出國報告(出國類別：短期專題研究)

105年度交通部選送人員赴國外專題研究—橋梁耐震補強計畫執行策略、設計、材料、工法實務研習

服務機關：交通部臺灣區國道高速公路局中區工程處

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摘要

目前本局正辦理高速公路後續路段橋梁耐震補強之規劃設計，從前期國道橋梁耐震補強之執行經驗及統計結果，基礎補強所需費用最高，施工最為困難，對地方道路之交通及河川公地影響也最大，期望藉由此次研習機會蒐集美國橋梁耐震補強策略、材料、工法、技術等新知與資料，作為國內橋梁耐震補強參考，若能在符合耐震安全標準下，儘量減少基礎補強，將可有效降低施工經費並縮短補強工程期程。

我國部頒橋梁設計規範主係參考美國AASHTO規範修訂，其中加州為美國地震最頻繁地區，經常有大地震發生。加州運輸署 Caltrans管轄之橋梁共計2萬5千（雙向分開計算）餘座橋，計有員工約2萬人，負責加州橋梁自辦設計、監造、檢測及維修與耐震補強設計，其橋梁耐震設計與補強作為向來居於領先地位，且與本局訂有技術交流協議。筆者於 105 年5月下旬獲人事室通知經奉局長推薦代表本局參加105年度交通部選送人員赴國外專題研究，須於本年7月31日前將出國專題研究執行計畫書函報交通部，且出國日期不得超過本年8月31日，因事屬突然時程緊迫，經與美國加州運輸署以電子郵件積極聯繫結果，同意安排約12週之研習課程，內容包括結構策略與創新、耐震設計及標準、材料工程及試驗、施工實務及經驗回饋、橋梁災損緊急應變作為、施工查驗等作業實務研習與交流。其中6週安排於加州洛杉磯運輸署辦公室，進行橋梁結構補強設計研習並參與實際設計作業，研習相關契約執行計畫，耐震設計標準制訂、強震儀器和耐震設計支援實務作業。後續6週再到北加州沙加緬度運輸署總部工地及附設之實驗室進行現場觀摩實習與技術交流，包括專門的實驗室和現場測試和檢驗觀摩實習、大地工程研習、土壤和岩石現場調查、岩土地震工程、基礎的建議、基礎設計和施工支援、邊坡滑動、落石，及橋梁沖刷和地震破壞的災損緊急應變作為觀摩實習。藉由前往加州吸取橋梁耐震補強策略、材料、工法、技術等新知，及相關延長橋梁壽命之新技術，期能回饋至國內，提昇國內高速公路橋梁耐震補強工程技術，達成國家整體防災計畫之永續發展總目標。

目次

壹、目的

高速公路為我國交通動脈，屬於國家交通關鍵基礎設施，為政府推動國家關鍵基礎設施防護（CIP）中重要的一環。目前，全球各國對於地震仍無法事先預測與阻止其發生。但對道路橋梁而言，補強工程確實可加強其對地震之抵抗力，不僅在地震來臨時能減少橋梁的損害，還可在救災時發揮緊急運輸的功能。因此，歐美及日本等先進國家的地震防災策略，偏重於考量生命線工程系統的風險管理理念，若有需要或能力可及，對高速公路橋梁的耐震補強工作皆不遺餘力。本局於民國88年921集集大地震後，考量高速公路為交通大動脈，為防範於未然，即積極研提建設計畫爭取經費，分階段逐年推動國道橋梁耐震補強工作。對於不符合最新耐震規範之橋梁進行補強，以期日後大地震侵襲時，能減少損害、避免傷亡。提供經濟發展所需之高安全性基礎交通建設，達成整體防災計畫『永續發展』總目標，並建構高效率的地震救災緊急道路系統。

目前本局已完成第一期工程之國道1號及國道2號全線，及第二期工程第一優先路段之國道3號北部汐止至香山路段及南部部分路段橋梁耐震補強工程，總計已完成國道1,155座橋梁之耐震補強工程，完成耐震補強之橋梁均已符合交通部頒最新耐震規範。其餘尚未辦理耐震補強之國道橋梁尚有1,169座，已於104年11月奉行政院核定辦理「高速公路後續路段橋梁耐震補強工程」建設計畫，該補強工程計畫分3個區段辦理，目前刻正辦理區段1之規劃設計中。從前期國道橋梁耐震補強之執行經驗及統計結果，基礎補強所需費用最高，施工最為困難，對地方道路之交通及河川公地影響也最大，期望藉由此次研習機會蒐集美國橋梁耐震補強策略、新材料、新工法、新技術等新知與資料，作為國內橋梁耐震補強參考，若能在符合耐震安全標準下，儘量減少基礎補強，將可有效降低施工經費並縮短補強工程期程。

貳、過程

一、 行程安排

我國部頒橋梁設計規範主係參考美國AASHTO規範修訂，其中加州為美國地震最頻繁地區，經常有大地震發生。加州屬加州運輸署 Caltrans負責加州橋梁自辦設計、監造、檢測及維修與耐震補強設計，其橋梁耐震設計與補強作為向來居於領先地位，且與本局訂有技術交流協議。筆者於 105 年5月下旬獲人事室通知經奉局長推薦代表本局參加105

年度交通部選送人員赴國外專題研究，須於本年7月31日前將出國專題研究執行計畫書函報交通部，且出國日期不得超過本年8月31日，因事屬突然時程緊迫，即刻與美國加州運輸署以電子郵件積極聯繫結果，同意安排約12週之研習課程，內容包括結構策略與創新、耐震設計及標準、材料工程及試驗、施工實務及經驗回饋、橋梁災損緊急應變作為、施工查驗等作業實務研習與交流。其中6週安排於加州洛杉磯運輸署辦公室，進行橋梁結構補強設計研習並參與實際設計作業，研習相關契約執行計畫，耐震設計標準制訂、強震儀器和耐震設計支援實務作業。後續6週再到北加州沙加緬度運輸署總部及附設之實驗室進行現場觀摩實習與技術交流，包括專門的實驗室和現場測試和檢驗觀摩實習、大地工程研習、土壤和岩石現場調查、岩土地震工程、基礎的建議、基礎設計和施工支援、邊坡滑動、落石，及橋梁沖刷和地震破壞的災損緊急應變作為觀摩實習。藉由前往加州吸取橋梁耐震補強策略、材料、工法、技術等新知，及相關延長橋梁壽命之新技術，期能回饋至國內，提昇國內高速公路橋梁耐震補強工程技術，達成國家整體防災計畫之永續發展總目標。

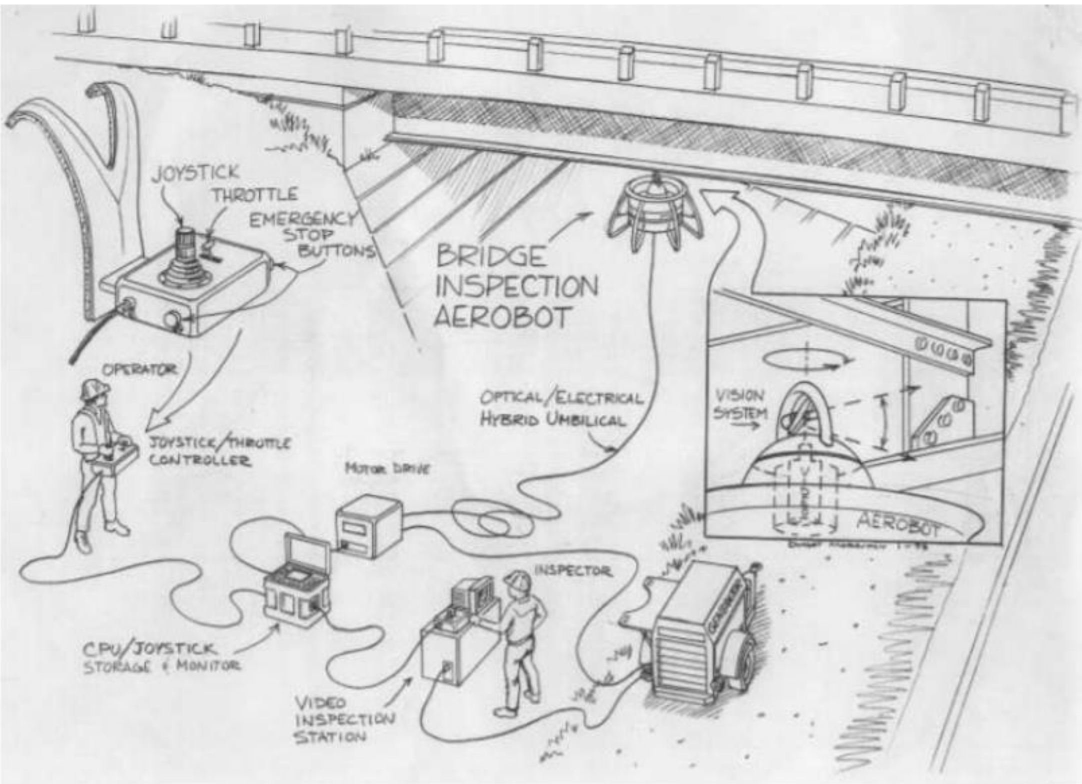
二、 研究實習內容

1. 橋梁檢測維修與災後應變

加州是美國面積第三大的州，僅次於阿拉斯加及德州。根據2015年的统計數據，加州GDP占全美的13.79%，達24,585億美元，為全美最高，若將其算成一個國家，則其GDP僅次于美國、中國、日本、德國、英國，是世界第6大經濟體，超過了法國。人均GDP為56,365美元，在全美各州中名列第12位。加州運輸署（簡稱Caltrans）位於加州首府Sacramento，其使命是提供一個安全，可持續，綜合和高效的交通系統，以加強加州的經濟和宜居性，有六個主要項目：航空，公路運輸，大眾運輸，交通規劃，行政和設備服務中心。負責管轄加州總長8萬多公里之高速公路、公路與鐵路，計有員工約2萬人，負責加州公路及橋梁自辦設計、監造、檢測及維修與耐震補強設計。Caltrans結構維護和調查單位Structural Maintenance and Investigation（SM＆I）擁有200名經過專門培訓的工程師，技術人員和行政協助人員負責對12,000多座州屬公路橋梁和當地政府機構擁有的約12,200座橋梁進行檢查，進行結構工作維修建議，確定所有橋梁的安全承載能力。每座橋梁都經過由具有橋梁專業知識的持照工程師定期進行的定期檢查。 大多數檢查每兩年進行一次。 跨越水道的鋼橋梁和結構得到額外的關注，由具備特殊（Fracture Critical）及水下橋檢隊，檢測包括鋼構元件和水下橋梁結構元件。 所有檢查都有一個目的：確保每個開放交通的橋梁的安全性和可靠性。每年依照聯邦公路總署制定之橋梁

檢測標準辦理橋梁檢測，檢測橋梁數量包括州及地方橋梁每年12000座以上。Caltrans每年花費約4.5億美元用於橋梁維護、檢查，其中大部分費用來自州高速公路營運和保護計劃the State Highway Operation and Protection Program（SHOPP）。在橋梁檢測方面，加州規定必須有註冊專業執照者，方可執行此任務，且需為專職，目前具專業執照之橋檢人員，以兩人為1組檢測完所有年度排定橋梁，並有專人負責抽查品管，再接受聯邦公路總署支複查。與國內目前尚無需執照制度大大不同，應可提昇檢測專業水準及檢測成果之可靠度，此外國內工程師橋梁檢測僅為其工作之一部份的現象甚為普遍，實有增加人力的必要，使其可專職辦理橋檢業務以利實務經驗之累積提升檢測品質。也可以使工程師對於所負責維管之橋梁能更為熟悉，可確實掌握維管橋梁之耐震易損性與特性，於震後可迅速動員經高度訓練與專業經驗之工程師來投入評斷橋梁之安全性並進行必要之緊急處置。此外，加州之橋梁大多數於設計建造時即有考量日後檢測及維修之可及性，對於較高橋墩上有裝設電梯等上下設備，並於上部結構設置檢測維修走道，減少檢測維修之困難。此外Caltrans在2008年曾經嘗試發展線控的飛行機器人來輔助橋梁檢測。

（如圖一~二）



圖一

圖二

加州橋檢頻率除災後特別檢測外，依需要分為三類：

（1）Routine定期檢測，依橋梁之實際狀況可彈性調整每2至4年一次，主要為掌握橋梁結構之健全　度、及早發現並評估造成功能減低之損傷及其原因，而於定期接近橋梁結構物實施之檢測。，其主要工作為：基本資料之收集、狀況評估、工作建議、照相、撰寫檢測報告。

（2）Fracture Critical斷裂臨界檢測，是設計為具有很少或沒有載荷路徑贅餘度的結構，當有主要荷載構件損傷破壞時即可能造成整體橋梁之崩落破壞，加州有238座斷裂臨界橋梁在州屬高速公路系統。Caltrans有一個斷裂臨界檢驗單位，使用專門接進和非破壞檢測設備識別任何潛在的徵兆，即使那些人眼不可見的。以檢測儀器針對有裂損或其他異狀之橋梁做較詳細之局部檢測，一般每2年一次。

（3）Underwater Inspection， 每5年一次以具專業檢測及潛水執照之人員潛入水中，檢測橋梁基礎之各項狀況。

為利於橋面板之檢測，一般混凝土橋面上規定不鋪築AC（圖三），檢測若發現橋面裂縫，影響橋梁長期之耐久性和承載力，進而滲水、鋼筋鏽蝕進一步劣化，更嚴重甚至可能造成橋板破裂掉落，在加州高分子甲基丙烯酸酯（Methacrylate）經過十多年來多次實驗測試及實作，目前已經被廣泛用作裂縫密封劑，加州用於涉及甲基丙烯酸酯在州屬橋梁上的維修花費每年可達數百萬美元。

圖三

Caltrans於地震過後的重要任務之一是在每次地震後，快速評估加州之橋梁及道路所受之影響衝擊，作為震後緊急決策及反應，以確保大眾安全並引導緊急救難車輛順暢及重新評估檢視建立重要維生道路路網。Caltrans在這十數年來已展現出其改善之震後緊急反應之應變能力（包含團隊和設備），CALTRANS結構部開發了一種程式化數據庫，具有所有24,000個州，縣和城市橋梁的坐標存儲。CALTRANS可以生成整個狀態或任何部分的映射狀態顯示橋梁，與主要斷層重疊的組合。這些地圖可以在電腦螢幕上查看或列印供設計者在篩選中用於識別高風險橋梁。步驟是非常簡單的使用計算機數據庫來定位所有公路橋梁狀態系統，定位所有地震斷層，然後確定這些結構處於高風險區。並發展可快速於震後反應之工具“Shake Cast”其主要特性為：

（1）可在網路執行的軟體系統。

（2）地震後數分鐘自動探索地震震動量數據和分析。

（3）依據震動資料進行每座橋梁性能特性之相關分析。

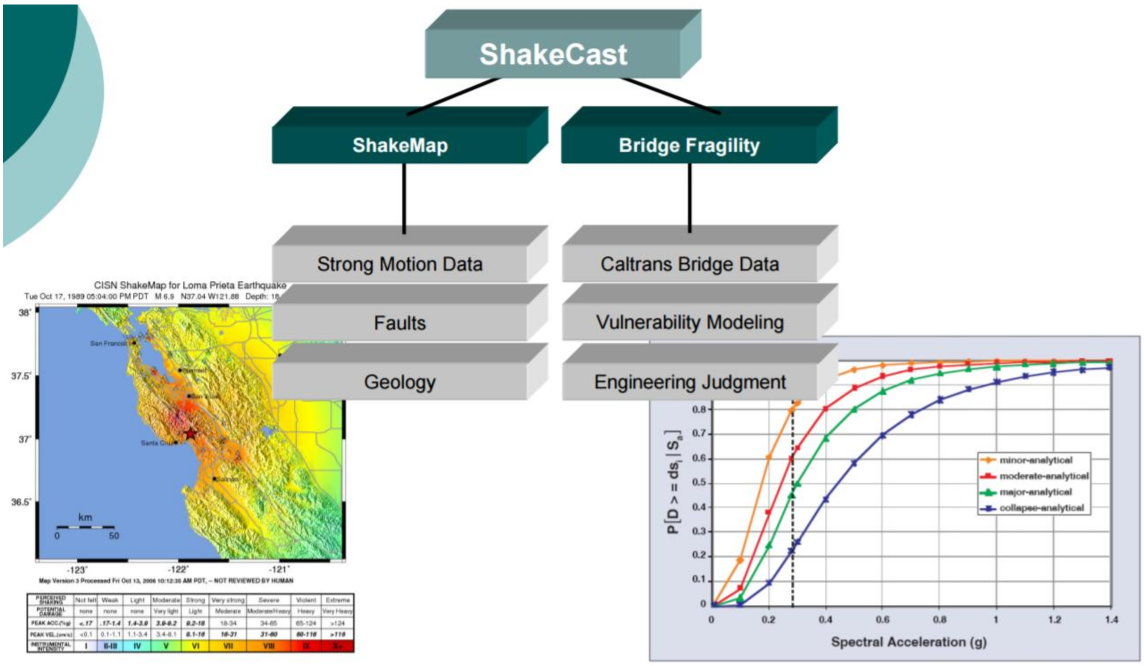
（4）自動提供最可能受影響衝擊橋梁之層次危害表及位置地圖。

（5）地震後10至15分內將分析結果自動E-mail通知應變處置人員。

（6）於“Shake Cast”網站上提供整套工具使用

“Shake Cast”是建立在Shake Map上，此地圖是從分佈加州的1900多個地震震動偵測器提供之數據並結合地質資料而創建，可顯示地面震動強度之地圖。這些地圖將提供詳細有

關地震震央、地質特性、斷層位置及震度等相關資料，其精密資料遠超過一般媒體所報導的。Shake Map利用先前輸入Map中之加州交通橋梁和公路庫存數據自動分析並提供參數輸入Shake Cast，產生橋檢優先順序表，利用此系統可使Caltrans很快知道震後那些公路或橋梁之潛在危險，可作及時救援反應。（詳圖四）



圖四

Caltrans於地震後第一週，對地方單位及結構維護單位應辦事項分別如下：

地方單位

（1）儘速消除民眾之恐慌。

（2）儘速恢復電力、供水及通訊。

（3）儘速改善交通。

（4）因應需求程度及修復經費開始增加。

（5）妥善規劃FHWA、總統、州長可能視察。

結構維護單位

（1）臨時支撐工作報告。

（2）震後重覆目視檢查。

（3）詳細檢測開始。

（4）主要線路之橋梁恢復通車。

（5）DES（Emergency Response Services）施工、設計人員於現場促進修復。

總結：Caltrans持續以多種指標性方法改善其應變的能力，其方法如下：

（1）應急準備和培訓。

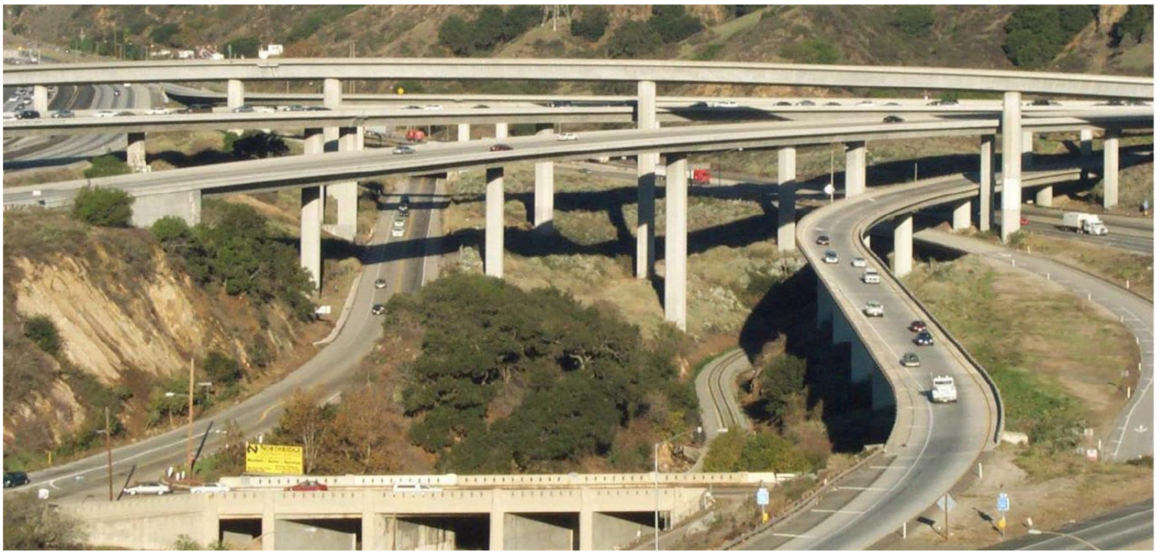
（2）改進評估及報告的方法。

（3）利用可靠的技術及進步的軟體。

（4）持續改進其設計及補強技術。

（5）推行相關的研究案。

由於加州現有大部分橋梁其墩柱與上部結構箱型梁係以整體式之固接接合，無需再設置連結上下部結構之支承墊，其伸縮縫一般係位於跨徑內之反曲點所設置外懸鉸接處（如圖五一~八），橋梁震動單元屬於靜不定度較高、耐震性能較佳之剛構架系統，且於震後可快速以目視方式檢測橋梁之狀況，評估橋梁之安全性。加州一般規定在大地震過後除上述緊急處置作為外，需要在4週內提出橋梁檢測調查報告。



圖五

圖六

圖七

圖八

2. 加州橋梁耐震補強歷史與策略

地震反應修正設備例如隔震支承、阻尼器或其他阻尼裝置和反力分散裝置等衝擊傳輸裝置等設備，在被用於Caltrans橋梁之前必須先進行預認證並被認可為經批准的新產品。

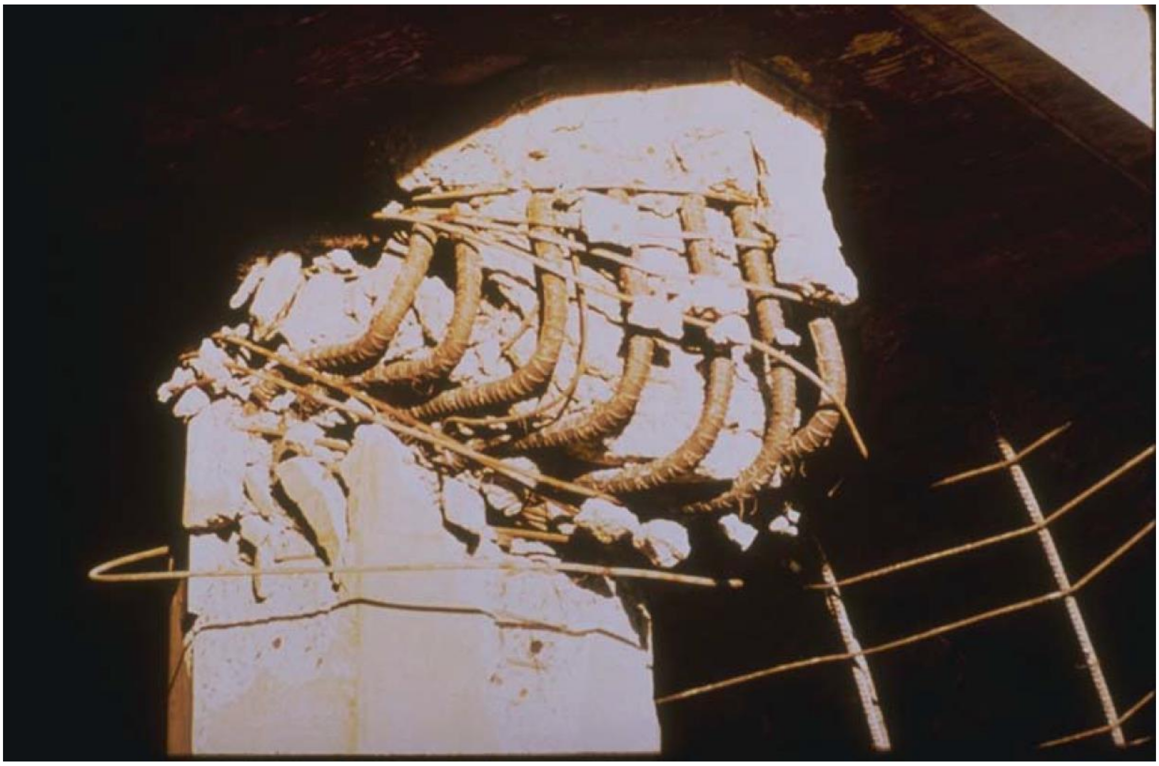
加州運輸署Caltrans維管之橋梁中有超過2/3約9千餘座是在1971年San Fernando聖費爾南多地震之前設計的，在當時有關橋梁耐震性能僅有考慮6%垂直載重為測向載重且不考慮結構非線性行為及韌性設計細節。結構韌性及損壞等知識尚在發展萌芽階段，在經歷了1971 San Fernando規模6.6的地震，造成洛杉磯地區數座橋梁嚴重損壞後（圖九~十五），橋梁工程師認識到橋梁結構耐震反應中細節和韌性的重要性，其後容量及韌性設計的概念被慢慢納入1974年版設計規範中。當時橋梁設計主要特性為：

（1）墩柱設計通常只配置非常少的橫向鋼筋，一般間距為30公分且未考慮柱之尺寸及強度。

（2）低估上部結構之位移量及設計載重，以至於橋台及外懸鉸接處伸縮縫之支承長度不足，尤其是skew斜交角較大之橋梁。（圖十六~十七）

圖九 橋梁落橋

圖十 橋梁完全崩塌



圖十一 橋柱頂剪力破壞

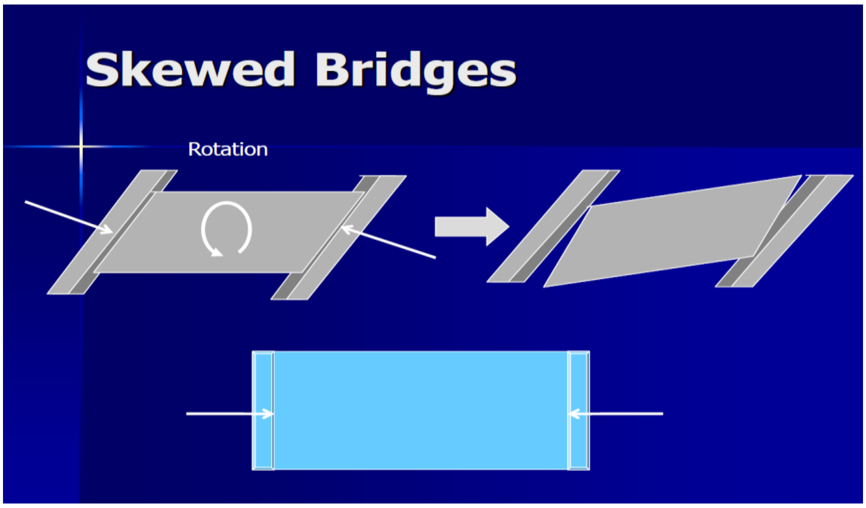
圖十二 橋柱剪力破壞

圖十三 橋柱剪力破壞

圖十四 橋柱主筋錨定長度不足破壞

圖十五 橋柱主筋挫屈破壞

圖十六 斜交角較大落橋



圖十七 Skewed Vs Straight Bridges

Caltrans於震後開始執行橋梁耐震安全補強計畫，依據橋梁地震受損之教訓，把重點置於單柱橋墩橋梁及活動端伸縮縫之改善補強，於伸縮縫加設防落設施或增加支承長度，也對於最具耐震危險性的單柱式橋墩進行鋼板包覆補強增加柱子的撓曲韌性及剪力容量。所有前述的補強於1989年完成，總計花費經費美金5,500萬元。而就在前期橋梁耐震安全補強計畫完成之同年10月加州又再度發生規模達7.0的Loma Prieta大地震，造成舊金山地區前所未有的橋梁破壞及生命損失，主要道路因部分橋梁倒塌而封閉（圖十八~二十）。880號州際高速公路於奧克蘭Cypress Street雙層高架橋倒塌造成42人死亡；舊金山到奧克蘭的海灣大橋其中一個跨經崩塌造成橋梁封閉30天，公路之關閉造成無法估量之經濟損失及中斷。Caltrans依據橋梁地震受損的教訓加速進行新的橋梁耐震補強相關研究與計畫，主要與加州大學柏克萊（UCB）及聖地牙哥（UCSD）兩個分校合作並且任命之名之專家學者組成了地震諮詢委員會Seismic Advisory Board。並根據研究成果大幅修正橋梁耐震設計標準，並依據研究成果及新的設計標準經耐震評估篩選了1039座州屬公路橋梁進行耐震補強，稱為橋梁耐震補強第1期計畫，以防止橋梁在日後發生之地震中倒塌或落橋。

圖十八 上部結構外懸鉸接損壞落橋

圖十九 雙層高架橋接頭剪力損壞落橋

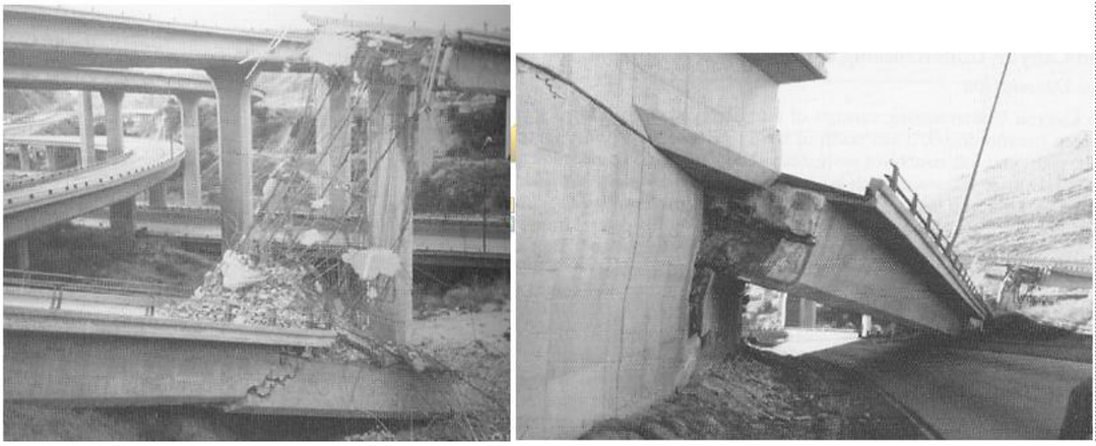
圖二十　橋梁上部結構穿孔損壞落橋

1994年1月南加州北嶺Northridge發生了規模6.7的地震，造成7座州屬公路橋梁倒塌，致洛杉磯西北部高速公路系統中斷，倒塌之7座橋梁中有5座已排定計畫需要進行耐震補強，而其中2座橋梁經耐震評估認定不需要進行耐震補強，因此當時相關耐震評估篩選程序經檢討仍有改進之必要。事實上，在此次地震中最大地表加速度（PGA）≧0.5g之強震範圍內506座橋梁中，僅有2座橋梁遭到Caltrans耐震評估篩選程序誤判，這個評估系統之可靠度實際上也算是難能可貴了。倒塌橋梁依設計及建造年份可概分為3組：第1組有3座橋為1971年San Fernando地震以前設計及建造的橋梁；第2組有2作為1971年以前設計但在1971年以後才建造；第3組2座則為1971年以後數年設計及建造的，但不符當時1974年新修訂之耐震設計規範。其他許多橋梁在強震中受損但沒有倒塌，損害程度從輕微裂縫及混凝土爆裂到更嚴重損壞需要封閉交通進行維修都有。在此區域內遭受強震橋梁中依當時1974年新修訂之耐震標準建造或補強的橋梁，大部分均僅有輕微損壞，全部橋梁均能維持安全正常通行。所有於此次地震受損之橋梁，從Northridge地震觀測之地殼運動紀錄來看是屬於可預期及合理的，這些老舊橋梁其設計地震力遠遠小於此次Northridge地震實際之地表運動，故橋梁損壞及倒塌均屬可以預期的。此次地震之橋梁倒塌型式經調查基本上與1989年Loma Prieta及1971年San Fernando地震，老舊橋梁損壞型式是一致的（圖二十一~二十四）。值得一提的是，所有依據1989年開始執行之第1期橋梁耐震補強計畫完成補強之橋梁，在此次Northridge地震計有24座已完成耐震補強橋梁位於強烈地表震動區域（PGA≧0.5g），另外60座位於最大地表加速度（PGA≧0.25g）區域內，其耐震性能均表現良好，完成耐震補強橋梁其耐震能力遠遠優於未補強橋梁。經加州地震諮詢委員會Seismic Advisory Board評估，這7座倒塌橋梁如有經過耐震補強，應該可以僅有些微損傷而不會倒塌。

圖二十一 橋柱剪力破壞

圖二十二 橋柱剪力破壞

圖二十三 伸縮縫處鉸接破壞落橋



圖二十四 伸縮縫處鉸接破壞落橋

1994年Northridge地震以後，加州通過了在交通建設資源配置上耐震安全應置於最優先之考慮，並加速辦理橋梁耐震補強計畫。Northridge地震時，幾乎所有單柱式橋墩橋梁耐震補強都已完成，僅有少部分尚在施工中。而多柱式橋墩橋梁僅有7%完成補強。Caltrans評估篩選1,155州管公路橋梁（大部分為多柱式橋墩），進行橋梁耐震補強，稱為橋梁耐震補強第2期計畫。此外，規模較大及重要性較高之加州收費橋梁，因所需經費龐大、規模及複雜性較高，耐震補強計畫進展較慢。但因這些橋梁之重要性，亦有必要於下次大地震來臨前，完成耐震補強以避免損壞之風險，實有必要加速辦理。Caltrans應評估篩選州內風險最高橋梁，儘速完成這些橋梁之耐震補強，以取代依橋梁分類來實施橋梁耐震補強。

由Northridge地震中證明了Caltrans至今所辦理的橋梁耐震補強計畫及施工方法，對於地震風險之降低效果顯著。Northridge地震對於Caltrans的耐震補強設計提供了很有價值對於高強度及中度規模地震實際試驗，但尚不包含加州將來可能面臨較大之長歷時地震。7座倒塌橋梁中有5座已經評估篩選，計劃計劃進行補強，而另外2座Misssion & Gothic穿越橋及I18州際公路的Bull Creek Canyon穿越橋，則經評估不屬於高風險性，未排定計畫進行補強。因此加州地震諮詢委員會Seismic Advisory Board建議Caltrans應對未包含於第1期耐震補強計畫內之橋梁，重新以在各種強度地震中均應避免倒塌為主要目標，來進行評估確認是否需要補強。依據1989年Loma Prieta地震後Caltrans所採行進化之耐震補強計畫實施計畫是合理可靠的，惟橋梁對於較長持續時間之地震經耐震性能評估，顯示仍有改進必要。Caltrans依據橋梁結構之易損性、地震風險及對經濟及社區之衝擊來分類，評估補強優先順序。每一類別中之各元素及權重似有未妥，Caltrans優選排序之程序應依最新地震獲得之教訓再檢討修正。重點應特別置於評估過程資料之品質，包括是否有非韌性橋柱、現地地質變化、一序列橋梁或互相聯結高速公路網是否容存在容易損壞之橋梁、其他特性及其權重亦應一併檢討修正。Caltrans橋梁設計標準分為重要橋梁及一

