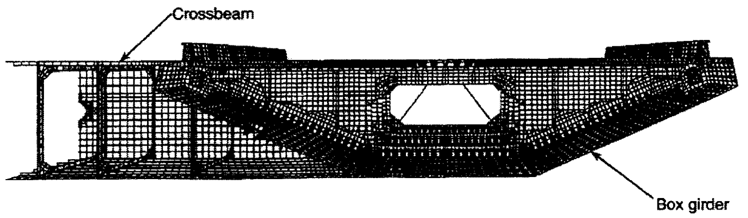

Figure 17. Perspective–cutaway view of box girder
and cross beam computer model

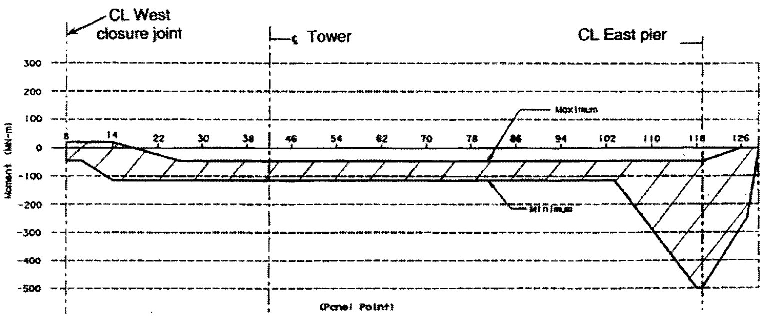

Figure 18. Envelope of design box girder dead load moments
vs. panel point location along girder

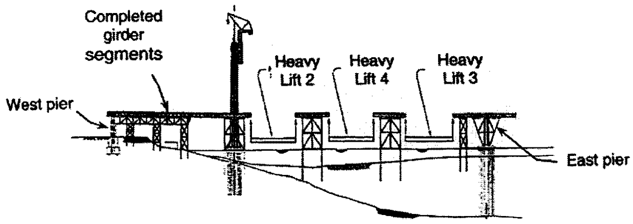

Figure 19. Erection of main span box girder. In this stage falsework piers
and various girder segments are already in place.