般橋梁二種類別。重要橋梁其性能目標為能在大地震後經快速檢視後立即開放正常通行；所有一般橋梁其耐震性能標準則為避免在大地震中倒塌，但允許有明顯損傷及提供有限度的服務。然而只要涉及二次安全、經濟衝擊、或緊急使用等三種特性需求，將使該橋分類提升為重要橋梁，所以橋梁之分類存在相當之灰色模糊地帶。Northridge地震後，一般大眾反應及建議將更多橋梁從現行分類中提升為重要橋梁。Caltrans應重新思考擴大定義重要橋梁及一般橋梁之合適結構耐震性能目標。日後地震風險評估應考慮橋址區域受盲逆衝斷層及結構特性受速度脈衝等之影響。此次Northridge地震部分鋼梁橋之端支承嚴重損壞，地震後有潛在風險，應加強鋼梁橋支承補強。

總結，預算、管理、法規和人事的限制是造成Caltrans在Northridge地震前，橋梁耐震補強進度不如預期之主要原因。其主要牽涉到議題為：

（1）能夠動員的人員數量及其技術水準。

（2）人員對合約管理及審查設計能力。

（3）工地監造人員支能力。

（4）如要加速辦理提高橋梁安全應再增加各項資源投入。

3. 透過持續研究改進橋梁耐震設計

Caltrans耐震研究計畫的目標是確定公路結構的地震易損性，並開發解決這些問題的工程解決方案。研究結果被納入到橋梁的耐震設計規範中，從而改善了耐震性能。Caltrans在Loma Prieta地震後於1989年發起了一項高度優先的研究計畫。該計畫的主要目標是確定公路交通結構的地震易損性，並製定解決這些問題的工程解決方案。Caltrans開始測試橋梁部件，以確定其抗震改造的能力和可行性。這些部件包括：在擬靜態負載下測試的不同的柱配置，基礎，抗彎柱帽，位移限制器和壁式橋墩。現在Caltrans使用更複雜的測試技術測試新的橋梁設計與細節。這些技術包括振動台測試，可測試整體橋梁系統而不是僅有橋梁構件。在1990年代，經由研究發現的成果實踐和開發更好的橋梁耐震補強工程技術的發展，引導Caltrans於1999年出版新的橋梁耐震設計標準，然後在2001年和2004年並再度更新。這些研究結果納入橋梁設計標準，以提高加州橋梁的耐震性能。

目前Caltrans耐震研究計畫的目標是：

（1）推動適用的以問題為中心的地震研究項目的開發。

（2）通過將研究結果納入Caltrans設計規範，指導材料和其他實施（從而提高運輸流動性，安全性，可靠性，性能和生產率），擴大研究結果的使用。

（3）提高橋梁耐震設計和補強實務，通過擴大研究工作，並將研究結果納入實務。

（4）驗證新的設計技術。

（5）驗證由工業引入的新材料/設備用於高速公路結構，以提高耐震性能。

在過去四十餘年中，加州和世界各地的地震提供了具體證據，使用舊的設計規範/假設建造的橋梁易受地震的損害。對加州橋梁的研究揭示了以下易損性：

- ◆ 墩柱細節不適當
  - ✧ 1971年之前Caltrans柱缺乏撓曲和剪力強度，這是由於不充分的圍束和搭接接頭。
  - ✧ 墩柱之主筋延伸進入上部結構和進入基礎其發展長度不足。
  - ✧ 擴頭墩柱的細節不適當。
  - ✧ 壁式橋墩的細節不適當。
  - ✧ 接頭剪力容量不足。
- ◆ 鉸接處支承長度不足
- ◆ 抗彎柱帽的剪力強度不足。
- ◆ 橋梁上部結構和橋台背牆之間的橋梁細節和間隙不足。
- ◆ 橋梁搖擺/支承的脆弱性。
- ◆ 現有基樁缺乏側向載荷能力。
- ◆ 基樁和樁帽之間的連接不足。
- ◆ 由於土壤側潰或液化導致之橋梁的脆弱性能。

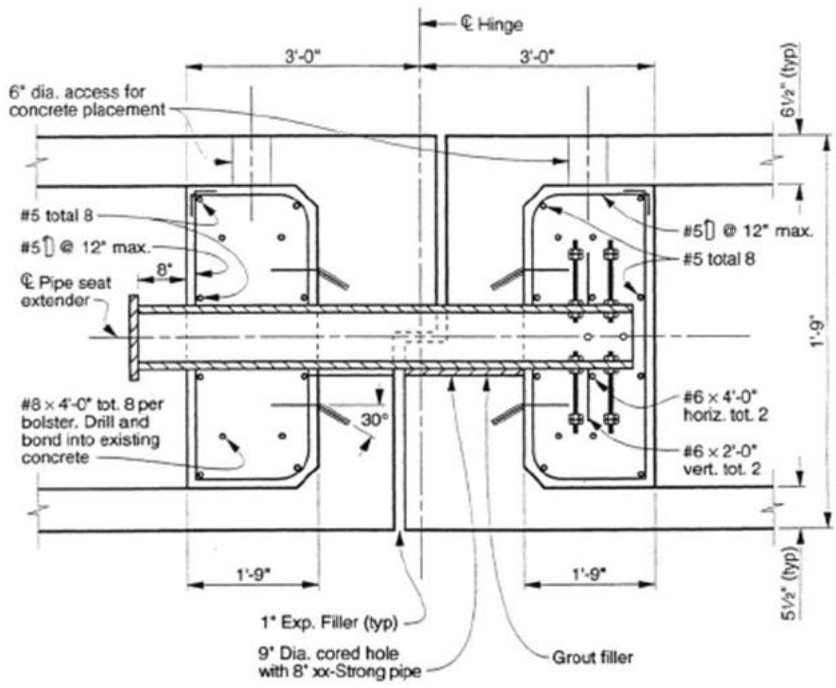
Caltrans地震設計標準（SDC）（Caltrans，2013）要求橋梁具有可以形成塑性鉸的柱，並且須具有足夠的韌性以承受甚至不可預期的大地震。這些橋梁中的大多數是現場澆築墩柱與後拉法預力箱型大梁之整體性抗彎構架。這種類型的橋梁在承包商已積累模具的同時，Caltrans在這種橋梁方面積累了經驗和信心，已在加州的高速公路佔據主導地位。由於Caltrans橋梁清單大多數均為抗彎構架橋梁。然而，這種類型的橋可能正在被加州淘汰。聯邦公路總管理署（FHWA）推動加速橋梁工法（ABC）和下一代橋梁（NGB），以加速橋梁建設，而不會破壞現有交通。另外，研究人員正在測試能夠保持相對完好的橋梁，並且可以在地震後不久恢復使用。Caltrans地震工程處（OEE）正在幫助編寫AASHTO耐震指南（AASHTO，2011），用於其他州使用，其重點是其他類型的橋梁。最終在加州採用本指南可能很方便。所有這些影響可能最終改變加州的橋梁目錄清單，伴隨著Caltrans SDC的變化，Caltrans工程師一直在努力使其他類型的橋梁能符合相同的地震標準。 近年來，Caltrans地震工程處（OEE）近來使用了可觀的資源和努力，發展設計中考量其他地震危害，改進分析程序，開發新的耐震補強程序，更好的加固細節，以及開發除韌性柱以外的抗震元件的標準。

◆ 固定接頭之改進

Caltrans已經花費了大量的努力來改進橋梁構件連接之可靠度。容量保護理念依賴於固定的柱連接，不會由於柱塑性鉸引致之剪力而損壞。當前的接頭剪力標準要求工程師確定接頭中的主應力。如果應力低，則僅需要一些額外的鋼筋，但是如果應力高，則接頭必須修改得更大。圍束鋼筋必須從柱子向上延伸到帽梁頂部鋼筋和向下伸入樁帽的底部鋼筋。墩柱的主筋必須完全發展延伸入頂部和底部接頭。對於大號鋼筋因直徑較大，需要較深帽梁以完全發展鋼筋強度，對於設計這可能會是一個問題。對於具有小的深度的接頭，需要使用“T”頭部主鋼筋，柱頭擴座連結帽梁或較小直徑的縱向鋼筋。

◆ 鉸接接頭之改進

Caltrans所偏好的橋型為墩柱與上部結構箱型梁以整體式之固接接合之剛構架橋梁，對於長度較長之橋梁，無可避免必須於橋墩間之跨徑內反曲點處設置外懸鉸接，一般以裝設鋼管來增加防落長度及束制變為（圖二十五）。此外若墩柱之直徑大於上部結構之深度，則梁柱結頭也許有需要改為鉸接。多柱式構架橋墩亦可能在柱底設置鉸接，以有效減少基礎的尺寸及橋梁建造經費。目前Caltrans委託內華達大學之研究計畫，仍持續在研究可靠的鉸接接頭設計，使其可承受一般橋梁之服務載重但不會在地震時受力而損壞。Caltrans使用鋼管插梢之鉸接接頭來減少斷面鋼筋將柱連接到帽梁。鉸接通常需要某種可更換的軸承表面和足夠的橫向鋼筋以保護鉸接處周圍的混凝土。



圖二十五 外懸鉸接增設限制位移之鋼管補強

◆　鋼筋續接的改進

Caltrans對鋼筋續接的使用有嚴格的規定。在塑鉸區（PHZ）中不允許續接，並且只有預先核准的極限續接可以在韌性構件的塑鉸區外部使用。大多數受容量保護的構件需要極限續接，而用於橋的其餘部分的鋼筋可使用服務載重續接或搭接。Caltrans很少在柱子中使用螺旋鋼筋，但是螺旋續接時，必須額外的180°搭接，穿過混凝土核心的對角彎鉤。大多數柱使用箍筋，需要使用極限載重續接。Caltrans耐震設計理念依賴於充足的連續性和發展，以在地震期間把各構件連結在一起。但因鋼筋通常最大長度只有18公尺，“無續接”的規定對於較長或高橋梁就需要例外性的放寬

◆　支承的改進

上部結構在橋台和墩柱構架處等下部結構之連接通常需使用支承。Caltrans有時會取消支承設置，改採以端隔梁和橋台整體固接墩柱構架，特別是在高地震風險區域。然而，非常長的橋通常需要於跨徑間設置鉸接座，以容許溫度、預力縮短和其它縱向位移。Caltrans要求這些支承長度必須比相鄰構架的位移（加上潛變，乾縮等）的平方根（SRSS）更長，且不得小於24英寸（600mm）。在橋台和墩柱帽梁的左端和右端設置剪力榫，以防止較小地震的橫向位移（和支承損壞）。最近的研究（Bozorgzadeh，2007）已經證明這些剪力榫比預期更強，因此已經設計了新的模組化剪力榫作為保險絲，其將在更重要的橋梁構件被損壞之前失效。對於跨徑中支外懸鉸接座，一般於接頭放置雙重強壯鋼管以防止橫向運動並提供更長的防落長度（圖二十五）。

◆　基礎的改進

一般柱的底部是連接於擴展基腳，樁帽或大直徑井筒。Caltrans使用兩種井筒基礎來支撐橋柱。 Type 1井筒與柱具有大約相同的直徑，並且被設計成在地表面下方形成塑鉸。Caltrans喜歡Type 1型井筒，因為它們具有高韌性和長的塑鉸長度。在UCLA的測試（Wallace，2001）顯示直徑1.8m（6ft）的井筒能具有20%位移比率。Type 2井筒具有比柱更大的直徑，以在地面上形成可靠的塑鉸。

基礎的設計取決於土壤條件，需要地質工程師和橋梁工程師之間的良好溝通。Caltrans成立了一個委員會，研究樁群在良好和不良土壤中的行為。他們發現，在良好的土壤中，樁基本上是受軸向力構件，並且廉價的標準樁可以鉸接方式與樁帽連接使用。然而，在差的土壤中，樁必須具有良好的韌性並且與樁帽使用固定連接。

擴展基礎允許在低到中度地面振動的區域中在良好地質上支撐橋台墩柱構架。設計

工程師有時會用抗拉拔構件（地錨或微型樁）來束制擴展基礎，以防止在地震過程中翻倒。 最近的研究（Deng，2010）表明，即使對於非常大的地震，這些擴展基礎也可以被允許搖滾。 然而，有一種自然的猶豫，允許橋梁來回搖擺，讓人很容易想像到橋梁會晃動不穩。

◆ 下部結構的改進

Caltrans偏好使用較具柔韌性墩柱甚於剛性的壁式橋敦，但仍然允許它們被建造。塑鉸構件和容量保護的相鄰構件的SDC耐震理念，因壁式橋敦比基礎還強壯，不能在橫向方向上形成塑性鉸而無法符合。Caltrans要求將壁式橋墩設計為用於設計加速度反應譜峰值（乘以安全係數）的剪力構件。一般壁式橋墩配置箍筋和繫筋，在大地震時可能鬆動。Caltrans需要一個交叉的繫筋，在一端有135度彎鉤，另一端有90度彎鉤交互配置，無論垂直和水平鋼筋在墩壁上匯合在一起。

◆ 橋梁型式的改變

Caltrans正在努力使更多類型的橋適符合使用SDC設計的“普通標準橋”的分類。

◆ 預鑄橋梁的發展

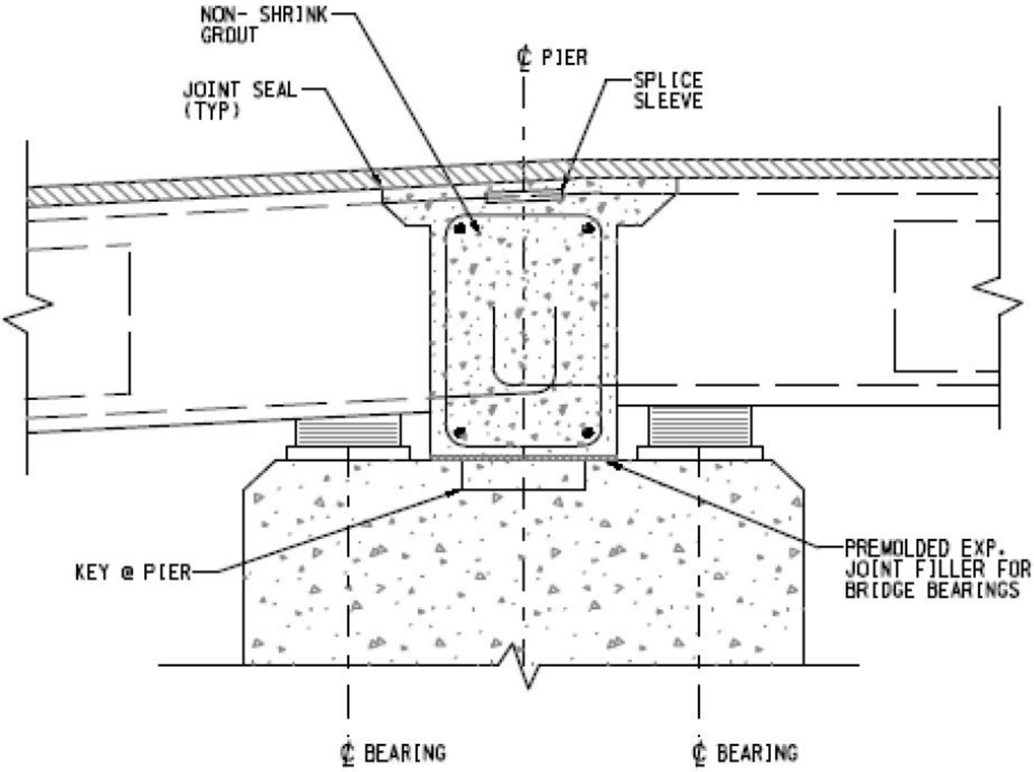
Caltrans開始設計預鑄梁橋，以滿足2000年對San Mateo Hayward Bridge拓寬的所有SDC設計要求。這條7.5公里（4.7英里）長的橋梁以墩柱構架支撐其上方預鑄大梁，設計用於提供所有鋼筋的完全連續性，以滿足柱塑性彎矩的接頭剪力要求。一旦施工開始，儘管在大梁和墩柱構架之間的複雜的連接，平均能建造每天超過30米的橋梁。

最近有幾個關於預鑄梁橋抗震性能的測試，重點是抗彎帽梁（Veletzos，2006）（Snyder，2011）。這些測試包括支撐不同類型預鑄梁的倒T形帽梁。Caltrans OEE將優選通過帽梁處具有梁正鋼筋，因為這最符合SDC要求並且確保所有損壞發生在柱塑鉸中。 Caltrans與愛荷華州立大學Sritharan教授正在合作探索相關設計。大梁將置放在倒置的'T'彎帽，底部預應力鋼筋纏繞4支＃11（直徑36mm）鋼筋。

Caltrans正在與內華達大學聯合探索的另一個選擇（Saiidi，2013）是加速橋梁建設（ABC）的下一代橋梁（NGB）組件。該項目於2007年開始，將附帶鋼筋續接器的預鑄柱與基礎的性能與Caltrans標準現場澆築（CIP）柱和基礎的性能進行比較。到目前為止，已經測試了五種配置：（1）CIP柱、（2）預鑄柱連接到無底座的敦頭續接器（HCNP）、（3）預鑄柱連接到沒有基座具延展性的鑄鐵灌漿套筒GCNP）、（4）連接到具有基座（HCPP）的敦頭續接器的預製柱，（5）連接到具有基座（GCPP）的具延展性的鑄鐵灌漿套筒的預製柱。預鑄柱是空心殼狀，在它們附接到基礎之後於內部填充自充填混凝土。基礎是將連接位置移動到塑鉸區域上方。到目前為止，測試一直令人鼓舞。 HCNP具有更好的

位移容量，但GCNP更容易組裝。測試將繼續。最終，Caltrans希望測試一個完整的預鑄基礎，預鑄柱，預鑄帽梁和預鑄大梁的裝配。Caltrans關心的一個嚴肅議題是，須確保這些預鑄構件不僅具有良好的抗震性能，而且是實用的，並且不會成為維護問題。

圖二十六 預鑄帽梁吊裝施工



圖二十七 預鑄大梁連續化示意圖

◆ 鋼橋設計的發展

鋼梁橋可以快速建造，對交通的干擾最小，這使它們成為加速橋梁建設的重要橋梁類型。鋼橋的抗震設計在Caltrans鋼橋抗震設計標準（Caltrans，2001）中提出。鋼梁橋的抗震設計在Caltrans SDC中提出。面臨的挑戰是將這些鋼橋的連接接頭設計為容量保

護構件。

◆ 長跨徑橋梁的發展

像舊金山奧克蘭海灣大橋東部跨度的項目給了Caltrans一個機會反思與長跨度結構有關的問題。塔需要在大地震後保持使用，因此塔腿之間的剪力連桿被設計成充當保險絲並保護塔免受損壞。然而，該地震抵抗構件ERE尚未經過廣泛測試，並且諸如焊縫、錨固及地震後的更換等均需要在剪力連桿成為橋塔標準設備之前進行廣泛研究確認。

Caltrans要求韌性容量安全係數至少為3，以防主要ERE損壞或發生不可預期的大地震。因此，Caltrans在長跨度橋梁上具有中空柱和塔的韌性和後屈服性能要求。最近對空心柱的測試已經顯示出可行性，但沒有像具有緊密間隔的大直徑箍的實心柱的韌性。對於更昂貴的橋梁，接受較小的強度和延展性似乎不合邏輯。Caltrans建議空心矩形柱在與非常強壯的隔梁連接的角落具有大的抗壓構件，因此在地震期間整個部分完全發揮作用。壓縮構件應延伸超過隔梁，由壓縮構件承受所受彎矩而不是由隔梁承受。其他要求包括：

（1）對穿緊筋為180度彎鉤和延伸9倍直徑長度。

（2）對於高塔，柱的最小壁厚必須為900mm（3'）。

（3）配置直徑24mm（＃8 US）內圈鋼筋。

（4）配置直徑43mm（＃14 US）外圈鋼筋。

（5）內圈鋼筋的支數必須至少為外圈鋼筋支數的50％。

（6）主鋼筋量包括內外圈鋼筋 ＞1％實心截面。

（7）在垂直鋼筋及具有箍筋處最小鋼筋間距200mm（8“）。

◆ 橋台束制

一個在普通橋梁上相當普遍使用的地震力抵抗構件ERE，有效利用橋台後面的土壤的被動土壓力來抵抗地震，且當土壤屈服時可作為消能作用。

◆ 橋梁隔震設計

目前所採用最具效果隔震設施如鉛心橡膠支承和摩擦單擺支承（AASHTO，2010）。它們一般可在比柱形成塑鉸時更小的力量下達到屈服，使得基礎可以設置得更小，並且防止柱損壞，使得橋梁可以在震後更快速回復正常使用。Caltrans成立了一個小組來發展使用隔震設施和其他ERE開發用於普通一般橋梁的設計標準，目標是要求不管橋梁選擇何種ERE或類似的作為，均應使得橋梁具有相同的安全水平。

（1）隔震橋應滿足Caltrans SDC的要求。

（2）使用AASHTO隔震指南來決定隔震橋梁之位移。

（3）使用SDC的附錄B（減少阻尼）來確定風險。

（4）隔震設施設計應有1.25倍位移需求的安全係數。

（5）所有下部結構構件必須具有大約相同的勁度和質量。

（6）所有隔震設施必須具有相同的勁度和位移容量。

（7）橋柱應設計於承受隔震設施側向力時保持彈性。

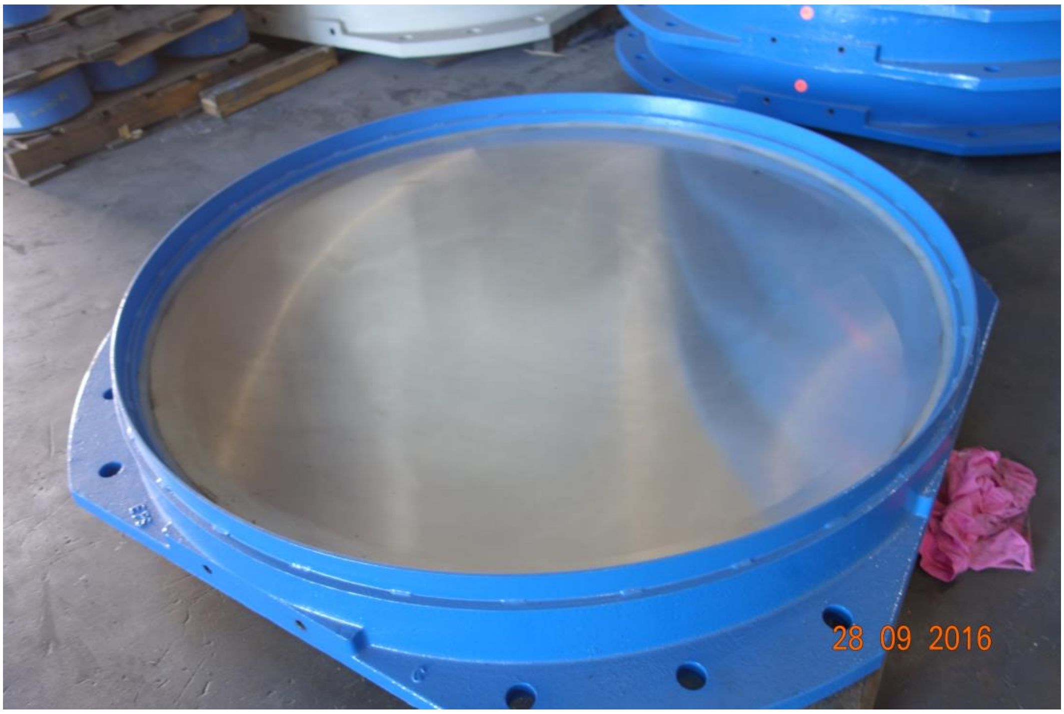
（8）橋柱設計為$Vu \geq 1.2 * F_{1.25 \Delta D}$（橫向力的1.2倍）。

（9）橋柱橫向容量> 0.15g（0.15倍靜載反應）。

（10）橋柱必須具有的位移容量（超過屈服）> 3.0。

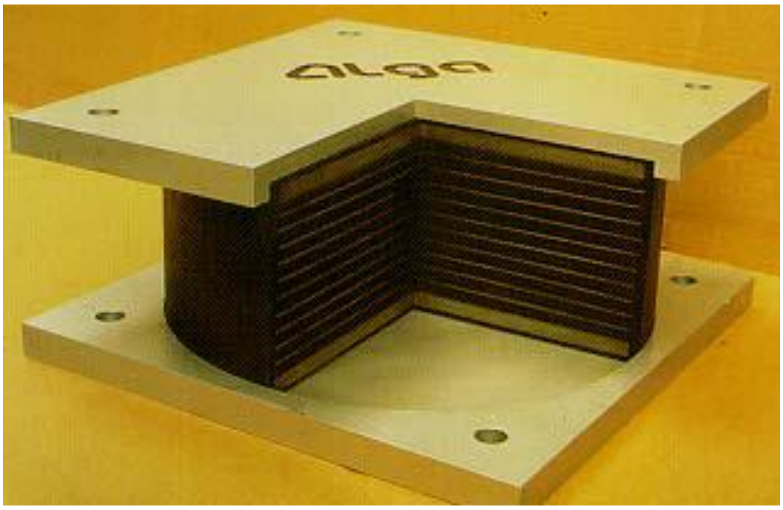
Caltrans制定規則以在隔震設施中提供足夠的強度以承受一般的服務和風荷載，同時還需要設置防落設施，作為隔震設施斷裂之保護裝置，對於伸縮縫則沒有特殊要求，允許可於設計地震斷裂（但可快速修復）。

圖二十八 Caltrans 6th Street 高架橋梁使用摩擦單擺支承



圖二十九 摩擦單擺支承下盤

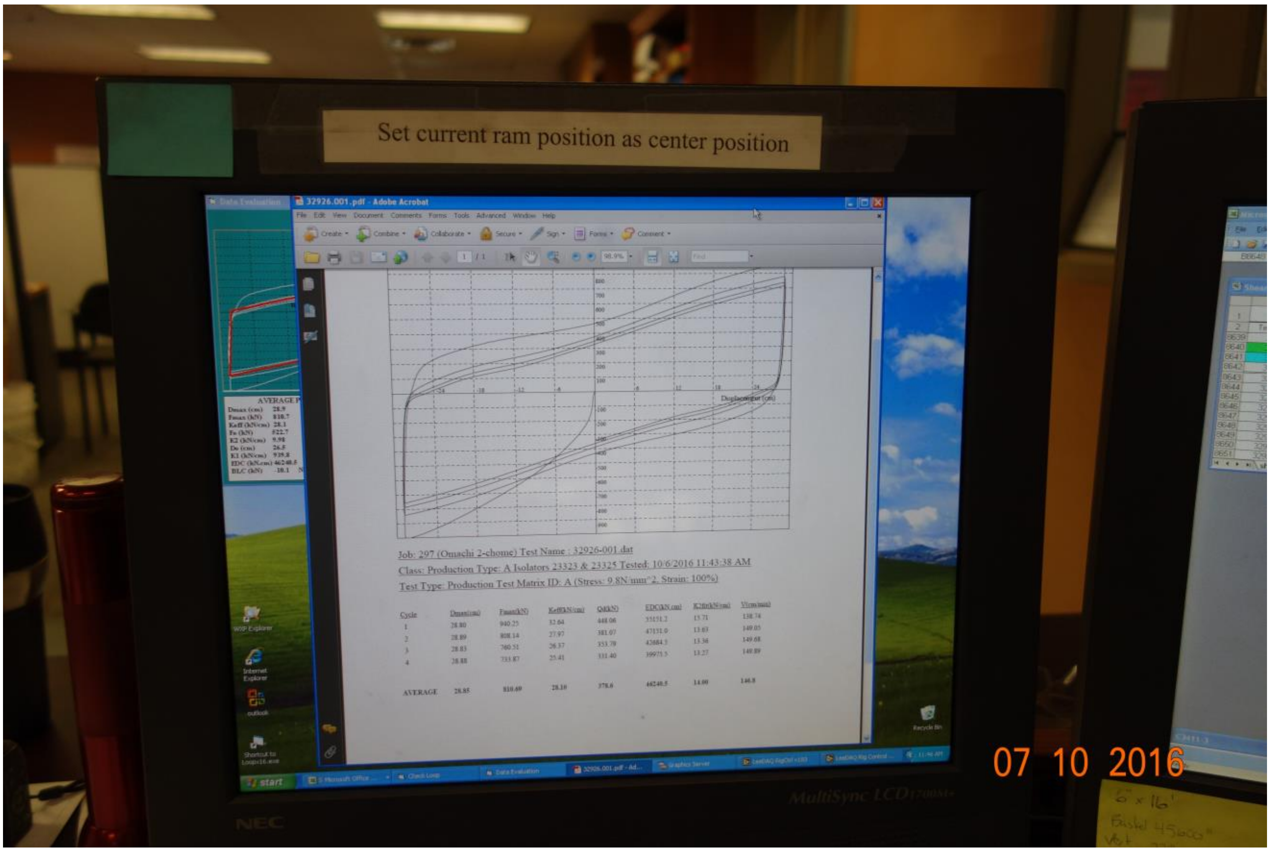
圖三十 摩擦單擺支承性能測試



圖三十一 鉛心橡膠支承

圖三十二 鉛心橡膠支承

圖三十三　鉛心橡膠支承性能測試



圖三十四　鉛心橡膠支承性能測試結果

◆　設置阻尼器

Caltrans曾經寄望粘性阻尼器被證明是一個有效的ERE，直到他們在幾個橋梁耐震補強工程計畫開始漏油。由於必須更換這些大型阻尼器的成本很高，Caltrans結構維護單位不願意把它們用在任何更多的州管橋梁上。最近的補強計畫（如Forest Hills Bridge）已經改用挫屈拘束支撐Buckling Restrained Braces（BRB），結果良好。此外，希望可以使用液體矽填充的阻尼器代替充油阻尼器，以消除所有的維護問題。Caltrans並已經資助對可能有一天被用作橋梁阻尼器的形狀記憶合金的研究。

◆　Rocking搖擺

Caltrans目前正在編寫程序，將允許搖擺作為新橋梁的可接受的ERE。Caltrans一直允許橋梁耐震補強採用搖擺，但新的橋梁的抗震性能要求更高 Caltrans已經資助了一些關於搖擺的研究項目，結果是正面的，足以開始討論允許於大地工程服務部門建議可採用擴展基礎短橋梁搖擺（Kutter，2010）（Panagiotou，2014）。類似於隔震支承，橋梁搖擺可減少基礎承受之地震力，可採用更小的基礎，並可使橋梁於震後更快速地恢復服務。

◆　自動回正橋柱

研究人員正在測試能夠保持相對完好的橋梁，並且可以在地震後不久恢復使用。華盛頓大學，史坦福大學和加州大學伯克萊分校正在研究預鑄橋柱，在中心有一個孔用於後拉預力鋼索，藉由預力可讓橋柱在地震後自動回正重新定位（Cohagen，2009）（Lee，2009）（Jeong，2008）。由於這些柱將更快地組裝，這將滿足Caltrans加速橋梁建造的目標。然而，Caltrans仍然關注這些柱的韌性，施工性和維護問題。

◆　改進的地震風險分析方法

Caltrans已經開發了新的方法來獲得由於地面震動，表面斷層，土壤側潰和海嘯風險對橋梁的耐震需求。根據橋梁位置，在分析中需要組合這些風險中的一些。

◆　地面震動風險

目前使用等效靜力分析（ESA）方法，彈性動力分析（EDA）方法或非線性歷時分析（NTHA）方法獲得地面振動風險的需求。無論使用何種方法，輸入地面運動都來自ARS在線網站上提供的反應譜（並在Caltrans SDC的附錄B中描述）。Caltrans計劃在2016年前僅使用基於概率的地面運動。

在最近的Caltrans地震諮詢委員會會議上，Ed Wilson教授嚴格地談到了EDA方法。他說：

・EDA方法僅適用於SDOF系統。

・它只產生正數位移和構件力。

・結果是在未知時間發生的最大可能值。

・短和長延時地震的處理方式相同。

・需求/容量比總是過度保守。

・EDA方法不提供對橋梁動態行為的洞察。

・結果不處於平衡狀態。

Caltrans希望提出對標準橋梁執行非線性歷時分析的程序。問題是提供地面運動的足夠的概率性導出的時間歷時，其作為反應譜在統計上是可靠的。目前在PEER中心有關於非線性分析的地面運動選擇和縮放的研究（Rezeaeian，2010），Caltrans將納入其新的分析程序中。該計畫是為橋址的特徵創建足夠的合成時間歷史，將時間歷程縮放到基本橋週期處的概率反應譜，並且使用這些縮放的時間歷時以30°的增量分析橋獲得對橋梁構件的最大要求。

◆ 地表斷層風險

基於加州地質調查（CGS）Alquist-Priolo地圖以及現場調查和文獻綜述，從全新世斷層獲得表面斷層危害 類似於地震動風險，確定性導出的斷層偏移量是基於使用統計或其他經驗關係的斷層特徵獲得的（Wells，1994）。概率導出的偏移量是使用舊金山公共事業委員會的報告（Abrahamson，2008）獲得的。一旦斷層被定位並且獲得偏移量，橋基礎被移動到偏移位置，並且從橋的3D模型獲得柱的位移。然後，在類似變形的狀態下獲得該橋的彈性形式的地面振動位移（Chopra，2008）。

◆ 土壤液化和側潰風險

Caltrans地工服務處目前正在製定“地工技術手冊”中評估地震災害的標準方法。基於Caltrans資助的研究（Shantz，2012）（Ashford，2010），一個新的程序用於確定橋梁由於土壤側潰的需求。Caltrans MTD 20-14和MTD 20-15為設計人員提供了用於橋梁設計之液化和側潰的簡單程序。

◆ 結論

在1989年Loma Prieta地震後Caltrans大大增加了耐震研究的資金，主要針對Caltrans現有橋梁耐震補強計畫，隨著橋梁耐震補強計畫接近完成，Caltrans將重點從現有橋梁的研究轉變為新橋梁的耐震設計。1999年，基於地震的經驗教訓和Caltrans研究計畫成果，Caltrans首次發布耐震設計標準SDC。儘管在多年沒有任何破壞性的地震，Caltrans仍然持續支持大量與耐震相關的研究計畫。

從CALTRANS多年來獲得的經驗回應自然災害，以及其對Loma Prieta災難的反應結果明確一個健全的，排練得當的應急計畫是任何一個必須的。因為緊急情況由各地方

機構進行的演習動員承包商，CALTRANS能夠動員工人、設備和材料進行即時瞬間修復工作，或已緊急臨時支撐方式來重新開放重要受損交通設施。Roberts（1990）和其他的CALTRANS發表論文。每個公共機構都應該有一個計畫準備和行使，以便人員準確地知道他們要報到的地方，及他們在發生自然災害時的職責。給定一個地震發生並在幾秒鐘內立即反應，完全沒有時間再進行規劃，必須基於先前的規劃和實踐練習。相關反應計畫必須包括冗餘路線和對重要道路和結構的評估與立即恢復交通等作為。

CALTRANS使用脆弱性對橋梁進行了優先級排序，確保最脆弱的橋梁優先安排進行耐震補強，將有限的資金，做最好的整體規劃和應用。對於防災及減災相關作為，Caltrans期望以Research→Implement→Improve→Prepare→Evuluate→ Report→Research之循環模式不斷求進步，增加震後應變能力。

4. 橋梁檢測與施工觀摩

圖三十五 參與 Caltrans 橋梁實地檢測作業

圖三十六 San Diego Coronado 橋梁檢測使用橋檢車

圖三十七 San Diego Coronado 橋梁檢測使用橋檢車

圖三十八 San Diego Coronado 橋梁檢測人員合影

圖三十九 San Diego Coronado 橋梁檢測塗裝剝落

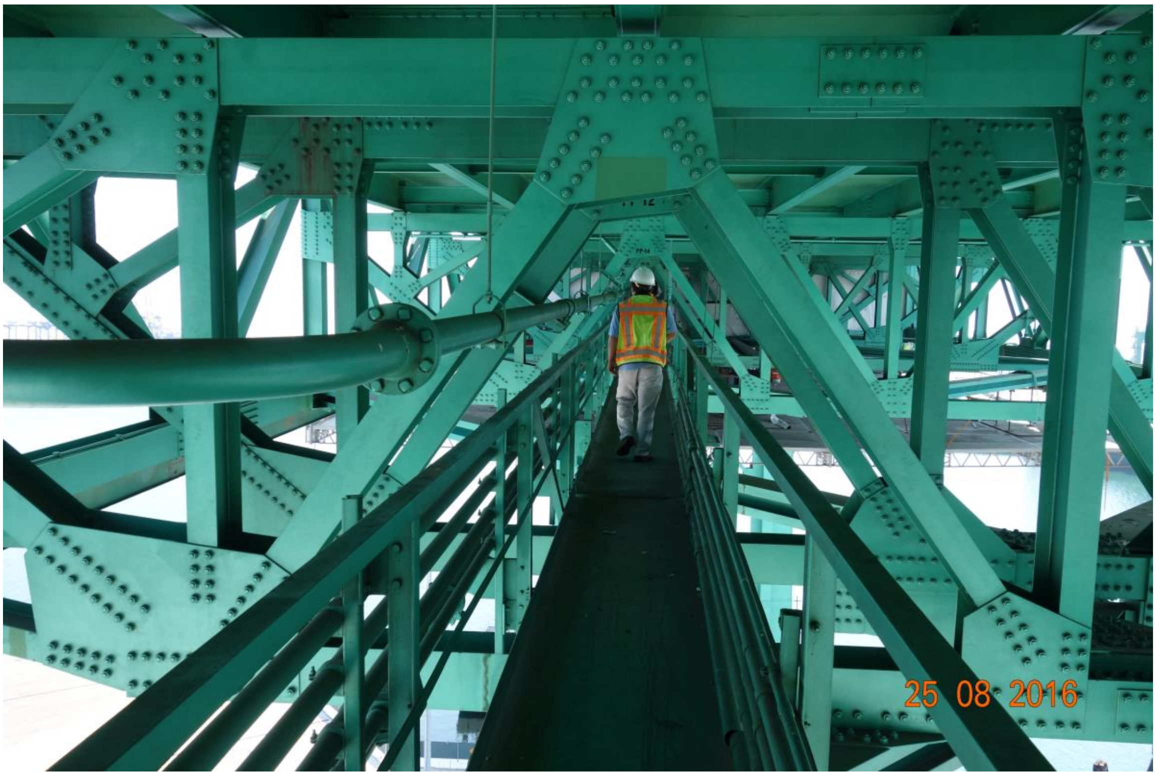
圖

四十 San Diego Coronado 橋梁檢測

圖四十一 San Diego Coronado 橋梁檢測平台及貓道

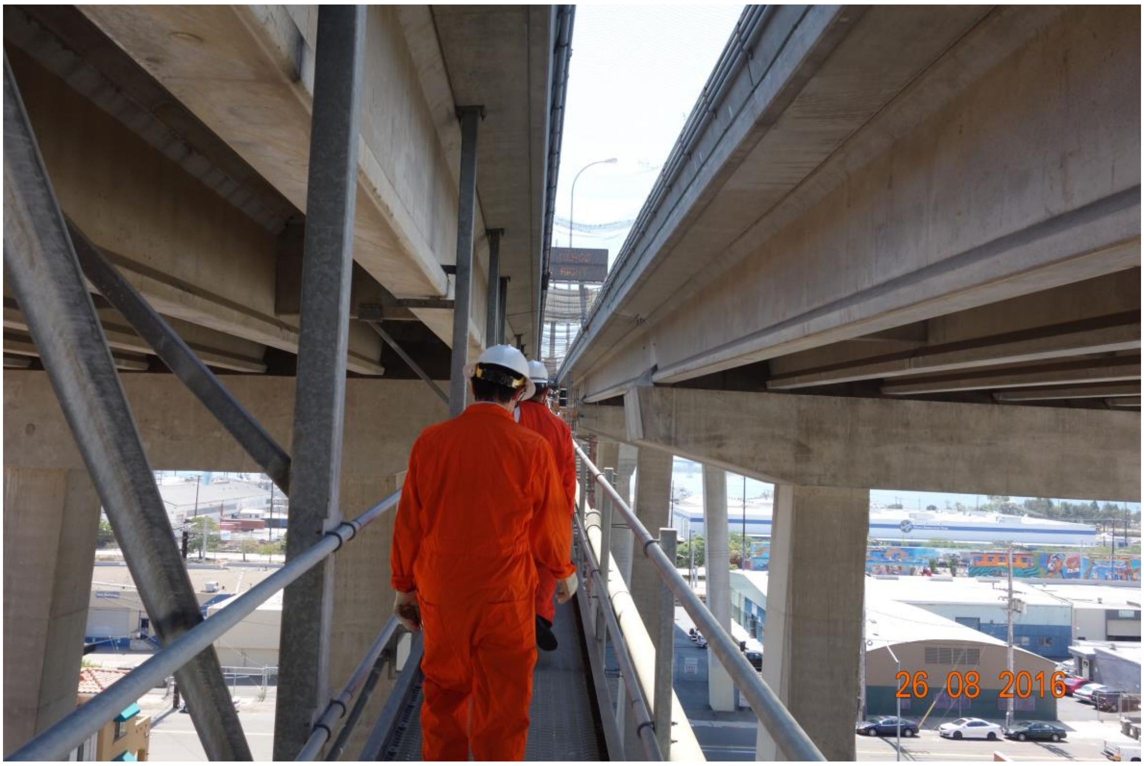
圖四十二 San Diego Coronado 橋梁檢測專用電梯

圖四十三 San Diego Coronado 橋梁檢測專用電梯



圖四十四 San Diego Coronado 橋梁檢測貓道

圖四十五 San Diego Coronado 橋梁維修用臨時吊裝平台



圖四十六 Los Angeles Harbor Vicenti Thomas Bridge橋梁檢測專用步道

圖四十七 Los Angeles Harbor Vicenti Thomas Bridge橋梁檢測專用步道

圖四十八 Los Angeles Harbor Vicent Thomas Bridge箱梁內檢測步道

圖四十九 Los Angeles Harbor Vicenti Thomas Bridge耐震補強阻尼器

圖五十 Los Angeles Harbor Vicenti Thomas橋梁維修用臨時吊裝平台

圖五十一 10號高速公路拓寬工程工地觀摩

圖五十二 10號高速公路拓寬工程工地觀摩

圖五十四 10號高速公路拓寬工程邊坡擋土牆裝設土釘

圖五十五 10號高速公路拓寬工程邊坡擋土牆裝排水用地工織物

圖五十六 排水用地工織物

圖五十七 10號高速公路拓寬工程基樁超音波檢測

圖五十八 10號高速公路拓寬工程進橋板下方裝設濾水管

圖五十九　橋梁耐震補強樁帽擴大增加樁並使用環氧樹酯防蝕鋼筋

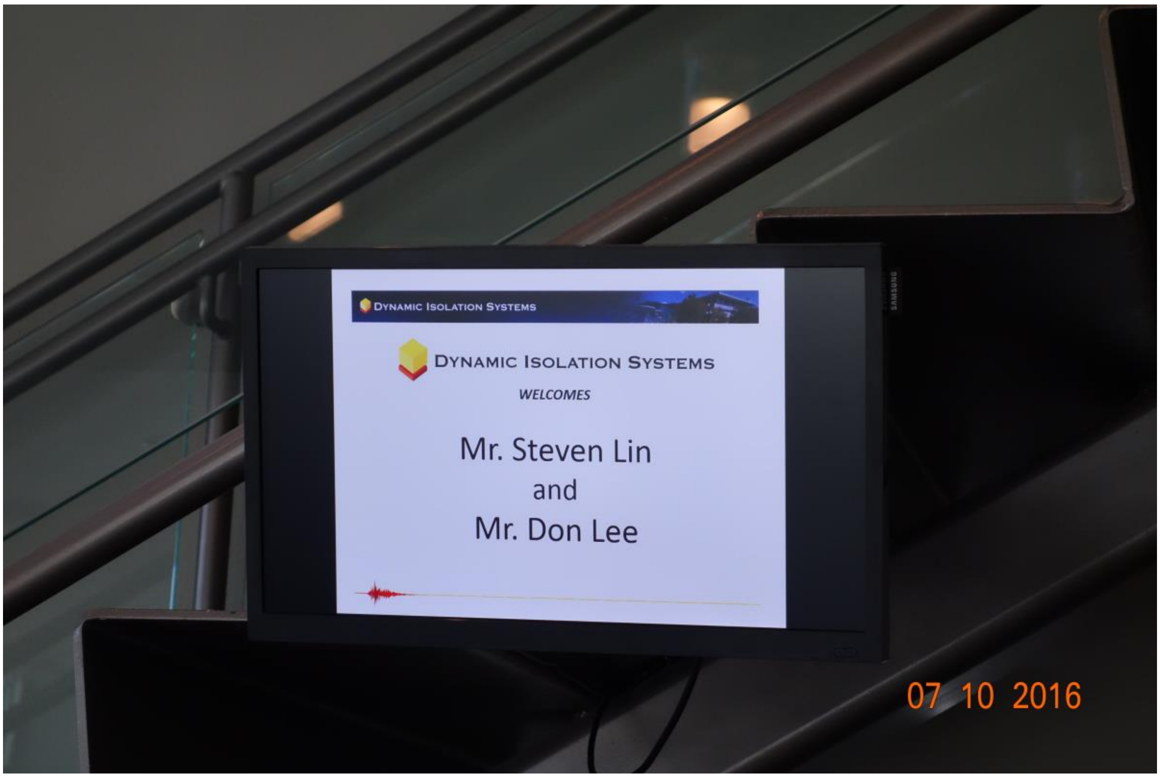
圖六十　支撐先進工法橋梁施工

圖六十一 觀摩加州地震諮詢委員會開會情形

圖六十二 向Dr. Mark Mahan請教加州橋梁耐震補強策略

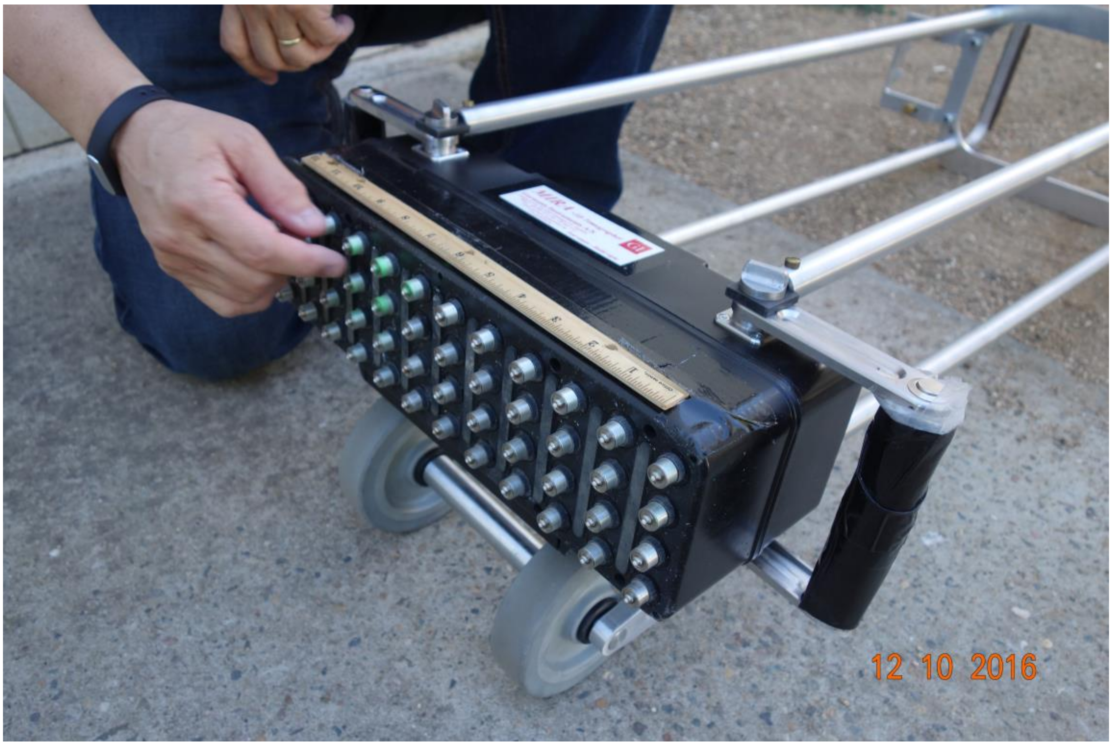
圖六十三 向Dr. Steve Mitchell請教加州橋梁耐震評估

圖六十四 參訪隔震支承工廠



圖六十五 參訪隔震支承工廠

圖六十六 與隔震支承研究人員共同研討



圖六十七 矩陣透地雷達探測器

圖六十八 矩陣透地雷達探測器

圖六十九 矩陣透地雷達探測器


CERTIFICATE OF RECOGNITION

The California Department of Transportation commends the

Engineering Service Center

for outstanding performance in meeting major challenges during the 1995-96 fiscal year. Especially notable is the uncompromising commitment and dedication of the staff. Exceptional work is recognized in the following areas:

- *Seismic Retrofit, Phase 2, Structure Design completion*
- *1995 Allocation Plan delivery assistance*
- *Significant progress executed in the Toll Seismic Retrofit Program*
- *Highest number of contract allotment ever awarded, $1.9 billion*

Dick Weaver, Deputy Director
Project Development

Caltrans

James W. van Loben Sels, Director
California Department of Transportation

01 11 2016

圖七十 參訪Caltrans工程服務中心

圖七十一 與Caltrans工程服務中心實驗室主任合影

圖七十二 參訪Caltrans工程服務中心實驗室

圖七十三 參訪Caltrans工程服務中心實驗室

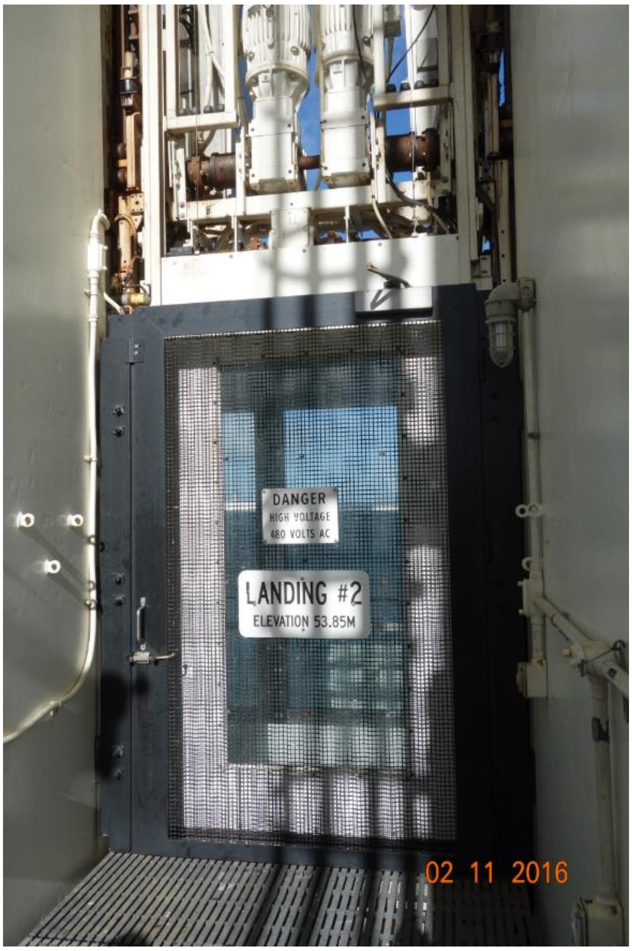
圖七十四 會同Caltrans工程師至工廠驗收鋼鉸索

圖七十五 會同Caltrans工程師至工廠驗收鋼鉸索

圖七十六 Okland新的海灣大橋參訪

圖七十七 Okland新的海灣大橋參訪

圖七十八 Okland新的海灣大橋參訪



圖七十九 Okland新的海灣大橋橋塔檢測電梯

圖八十 於Okland新的海灣大橋橋塔上留影

參、心得

加州運輸署（簡稱Caltrans）計有員工約2萬人組織龐大。負責管轄加州總長8萬多公里之高速公路、公路與鐵路自辦測量、大地調查、地質專探、規劃、設計、監造、養護、檢測及維修、規範修訂、耐震補強等作業。分工精細，工程師可專心於本業累積豐富實務經驗，對於所負責維管之橋梁能更為熟悉，可確實掌握維管橋梁之耐震易損性與特性，於震後可迅速動員經高度訓練與專業經驗之工程師來投入評斷橋梁之安全性並進行必要之緊急處置。惟因組織規模龐大，人員管理、效率及資源分配與有效運用及管理等問題，也是值得進一步去研究探討的課題。在橋梁檢測方面，加州規定必須有註冊專業執照者，方可執行此任務，且需為專職，目前Caltrans具專業執照之橋檢人員約75名，需於2年內以兩人為1組檢測完2萬5千座橋梁，每人每年需完成之橋梁檢測約160座，並有專人負責抽查品管。

Caltrans對於新進的工程師採取於各個部門輪調的方式來進行人員培訓，使得工程師能有較全面的學習機會，如果將來作為專職的設計工程師，所設計的成果較能以施工者的角度去設想，施工性較佳，無形中可節省很多看不到的成本。此外，藉由對新工與養護的工程師採取輪調作為，使設計工程師能多加考慮完工後之使用及維管等需求，也有助於提升設計施工的品質，落實全生命週期之設計目標，值得各工程專責單位參考學習。

Caltrans對於設計審查工作，為了避免因人而異沒有一定的審查標準，制定了各種工程的審查項目列表Check List，由審查工程師逐一進行核對，可避免審查重點不一及重要審查項目遺漏，提高設計審查品質。

Caltrans各專業部門每個月定期召開專題經驗分享討論會議，討論規範各章節，由各工程師輪流主講發表、共同討論，由於Caltrans各單位分散在全加州，會議是以視訊方式由各分區代表參加，亦是有效提升工程師個專業能力之學習訓練管道。

Caltrans對於重要橋梁均裝設強震及相關監測設施，監測橋梁動力特性，進行分析評估，於大地震後可評估橋梁受損情況與安全性。

目前高速公路橋梁耐震補強計畫所採用耐震評估方法與補強工法，大致上均有跟上Caltrans的腳步，相關增設上部結構的變位束制及增加橋柱韌性及強度的工法，也大致相仿。惟加州由於高速公路路網完整，替代道路選擇多較具彈性，故除了少數海灣地區的收費橋梁，因橋梁規模較大複雜度高且替代道路繞到較遠，列為重要橋梁，大部分高速公路橋梁僅列為一般橋梁，其耐震補強標準只要求於大地震時橋梁不得崩塌或落橋之生命安全需求，但允許有明顯損壞及提供有限度的服務。對於既有橋梁的補強，Caltrans

允許基礎產生搖擺Rocking，對於樁基礎如果土壤可能產生液化的狀況下，允許基樁產生第1個塑鉸，故可減少下部結構基礎的補強，加速耐震補強計畫之完成並節省可觀耐震補強經費。

Caltrans對於新材料、新工法、專利性或獨家產品，可預先經由所屬實驗室或委託學術專業機構進行試驗認證合格，即可列入加州合格產品清單Qualified Products List內，相關單位亦會對清單內產品採取定時之查驗，工程師在設計時，經評估比較某項材料確實屬於經濟有效，如該項產品有列於加州合格產品清單內及可挑選使用，免除冗長之試驗及申請手續。而廠商為維持產品可持續列於清單內爭取龐大商機，亦會盡力維持產品之品質，以免品質不穩遭到剃除。

Caltrans對於橋梁拓寬工程新舊混凝土相接介面，一般只要打毛清理乾淨即可，無需於舊混凝土面塗刷環氧樹脂。閉合塊在新施作主梁支撐拆除60天後，所有橋面板以上之靜載重均加載上去才可澆築，可減少閉合塊相接介面處支開裂與變形。閉合塊厚度以30公分以上為佳。

Caltrans對於既有橋梁之拓寬於拓寬部分用最新規範來設計，舊橋部分則採用原有規範來檢核，對於達到年限橋梁，則進行安全評估，安排改建計畫。

舊金山-奧克蘭海灣大橋（San Francisco-Oakland Bay Bridge），當地多簡稱為海灣大橋（Bay Bridge），又譯為灣區大橋。興建時因人工問題整個鋼橋發包給中國上海的鋼構廠製造，據前往監造的美方人員表示，由於品管觀念不同，當地的品管人員只願意對被抽查到的施工缺陷進行改善，沒抽查到的地方，費了很大的溝通工夫，最後鋼構廠才願意進一步加強自主品管改善缺失。

Caltrans為延長橋梁結構使用壽命，加強鋼筋的防蝕已普遍使用環氧樹脂保護Epoxy Coating鋼筋，計分為ASTM 775藍色塗裝，塗佈後鋼筋可彎曲加工，及ASTM 934紫色塗裝，塗佈後鋼筋不可彎曲加工，但更為堅固耐久。

肆、建議事項

目前國內高速公路橋梁一般仍能維持在甚佳之狀況，惟隨著橋齡的增加及材料老裂化，不可避免終將面對橋梁補強延壽或改建等難題。為延長公路橋梁之使用壽命，確保基礎設施之安全、可靠及效率，應與學術或專業單位合作以利用光電等高科技於橋梁變位、頻率變化或預力鋼腱腐蝕檢測等相關研究，以藉由客觀及精確的量測，協助維管單位確實掌握橋梁之損壞狀況，及早進行預防性修復避免橋梁急劇劣化，儘量延長橋梁使用壽命。

國內近數十年來新建橋梁因考慮美觀及及大跨經之力學需求，大多數均採用預力混凝土或鋼箱型梁橋，且為顧及行車舒適性，一般均在橋面版上舖築AC，於橋樑檢測時不易或無法及早檢察出橋面版之劣化狀，建議應編列相關費用要求檢測工程師定期進入箱梁內進行檢測，以期能確實掌握橋面版之結構狀況，維護行車安全。

國內在政府組織精簡原則下，高公局因新的高速公路逐漸完工接管的路段越來越長，維管的橋梁數量也急遽增加，但相關技術人力卻不被允許配合增加，在橋梁檢測維修的人力嚴重不足的情況下，橋梁檢測維修工作目前已完全委外辦理。負責橋梁業務的工程司除辦理橋梁檢測維修採購及履約管理等業務外，尚須兼辦甚多其他業務的現象甚為普遍，實有增加人力的必要，使其可專職辦理橋檢業務，並可以有較多的時間實際執行部分橋梁檢測業務，作為品管抽檢來實質監督規範執行者，提升橋梁檢測品質及有利實務經驗之累積。

為有效減輕橋檢業務之工作之負荷，建議可對橋梁狀況進行評估分級，對於狀況較差之橋梁增加檢測之頻率，狀況較佳之橋梁則減少檢測頻率，也期望高公局與國工局兩局整併可早日完成，以補充養護技術人力之不足。

建議各項橋梁常用之各種構件如支承、伸縮縫、預力系統端錨，隔震設施及阻尼器等，能由權管上位工程單位負責進行試驗認證，列入合格產品清單，方便各工程單位設計採用，避免各單位標準不一，也可節省廠商及工程單位之成本，提升競爭力。

Caltrans對於橋梁支承，有逐漸增加採用隔震支承的趨勢，除了鉛心橡膠及摩擦單擺支承外，亦有推薦彈性高阻尼橡膠支承，可有效分散及吸收橋梁所承受地震力，減輕下部結構及基礎之負擔，可降低橋梁耐震補強或新建工程，於基礎施工及用地的費用。

伍、附錄

一、 ShakeCast：美國加州震後快速反應及決策輔助工具軟體

二、 加州橋梁結構策略方向

三、 Seismic Retrofit Guidelines For Bridges In California 加州橋梁耐震補強指針

四、 加州新材料評估流程

ShakeCast: A Tool for Post-Earthquake Decision-making and Response

Presentation for the Research Connection

Loren Turner
Caltrans
Division of Research & Innovation

David Wald
Kuo-Wan Lin
U.S. Geological Survey

Overview

- Project Background
- Development of ShakeCast
- Demonstration of Features

Motivation for the Project

- Following a major earthquake, the Department must rapidly assess the condition of its bridges.

- Timely response is critical to:
  - Ensure public safety
  - Aid routing of emergency vehicle traffic
  - Re-establish critical lifeline routes.

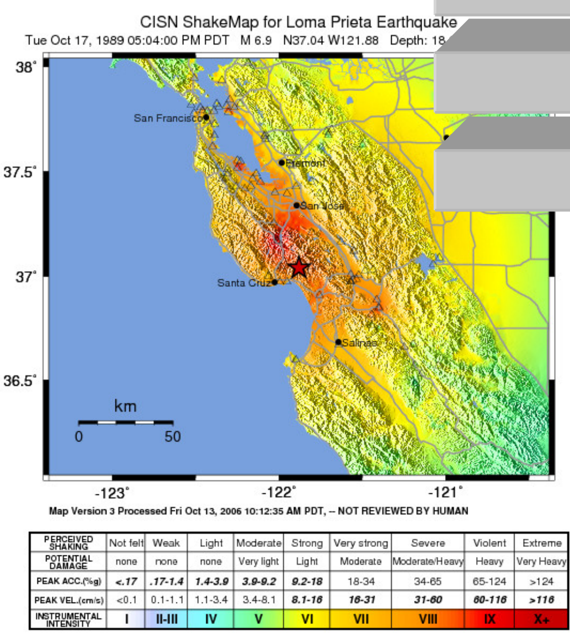
What is ShakeCast?

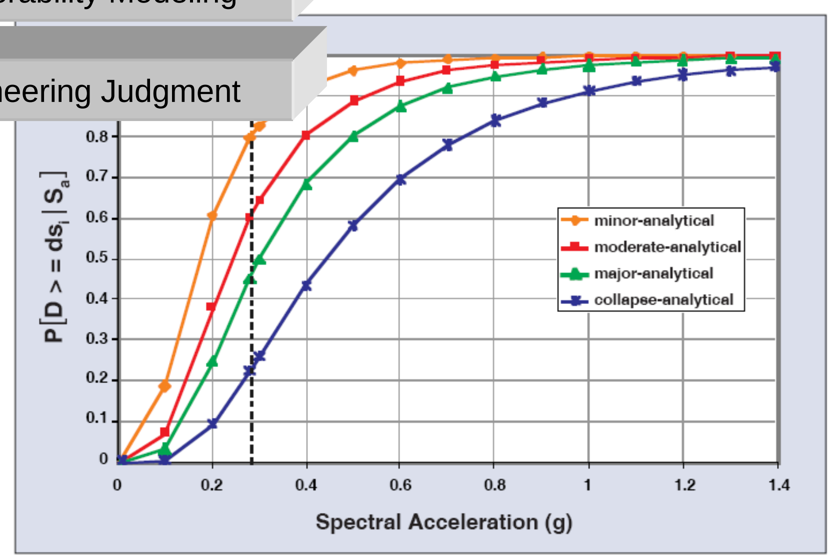
- ShakeCast
  - Software system that runs on a web server
  - Retrieves measured shaking data within minutes after an earthquake
  - Compares shaking distribution with unique bridge vulnerabilities
  - Provides hierarchical lists and maps of bridges most likely impacted
  - Emails info to responders
  - Provides suite of tools on website
- ShakeCast = *Shake*Map Broad*Cast*

Foundation for ShakeCast

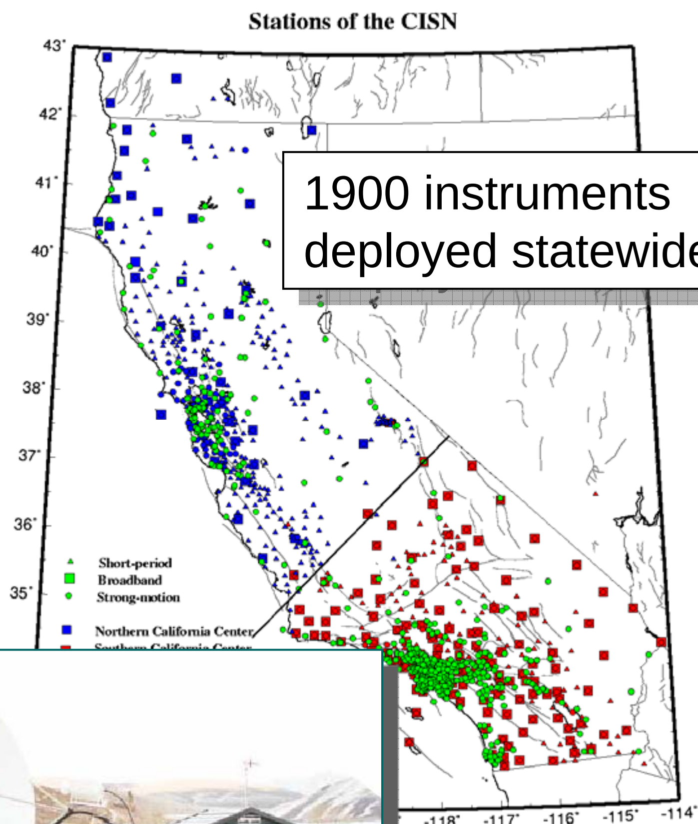
ShakeCast

- **ShakeMap**
  - Strong Motion Data
  - Faults
  - Geology
- **Bridge Fragility**
  - Caltrans Bridge Data
  - Vulnerability Modeling
  - Engineering Judgment

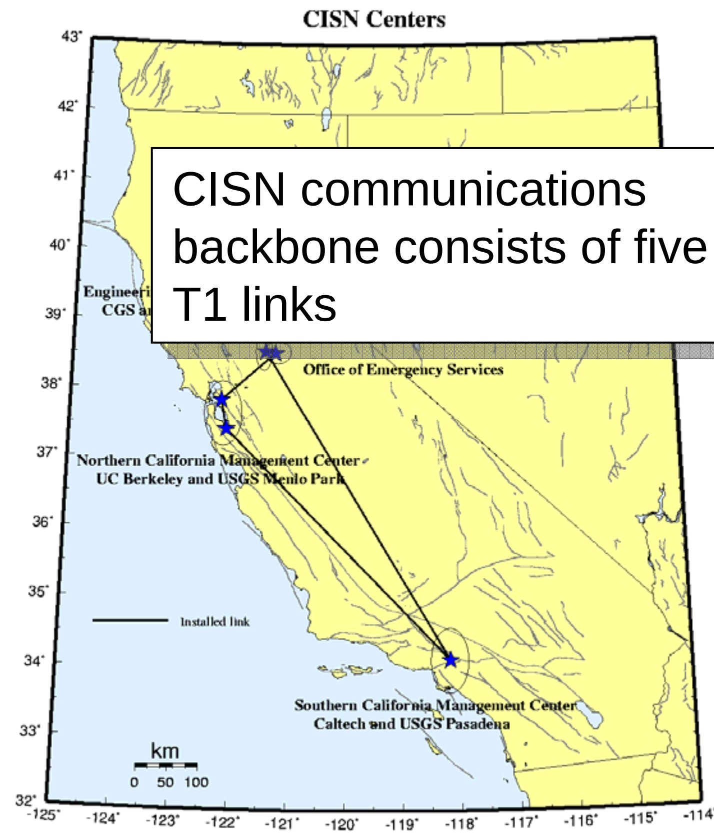




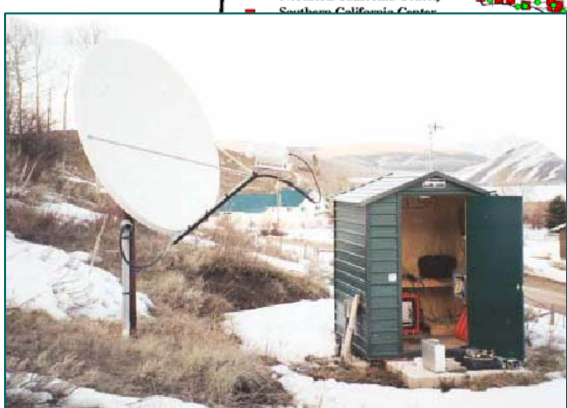
California Integrated Seismic Network (CISN)



1900 instruments deployed statewide



CISN communications backbone consists of five T1 links



CGS USGS Caltech OES UC Berkeley


ShakeMap: Northridge (Mag. 6.7)
GORMAN
PALMDALE
CASTAIC
OJAI
WRIGHTWOOD
VENTURA
NORTHRIDGE
PASADENA
SANTA CRUZ IS.
LOS ANGELES
SANTA MONICA
LONG BEACH
IRVINE
miles
0
10
20
30
SHAKING:
WEAK
STRONG
SEVERE

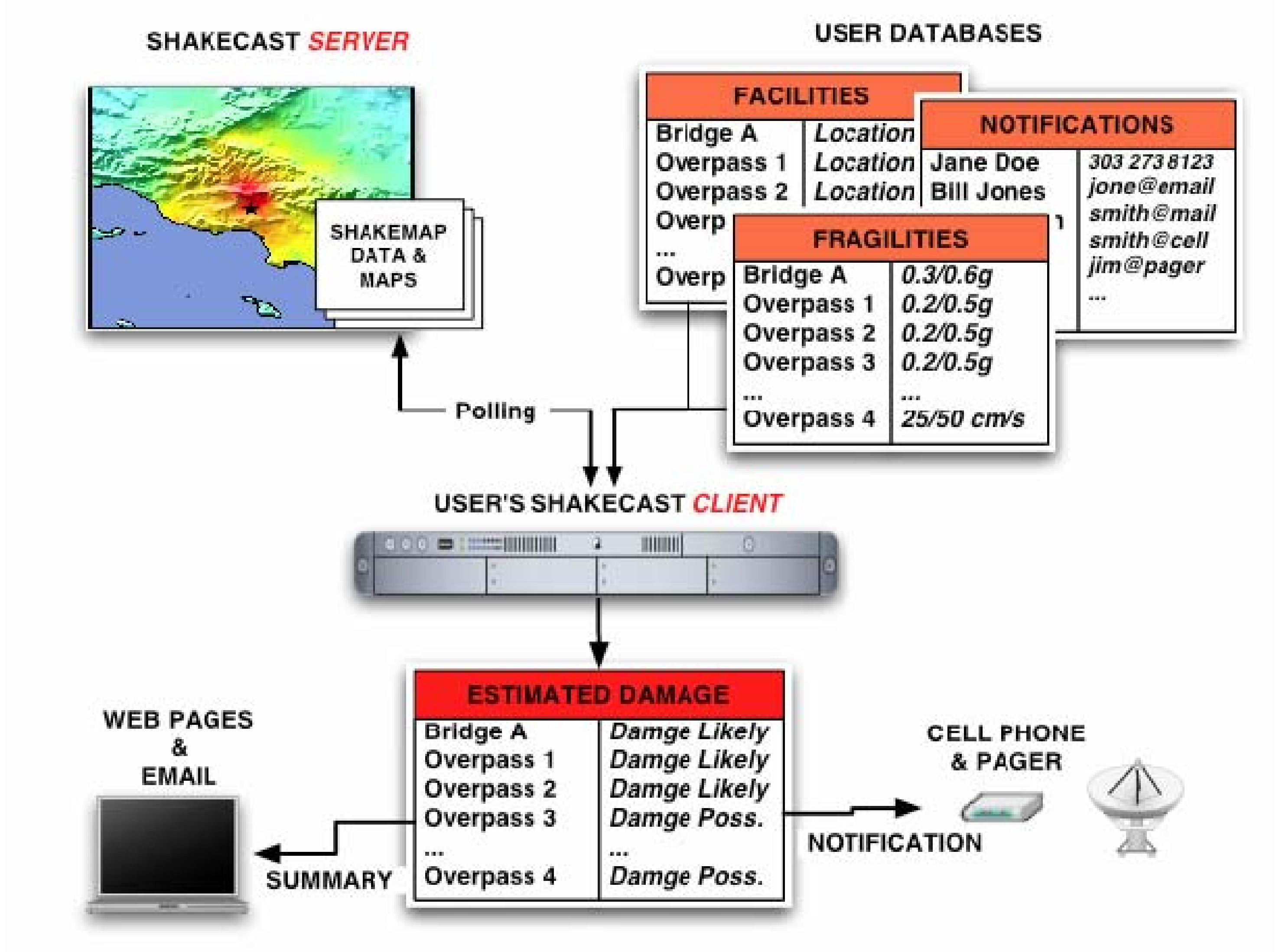
History

- 1996 – ShakeMap introduced
- 1999 – NT&R proposes “simple” method for assessing bridge impacts using ShakeMap and ArcView
- 2003 – ShakeCast Version 1 developed; Caltrans beta tester
- 2005 – Caltrans contracts with USGS
- 2008 – ShakeCast Version 2 released

Project Details

- Contract with the United States Geological Survey (USGS) commenced in 2006.
- Scope of Work – develop ShakeCast system that provides:
  - Automated earthquake and bridge performance analysis
  - Produce maps and bridge inspection priority lists.
  - Web-based interface to administer system and provide suite of products to users.

System Overview



ShakeCast Technologies

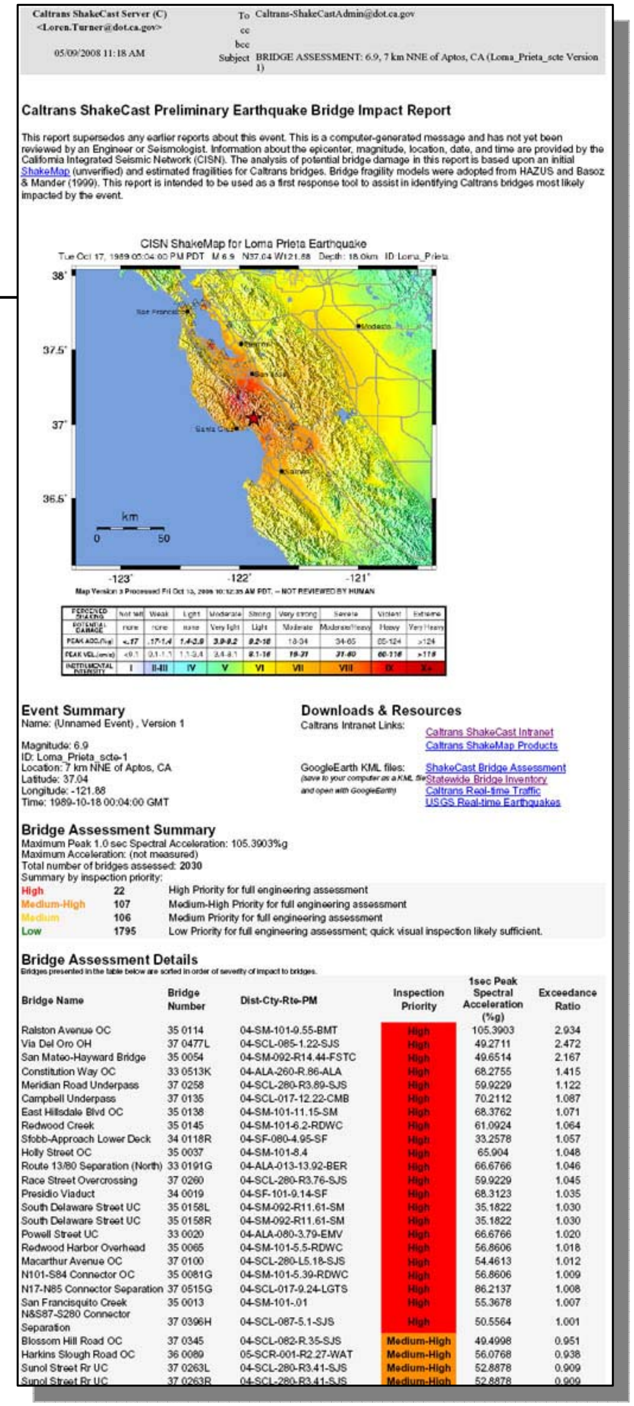
- WAMP
  - **W**indows (Server 2003)
  - **A**pache
  - **M**ySQL
  - **P**HP
- Perl
- Javascript
- RSS
- GoogleMaps
- GoogleEarth

ShakeCast Products

- Email notification
- GoogleEarth products
- ShakeCast Website

ShakeCast Email

- Automatic delivery of earthquake products to Caltrans.
- Automatic analysis of potential bridge damage state.
- Email bridge inspection prioritization lists.



Caltrans ShakeCast Server (C) <Loren.Turner@dot.ca.gov>		To	Caltrans-ShakeCastAdmin@dot.ca.gov
		cc	
		bcc	
05/09/2008 11:18 AM		Subject	BRIDGE ASSESSMENT: 6.9, 7 km NNE of Aptos, CA (Loma_Prieta_scte Version 1)

Caltrans ShakeCast Preliminary Earthquake Bridge Impact Report

This report supersedes any earlier reports about this event. This is a computer-generated message and has not yet been reviewed by an Engineer or Seismologist. Information about the epicenter, magnitude, location, date, and time are provided by the California Integrated Seismic Network (CISN). The analysis of potential bridge damage in this report is based upon an initial ShakeMap (unverified) and estimated fragilities for Caltrans bridges. Bridge fragility models were adopted from HAZUS and Basoz & Mander (1999). This report is intended to be used as a first response tool to assist in identifying Caltrans bridges most likely impacted by the event.

PERCEIVED SHAKING	Not felt	Weak	Light	Moderate	Strong	Very strong	Severe	Violent	Extreme
POTENTIAL DAMAGE	none	none	none	Very light	Light	Moderate	Moderate/Heavy	Heavy	Very Heavy
PEAK ACC.(%g)	<.17	.17-1.4	1.4-3.9	3.9-9.2	9.2-18	18-34	34-65	65-124	>124
PEAK VEL.(cm/s)	<0.1	0.1-1.1	1.1-3.4	3.4-8.1	8.1-16	16-31	31-60	60-116	>116
INSTRUMENTAL INTENSITY	I	II-III	IV	V	VI	VII	VIII	IX	X+

Event Summary

Name: (Unnamed Event) , Version 1

Magnitude: 6.9
ID: Loma_Prieta_scte-1
Location: 7 km NNE of Aptos, CA
Latitude: 37.04
Longitude: -121.88
Time: 1989-10-18 00:04:00 GMT

Downloads & Resources

Caltrans Intranet Links: Caltrans ShakeCast Intranet; Caltrans ShakeMap Products

GoogleEarth KML files: (save to your computer as a KML file and open with GoogleEarth) ShakeCast Bridge Assessment; Statewide Bridge Inventory; Caltrans Real-time Traffic; USGS Real-time Earthquakes

Bridge Assessment Summary

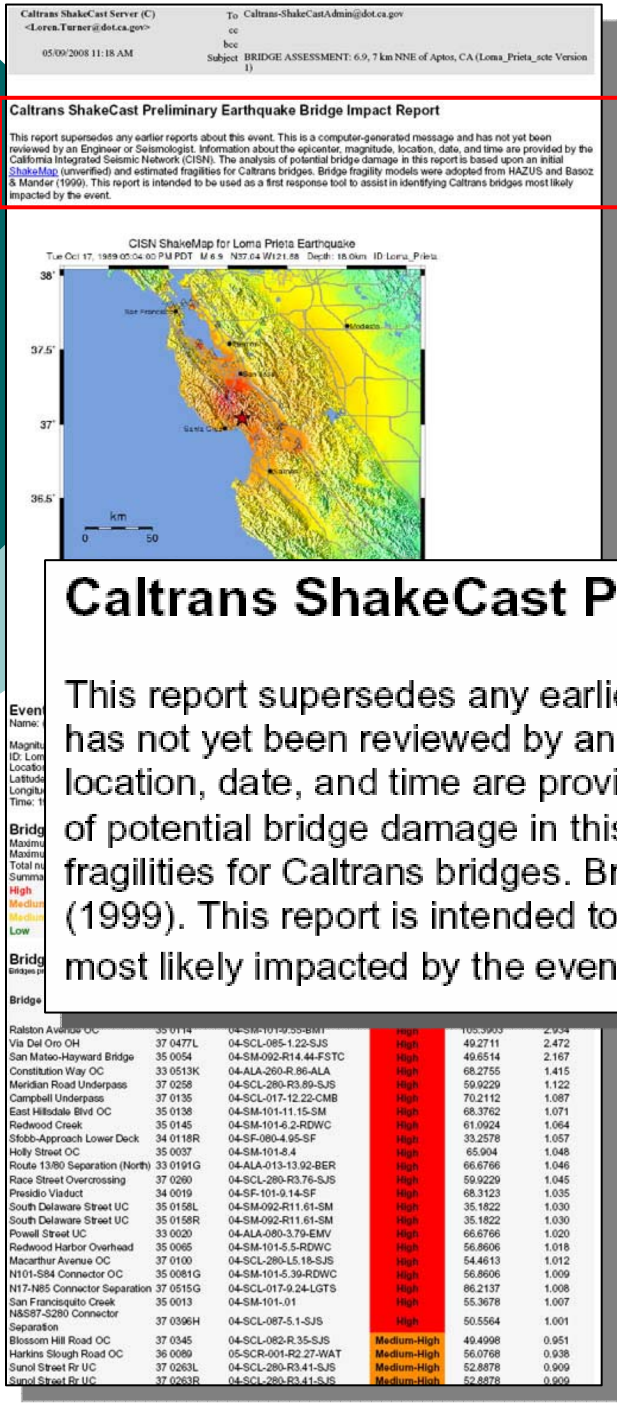
Maximum Peak 1.0 sec Spectral Acceleration: 105.3903%g
Maximum Acceleration: (not measured)
Total number of bridges assessed: **2030**
Summary by inspection priority:

High	22	High Priority for full engineering assessment
Medium-High	107	Medium-High Priority for full engineering assessment
Medium	106	Medium Priority for full engineering assessment
Low	1795	Low Priority for full engineering assessment; quick visual inspection likely sufficient.

Bridge Assessment Details

Bridges presented in the table below are sorted in order of severity of impact to bridges.

Bridge Name	Bridge Number	Dist-Cty-Rte-PM	Inspection Priority	1sec Peak Spectral Acceleration (%g)	Exceedance Ratio
Ralston Avenue OC	35 0114	04-SM-101-9.55-BMT	High	105.3903	2.934
Via Del Oro OH	37 0477L	04-SCL-085-1.22-SJS	High	49.2711	2.472
San Mateo-Hayward Bridge	35 0054	04-SM-092-R14.44-FSTC	High	49.6514	2.167
Constitution Way OC	33 0513K	04-ALA-260-R.86-ALA	High	68.2755	1.415
Meridian Road Underpass	37 0258	04-SCL-280-R3.89-SJS	High	59.9229	1.122
Campbell Underpass	37 0135	04-SCL-017-12.22-CMB	High	70.2112	1.087
East Hillsdale Blvd OC	35 0138	04-SM-101-11.15-SM	High	68.3762	1.071
Redwood Creek	35 0145	04-SM-101-6.2-RDWC	High	61.0924	1.064
Sfobb-Approach Lower Deck	34 0118R	04-SF-080-4.95-SF	High	33.2578	1.057
Holly Street OC	35 0037	04-SM-101-8.4	High	65.904	1.048
Route 13/80 Separation (North)	33 0191G	04-ALA-013-13.92-BER	High	66.6766	1.046
Race Street Overcrossing	37 0260	04-SCL-280-R3.76-SJS	High	59.9229	1.045
Presidio Viaduct	34 0019	04-SF-101-9.14-SF	High	68.3123	1.035
South Delaware Street UC	35 0158L	04-SM-092-R11.61-SM	High	35.1822	1.030
South Delaware Street UC	35 0158R	04-SM-092-R11.61-SM	High	35.1822	1.030
Powell Street UC	33 0020	04-ALA-080-3.79-EMV	High	66.6766	1.020
Redwood Harbor Overhead	35 0065	04-SM-101-5.5-RDWC	High	56.8606	1.018
Macarthur Avenue OC	37 0100	04-SCL-280-L5.18-SJS	High	54.4613	1.012
N101-S84 Connector OC	35 0081G	04-SM-101-5.39-RDWC	High	56.8606	1.009
N17-N85 Connector Separation	37 0515G	04-SCL-017-9.24-LGTS	High	86.2137	1.008
San Francisquito Creek	35 0013	04-SM-101-.01	High	55.3678	1.007
N&S87-S280 Connector Separation	37 0396H	04-SCL-087-5.1-SJS	High	50.5564	1.001
Blossom Hill Road OC	37 0345	04-SCL-082-R.35-SJS	Medium-High	49.4998	0.951
Harkins Slough Road OC	36 0089	05-SCR-001-R2.27-WAT	Medium-High	56.0768	0.938
Sunol Street Rr UC	37 0263L	04-SCL-280-R3.41-SJS	Medium-High	52.8878	0.909
Sunol Street Rr UC	37 0263R	04-SCL-280-R3.41-SJS	Medium-High	52.8878	0.909



Caltrans ShakeCast Server (C) <Loren.Turner@dot.ca.gov>
05/09/2008 11:18 AM
To Caltrans-ShakeCastAdmin@dot.ca.gov
cc
bcc
Subject BRIDGE ASSESSMENT: 6.9, 7 km NNE of Aptos, CA (Loma_Prieta_scte Version 1)

Caltrans ShakeCast Preliminary Earthquake Bridge Impact Report

This report supersedes any earlier reports about this event. This is a computer-generated message and has not yet been reviewed by an Engineer or Seismologist. Information about the epicenter, magnitude, location, date, and time are provided by the California Integrated Seismic Network (CISN). The analysis of potential bridge damage in this report is based upon an initial ShakeMap (unverified) and estimated fragilities for Caltrans bridges. Bridge fragility models were adopted from HAZUS and Basoz & Mander (1999). This report is intended to be used as a first response tool to assist in identifying Caltrans bridges most likely impacted by the event.

Ralston Av[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
Via Del Oro OH	37 0477L	04-SCL-085-1.22-SJS	High	49.2711	2.472
San Mateo-Hayward Bridge	35 0054	04-SM-092-R14.44-FSTC	High	49.6514	2.167
Constitution Way OC	33 0513K	04-ALA-260-R.86-ALA	High	68.2755	1.415
Meridian Road Underpass	37 0258	04-SCL-280-R3.89-SJS	High	59.9229	1.122
Campbell Underpass	37 0135	04-SCL-017-12.22-CMB	High	70.2112	1.087
East Hillsdale Blvd OC	35 0138	04-SM-101-11.15-SM	High	68.3762	1.071
Redwood Creek	35 0145	04-SM-101-6.2-RDWC	High	61.0924	1.064
Sfobb-Approach Lower Deck	34 0118R	04-SF-080-4.95-SF	High	33.2578	1.057
Holly Street OC	35 0037	04-SM-101-8.4	High	65.904	1.048
Route 13/80 Separation (North)	33 0191G	04-ALA-013-13.92-BER	High	66.6766	1.046
Race Street Overcrossing	37 0260	04-SCL-280-R3.76-SJS	High	59.9229	1.045
Presidio Viaduct	34 0019	04-SF-101-9.14-SF	High	68.3123	1.035
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Macarthur Avenue OC	37 0100	04-SCL-280-L5.18-SJS	High	54.4613	1.012
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N17-N85 Connector Separation	37 0515G	04-SCL-017-9.24-LGTS	High	86.2137	1.008
San Francisquito Creek	35 0013	04-SM-101-.01	High	55.3678	1.007
N&S87-S280 Connector Separation	37 0396H	04-SCL-087-5.1-SJS	High	50.5564	1.001
Blossom Hill Road OC	37 0345	04-SCL-082-R.35-SJS	Medium-High	49.4998	0.951
Harkins Slough Road OC	36 0089	05-SCR-001-R2.27-WAT	Medium-High	56.0768	0.938
Sunol Street Rr UC	37 0263L	04-SCL-280-R3.41-SJS	Medium-High	52.8878	0.909
Sunol Street Rr UC	37 0263R	04-SCL-280-R3.41-SJS	Medium-High	52.8878	0.909

Caltrans ShakeCast Preliminary Earthquake Bridge Impact Report

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Caltrans ShakeCast Server (C) <Loren.Turner@dot.ca.gov>
05/09/2008 11:18 AM

To Caltrans-ShakeCastAdmin@dot.ca.gov
cc
bcc
Subject BRIDGE ASSESSMENT: 6.9, 7 km NNE of Aptos, CA (Loma_Prieta_scte Version 1)

Caltrans ShakeCast Preliminary Earthquake Bridge Impact Report

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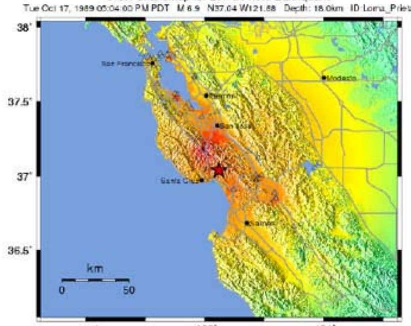
CISN ShakeMap for Loma Prieta Earthquake

Tue Oct 17, 1989 05:04:00 PM PDT M 6.9 N37.04 W121.88 Depth: 18.0km ID:Loma_Prieta



Map Version 3 Processed Fri Oct 13, 2006 10:12:35 AM PDT, -- NOT REVIEWED BY HUMAN

PERCEIVED SHAKING	Not felt	Weak	Light	Moderate	Strong	Very strong	Severe	Violent	Extreme
POTENTIAL DAMAGE	none	none	none	Very light	Light	Moderate	Moderate/Heavy	Heavy	Very Heavy
PEAK ACC.(%g)	<.17	.17-1.4	1.4-3.9	3.9-9.2	9.2-18	18-34	34-65	65-124	>124
PEAK VEL.(cm/s)	<0.1	0.1-1.1	1.1-3.4	3.4-8.1	8.1-16	16-31	31-60	60-116	>116
INSTRUMENTAL INTENSITY	I	II-III	IV	V	VI	VII	VIII	IX	X+

Event Summary

Name: (Unnamed Event) , Version 1

Magnitude: 6.9
ID: Loma_Prieta_scte-1
Location: 7 km NNE of Aptos, CA
Latitude: 37.04
Longitude: -121.88
Time: 1989-10-18 00:04:00 GMT

Downloads & Resources

Caltrans Intranet Links: Caltrans ShakeCast Intranet
Caltrans ShakeMap Products

GoogleEarth KML files: ShakeCast Bridge Assessment
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Caltrans Real-time Traffic
USGS Real-time Earthquakes

Bridge Assessment Summary

Maximum Peak 1.0 sec Spectral Acceleration: 105.3903%g
Maximum Acceleration: (not measured)
Total number of bridges assessed: **2030**
Summary by inspection priority:

High	22	High Priority for full engineering assessment
Medium-High	107	Medium-High Priority for full engineering assessment
Medium	106	Medium Priority for full engineering assessment
Low	1795	Low Priority for full engineering assessment; quick visual inspection likely sufficient.

Bridge Assessment Details

Bridges presented in the table below are sorted in order of severity of impact to bridges.

Bridge Name	Bridge Number	Dist-Cty-Rte-PM	Inspection Priority	1sec Peak Spectral Acceleration (%g)	Exceedance Ratio
Ralston Avenue OC	35 0114	04-SM-101-9.55-BMT	High	105.3903	2.934
Via Del Oro OH	37 0477L	04-SCL-085-1.22-SJS	High	49.2711	2.472
San Mateo-Hayward Bridge	35 0054	04-SM-092-R14.44-FSTC	High	49.6514	2.167
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Campbell Underpass	37 0135	04-SCL-017-12.22-CMB	High	70.2112	1.087
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Holly Street OC	35 0037	04-SM-101-8.4	High	65.904	1.048
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Race Street Overcrossing	37 0260	04-SCL-280-R3.76-SJS	High	59.9229	1.045
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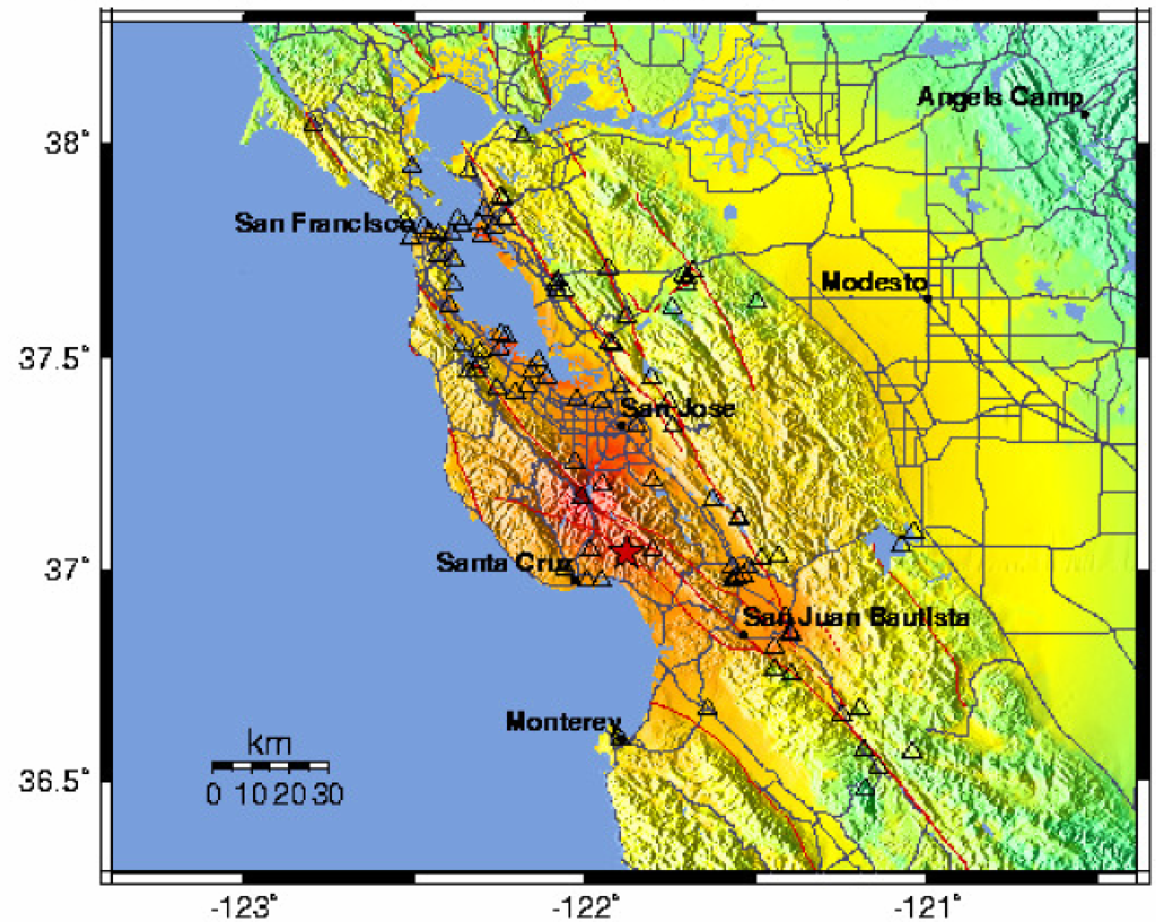
Sunol Street Rr UC	37 0263L	04-SCL-280-R3.41-SJS	Medium-High	52.8878	0.909
Sunol Street Rr UC	37 0263R	04-SCL-280-R3.41-SJS	Medium-High	52.8878	0.909

CISN Rapid Instrumental Intensity Map for Loma Prieta Earthquake

Tue Oct 17, 1989 05:04:00 PM PDT M 6.9 N37.04 W121.88 Depth: 18.0km ID:Loma_Prieta



PROCESSED: Thu Nov 6, 2003 05:53:05 AM PST,

PERCEIVED SHAKING	Not felt	Weak	Light	Moderate	Strong	Very strong	Severe	Violent	Extreme
POTENTIAL DAMAGE	none	none	none	Very light	Light	Moderate	Moderate/Heavy	Heavy	Very Heavy
PEAK ACC.(%g)	<.17	.17-1.4	1.4-3.9	3.9-9.2	9.2-18	18-34	34-65	65-124	>124
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05/09/2008 11:18 AM

To Caltrans-ShakeCastAdmin@dot.ca.gov
cc
bcc
Subject BRIDGE ASSESSMENT: 6.9, 7 km NNE of Aptos, CA (Loma_Prieta_scte Version 1)

Caltrans ShakeCast Preliminary Earthquake Bridge Impact Report

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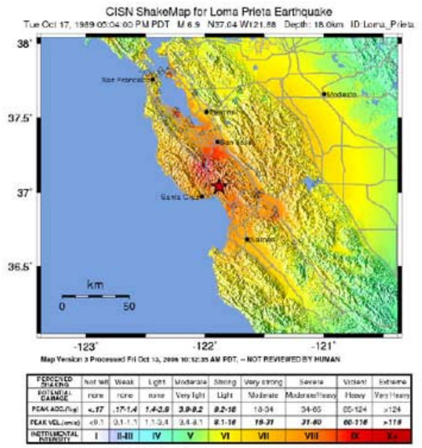

Event Summary
Name: (Unnamed Event) , Version 1

Magnitude: 6.9
ID: Loma_Prieta_scte-1
Location: 7 km NNE of Aptos, CA
Latitude: 37.04
Longitude: -121.88
Time: 1989-10-18 00:04:00 GMT

Downloads & Resources
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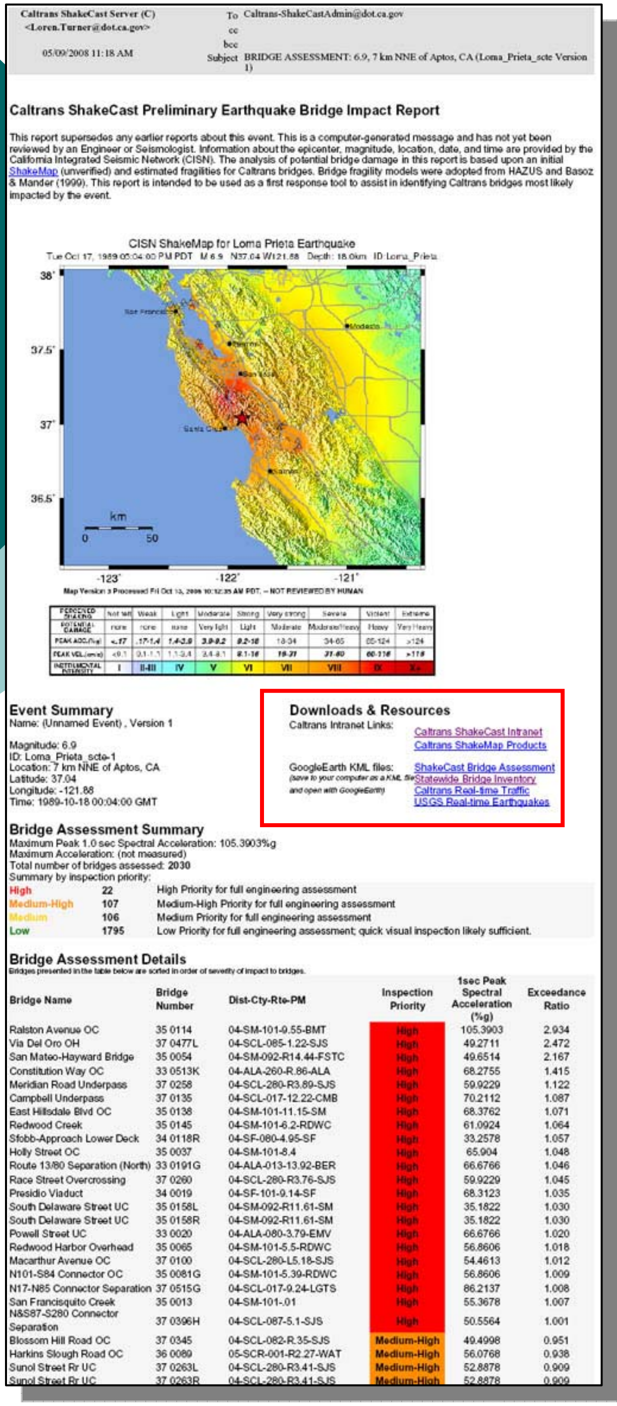
Presidio Viaduct	34 0019	04-SF-101-9.14-SF	High	68.3123	1.035
South Delaware Street UC	35 0158L	04-SM-092-R11.61-SM	High	35.1822	1.030
South Delaware Street UC	35 0158R	04-SM-092-R11.61-SM	High	35.1822	1.030
Powell Street UC	33 0020	04-ALA-080-3.79-EMV	High	66.6766	1.020
Redwood Harbor Overhead	35 0065	04-SM-101-5.5-RDWC	High	56.8606	1.018
Macarthur Avenue OC	37 0100	04-SCL-280-L5.18-SJS	High	54.4613	1.012
N101-S84 Connector OC	35 0081G	04-SM-101-5.39-RDWC	High	56.8606	1.009
N17-N85 Connector Separation	37 0515G	04-SCL-017-9.24-LGTS	High	86.2137	1.008
San Francisquito Creek	35 0013	04-SM-101-.01	High	55.3678	1.007
N&S87-S280 Connector Separation	37 0396H	04-SCL-087-5.1-SJS	High	50.5564	1.001
Blossom Hill Road OC	37 0345	04-SCL-082-R.35-SJS	Medium-High	49.4998	0.951
Harkins Slough Road OC	36 0089	05-SCR-001-R2.27-WAT	Medium-High	56.0768	0.938
Sunol Street Rr UC	37 0263L	04-SCL-280-R3.41-SJS	Medium-High	52.8878	0.909
Sunol Street Rr UC	37 0263R	04-SCL-280-R3.41-SJS	Medium-High	52.8878	0.909

Event Summary

Name: (Unnamed Event) , Version 1
Magnitude: 6.9
ID: Loma_Prieta_scte-1
Location: 7 km NNE of Aptos, CA
Latitude: 37.04
Longitude: -121.88
Time: 1989-10-18 00:04:00 GMT

Downloads & Resources

Caltrans Intranet Links:	Caltrans ShakeCast Intranet Caltrans ShakeMap Products
GoogleEarth KML files: *(save to your computer as a KML file and open with GoogleEarth)*	ShakeCast Bridge Assessment Statewide Bridge Inventory Caltrans Real-time Traffic USGS Real-time Earthquakes



Caltrans ShakeCast Server (C) <Loren.Turner@dot.ca.gov>

05/09/2008 11:18 AM

To Caltrans-ShakeCastAdmin@dot.ca.gov

cc

bcc

Subject BRIDGE ASSESSMENT: 6.9, 7 km NNE of Aptos, CA (Loma_Prieta_scte Version 1)

Caltrans ShakeCast Preliminary Earthquake Bridge Impact Report

This report supersedes any earlier reports about this event. This is a computer-generated message and has not yet been reviewed by an Engineer or Seismologist. Information about the epicenter, magnitude, location, date, and time are provided by the California Integrated Seismic Network (CISN). The analysis of potential bridge damage in this report is based upon an initial ShakeMap (unverified) and estimated fragilities for Caltrans bridges. Bridge fragility models were adopted from HAZUS and Basoz & Mander (1999). This report is intended to be used as a first response tool to assist in identifying Caltrans bridges most likely impacted by the event.

CISN ShakeMap for Loma Prieta Earthquake

Tue Oct 17, 1989 05:04:00 PM PDT M 6.9 N37.04 W121.88 Depth: 18.0km ID:Loma_Prieta

Map Version 3 Processed Fri Oct 13, 2006 10:12:35 AM PDT, -- NOT REVIEWED BY HUMAN

PERCEIVED SHAKING	Not felt	Weak	Light	Moderate	Strong	Very strong	Severe	Violent	Extreme
POTENTIAL DAMAGE	none	none	none	Very light	Light	Moderate	Moderate/Heavy	Heavy	Very Heavy
PEAK ACC.(%g)	<.17	.17-1.4	1.4-3.9	3.9-9.2	9.2-18	18-34	34-65	65-124	>124
PEAK VEL.(cm/s)	<0.1	0.1-1.1	1.1-3.4	3.4-8.1	8.1-16	16-31	31-60	60-116	>116
INSTRUMENTAL INTENSITY	I	II-III	IV	V	VI	VII	VIII	IX	X+

Event Summary

Name: (Unnamed Event) , Version 1

Magnitude: 6.9
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Bridge Assessment Summary

Maximum Peak 1.0 sec Spectral Acceleration: 105.3903%g
Maximum Acceleration: (not measured)
Total number of bridges assessed: **2030**
Summary by inspection priority:

High	22	High Priority for full engineering assessment
Medium-High	107	Medium-High Priority for full engineering assessment
Medium	106	Medium Priority for full engineering assessment
Low	1795	Low Priority for full engineering assessment; quick visual inspection likely sufficient.

Bridge Assessment Details

Bridges presented in the table below are sorted in order of severity of impact to bridges.

Bridge Name	Bridge Number	Dist-Cty-Rte-PM	Inspection Priority	1sec Peak Spectral Acceleration (%g)	Exceedance Ratio
Ralston Avenue OC	35 0114	04-SM-101-9.55-BMT	High	105.3903	2.934
Via Del Oro OH	37 0477L	04-SCL-085-1.22-SJS	High	49.2711	2.472
San Mateo-Hayward Bridge	35 0054	04-SM-092-R14.44-FSTC	High	49.6514	2.167
Constitution Way OC	33 0513K	04-ALA-260-R.86-ALA	High	68.2755	1.415
Meridian Road Underpass	37 0258	04-SCL-280-R3.89-SJS	High	59.9229	1.122
Campbell Underpass	37 0135	04-SCL-017-12.22-CMB	High	70.2112	1.087
East Hillsdale Blvd OC	35 0138	04-SM-101-11.15-SM	High	68.3762	1.071
Redwood Creek	35 0145	04-SM-101-6.2-RDWC	High	61.0924	1.064
Sfobb-Approach Lower Deck	34 0118R	04-SF-080-4.95-SF	High	33.2578	1.057
Holly Street OC	35 0037	04-SM-101-8.4	High	65.904	1.048
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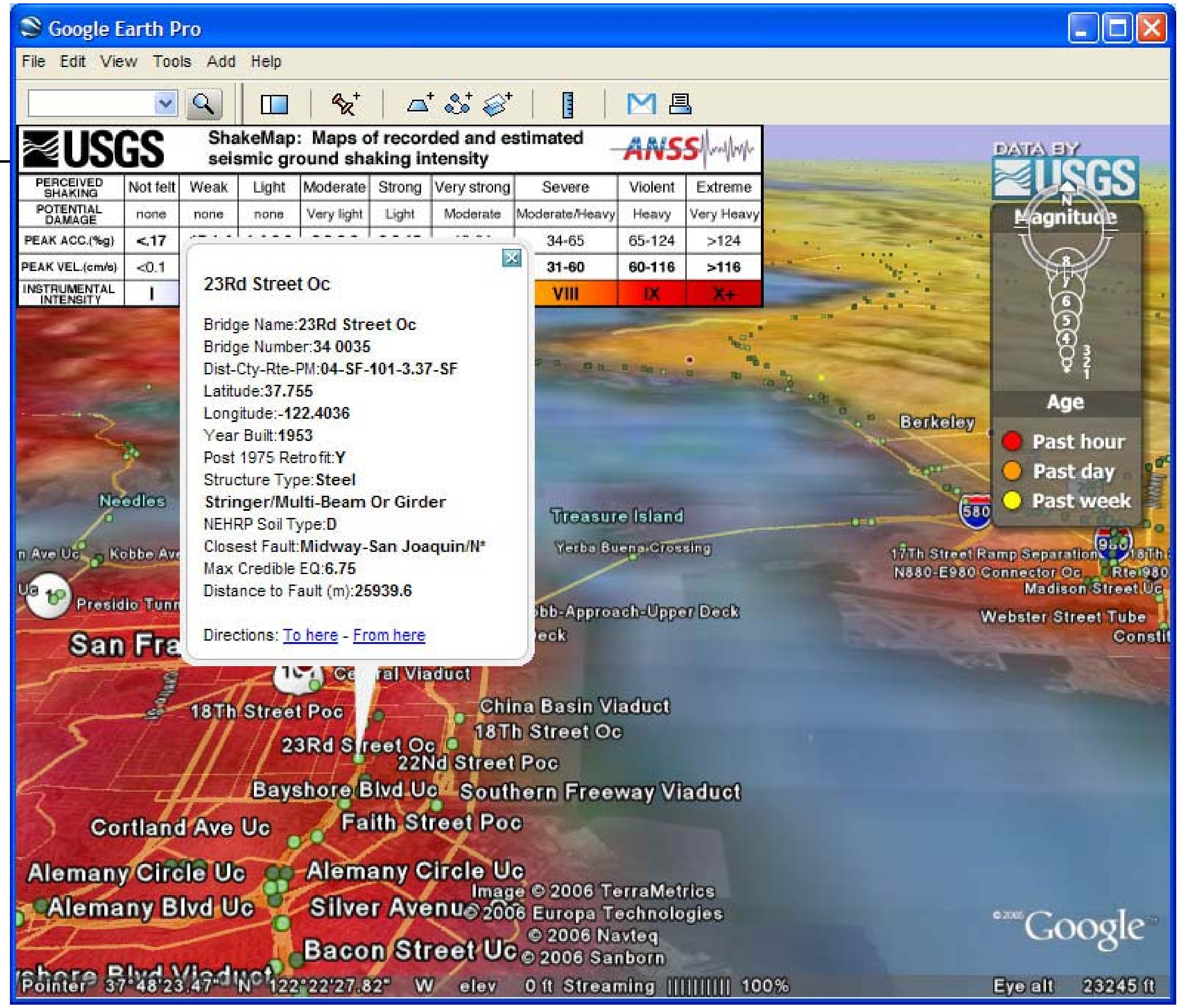
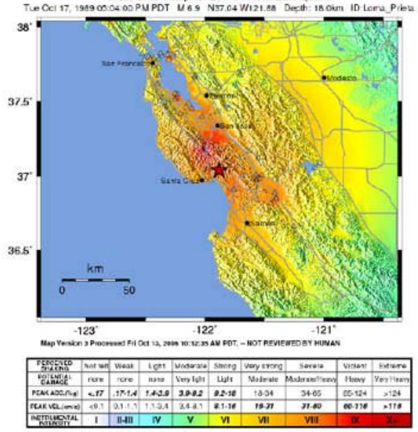


Caltrans ShakeCast Server (C) <Loren.Turner@dot.ca.gov>
05/09/2008 11:18 AM

To Caltrans-ShakeCastAdmin@dot.ca.gov
cc
bcc
Subject BRIDGE ASSESSMENT: 6.9, 7 km NNE of Aptos, CA (Loma_Prieta_scte Version 1)

Caltrans ShakeCast Preliminary Earthquake Bridge Impact Report

This report supersedes any earlier reports about this event. This is a computer-generated message and has not yet been reviewed by an Engineer or Seismologist. Information about the epicenter, magnitude, location, date, and time are provided b[y] California Integrated Seismic Network (CISN). The analysis of potential bridge damage in this report is based upon an initial ShakeMap (unverified) and estimated fragilities for Caltrans bridges. Bridge fragility models were adopted from HAZUS and B[...] & Mander (1999). This report is intended to be used as a first response tool to assist in identifying Caltrans bridges most likely impacted by the event.



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Maximum Peak 1.0 sec Spectral Acceleration: 105.3903%g
Maximum Acceleration: (not measured)
Total number of bridges assessed: **2030**
Summary by inspection priority:

High	22	High Priority for full engineering assessment
Medium-High	107	Medium-High Priority for full engineering assessment
Medium	106	Medium Priority for full engineering assessment
Low	1795	Low Priority for full engineering assessment; quick visual inspection likely sufficient.

Bridge Assessment Summary

Maximum Peak 1.0 sec Spectral Acceleration: 188.76%g
Maximum Acceleration: (not measured)
Total number of bridges assessed: **3133**
Summary by inspection priority:

High	**119**	High Priority for full engineering assessment
Medium-High	**156**	Medium-High Priority for full engineering assessment
Medium	**152**	Medium Priority for full engineering assessment
Low	**2706**	Low Priority for full engineering assessment; quick visual inspection likely sufficient.

Bridge Assessment Details

Bridges presented in the table below are sorted in order of severity of impact to bridges.

Bridge Name	Bridge Number	Dist-Cty-Rte-PM	Inspection Priority	1sec Peak Spectral Acceleration (%g)	Exceedance Ratio
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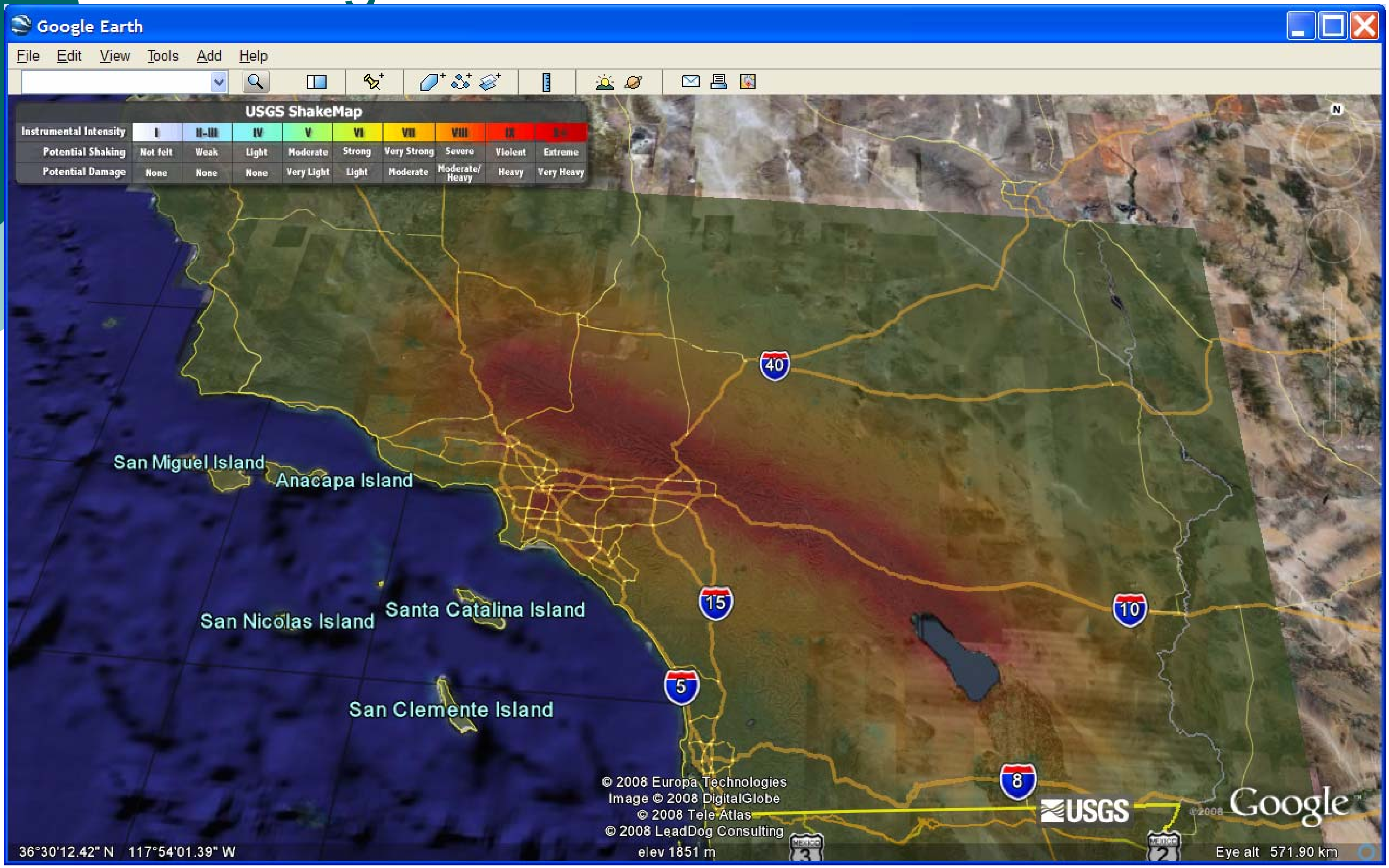
Harkins Slough Road OC	36 0089	05-SCR-001-R2.27-WAT	Medium-High	56.0768	0.938
Sunol Street Rr UC	37 0263L	04-SCL-280-R3.41-SJS	Medium-High	52.8878	0.909
Sunol Street Rr UC	37 0263R	04-SCL-280-R3.41-SJS	Medium-High	52.8878	0.909
Winchester Boulevard OC	37 0195	04-SCL-280-4.57-SJS	Medium-High	55.327	0.898
Lincoln Avenue UC	37 0262L	04-SCL-280-R3.51-SJS	Medium-High	52.8878	0.896
South Gilroy OH	37 0305L	04-SCL-101-R5.1	Medium-High	43.2728	0.896

GoogleEarth and ShakeCast



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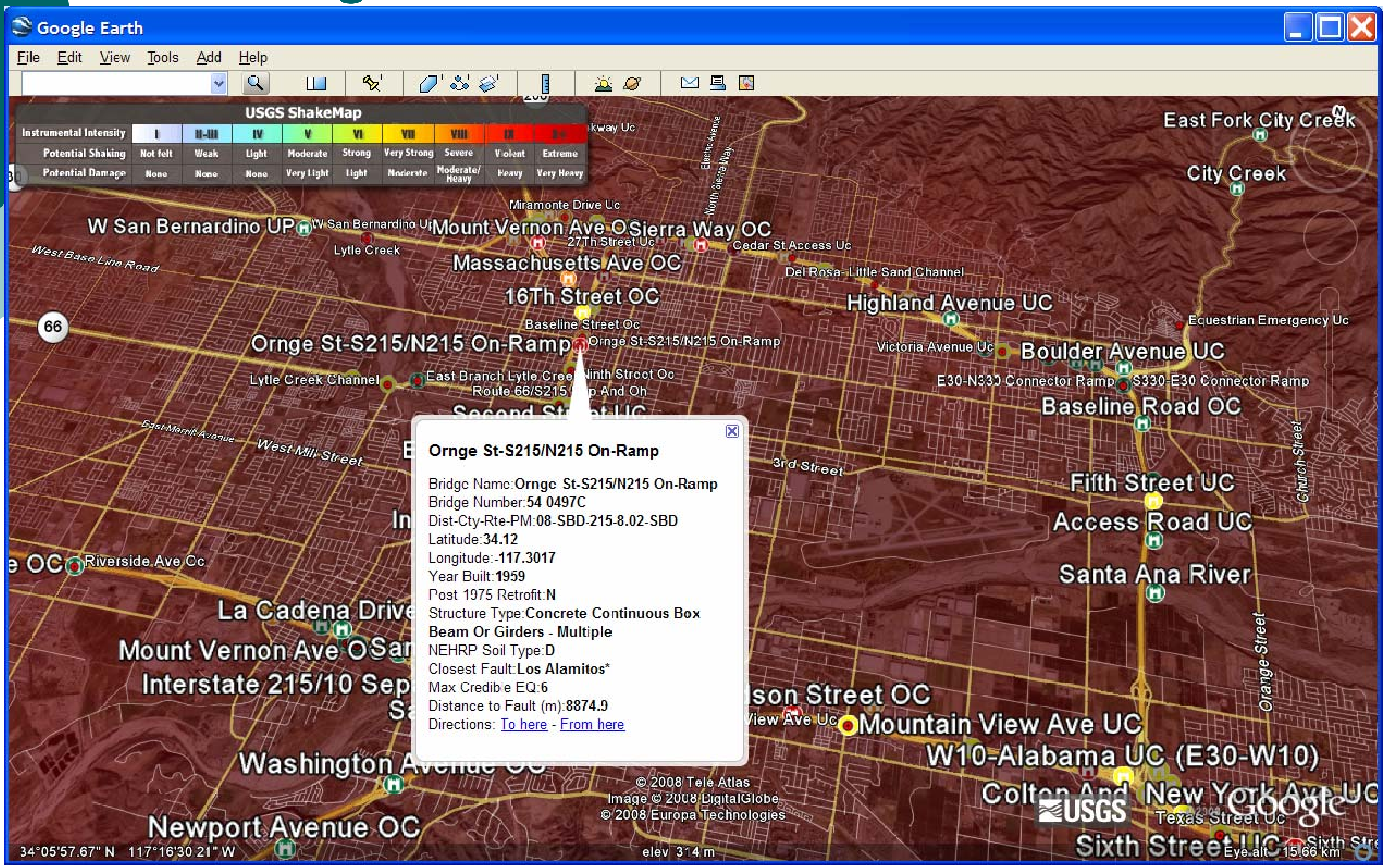

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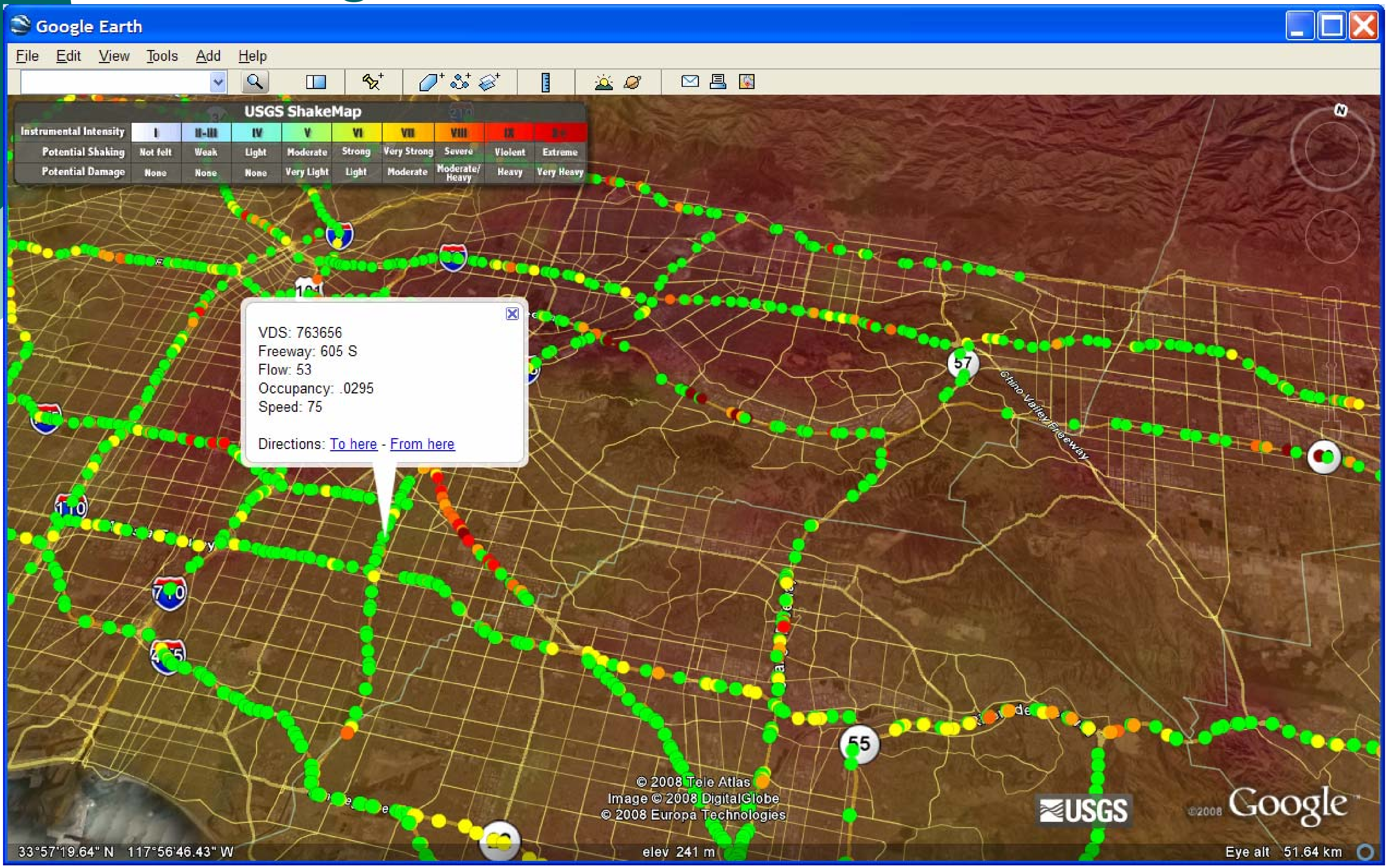
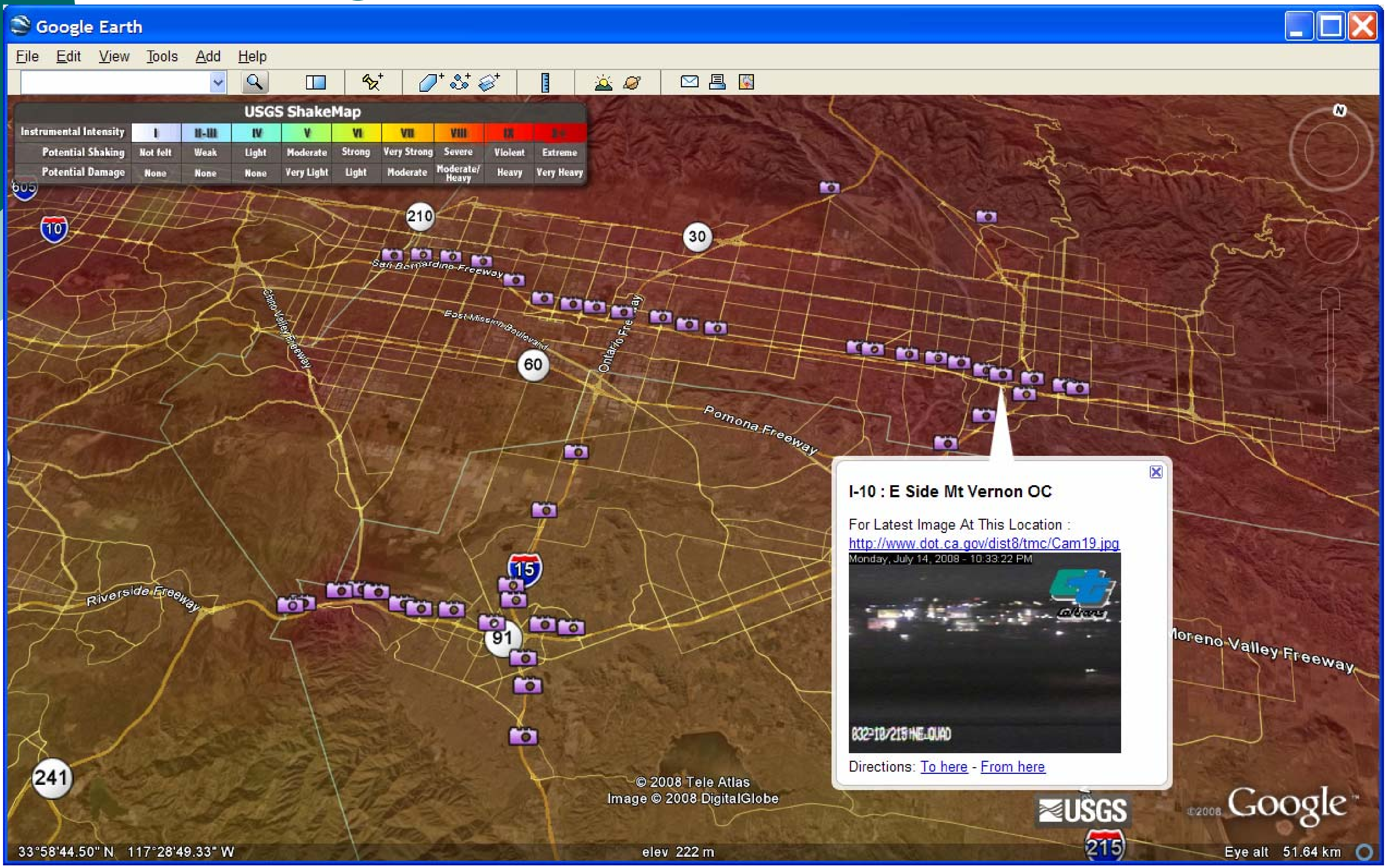
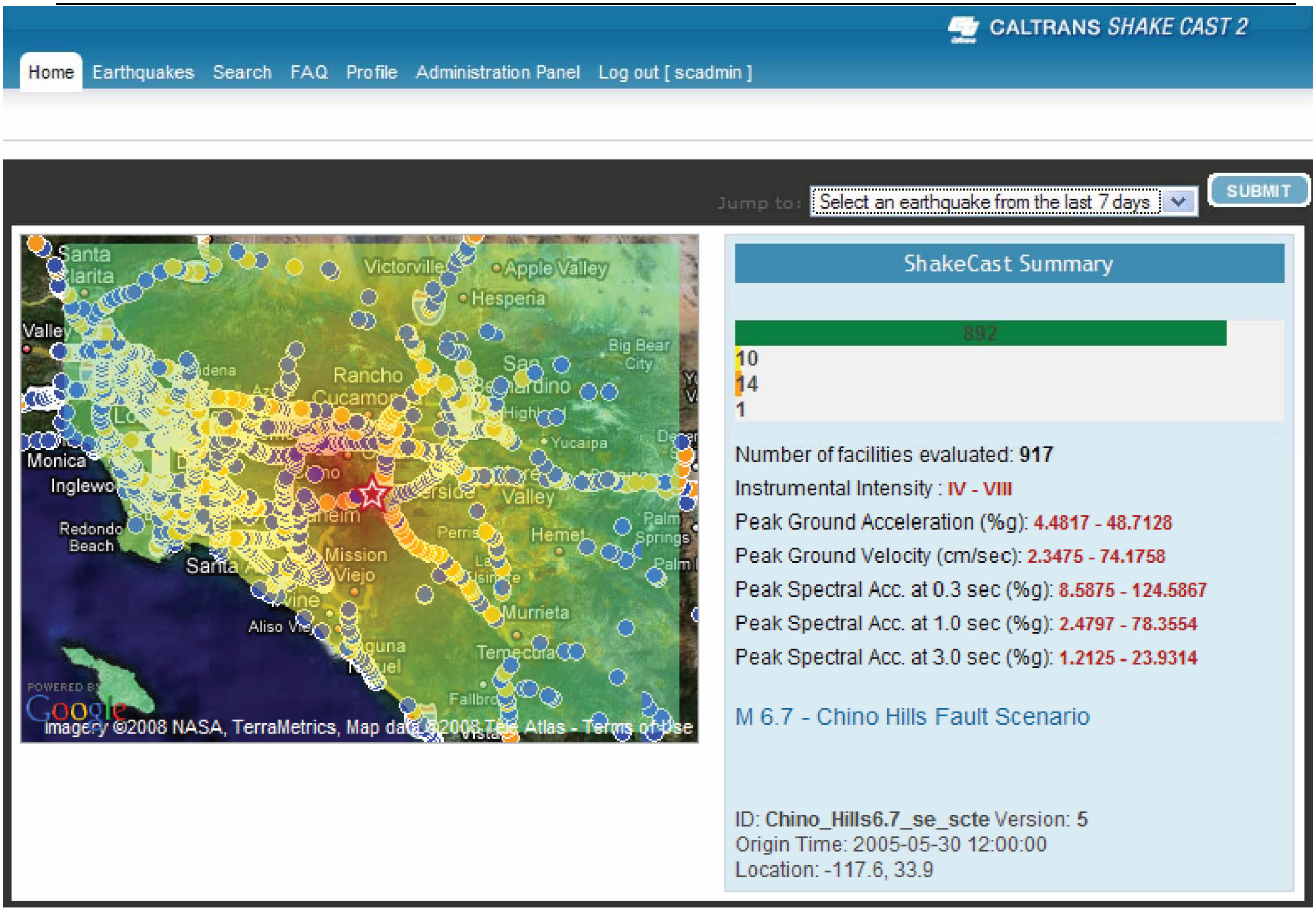


GoogleEarth and ShakeCast



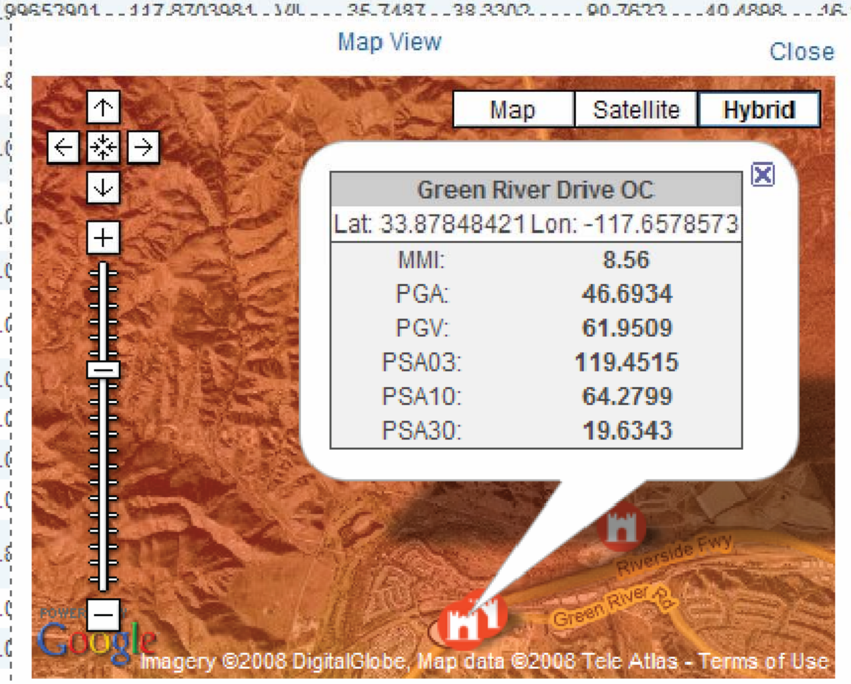
ShakeCast Website

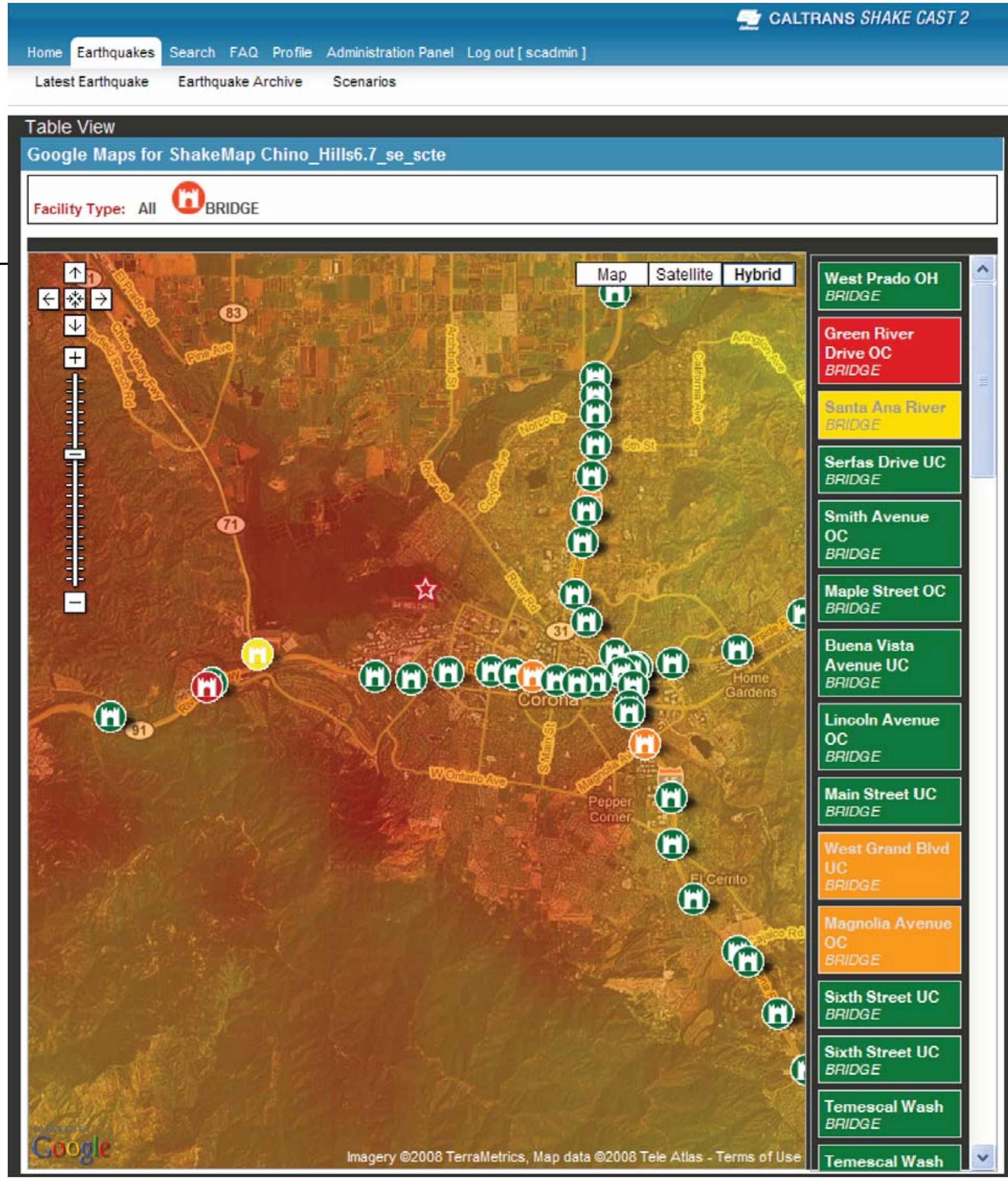


Map View

M 6.7 - Chino Hills Fault Scenario (ID: Chino_Hills6.7_se_scte - 5)

Facility ID	Type	Description	Inspection Priority	Latitude	Longitude	MMI	PGA (%g)	PGV (cm/sec)	PSA03 (%g)	PSA10 (%g)	PSA30 (%g)
56 0633	BRIDGE	Green River Drive OC	High	33.87848421	-117.6578573	VIII	46.6934	61.9509	119.4515	64.2799	19.6343
54 0748	BRIDGE	Benson Avenue OC	Medium-High	34.03032662	-117.6804218	VIII	37.8311	42.8441	96.2983	45.2159	16.1476
54 0747	BRIDGE	Central Avenue OC	Medium-High	34.03026777	-117.6891927	VIII	37.8311	42.8441	96.2983	45.2159	16.1476
53 1873G	BRIDGE	E60-N57 Connector OC	Medium-High	34.02202039	-117.8133506	VIII	39.693	47.723	101.3087	50.4097	17.9044
53 1788	BRIDGE	Fairway Drive UC	Medium-High	33.99652901	-117.8703981	VII	35.7487	38.3302	90.7633	40.4898	16.1639
56 0497	BRIDGE	Magnolia Avenue OC	Medium-High	33.8 [illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]711
54 0746	BRIDGE	Monte Vista Avenue OC	Medium-High	34.0 [illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]788
54 0744	BRIDGE	Pipeline Avenue OC	Medium-High	34.0 [illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]411
53 1873	BRIDGE	Prospectors UC	Medium-High	34.0 [illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]044
54 0745	BRIDGE	Ramona Avenue OC	Medium-High	34.0 [illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]788
53 1933	BRIDGE	Spadra OH	Medium-High	34.0 [illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]109
53 2106	BRIDGE	State Street OC	Medium-High	34.0 [illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]234
53 2078K	BRIDGE	Valley Blvd UC	Medium-High	34.0 [illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]962
53 2078	BRIDGE	Valley Blvd UC	Medium-High	34.0 [illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]962
56 0445	BRIDGE	West Grand Blvd UC	Medium-High	33.8 [illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]688
53 2081R	BRIDGE	Garey Ave UC	Medium	34.0 [illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]851
53 2081L	BRIDGE	Garey Ave UC	Medium	34.0 [illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]851
53 1022L	BRIDGE	Gibson OH (Eb&Wb Buswy)	Medium	34.07230404	-118.0327693	VI	16.3676	13.2602	42.7377	14.0073	6.6483

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Scenarios
Table View
Google Maps for ShakeMap Chino_Hills6.7_se_scte
Facility Type: All
BRIDGE
Map
Satellite
Hybrid
Corona
Home Gardens
Pepper Corner
El Cerrito
W Ontario Ave
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West Prado OH
BRIDGE
Green River Drive OC
BRIDGE
Santa Ana River
BRIDGE
Serfas Drive UC
BRIDGE
Smith Avenue OC
BRIDGE
Maple Street OC
BRIDGE
Buena Vista Avenue UC
BRIDGE
Lincoln Avenue OC
BRIDGE
Main Street UC
BRIDGE
West Grand Blvd UC
BRIDGE
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Sixth Street UC
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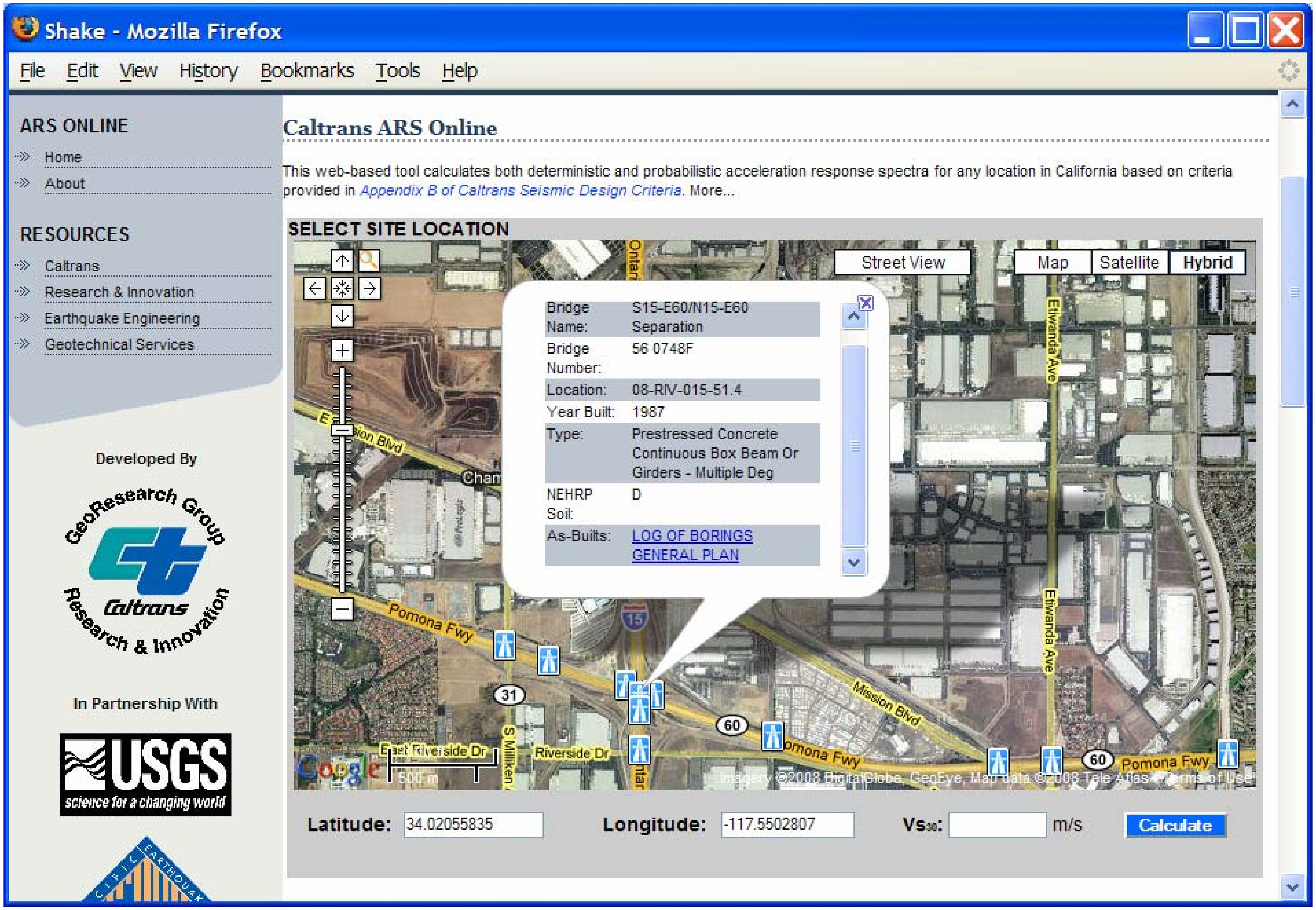
Temescal Wash
BRIDGE
Temescal Wash

Upcoming Features for Website



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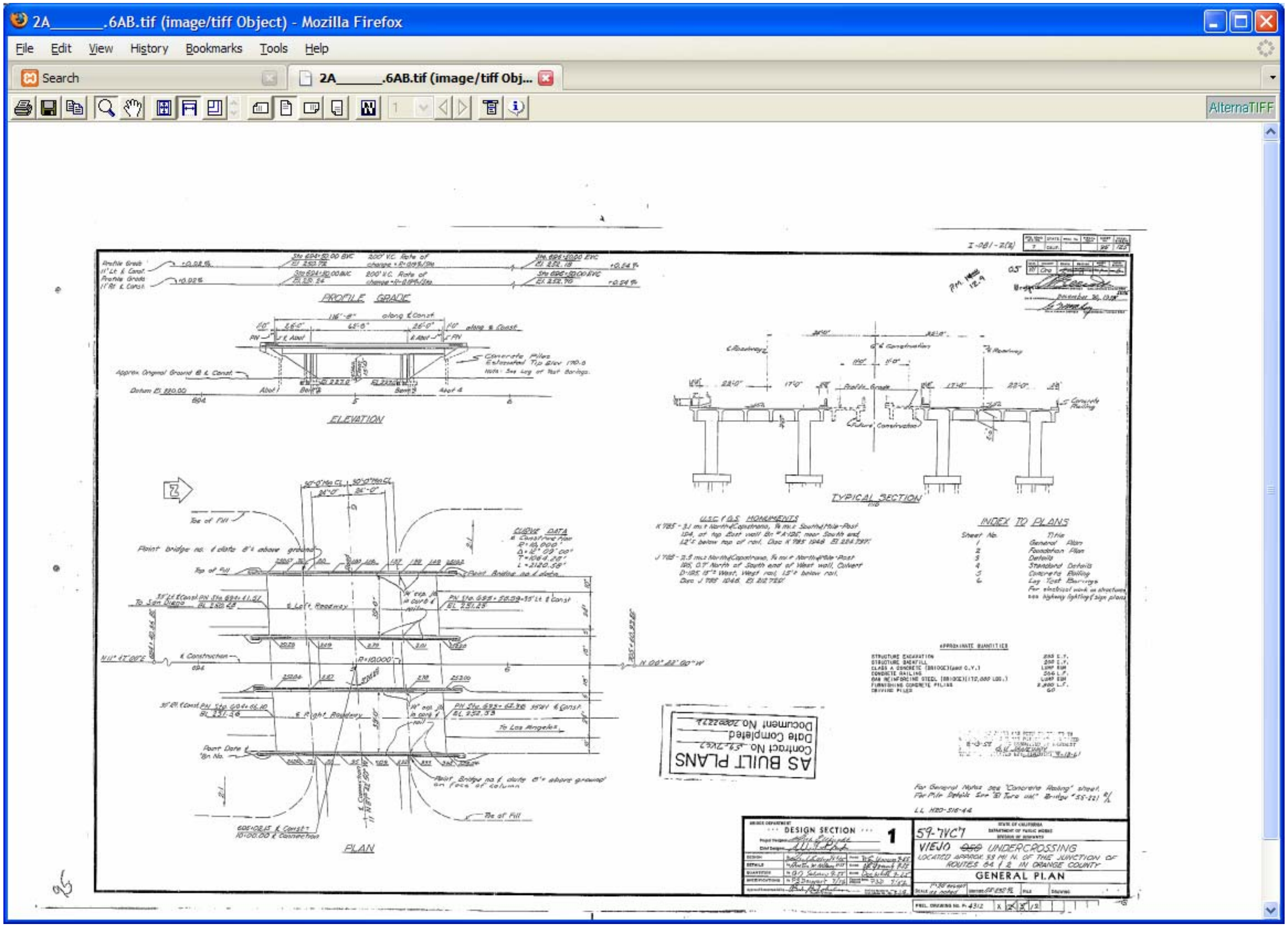

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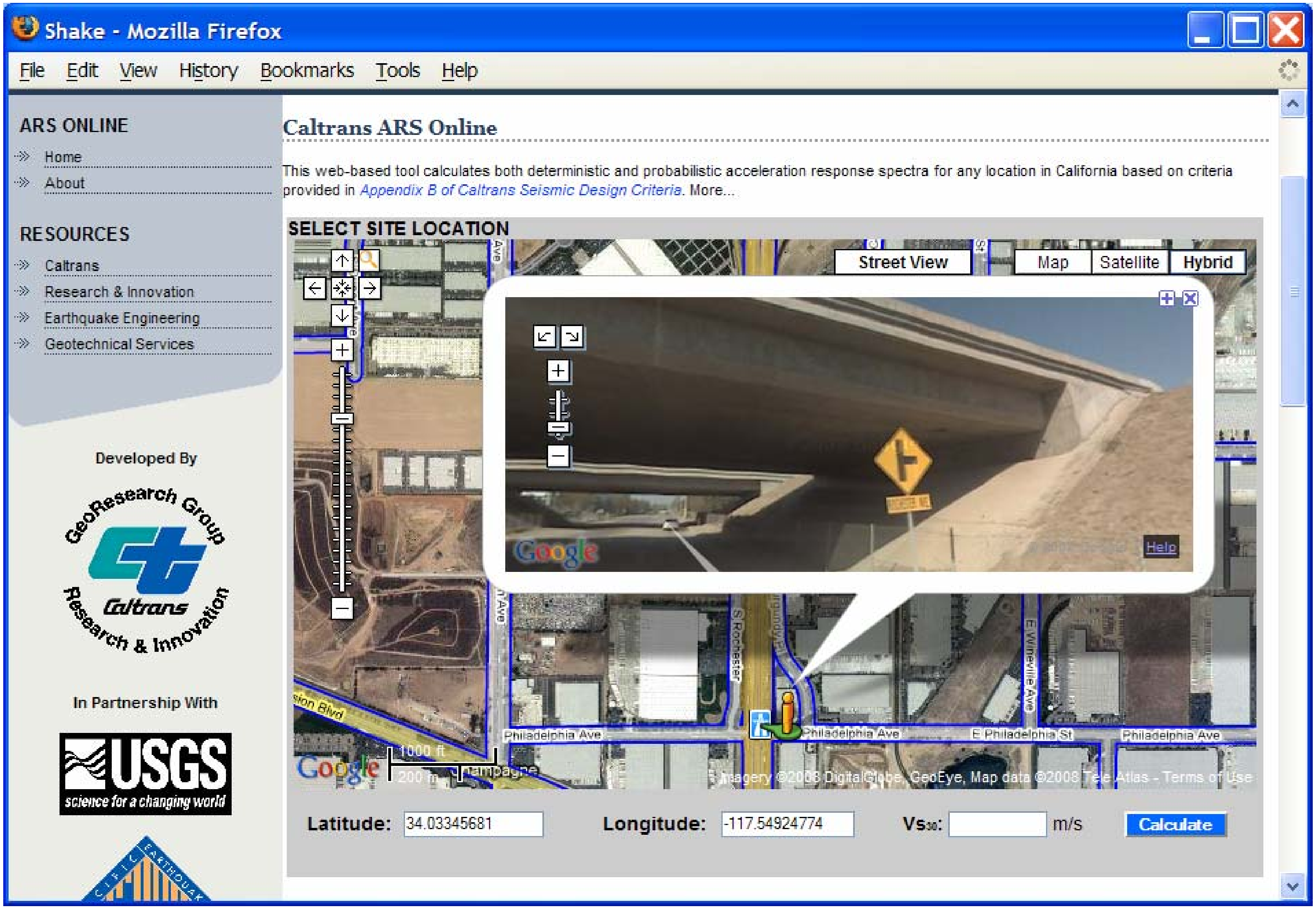


Summary

- Raises situational awareness after earthquake.
- Represents the most reliable information within the first hours following an event.
- Responders get information 10 to 15 minutes following an earthquake via email.

More Information

On the internet:

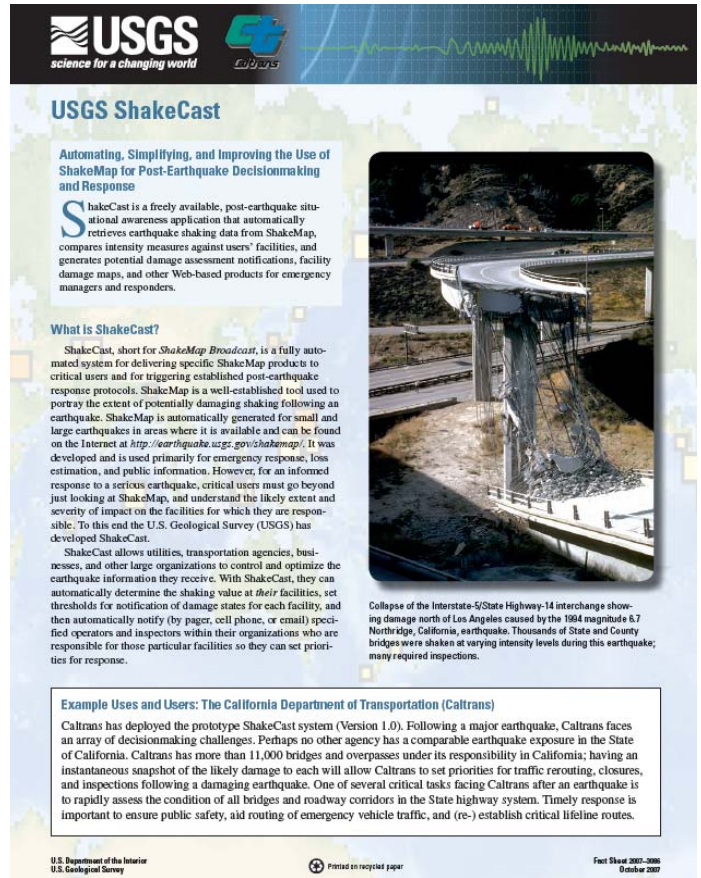
http://earthquake.usgs.gov/resources/software/shakecast/

In print:

Earthquake Spectra, May 2008, Volume 24, Issue 2

"ShakeCast: Automating and Improving the Use of ShakeMap for Post-Earthquake Decision-Making and Response"

EERI EARTHQUAKE SPECTRA



USGS science for a changing world

Caltrans

USGS ShakeCast

Automating, Simplifying, and Improving the Use of ShakeMap for Post-Earthquake Decisionmaking and Response

ShakeCast is a freely available, post-earthquake situational awareness application that automatically retrieves earthquake shaking data from ShakeMap, compares intensity measures against users' facilities, and generates potential damage assessment notifications, facility damage maps, and other Web-based products for emergency managers and responders.

What is ShakeCast?

ShakeCast, short for *ShakeMap Broadcast*, is a fully automated system for delivering specific ShakeMap products to critical users and for triggering established post-earthquake response protocols. ShakeMap is a well-established tool used to portray the extent of potentially damaging shaking following an earthquake. ShakeMap is automatically generated for small and large earthquakes in areas where it is available and can be found on the Internet at *http://earthquake.usgs.gov/shakemap/*. It was developed and is used primarily for emergency response, loss estimation, and public information. However, for an informed response to a serious earthquake, critical users must go beyond just looking at ShakeMap, and understand the likely extent and severity of impact on the facilities for which they are responsible. To this end the U.S. Geological Survey (USGS) has developed ShakeCast.

ShakeCast allows utilities, transportation agencies, businesses, and other large organizations to control and optimize the earthquake information they receive. With ShakeCast, they can automatically determine the shaking value at *their* facilities, set thresholds for notification of damage states for each facility, and then automatically notify (by pager, cell phone, or email) specified operators and inspectors within their organizations who are responsible for those particular facilities so they can set priorities for response.

Collapse of the Interstate-5/State Highway-14 interchange showing damage north of Los Angeles caused by the 1994 magnitude 6.7 Northridge, California, earthquake. Thousands of State and County bridges were shaken at varying intensity levels during this earthquake; many required inspections.

Example Uses and Users: The California Department of Transportation (Caltrans)

Caltrans has deployed the prototype ShakeCast system (Version 1.0). Following a major earthquake, Caltrans faces an array of decisionmaking challenges. Perhaps no other agency has a comparable earthquake exposure in the State of California. Caltrans has more than 11,000 bridges and overpasses under its responsibility in California; having an instantaneous snapshot of the likely damage to each will allow Caltrans to set priorities for traffic rerouting, closures, and inspections following a damaging earthquake. One of several critical tasks facing Caltrans after an earthquake is to rapidly assess the condition of all bridges and roadway corridors in the State highway system. Timely response is important to ensure public safety, aid routing of emergency vehicle traffic, and (re-) establish critical lifeline routes.

U.S. Department of the Interior
U.S. Geological Survey

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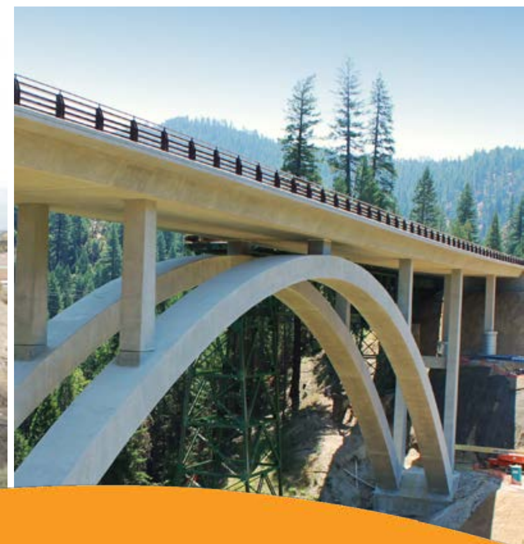
Fact Sheet 2007–3086
October 2007

California Bridges and Structures Strategic Direction

2016



Caltrans

ACKNOWLEDGEMENTS

This California Bridges and Structures Strategic Direction (hereafter referred to as "Strategic Direction") is the result of many hours of hard work that began with the Caltrans Division of Engineering Services (DES) Structure Policy Board (SPB) in 2011. The SPB assembled a diverse Task Force comprised of Caltrans managers who met in multiple three-day brainstorming workshops to create the 2014 Strategic Direction. The DES, Structure Policy and Innovation - Office of Structure Quality Management provided the 2016 update of the Strategic Direction. The 2016 Strategic Direction was approved by the Caltrans DES SPB on May 2, 2016.

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Carquinez Bridge, Hwy 80, Solano County, Caltrans

EXECUTIVE SUMMARY

The public deserves bridges and structures that are safe, sustainable, cost effective, well-built, and maintained in compliance with all applicable regulations. The California Bridges and Structures Strategic Direction (Strategic Direction) is a roadmap for the integrated delivery and management of ALL bridges and structures located on public roads in California. Through an integrated delivery and management approach, Caltrans and transportation partners can more effectively address California's bridge and structure needs to best serve the traveling public.

The State of California is faced with a number of challenges that influence our ability to effectively manage the complex bridge and structure infrastructure. Major challenges include:

- AGING INFRASTRUCTURE
- POPULATION GROWTH
- MULTIPLE STAKEHOLDERS
- CHANGING TRANSPORTATION NEEDS
- INADEQUATE FUNDING
- COMPETING INTERESTS
- ENVIRONMENTAL CONSTRAINTS
- LEGISLATIVE MANDATES
- SUCCESSION PLANNING

Bridgeport Bridge, Nevada County, Caltrans

In light of these challenges, it is in California's best interest that all stakeholders involved in the delivery and management of bridges and structures collaborate to meet shared goals, independent of ownership or funding sources. The Strategic Direction identifies 12 objectives that maximize safety, efficiency, sustainability, value, quality and innovation through integrated delivery and management of bridges and structures.

Regardless of the method of procurement or implementing agency, through integrated delivery and management, the Strategic Direction will balance asset performance and total lifecycle costs.

BACKGROUND

San Francisco-Oakland Bay Bridge East Spans, Hwy 80, Caltrans

California's highway system and complex bridge infrastructure are the lifeline of the California economy. The general public, businesses, and travelers from around the world utilize this vital system to go about their daily lives and carry out their business. Caltrans and local agencies manage more than 26,000 bridges on California's roads and highways. This infrastructure is a legacy system largely built by Caltrans during the 1950s, '60s and '70s utilizing a design-bid-build model. The model worked well as the State of California systematically created one of the most advanced transportation systems in the world during a period of tremendous economic expansion.

The world and environment that we live and work in has changed. We are now in an era that prioritizes environmental sustainability, quality of life, and preserving the highway system that was largely created decades ago. Many new players have entered the arenas of bridge and structure planning, design, construction, and management. In addition, projects are now delivered through several delivery methods (design-bid-build, design-build, private-public-partnerships, construction manager/general contractor, etc.) and paid for by numerous funding sources. The uniformity and quality afforded by a single provider (procurement, delivery, application of legal mandates and guidance) has changed. While this change is not necessarily negative, it does introduce the risk of inconsistent safety, performance, quality and durability, as well as other potential impacts to the public.

There is a need for uniform direction to better deliver and manage bridge and structure assets to reflect the current environment we live in. Decisions regarding the planning, design, construction, and maintenance strategies for bridges and structures need to be made in an integrated manner that considers the entire lifecycle of the assets and does not adversely affect the quality or safety of ALL bridges and structures located on public roads in California, regardless of who does the work.

Bridge and structure owners – whether they are state or local agencies – are responsible for the planning, design, construction, and maintenance of California's bridges and structures. They need uniform direction and guidance to ensure that decisions are made in an integrated and consistent manner. If decisions are not integrated and consistent, the consequences of error can be significant. Increased lifecycle costs of these assets – including project support, initial capital costs, and long-term maintenance – may result, which will ultimately adversely impact the traveling public.

Nationally, the Federal Highway Administration and the transportation community have recognized a shift in focus away from building new transportation systems to preserving and improving existing systems, as evidenced in recent legislation such as Moving Ahead for Progress in the 21st Century (MAP-21) and Fixing America's Surface Transportation (FAST) Act. Similarly, California is shifting its focus toward asset preservation, sustainability and management.

U.S. Route 40, Rainbow Bridge, Placer County, Caltrans

REGULATIONS

This Strategic Direction is a guiding document intended to comply with all corresponding federal, state and local laws, regulations, and governing codes for the National Highway System (NHS), State Highway System (SHS), non-NHS, non-SHS, and local streets and roads. Major governing codes and regulations include:

- U.S. Code of Federal Regulations[1]
- California Streets and Highways Code[2]
- Various Caltrans Deputy Directives[3,4]

Sources:

1. U.S. Code of Federal Regulations Title 23 – Highways, Part 625 – Design Standards for Highways
2. California Streets and Highways Code Sections 137 and 141
3. Caltrans Deputy Directive 23 R1: Roles and Responsibilities for Development of Projects on the State Highway System
4. Caltrans Deputy Directive 44: Federal-Aid and State Funded Highway Local Assistance

Pitkins Curve Bridge and Rockshed, Hwy 1, Monterey County, Caltrans

THE STRATEGIC DIRECTION

The Strategic Direction is a roadmap for delivering and managing ALL public bridges and structures in California to ensure that they are safe, durable, and cost effective through integrated delivery and management, independent of ownership or financial funding. Objectives were written to ensure that bridges and structures delivered by the aforementioned various delivery methods are consistent in all aspects and seamless in performance and value to the traveling public. The intent is not to solve all the challenges of delivering and managing bridges and structures in the 21st Century, but rather to deliberately and transparently establish a clear direction that the numerous partners in the transportation community can embrace and follow.

This roadmap clarifies what is important and integrates decision making to ensure greater consistency. When bridge managers are considering a decision, they need to weigh the impacts to the Strategic Direction objectives. Ultimately, if a decision adversely affects one of the objectives, it is probably not the best choice, and the associated risks need to be carefully considered. The Strategic Direction is a litmus test, and should be used for that purpose.

PURPOSE STATEMENT: PROVIDE A VEHICLE TO AFFECT IMPROVEMENTS ON HOW ALL PUBLIC BRIDGES AND STRUCTURES IN CALIFORNIA ARE PLANNED, DESIGNED, CONSTRUCTED, AND MAINTAINED.

Retaining Wall, Caltrans

EXPECTED OUTCOMES

The following results are intended to be delivered by this Strategic Direction approach:

- ✓ Integrated planning, design, construction, and maintenance decision-making regardless of the method of procurement or implementing agency
- ✓ Safe and sustainable bridges and structures
- ✓ Consistent and appropriate quality and management of risk
- ✓ Reduced project delivery costs and delays
- ✓ Balanced asset performance and total lifecycle costs
- ✓ Improved tools and training
- ✓ Effective use of emerging technologies (i.e. research, new materials, etc.)

The Strategic Direction focuses on long-term, cost-effective and sustainable strategies that address:

- ✓ Structure Design (loadings, geotechnical, seismic, and hydraulics)
- ✓ Structure Construction (specifications and contract administration)
- ✓ Asset Management (inspection and maintenance programming priorities, and preservation)
- ✓ Resources and Tools (policies, standards and guidance, staff skills, and software)
- ✓ Innovation (research, new materials and structural systems, technologies, and construction methods)
- ✓ Quality and Risk Management (including lessons learned)

Devil’s Slide Tunnel, State Route 1, San Mateo County, Caltrans

OBJECTIVES

The Strategic Direction identifies **12 objectives** that maximize safety, efficiency, sustainability, value, quality and innovation through integrated planning, design, construction, and maintenance of bridges and structures (hereafter referred to as "structures") in California.

California will enhance its economy and livability by investing in its structures in a manner that will:

1. **Minimize accidents**

   Work zone accidents and vehicle crashes must be minimized in order to provide a safer transportation system. Structures must be delivered and maintained in a way that ensures public safety and reduces worker and motorist exposure to injuries and fatalities.

2. **Minimize traffic delays**

   Traffic delays must be minimized in order to maximize system performance. The delivery of structures must aim to minimize delays to the traveling public and movement of goods during normal operations as well as during construction and maintenance activities.

5/14 Interchange in LA County, Caltrans

3. Ensure reliability and structural integrity

Reliability and structural integrity are paramount in order to ensure safe operations. Structures shall be constructed and maintained in a way that ensures safety, functionality, and durability while optimizing service life.

4. Optimize flexibility in meeting future intermodal transportation needs

Structures must be adaptable to future transportation needs to ensure that public funds are wisely invested. The planning and design of structures must consider attributes that provide for flexibility to address changing needs.

Tower Bridge, Sacramento, Caltrans

5. Meet established standards and policies consistent with laws, regulations, codes and agreements

   It is imperative to develop and adhere to standards and policies for structures that follow current laws, regulations, and codes to ensure the integrity of the transportation system and promote public trust.

6. Assure quality

   The consequences of poor quality are of great concern due to the critical nature and significant cost of structures. Therefore, it is important to establish and enforce quality management standards to protect the public's investment in structures.

7. Ensure open communication between all stakeholders

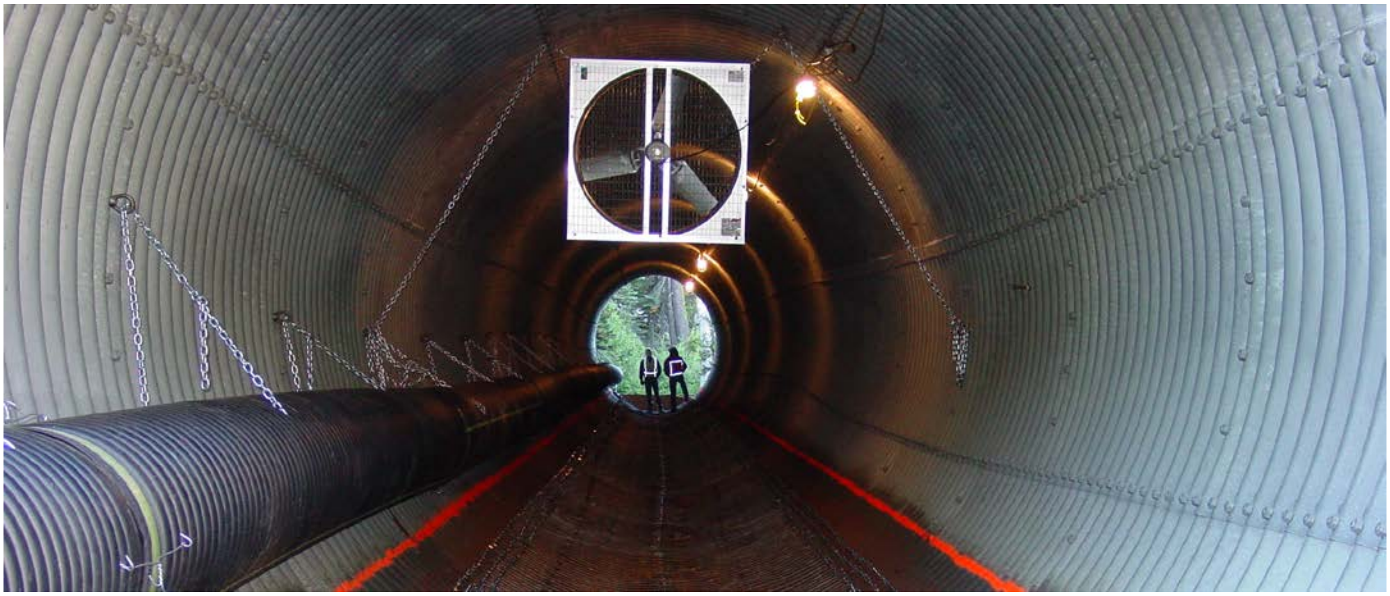
   The delivery and management of structures involves many different entities. In order to ensure that these assets are delivered and managed effectively, continuous communication among these entities is vital.

8. Balance performance, lifecycle cost, time, delivery, and risk to optimize total value

   The delivery and management of structures should optimize the public's return on investment. Therefore, decisions must be framed to promote the best value over the life of the asset while integrating risk-based thinking into decision-making.

9. Preserve the environment and minimize impacts

   Structures often play a significant role in either positively or negatively impacting the environment. Therefore, structures should be delivered and managed in a manner that minimizes impacts and preserves natural and cultural resources.



Culvert Invert Repair, Caltrans

10. Ensure transparency and accountability

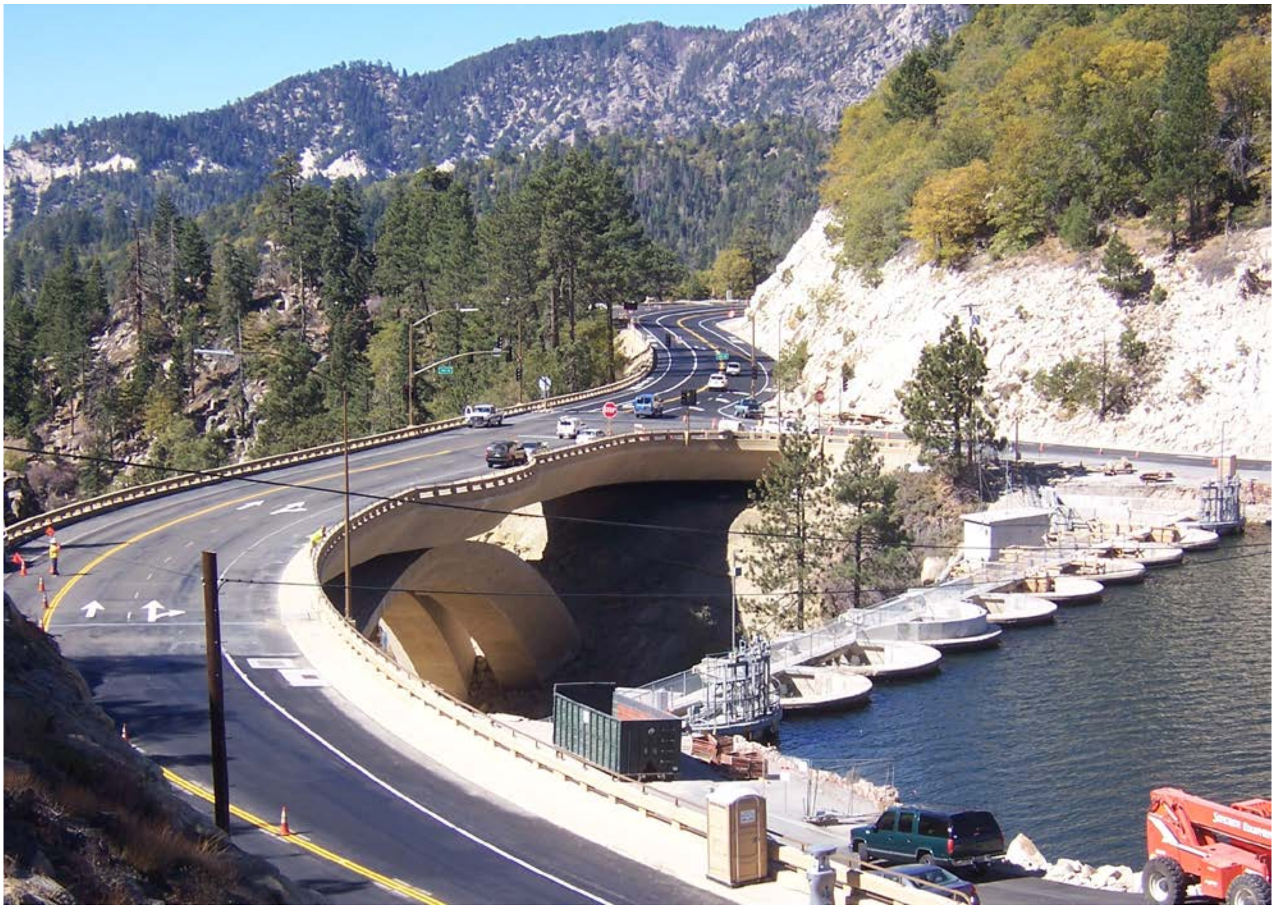
    Demonstrate prudent management of public funds by maintaining transparency and accountability in decisions and data related to structure assets.

11. Cultivate knowledge and experience

    The public's expectation is that experienced and knowledgeable experts are responsible for delivering and managing structures. To support this, a culture of continuous improvement that fosters the sharing and retention of knowledge and experience is essential.

12. Encourage innovative solutions

    Innovation is the catalyst for developing better transportation solutions. Emphasis should be placed on supporting an environment that encourages creative problem solving and intelligent risk taking.



Big Bear Bridge, Hwy 18, Caltrans

The mission and vision stated in the 2015 Caltrans' Strategic Management Plan (SMP) are supported by five goals. These five goals are directly supported by the twelve objectives in this Strategic Direction as shown below:

Caltrans' SMP Goals		Safety and Health	Stewardship and Efficiency	Sustainability, Livability and Economy	System Performance	Organizational Excellence
California Bridges and Structures Strategic Direction Objectives	1. Minimize accidents	X	X	X	X	
	2. Minimize traffic delays		X	X	X	
	3. Ensure reliability and structural integrity		X	X	X	
	4. Optimize flexibility in meeting future intermodal transportation needs		X	X	X	
	5. Meet established standards and policies consistent with laws, regulations, codes and agreements	X	X		X	X
	6. Assure quality	X	X	X	X	X
	7. Ensure open communication between all stakeholders		X			X
	8. Balance performance, lifecycle cost, time, delivery and risk to optimize total value		X	X	X	X
	9. Preserve the environment and minimize impacts		X	X		X
	10. Ensure transparency and accountability		X			X
	11. Cultivate knowledge and experience					X
	12. Encourage innovative solutions		X			X

The principal strategy used to accomplish many of these objectives will be implementation of ISO 9001 Quality Management Systems.

20-4 SEISMIC RETROFIT GUIDELINES FOR BRIDGES IN CALIFORNIA

Introduction

Caltrans *Memo to Designers* (MTD) 20-4 describes policies and procedures for the seismic retrofit of California's bridges.[1] Caltrans, *Bridge Design Aids* (BDA) 14-5 and *Bridge Standard XS Detail Sheets* Section 7 include common retrofits that can be used by designers. The Federal Highway Administration has published a bridge retrofitting manual (Buckle, 2006), with examples of common retrofits. This manual is a useful reference, however the specifications and details are not approved by Caltrans.

While MTD 20-4 is intended to provide guidelines for retrofitting existing structures, it is not possible to anticipate every situation that may be encountered. It is the designer's responsibility to accurately assess the performance of the existing structure, to show a collapse mechanism if it exists, and to develop retrofit strategies that ensure the structure meets the no collapse performance standard.

Expected Performance

The primary performance standard for retrofitting bridges is to prevent the structure from reaching the collapse limit state[2] for the Design Earthquake[3]. The goal of this "No Collapse" performance standard is to protect human life and there are no serviceability expectations for retrofitted bridges.

An acceptable determination of collapse is captured through an analysis of the bridge model subject to the Design Earthquake. However, determining collapse is different than simply determining that demand exceeds capacity. First of all, capacity is more conservatively

1. This memo is intended to apply to Ordinary Standard state and local bridges. In cases where this memo does not apply, the designer is referred to MTD 20-1 and 20-11.
2. The collapse limit state is defined as the condition where any additional deformation will potentially render a bridge incapable of resisting the loads generated by its self-weight. The "No Collapse" performance standard prevents failure of this type while allowing for the possible localized failure of some individual components (typically redundant or secondary components that are not necessary for structural stability).
3. In this memo the 'Design Earthquake' is substituted for the term 'Design Seismic Hazards' used by Caltrans Geotechnical Services to refer to the collection of seismic hazards at the bridge site used for the design of bridges.

defined for new than for existing bridges. Secondly, exceeding a single member capacity may not lead to system collapse. Collapse means that the demand is so large that the bridge will become unseated, that it will break a significant load bearing element, or it will cause some other collapse mechanism that will positively bring down the bridge. An equally valid solution is to demonstrate through analysis that a collapse will not occur. This would be the preferred alternative since construction (with its costs and risks) would not be required.

There are several reasons why seismic performance requirements are higher for new bridges. Designers can provide additional seismic resiliency on new bridges whereas they are often constrained by geometry or structural configuration with an existing bridge. Moreover, the seismic demands for existing bridges (with a shorter remaining life) can be less conservative than for new bridges and still provide an acceptable level of risk. MTD 20-4 only requires the minimum seismic retrofit to prevent collapse while Caltrans *Seismic Design Criteria* (SDC) for new bridges has additional requirements that provide a larger safety factor against collapse. Therefore existing bridges are allowed to have behavior that is discouraged for new bridges. For instance, rocking of existing bridge foundations is acceptable for ground shaking hazards and more drift is allowed on piles and shafts of existing bridges in laterally spreading soil.

Currently the Structure Replacement and Improvement Needs (STRAIN) Report identifies bridges with many needs including seismic retrofit and each district chooses projects from the report. When post event structural serviceability is a design requirement, this memo will not apply, and a more conservative approach based on project specific performance standards must be followed. MTD 20-11 (Caltrans, 1999) must be used to establish this criterion.

The Design Earthquake

Ground shaking is the one seismic hazard that can occur at every bridge site. The designer must carefully read Caltrans MTD 20-17 "*Understanding Directionality Concepts in Seismic Analysis*" (Caltrans, 2014) to understand how ground shaking demands are obtained for different methods of analysis. All of these methods originate from the Design Spectrum described in Caltrans SDC Section 2.1 and in Appendix B (Caltrans, 2013). Amplification of the ground shaking hazard for near fault and basin effects is accomplished by increasing the long period response. Caltrans Design Spectrum is also used to produce time histories of ground motion that include these effects.

In rare cases a bridges may need to be analyzed for two or even three seismic hazards. Designers are notified of all seismic hazards at the bridge site in the *Preliminary Foundation Report*. Caltrans MTD 20-8 and MTD 20-10 provide methods for determining the surface faulting hazard and the resulting demands on bridges. MTD 20-13 provides a method for determining the hydrodynamic forces due to the tsunami hazard which are used to determine the demands on bridges. MTD 20-14 discusses how to proceed when liquefiable soil may be an issue. MTD 20-15 provides a method for analyzing bridges for lateral spreading. However, these memos were written for the design of new bridges. It is overly conservative to design existing bridges for several simultaneously occurring seismic hazards with a 5% in 50 year probability of occurrence. The chances that the largest ground shaking, liquefaction, and lateral spreading hazards in 975 years will occur simultaneously is extremely unlikely and even more unlikely for a bridge with a small remaining service life. For existing bridges the designer must analyze for each hazard separately, determine if any of them can cause the bridge to collapse, and achieve a retrofit design (if needed) that will accommodate the effects of each hazard on the bridge[4].

Caltrans uses the larger of the deterministic and probabilistic (for a 1000 year return period) derived seismic hazards for both new and existing bridges. However, bridges that will remain in service for less than five years only need to be analyzed for the hazards that are likely to occur during a 100 year return period (10% in 10 years). For instance, if there is a delay in the replacement of a vulnerable existing bridge, an interim retrofit for the smaller return period of 100 years may be performed to reduce the risk to the public at a reasonable cost. MTD 20-2 "*Site Seismicity for Temporary Bridges and Stage Construction*" provides the rules for the seismic design of new and existing temporary bridges. Of course, an acceptable alternative for 'interim' retrofits is to do nothing if a collapse mechanism does not form for this smaller hazard.

Our understanding of seismic hazards and bridge earthquake response has increased since MTD 20-4 was first published in 1990. Larger ground motions as well as previously unconsidered seismic hazards means that retrofits done in the 1990s may need to be revisited. However, because Caltrans has to prioritize the many life safety concerns on state highways and locally owned bridges, undue conservatism is not appropriate for the seismic retrofit of ordinary bridges.

Background Work And Review

As a preliminary step in determining if a structure requires a retrofit, the designer must verify the existing conditions. This would include a review of all the as-built plans including any previous work done on the structure, checking *Structure Maintenance and Investigations* (SM&I) records, obtaining site seismicity and geological conditions, and visiting the site (if possible) to compare as-built and current site (including traffic and utility) constraints. When evaluating a state highway bridge, the designer must also review the STRAIN to

---

4. Long term scour is combined with seismic loads for existing bridges. See the appropriate memo for rules on combining hazards.

assess the need to combine retrofit work with other work such as deck rehabilitation, barrier replacement, etc. wherever possible. This must be done as early in the project development phase as possible in order to properly scope the project. The designer should contact the SM&I bridge program coordinator to discuss the STRAIN recommendations.

Initial Assessment of Structure

Careful consideration must be given to assess the structural response of the entire system for the Design Earthquake (as provided in the *Geotechnical Services Foundation Report*) in order to develop an effective seismic retrofit strategy. Prescribed procedures may not apply to every situation. For example, yielding of a single element may not be sufficient to create a collapse mechanism. The redistribution of additional load in a structural system after incremental yielding will be different for each structure. Table 1 provides the maximum ductility demand values that are allowed for poorly reinforced substructures that were built before the 1971 San Fernando earthquake. These values are based on tests of older columns and piles (Priestley, 1991) and of pier walls (Haroun, 1993) that were done during the legislative-mandated retrofit program in the 1990s and can be used for an initial assessment of older bridges. The table represents the tested performance of columns with continuous reinforcement, and also notes the maximum ductility capacity observed when a member contains poorly confined lap splices in main reinforcement[5]. When analyzing older columns, after the substructure elements have reached their maximum ductility, a pinned connection can be substituted for the fixed connection and the push-over analysis can be continued.

Table 1. Maximum allowable displacement ductility capacity, $\mu_{c,max}$ for poorly confined members

	Poorly Confined/No Retrofit		Steel/Fiber Casing Retrofit	
Substructure Member Type	**lapped main bars**	**cont. main bars**	**lapped main bars**	**cont. main bars**
Round Columns	1.5	3.0	5.0	8.0
Rectangular Columns	1.0	3.0	6.0	8.0
Pile/Shaft Extensions	1.5	3.0	5.0	8.0
Pier Walls in weak direction	1.0	4.0	5.0	8.0

5. If the starter bars have an effective lap beyond the plastic hinge, approximately equal to the wall thickness, then it will act as a continuous main bar.

The designer must evaluate the global bridge model for collapse rather than the failure of individual elements. This 'diagnostic model' is created to analyze the structure in the as-built condition and identify the different collapse scenarios that can occur. Then an incremental approach is used to determine the level of retrofit necessary to develop a retrofit strategy that achieves the most economical retrofit design while meeting the "No Collapse" performance standard. For modeling and analysis guidelines, the designer can refer to the Caltrans SDC:

- Section 2.1 for determining the maximum demands due to the Design Spectrum
- Section 2.1.5 for damping factors
- Section 5.2 – 5.5 for analytical methods.
- Section 5.3 for global analysis modeling including bridges with irregular geometry
- Section 5.6.1 for effective section properties
- Section 6.1 for site seismicity and analyzing for different seismic hazards
- Section 7.8 for abutment response (existing bridges can take greater advantage of abutment stiffness to protect weak columns and piers)

Note that acceptable limit states for assessment may be different from those in the SDC. For instance new columns have a target displacement ductility demand of 4 to 5, well short of their actual capacity, while retrofitted columns are allowed a target ductility demand of up to 8 (based on Priestley, 1991). Similarly, the shear strength of new columns is based on nominal properties but it is based on expected properties for existing columns. The shear model used in SDC is relatively conservative compared to results of experimental testing of existing and new columns. In certain situations the UC San Diego shear model can be utilized to compute higher capacities on existing columns (Priestley, 1991). Similarly, pier wall shear capacity in the weak direction may be overly conservative using the SDC column shear degradation model at moderate levels of ductility (Haroun, 1994). The use of alternative shear models must be approved at the strategy meeting.

The designer must estimate various modeling parameters, such as abutment stiffness, cracked section properties, etc., and run the diagnostic model assuming structural integrity is maintained. The resulting displacement demands are then compared with member capacities. Some of the Demand/Capacity ratios the designer must check include (but are not limited to) ultimate displacement, shear, pile capacities, and seat length. For some pile types such as 'Raymond' step tapered or timber piles, the capacities are usually assumed to be zero. The initial modeling assumptions, such as abutment stiffness, etc., used in the diagnostic model are then verified. If necessary, the model is rerun with revised assumptions, and then checked again. This process is repeated until the results converge with the assumed modeling parameters.

Material Properties For Existing Bridges

Stresses and strains for structural steel, concrete, and steel reinforcement have changed over time. The Concrete Reinforcing Steel Institute published a report (CRSI, 2001) with rebar specifications from 1900 to 2001. The expected compressive strength of portland cement concrete in good condition can be taken as 5000 psi. The properties of bar reinforcement that are not in the table (or in the references) must be established on a project specific basis.

Table 2. Properties for Moment Curvature Analysis

Property	Symbol	A706	A615 Gr 60	A615 or older Gr 40
Specified Minimum Yield Stress	F_y min	60 ksi	60 ksi	40 ksi
Specified Maximum Yield Stress	F_y max	78 ksi	NA	NA
Expected Yield Stress	F_{ye}	68 ksi	68 ksi	48 ksi
Specified Minimum Tensile Stress	F_u	80 ksi	90 ksi	60 ksi
Expected Tensile Stress	F_{ue}	95 ksi	95 ksi	68 ksi
Nominal Yield Strain	ε_y	0.0021	0.0021	0.00138
Expected Yield Strain	ε_{ye}	0.0023	0.0023	0.00166
Ultimate Tensile Strain #4 to #10 #11 to #18	ε_{su}	0.120 0.090	0.090 0.060	0.120 0.090
Reduced Ultimate Tensile Strain #4 to #10 #11 to #18	ε_{su}^R	0.090 0.060	0.060 0.040	0.090 0.060
Onset of Strain Hardening #8 and smaller #9 #10 and #11 #14 #18	ε_{sh}	0.0150 0.0125 0.0115 0.0075 0.0050	0.0150 0.0125 0.0115 0.0075 0.0050	$14\ \varepsilon_y$ = 0.0193

Development Of Retrofit Strategy

If the diagnostic model indicates that a collapse mechanism exists then the designer must estimate the minimum amount of retrofit required[6] to meet the "No Collapse" performance standard. The diagnostic model with the proposed retrofit is then run. If a collapse mechanism for the structural system still exists, additional retrofit measures are required. If the retrofit model indicates there is no collapse mechanism and that the associated member demands are significantly less than their capacities, the designer must consider reducing the amount of retrofit and re-running the model. This procedure is repeated until an optimal, or "preferred" retrofit strategy is obtained.

The designer must consider costs when developing a retrofit bridge model. For instance, the abutment and superstructure can sometimes be modified to reduce demands to the columns at considerable savings over a column and foundation retrofit. To obtain the cost codes for contract items the designer can go to: http://www.dot.ca.gov/des/oe/construction-contract-standards.html. The codes are input at http://sv08data.dot.ca.gov/contractcost/, which provides costs for retrofit and other construction items. This can be useful for estimating costs (although final costs will be supplied by Caltrans Structure Office Engineer).

The designer must also consider the hierarchy of different retrofit strategies. Large seats have the most direct effect on preventing collapse. Increasing column ductility with casings is a common strategy. Increasing column shear strength is also very effective. Strengthening foundations may have little effect unless liquefaction with lateral spreading is a threat. Even when poor soil is a problem, it is usually more effective (and less expensive) to turn the superstructure into a strut that uses the abutments to restrain movement. Single column bents are more vulnerable to collapse and may benefit from foundation work (see Section 8). On a shorter bridge, putting timber blocking between the abutment backwall and the superstructure (if there is a gallery) is sometimes sufficient to reduce displacements and protect vulnerable elements.

Seismic design is a balance between strength and ductility. Increasing the ductility of existing bridges is usually the most straightforward retrofit. When strength is added to existing bridges, other members in the load path must be rechecked to ensure the reliability of the retrofit scheme. It is better not to add strength as it usually just makes the seismic demands larger.

The designer must try to use standard retrofit details as much as possible. *Bridge Standard Detail Sheets* (XS Sheets) Section 7 (Caltrans, 2014) provides the most common retrofit details that have been tested and known to be effective. Other seismic retrofit details can be

6. The minimum amount of retrofit is typically the retrofit alternative that satisfies the project report and environmental document and can be constructed for the lowest cost. Future maintenance costs must also be considered.

found in Caltrans BDA 14-5 *Example Seismic Retrofit Details* (Caltrans, 2008) and in the different parts of MTD Section 20. Any deviation from these standard retrofits requires a design exception as described in Caltrans MTD 20-11.

For any alternative retrofit strategy, the designer must clearly demonstrate that the strategy is the minimum that meets the "No Collapse" performance standard. The designer must also develop sufficient conceptual details for the strategy in order to show that the strategy is feasible. Each strategy must address geotechnical, hydraulic, aesthetic, highway, environmental, constructability, utility, and other relevant issues. During the strategy development phase, the Lead Office must consult with the Office of Earthquake Engineering (OEE) for complex strategies.

Following the development of the retrofit strategy, the respective Lead Office must schedule a Retrofit Strategy Meeting. Other relevant Functional Offices must be present at the meeting.

Lead Offices

- Offices of Structure Design
- Office of Special Funded Projects/Structures Local Assistance (SFP/SLA)

Functional Offices

- Earthquake Engineering
- Geotechnical Design Offices within Geotechnical Services
- Structure Design (for in-house designs and SFP/SLA projects)
- Structure Maintenance and Investigations
- Structure Office Engineer (as needed)
- Structure Construction
- Bridge Architecture and Aesthetics (as needed)
- Structure Hydraulics (as needed)

The Lead Office must provide a Strategy Report to the meeting attendees at least one week prior to the Strategy Meeting for simple projects and at least two weeks prior to the Strategy Meeting for complicated bridges with multiple frames and/or with multiple hazards. As a minimum, the report must include:

- A General Plan indicating the retrofit work for each alternative
- All pertinent as-built plans for the existing bridge

- A summary of demand/capacity ratios (μ_D/μ_C), structural vulnerabilities, potential collapse mechanisms, and modeling assumptions for the diagnostic model and each retrofit alternative. If special retrofit requirements are a result of the findings of the Project Report or Environmental Document, they should be shown on the Strategy Report
- Preliminary Foundation Report for bridges including geotechnical seismic recommendations with ground shaking plus liquefaction or for other multiple hazards
- Conceptual details that show the retrofit alternatives are feasible
- A cost estimate for each alternative

In addition, the designer must be prepared to discuss the analysis methods used to evaluate the existing structure as well as all retrofit alternatives.

Caltrans OEE provides a key role before the strategy meeting and must approve the earthquake retrofit strategy. The use of pre-strategy consultations with the Office of Earthquake Engineering is essential for projects with multiple hazards, as seismic criteria and engineering practice are still evolving.

While it is the responsibility of the designer to accurately assess the seismic performance of the existing structure, and to develop the retrofit strategy, a successful Strategy Meeting achieves consensus among all attendees and confirms that the retrofitted structure meets the required performance standard[7]. Unusual retrofit strategies or performance standards require a design exception.

The Lead Office Chief will give final approval of the retrofit strategy and grant exceptions to retrofit requirements when necessary. When disagreements occur between OEE and the Lead Office, they will be resolved by the OEE Chief. After approval the Seismic Retrofit Assessment Form (MTD 20-4 Attachment A)[8] must be completed by the designer and included in the Final Strategy Report. The Lead Office must also submit a copy to the OEE Chief, for incorporation into the permanent bridge records.

Structures may require seismic evaluation and retrofit when modified (widening, rehabilitation, etc.) as discussed in MTD 20-12 (Caltrans, 2013) and MTD 9-3 (Caltrans, 2010). In these cases, the Strategy Meeting may be combined with the Type Selection Meeting (See MTD 1-29). The designer is required to demonstrate that the new or widened portion

7. The minimum required performance standard is “No Collapse” unless directed otherwise by the Lead Office Chief with concurrence from the Chief of OEE.

8. The purpose of the Seismic Retrofit Assessment form is to keep a record of previous seismic evaluations for future reference. Sometimes an APS or Strategy Meeting concludes that no retrofit is required. This conclusion should be documented on a Seismic Retrofit Assessment form.

of the structure meets the SDC requirements while the combined structure meets the "No Collapse" performance standard. (See MTD 9-3 for additional guidelines and information). For complex strategies, the Lead Office may consider meeting with OEE prior to the Type Selection/Strategy Meeting in order to gain consensus on the recommended seismic retrofit strategies. In cases where there is an adjacent structure with potential seismic vulnerabilities similar to the bridge being modified (for example left and right bridges), it is important to ensure the adjacent structure is either retrofitted or programmed for future retrofit assessment. This must be accomplished by submitting a Seismic Retrofit Assessment Form (Attachment A) to the Office of Earthquake Engineering.

Retrofit Design Considerations

In order to meet the goal of the "No Collapse" performance standard, the designer must consider the most common vulnerabilities that may lead to collapse mechanisms and are described below.

Single Column Bents

Prior to 1971, single column bents were constructed with dowels protruding from the top of the footing (called 'starter bars'). The column cage was then connected to the dowels by lap splices. These lap splices usually had insufficient length and confinement to maintain enough fixity to develop the plastic capacity of the column.

Slippage of the lap splice at the bottom of the column may compromise the fixity and affect the overall stability of the structure. When retrofitting a column to maintain flexural capacity, the column's overstrength moment ($M_o^{col} = 1.2 \times M_p^{col}$) will be transferred to the footing and consideration must be given to strengthening the footing in order to resist the resulting moment. However, rotation of a footing is not necessarily a collapse mechanism. Axial displacement of a pile through the competent soil will dissipate energy during the earthquake. Therefore, it may not be necessary to ensure fixity at every column/footing connection. Slipping of the lap splices may be permitted provided the vertical load carrying capacity of the column is not compromised. Retrofit design allows some lap splices to release provided there is sufficient strength in the frame to prevent collapse.

When instability of a single column bent could result in a bridge collapse, the designer should consider using a Class F column casing to protect the column and the connection to the foundation.

Multi-Column Bents

In multi-column bents, the columns are typically pinned at the base. In these cases, the designer must check that the footing can resist moments and forces based on the shear capacity of the pin. If the column/footing connection is fixed, the designer must consider the consequences if the fixed condition is lost during the earthquake. If the fixed condition is necessary for structural stability, the designer must take appropriate measures (such as Class F casings and footing retrofits) to prevent collapse.

Foundations

Damage to abutments and footing piles is acceptable provided this does not lead to a collapse of the structure. In a pile type foundation, if a fixed column condition is not required, foundation damage that could result in a substantial loss of fixity of the column may be acceptable. However, there must be a sufficient number of piles in the resulting effective foundation region to maintain the vertical capacity of the structure. The effective foundation region is assumed to be an area bounded by the column and one half of the footing depth on either side of the column. Similarly for spread footings, the effective area under the column must be sufficient to maintain vertical load carrying capacity.

Pile Extensions

In the case of relatively short slab bridges (typically 4 spans or less) on pile extensions, the diaphragm-type abutments typically provide most of the lateral resistance. The pile extensions may exceed their ultimate displacement capacities provided they maintain their vertical load carrying capacity.

Transverse Reinforcement

Shear failures are brittle, and therefore the shear demand/capacity ratio must remain below 1.0. For structures with minimal and poorly detailed (#4 ties at 12 inches) transverse lapped reinforcement, the designer must assume that only the concrete provides shear resistance. In this case the bridge should be modeled as unconfined concrete.

For bridges that have improved transverse column reinforcement details, it may be assumed that both concrete and steel provide shear resistance. The designer may refer to "*Seismic Assessment and Retrofit of Bridges*" (Priestley, 1991) for help evaluating the shear capacity of older columns. The shear capacity of existing columns may be determined with the methods described in SDC Section 3.6 using expected properties instead of the nominal properties that are required for new bridges. Refer to Section 5 of this memo for more information on evaluating the shear capacity of columns.

Abutments

On shorter bridges (typically 4 spans or less), the abutments may provide significant resistance to longitudinal movement. Using methods discussed in SDC Section 7.8.1, the designer may apply longitudinal abutment springs to structural models. Typically on seat type abutments, the shear keys and backwalls will fail at the Design Earthquake. It may also be worthwhile to increase the damping of shorter bridges by following the procedure in Caltrans SDC Section 2.1.5.

Bent Caps

In bridges with multi-column bents, hinging could occur in the bent cap. While this is not desirable, it may not necessarily lead to a collapse of the structure. For box girder bridges, the bent cap remains effective as long as its displacement ductility capacity (measured in rotation, curvature, or displacement) is greater than the displacement demand from the Design Earthquake. For other types of bridges, as long as the transverse displacement of the bent is less than two times the displacement that causes the bent cap to yield, and there is sufficient shear reinforcement (V_S) in the cap to resist the shear due to the plastic moment of the bent cap and dead load (V_P+V_{DL}), they remain effective in preventing collapse. When there are tightly spaced stirrups in the cap (to prevent excessive cracking), V_C may also be considered when determining the shear capacity of the bent cap. The effective width of the bent cap for considering its flexural and shear capacity is the cap width plus 12 times the top or bottom slab thickness (as illustrated in SDC Section 7.3.1.1).

At displacement ductility ratios above 2.0, the designer must demonstrate that even if the bent cap hinge degrades to a natural hinge (pin), adjacent elements like columns and abutments will continue to support the superstructure and prevent collapse.

P-Δ Effects

The P-Δ check is intended to ensure adequate results when using the equal displacement principle (between linear and nonlinear systems). The SDC treats P-Δ at the local level and the limit of 0.2 in SDC Section 4.2 was adopted to be on the conservative side for new bridges. MTD 20-4 treats P-Δ as a system parameter that is often addressed by ensuring continuity of the superstructure. Therefore the P-Δ limit for existing bridges can vary from 0.2 to 0.3. For movements in the longitudinal direction, the soil mass behind the abutment may be sufficient to prevent additional movement caused by P-Δ (the soil mass acts as a restoring force).

For cases where the P-Δ effect is a concern, the designer may evaluate the marginal increase in the displacement demand due to second order effects using time history methods of analyses that include geometric nonlinearity. The designer should consult with the Office of Earthquake Engineering (or their liaison engineer) for more information.

Pier Walls

Pier walls must be analyzed as columns in the weak direction, and as a shear element in the strong direction. For bending in the weak direction (given continuous main reinforcement in plastic hinges) the calculated displacement ductility demand is capped at 4.0. For lapped starter bars, the ductility of all structural members must be limited to $1.5\mu_C$ (but see footnote 5). More information on the behavior of pier walls is available from a series a tests that were done at UC Irvine (Haroun, 1993). For existing bridges, the shear demand of pier walls in the strong direction can be calculated as the peak of the Design Spectra while the capacity can be determined from the less conservative UCSD shear equation. Damage to piles is acceptable in the strong direction provided the stability of the pier wall is not compromised in the weak direction.

Unbalanced Bents and Frames

Previous earthquakes have demonstrated the vulnerability of unbalanced columns in a bent and unbalanced bents in a frame. In these systems there is unequal sharing of the seismic demand. The stiffer elements will carry more of the inertial load and be unable to displace as much as the other members and they can break. It is difficult to modify an unbalanced system. The best solution is to provide isolation casings in the soil around stiffer elements to give them a greater displacement capacity. Column casings and isolation bearings have also been used to increase the displacement capacity of stiffer elements.

Unbalanced frames have out-of-phase motion that can result in the frames moving away from each other and dropping a span at the hinge. The solution for these situations is provide pipe seat extenders or other devices to prevent unseating.

Expansion Joints

On longer bridges with continuous superstructures, expansion joints are used to allow for thermal expansion. The designer must ensure that the hinge has sufficient seat length to accommodate differential movements between adjacent frames for the Design Earthquake. Caltrans SDC Section 7.2.5.4 provides guidance for determining adequate seat length, however, the 24-inch minimum seat length required by Caltrans SDC does not apply to retrofits. When in-span hinge seats are less than twelve inches, the seat must be retrofitted with pipe seat extenders. Use of cable restrainers instead of pipe seat extenders to prevent unseating requires a design exception.

When it is necessary to core through hinge diaphragms or bent caps in order to place pipe seat extenders or hinge restrainers, the designer is cautioned to avoid structurally critical elements such as pre-stressing steel or shear reinforcement.

On some existing cable restrainer systems, the cables were grouted into the openings, essentially reducing the effective length of the cables to a few inches. The designer must refer to the as-built plans to determine if the existing cables were grouted. The designer must consider that in a seismic event, grouted cable restrainers could fail at small movements thus leaving the hinge unrestrained, and therefore take appropriate measures such as pipe seat extenders.

Simple Spans

On bridges with simple span superstructures, the designer must ensure that the spans remain seated on the abutments and bents for the Design Earthquake. Often, it is not practical to place pipe seat extenders in these situations. Catcher blocks and shear keys are an effective means of retrofit for these situations and typical details may be found in BDA 14-5. Use of cable restrainers to prevent unseating of bridges with simple spans requires an approved design exception.

Rocker Bearings

On some structures, tall rocker bearings were used at the abutments and at the bent caps on simple span configurations. For the Design Earthquake these bearings could fail and result in a drop of the superstructure. While a drop of six inches or less is not typically catastrophic, a potential drop greater than this must be investigated in order to ensure that the structure is not vulnerable. When the height of the rocker bearing is greater than 2/3 of the seat length, the superstructure could become unseated and the designer must consider appropriate retrofit measures.

Flared Columns on Multi-Column Bents

Flares on columns are an architectural feature on some bridges in California. It is desirable for plastic hinges to form at the top and bottom of the column as this minimizes its plastic shear and rotational demands. However, flares on multi-column bents typically cause a hinge to form at the base of the flare rather than at the top of the column thus increasing the column’s plastic shear demand in the prismatic portion and potentially exceeding its rotational capacity.

Liquefaction

Liquefaction (the loss of strength of saturated cohesionless soils during earthquakes) can damage a bridge due to reduced lateral resistance, excessive foundation settlement, or due to increased axial loading (as a result of downdrag forces). The designer must determine if liquefaction will result in collapse. If a potential collapse mechanism exists, either footing modification or soil improvement is usually required to meet the "No Collapse" performance standard. In these situations, the designer is referred to MTD 20-14 and 20-15 for guidance.

Lateral Spreading

A bridge can be damaged by lateral soil movement caused by a combination of sloping ground, horizontal shaking, and reduced soil strength. If foundations are not sufficiently stiff or strong enough to resist these lateral displacement demands, damage may occur in the form of superstructure unseating and/or excess deformation of columns and foundation elements. MTD 20-15 addresses lateral spreading for new bridges and requires that the lateral spreading demand should be combined with the demand due to ground shaking. Following the policy previously stated in this memo, lateral spreading and the ground shaking are considered separately for existing bridges and the designer must ensure that the bridge will not collapse for either of these demands.

Scour

Scour is the transportation of the soil supporting bridge foundations in streams and rivers. Although most hazards are considered separately for retrofit design, scour must be considered in combination with seismic hazards. Caltrans SDC Section 2.2.5 provides the rules for considering scour in combination with different seismic hazards on new bridges. For existing bridges the seismic evaluation must be based on long term scour plus each seismic hazard (considered separately) where long term scour considers the remaining life of the bridge and the hydraulics report.

Joint Shear

Since the early 1990's, greater emphasis has been placed on joint shear considerations in the seismic design of bridges. Previously, joints were modeled as either fixed or pinned if demands exceeded the elastic joint shear capacity. As a joint is cycled at high ductilities during a seismic event, it may lose some of its ability to carry moment and degrade to a rotational spring or pin. Degradation models for modeling column/beam joints as a spring are available. A procedure and example for determining the effects of joint shear may be found in the BDA 14-4.

While joint shear is not typically a collapse mechanism and retrofit is not usually required, on long viaducts a large number of adjacent joints that form pins could potentially lead to instability of the structure. In these situations, with the concurrence of the Lead Office Chief, the designer must demonstrate that a potential collapse mechanism exists and retrofit the minimum number of joints to ensure structural stability.

The procedure for determining joint shear on pre-1994 structures was developed from research (Mazzoni, 2004). The procedure may require modification as the knowledge base increases. The proof test for the joint shear retrofit strategy on existing bridges is still pending. Therefore, the Lead Office must obtain approval for the design and details for joint retrofit from OEE.

Common Retrofit Measures For Existing Bridges

Steel Column Casing

The most common column retrofit is to encase the column with a steel casing to increase the confinement and to improve the flexural ductility and shear capacity of the column. There are two classes of steel column casing retrofit currently in use, Class F and Class P/F. These types of casings must be circular for square and round columns, and elliptical for rectangular columns (refer to BDA 14-2 for casing and radius requirements). However, when retrofitting for shear only, it is not necessary to maintain a circular or elliptical shape. Flat plates may be used when required due to limited horizontal clearance.

In the Class F retrofit, no gap is provided in the space between the column and the steel casing resulting in full-length confinement of the column. This limits the dilation of the concrete and prevents lap splices from slipping thus ensuring the fixed condition of the column/footing connection remains intact. The supporting footing must be stronger than 1.2 M_p of the column if the bridge system requires successful plastic hinging at this location.

In the Class P/F retrofit, a gap between the column and steel casing is provided around the plastic hinge region near the bottom of the column. This allows the concrete to dilate and the lap splices to slip and ensures that a pin will form at the bottom of the column. The Class P/F column casing just allows the column's nominal moment capacity (M_n) to be transferred to the footing, often eliminating the need for a footing retrofit. However, the column shear capacity in the lap splice region is limited to the capacity of the steel casing. Details for column casings (Both Class F and Class P/F) can be found in BDA 14-2 and the *Bridge Standard Detail Sheets* XS7-010.

Footings

When Class F column shells are used in single column bents, it is assumed that the footing (including pile caps) resists the column's overstrength moment or the controlling foundation moment capacity. For structures designed prior to 1971, the following vulnerabilities may exist in the footings:

- No top mat of reinforcing steel.
- Inadequate tension ties connecting the pile and the footing.
- Inadequate pile capacity for the column's plastic moment[9].
- Insufficient shear strength in the piles to resist the column's plastic shear.

Composite Column Casings

Occasionally, space or clearance considerations do not allow steel column casings to be used for retrofit. In some of these cases, Fiber Reinforced Polymer (FRP) composite casings may be used instead. The primary column retrofit for flexural and shear issues is steel casings. However, FRP has been proven to be effective under certain conditions. Caltrans has limited test data for the shear capacity of FRP wrapped columns, but these results show it to be an effective retrofit strategy (Pulido, 2002). See BDA 14-3 for procedures and specifications when using this alternative.

Infill Walls

In multi-column bents, the infill wall is an inexpensive and effective retrofit for addressing transverse vulnerabilities both in the columns and in the bent cap. Research has shown that infill walls perform the same whether the concrete is poured up to the soffit or a six inch gap is left between the top of the wall and the soffit (Haroun, 2001). Doweling into the soffit of the bent cap does not provide any additional capacity and thus is not recommended. Typical details for the in-fill wall may be found in BDA 14-5. In the longitudinal direction infill walls act as a catcher to prevent collapse. Because infill walls are shear-critical elements their use is discouraged when a flexural system with larger displacement capacity is feasible.

9. Typical details for a footing retrofit may be found in BDA 14-5.

Abutment Strengthening

On short bridges, mobilizing the soil behind the abutments may be sufficient to reduce displacement demands below the structure's displacement capacity. This may be accomplished by strengthening the abutment diaphragm, or in the case of seat type abutments, connecting the superstructure end diaphragm to the seat. When a large gap exists between the end diaphragm of the superstructure and the abutment backwall, the soil behind the backwall can be mobilized by eliminating the gap with concrete or timber blocking. The designer is cautioned to leave a small gap that still allows for service load and temperature movements of the structure.

Catcher Blocks

Abutment bearings frequently fail during seismic events. However, such localized failure is not generally catastrophic unless the drop exceeds six inches. Seat catchers are an effective and inexpensive method of limiting superstructure drop and providing additional seat length as well. Catchers may also be used on bent caps for simply supported spans.

Cable Restrainers

Use of restrainers are at the discretion of the designer. They are effective in limiting the displacements for small to moderate seismic events. Restrainers are not considered to be effective at preventing unseating, and so their use to reduce displacement demand in a seismic retrofit requires an approved design exception. If existing restrainers are retained, anchorages must be checked for proper gapping and anchor nuts secured with a thread locking system.

Pipe Seat Extenders

Pipe seat extenders are effective in preventing collapse of a hinge span; however, the bridge may not be serviceable when the hinge opens sufficiently to engage the extenders. Therefore when pipe seat extenders are used for retrofit, consideration must be given to placing cable restrainers through the pipe and anchoring them to the adjacent bent cap. Restrainers may limit the differential movement in the hinge during moderate events and reduce damage to the bearing pads and expansion joints.

The typical detail, found in BDA 14-5 and *Bridge Standard Detail Sheets* (XS Sheets) Section 7, for a pipe seat extender makes use of Pipe XX-Strong (ASTM A-53 Grade B). The allowable load that can be carried by pipe seat extenders depends on the anticipated displacement demand, Δ_D and the seat length (See Figure 1).

For an 8-inch seat (or less), each pipe can carry 300 kips at unseating, 200 kips at $\Delta_D = 12$ inches, and 135 kips at $\Delta_D = 18$ inches.

For a 12-inch seat each pipe can only carry 200 kips at unseating and 135 kips at $\Delta_D = 18$ inches.

For an 18-inch seat each pipe can only carry 135 kips at unseating.

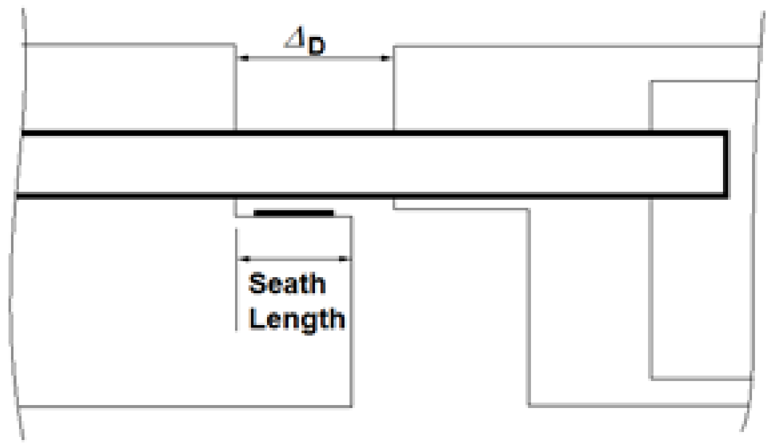


Figure 1. Unseating at pipe seat extender

Pipe seat extenders must be installed so that movement of the bridge under service conditions is not restricted (typically the extenders must be placed parallel to the girders). In addition, the designer must evaluate the capacity of the supporting hinge diaphragm. Pipe seat extenders are also effective as shear keys for existing bridges.

Foundation Retrofit

Typically, footings are strengthened by the addition of a top mat of reinforcing steel and additional piles. A foundation retrofit is usually costly and careful consideration must be given to retrofitting only the minimum number required to meet the "No Collapse" performance standard. Past foundation retrofits have included tie-downs (as an alternative to piles in tension) and micropiles (handy when working under the superstructure). Typical details for footing retrofits may be found in BDA 14-5.

Flare Isolation

Isolating a column flare is an inexpensive and effective method of eliminating the potential hinge formation at the base of the flare. Flares may be isolated by cutting the flare steel. However, the designer must ensure that the steel being cut is not necessary for structural integrity, and in any case, the main column reinforcement must not be cut or damaged. If the flare steel is main column reinforcement, other retrofit measures must be used. In addition to cutting the steel, the top four inches of concrete is removed in order to allow the top of the column to rotate freely (Sanchez, 1997). The removal of the concrete will increase the span length of the bent cap and the designer must ensure that the modified bent cap meets service load requirements.

Seismic Isolation

Occasionally, a situation is encountered where physical constraints prevent the use of more conventional measures for retrofitting the substructure of a bridge. In these cases, isolation may be used as an alternate method by reducing the seismic forces transmitted to the substructure from the superstructure and reducing the need for substructure retrofit. However, the force transfer through the isolation device may overload an existing column with poor confinement in which case a substructure retrofit will still be required. Seismic isolation may also be used to improve the mass/stiffness ratio of adjacent frames. However, when using seismic isolators, there must be sufficient clearance between the soffit of the superstructure and the top of the bent cap in order to place the isolators. In addition, the superstructure must be free to move a sufficient amount for the isolators to be effective. The designer is referred to the AASHTO *Guide Specifications for Seismic Isolation Design* for more information (AASHTO, 2014).

Other Retrofit Measures

While these retrofit measures are the most commonly used by Caltrans, there are many other methods available to the designer for retrofitting highway structures. In developing alternative retrofit measures, the designer must ensure that these measures address the vulnerabilities identified in the diagnostic model, and that the retrofitted structure meets the "No Collapse" performance standard. See BDA 14-5 for common seismic vulnerabilities and typical details for common seismic retrofits. Alternative retrofit measures require an exception from Caltrans OEE.

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Original signed by Mark Mahan

Mark Mahan, Chief

Office of Earthquake Engineering, Analysis, and Research

Structure Policy and Innovation

Division of Engineering Services

CALIFORNIA DEPARTMENT OF TRANSPORTATION (Caltrans)

NEW PRODUCT EVALUATION PROCESS FLOWCHART

MARCH 2008

Vendor submits New Product Submittal Package (NPSP) to New Products Coordinator (NPC).

NPC reviews NPSP for completeness, sends acknowledgement letter to vendor, and sends NPSP to appropriate Technical Committee.

If applicable, NPSP is sent concurrently to Office of Health and Safety Services for approval of MSDS. If MSDS is rejected, Safety Committee notifies NPC of **rejection**; NPC notifies vendor and Technical Committee in writing. **FILE CLOSED.**

Technical Committee determines and/or verifies whether specifications exist.

If applicable, NPSP is sent concurrently [by Technical Committee] to Environmental Committee for environmental approval of new product. If new product is rejected, Environmental Committee notifies Technical Committee of **rejection**; Technical Committee notifies NPC; NPC notifies vendor in writing. **FILE CLOSED.**

No specifications exist (Type I).

Technical Committee determines whether Caltrans has a need for product.

Need exists.

Technical Committee notifies NPC; NPC notifies vendor and Lead Division, in writing, that need exists, and that evaluation will be conducted.

Lead Division conducts evaluation.

Product approved.

Lead Division notifies vendor and NPC of **approval** in writing. **FILE CLOSED.**

Lead Division develops or modifies specifications and standards; notifies Caltrans Districts/Divisions of product availability; and adds product to QPL (if a QPL exists for the product category).

Product rejected.

Lead Division notifies vendor and NPC of **rejection** in writing. **FILE CLOSED.**

No need exists.

Technical Committee notifies NPC of **rejection**; NPC notifies vendor in writing. **FILE CLOSED.**

Specifications exist (Type II).

Technical Committee determines whether product meets existing specifications.

Appears to meet specifications.

Technical Committee determines whether evaluation is required.

Evaluation not required.

Technical Committee notifies NPC of **acceptance**; NPC notifies vendor in writing. **FILE CLOSED.**

Vendor presents acceptance letter to contractor.

Evaluation required.

Technical Committee notifies NPC; NPC notifies vendor and Lead Division, in writing, that evaluation is required.

Lead Division conducts evaluation.

Product approved.

Lead Division notifies vendor and NPC of **approval** in writing. Lead Division **adds product to QPL.** **FILE CLOSED.**

Product rejected.

Lead Division notifies vendor and NPC of **rejection** in writing. **FILE CLOSED.**

Does not meet specifications.

Technical Committee notifies NPC of **rejection**; NPC notifies vendor in writing. **FILE CLOSED.**

DIVISION OF ENGINEERING SERVICES

August 2016

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- Project Management tools & training
- Project Delivery Status
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- Independent Quality Assurance of Locally-Funded Highway Bridges
- Structural Technical Expertise for Local Assistance Projects
- Portfolio Management for District 1, 2, 3 & 4 Projects
- Construction contract advertisement, award, & approval
- Construction contracting scheduling, reporting, & funding confirmation
- Construction contract standards development & maintenance
- Construction contract Quality Management (Pre-Award Independent Quality Assurance & Post-Construction Independent Assurance)

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- Represent the Department on AASHTO issues related to bridges & structures
- Specialized Structure Analysis including Seismic Hazard Analysis & Earthquake Engineering
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- Technical support for the design & construction of falsework & shoring
- Constructability plan reviews
- Portfolio Management for Districts 7 & 12 Projects

Materials Engineering & Testing Services & Geotechnical Services
Roberto Lacalle
(Acting)
(916) 227-7254

- Specialized technical expertise & advice regarding all aspects of materials engineering for design, construction, & maintenance
- Assistance to investigate, determine probable cause, & provide alternatives for resolution of pavement failures
- Source inspection for sophisticated electronics systems & electrical products
- Source inspection & material testing related to all structural materials
- Coordination of the Caltrans Independent Assurance Program
- Geotechnical investigations, analysis & recommendations
- Geotechnical Design Reports & Structure Foundation Reports
- Specialized geotechnical engineering & engineering geology services; geotechnical earthquake engineering; field instrumentation; pile testing; soils testing; groundwater, reinforced earth, rock fall, & landslide studies; & and drilling services
- Portfolio Management for Districts 8 & 11 Projects

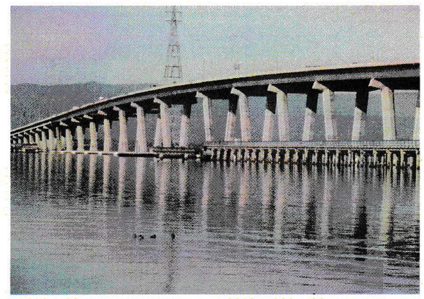
"Provide a safe, sustainable, integrated and efficient transportation system to enhance California's economy and livability."

Earthquake Protection Systems, Inc.

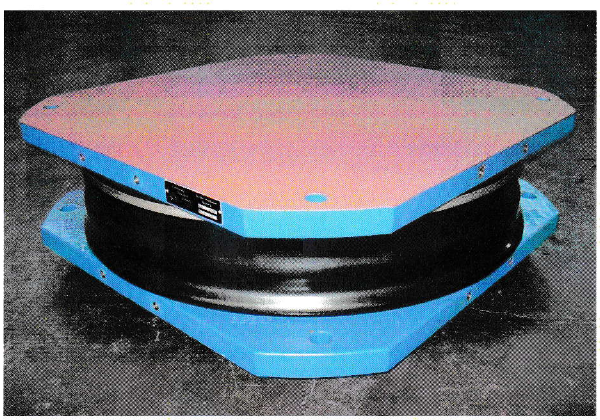
EPS engineers are the world's leading seismic isolation engineering experts. Our state-of-the-art seismic isolation solutions have substantially improved the seismic performance for many of the world's most important seismically isolated structures, while significantly reducing construction costs.

LNG Storage Tanks, Manzanillo, Mexico

Sabiha Gökçen International Airport, Turkey



Dumbarton Toll Bridge, California

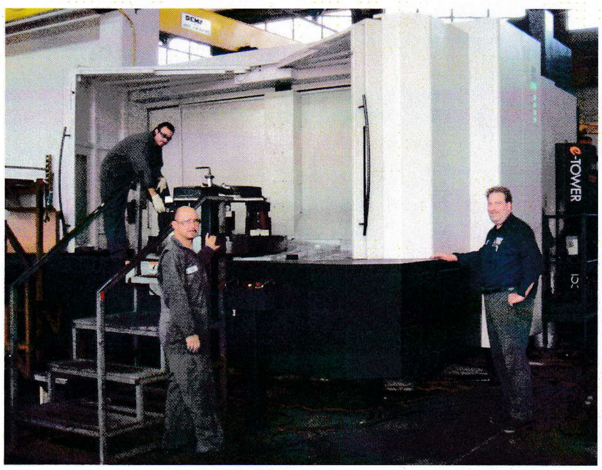


Triple Pendulum™ Bearing

EPS engineers apply advanced seismic isolation engineering to the design and construction of resilient and sustainable structures that retain their ability to function after an earthquake. Our Friction Pendulum™ bearings are the premier seismic isolation products, providing the best seismic performance at the lowest installed cost. Our advanced, Triple Pendulum bearing is a multi-stage bearing that optimizes seismic performance during small, medium and large earthquake events, while reducing the costs of accommodating the largest earthquakes.

Advances in EPS Friction Pendulum™ technology have made it economical to construct facilities that protect contents, non-structural components, and structures from damage during the most severe earthquakes. With EPS engineered and manufactured seismic isolation solutions, new construction can cost less than minimum code designs based on traditional seismic design methods. Consequently, designing for continued functionality is now an economical and practical alternative to code designs that implement only the minimum collapse prevention protection as required by law.

EPS offers the highest engineering qualifications, superior products, the broadest implementation experience, and the most comprehensive manufacturing and testing. EPS has three principal structural engineers, each with more than 25 years experience in the design and implementation of seismic isolation. We offer seismic isolation solutions and provide engineering support services to project engineers to help them evaluate the technical and cost benefits that EPS expert seismic isolation engineering and products can bring to a construction project. We develop application-specific seismic isolation solutions, isolation system designs, and bearing designs with guaranteed prices.



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Fax: (707) 644-5995
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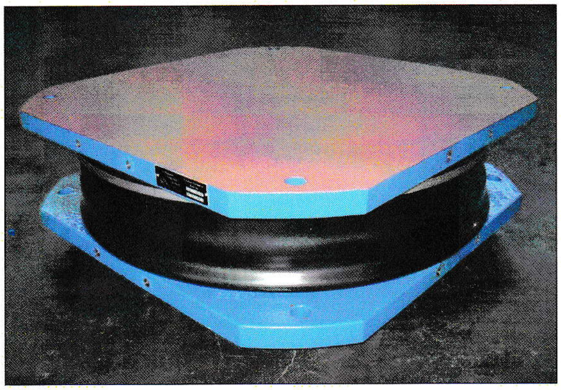
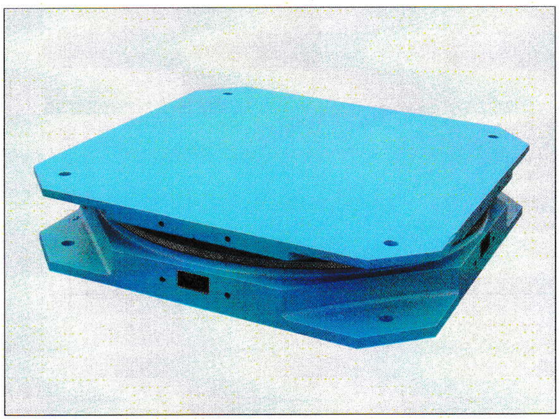
Friction Pendulum™ Seismic

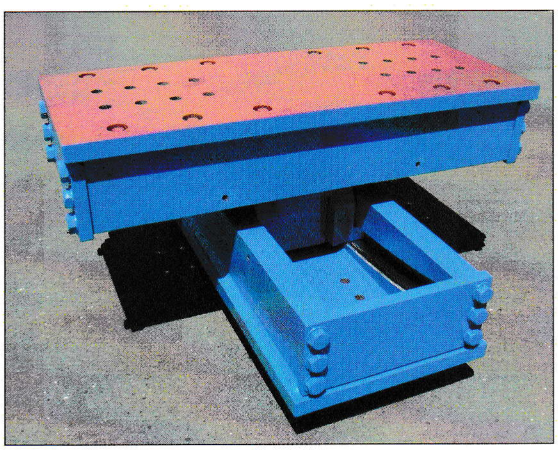
Seismic Isolation Bearings for the protection of buildings, bridges and industrial facilities



Triple Pendulum™ Bearing



Single Pendulum Bearing



Tension Capable Bearing

Friction Pendulum™ bearings are seismic isolators that are installed between a structure and its foundation to protect the supported structure from earthquake ground shaking. Using Friction Pendulum™ technology, it is cost-effective to build structures to elastically resist earthquake ground motions without structural damage.

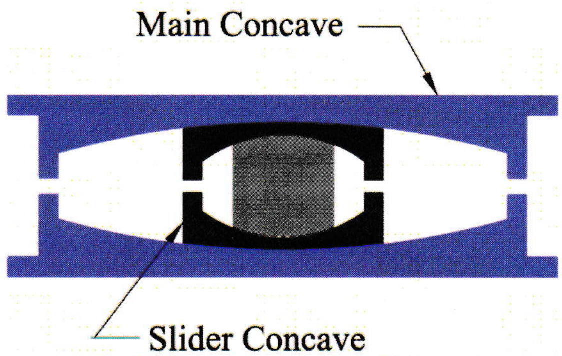
Friction Pendulum™ bearings use the characteristics of a pendulum to lengthen the natural period of the isolated structure so as to avoid the strongest earthquake forces. During an earthquake, the supported structure moves with small pendulum motions. Since earthquake induced displacements occur primarily in the bearings, lateral loads transmitted to the structure are greatly reduced.

The Single Pendulum Bearing is the original Friction Pendulum™ bearing. The single slider maintains the vertical load support at the center of the structural member. This offers construction cost advantages if one structural system is weaker, either above or below the bearing. The bearing also has a low height, which can be advantageous in some installations.

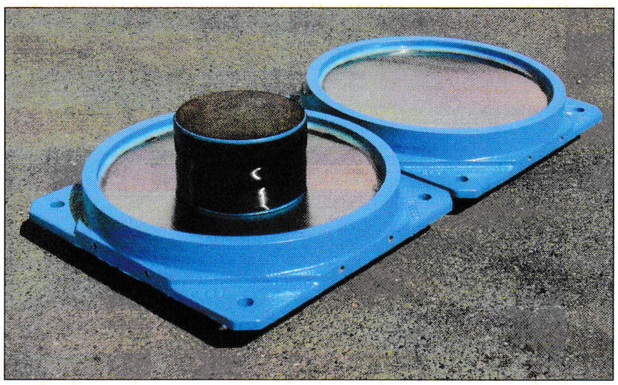
The Triple Pendulum™ bearing incorporates three pendulums in one bearing, each with properties selected to optimize the structure's response for different earthquake strengths and frequencies.

The Tension Capable Bearing can accommodate structure vertical loads that vary from compression to tension during seismic movements. This bearing can substantially reduce structural framing costs by preventing uplift of a primary structural member, and can eliminate concerns regarding potential structure overturning or large vertical earthquake motions.

Triple Pendulum™ Bearing



Cross Section of Triple Pendulum™ Bearing



Concaves and Slider Assembly

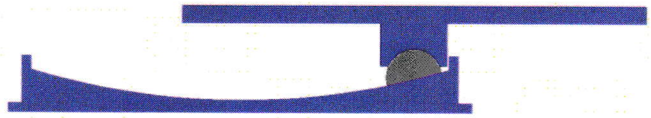
Concaves and Slider Components

The Triple Pendulum™ bearing offers better seismic performance, lower bearing costs, and lower construction costs as compared to conventional seismic isolation technology. The properties of each of the bearing's three pendulums are chosen to become sequentially active at different earthquake strengths. As the ground motions become stronger, the bearing displacements increase. At greater displacements, the effective pendulum length and the effective damping increase, resulting in lower seismic forces and bearing displacements.

The Triple Pendulum™ bearing's inner isolator consists of an inner slider that slides along two inner concave spherical surfaces. Properties of the inner pendulum are typically chosen to reduce the peak accelerations acting on the isolated structure and its contents, minimize the participation of higher structure modes, and reduce structure shear forces that occur during service level earthquakes.

The two slider concaves, sliding along the two main concave surfaces, comprise two more independent pendulum isolators. Properties of the second pendulum are typically chosen to minimize the structure shear forces that occur during the design basis earthquake. This reduces construction costs of the structure. Properties of the third pendulum are typically chosen to minimize bearing displacements that occur during the maximum credible earthquake. This reduces the size and cost of the bearings, and reduces the displacements required for the structure's seismic gaps.

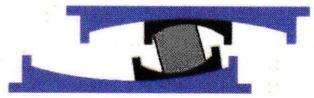
Single Pendulum Bearing
Cross Section



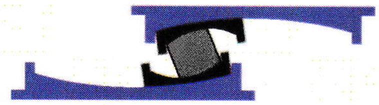
Single Pendulum Bearing
Maximum Credible Earthquake

Triple Pendulum Bearing
Center Position

Inner Pendulum Motion
Service Level Earthquake

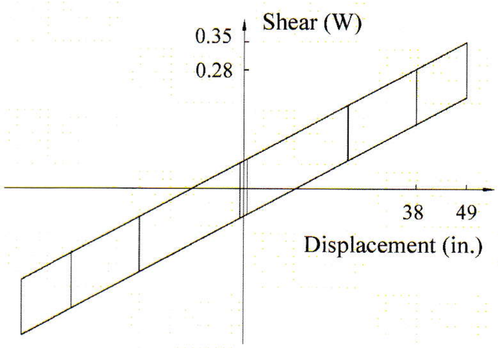


Lower Pendulum Motion
Design Basis Earthquake

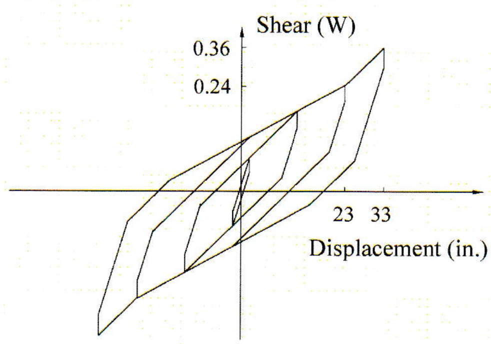


Upper Pendulum Motion
Maximum Credible Earthquake

Comparison of Triple Pendulum and Single Pendulum Bearing Sizes and Responses to Earthquake Motion



Single Pendulum Force-Displacement Hysteretic Loop



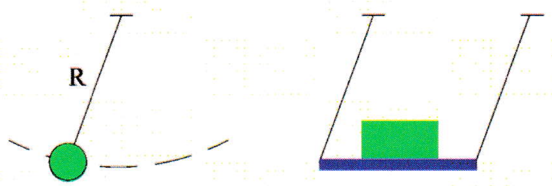
Triple Pendulum Force-Displacement Hysteretic Loop

The Single Pendulum bearing maintains constant friction, lateral stiffness, and dynamic period for all levels of earthquake motion and displacements. In the Triple Pendulum™ bearing, the three pendulum mechanisms are sequentially activated as the earthquake motions become stronger. The small displacement, high frequency ground motions are absorbed by the low friction and short period inner pendulum. For the stronger Design Level Earthquakes, both the bearing friction and period increase, resulting in lower bearing displacements and lower structure base shears. For the strongest Maximum Credible Earthquakes, both the bearing friction and lateral stiffness increase, reducing the bearing displacement. When designed for a severe Maximum Credible Earthquake, the plan dimensions of the Triple Pendulum™ bearing are approximately 60% that of the equivalent Single Pendulum bearing.

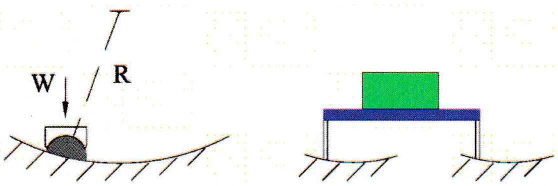
Principles of Friction Pendulum™ Seismic Isolation

The period of the Friction Pendulum™ bearing is selected simply by choosing the radius of curvature of the concave surface. It is independent of the mass of the supported structure. The damping is selected by choosing the friction coefficient. Torsion motions of the structure are minimized because the center of stiffness of the bearings automatically coincides with the center of mass of the supported structure. The bearing's period, vertical load capacity, damping, displacement capacity, and tension capacity, can all be selected independently. For the Triple Pendulum™ bearing, three effective radii and three friction coefficients are selected to optimize performance for different strengths and frequencies of earthquake shaking. This allows for maximum design flexibility to accommodate both moderate and extreme motions, including near-fault pulses.

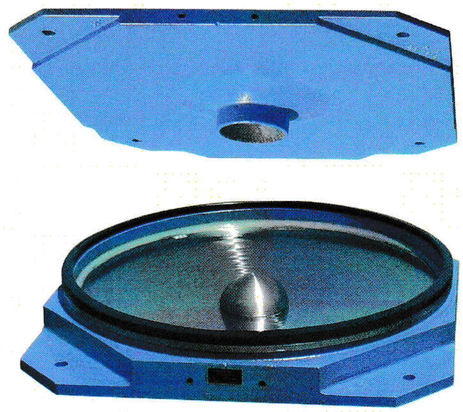
Period $T=2\pi\sqrt{R/g}$ Stiffness $K=W/R$



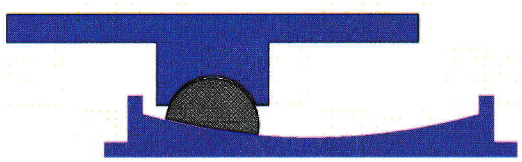
Pendulum Motion



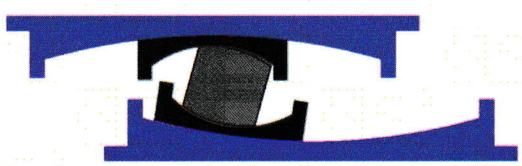
Sliding Pendulum Motion



Concave, Slider & Housing for Single Pendulum Bearing



Single Pendulum Operation



Triple Pendulum™ Operation

20 million lbs. Vertical Load Capacity Concave & Slider

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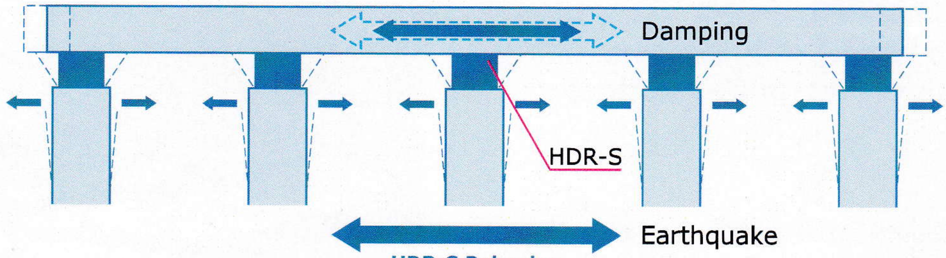
Website: www.earthquakeprotection.com

Performance

The HDR-S has larger deformation capability. Its long-term serviceability and highly reliability are verified by various performance tests.



HDR-S Behavior

The HDR-S is an ideal seismic device with its restoring performance by rubber spring, high damping effect and high durability as well as its environmentally friendliness. The characteristics of HDR-S bearings are shown in the following figures (Table-1,2 and 3).

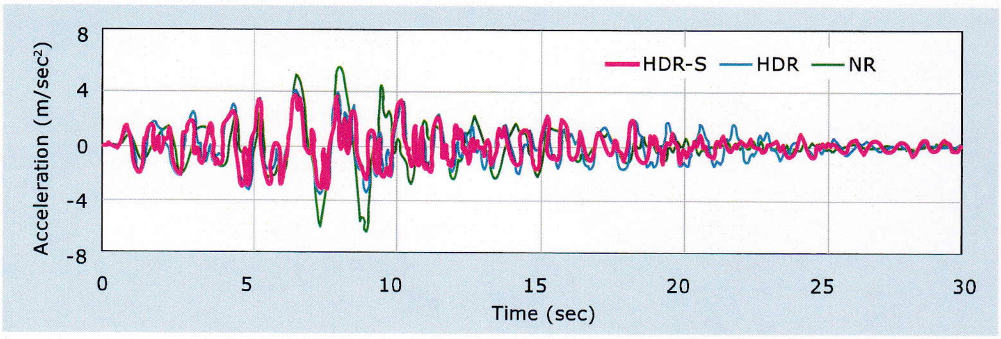


Figure-1. Acceleration of Superstructure

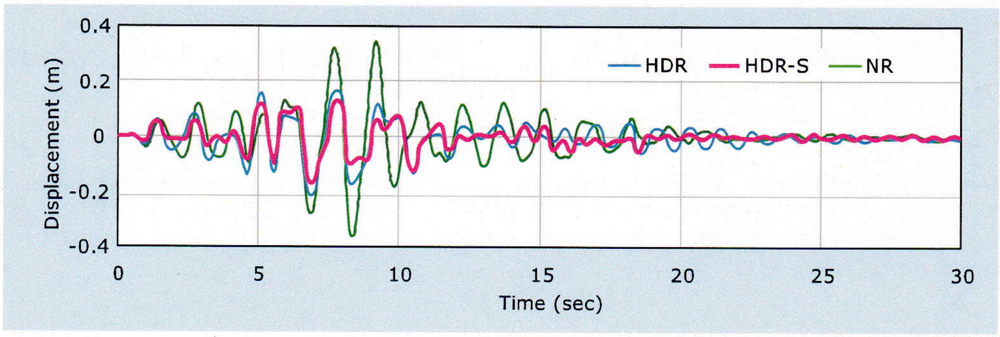


Figure-2. Superstructure Displacement

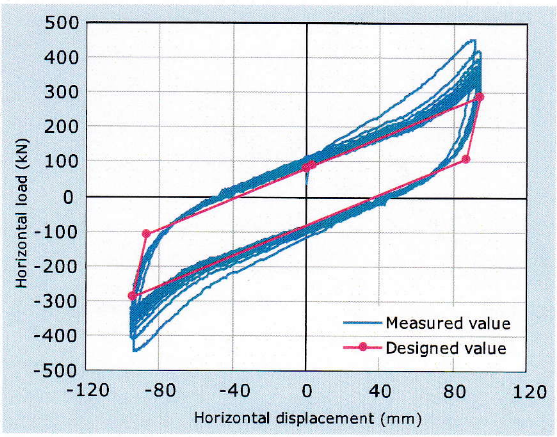


Figure-3. Hysteretic Curve

Fracture Test

Kawakin

Kawakin Core-Tech Co., Ltd.

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Tel. +81-48-259-1117 Fax. +81-48-259-1137
Branch/Plant : Ibaraki / Tokyo / Osaka / Sendai / Fukuoka / Sapporo
Representative Office : Hanoi (Vietnam)

Tomorrow's Technology, Today.

Kawakin Holdings Group

Issued 04/13

HDR-S Super-High Damping Rubber Bearing

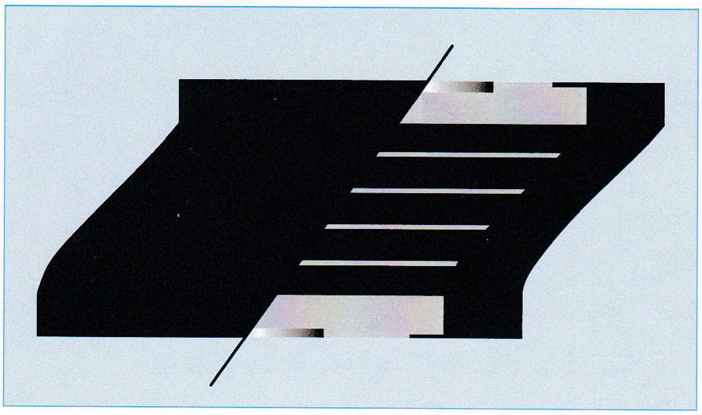
Kawakin

Fields of Application

Suitable for a variety of bridges including highway bridges and railway bridges

Function

- **Supports vertical loads**
- **Superior damping effect**



Sheared HDR-S Bearing Section

◎ High Performance

- Super high damping performance
- Stable performance against repeated deformation by large-scaled earthquakes

◎ High Quality

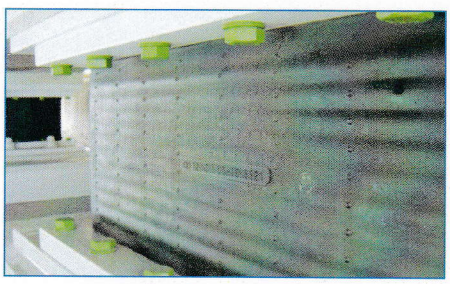
- Accurate quality control by rigorous performance tests

◎ Size Line-up

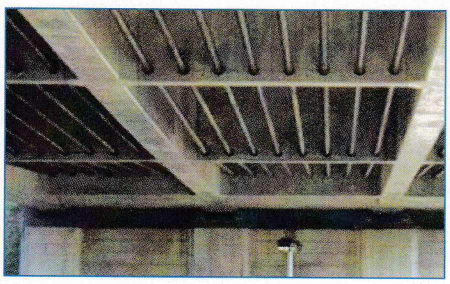
- A broad line-up of HDR-S bearings for supporting small to large vertical loads
- High vertical load capacity of as large as around 20,000 kN

The Super High Damping Rubber Bearing (HDR-S) is an improved version of High Damping Rubber Bearings (HDR). Its damping performance is 20% higher than that of HDR. It consists of the same alternate layers of rubber plates and steel plates as the HDR. The steel plates are installed to prevent rubber bulging and provide high vertical stiffness, while horizontal stiffness is controlled by low elastic shear modulus of the rubber.

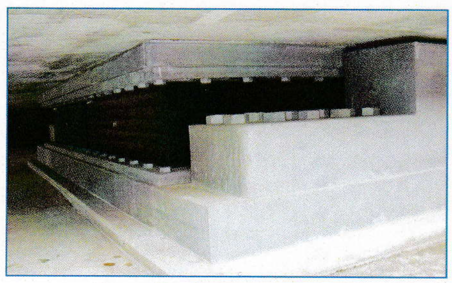
Application Examples



Product Detail



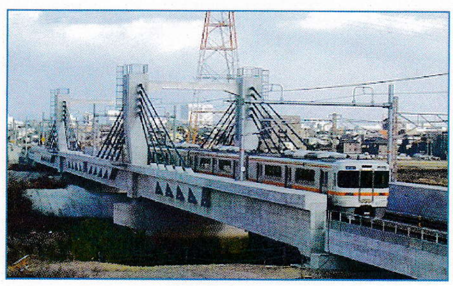
Installation Example-1



Installation Example-2

Imokawa Bridge

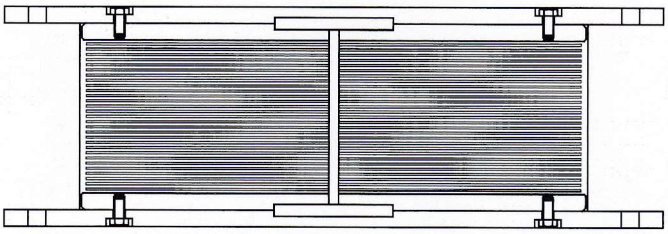
Tenpaku Bridge



Sakaigawa Bridge

KNRH Laminated Natural Rubber Bearing

KNRH is a rubber bearing capable of supporting high vertical load. It effectively protects the building from earthquakes by preventing the transmission of seismic forces to the structure.



Compression shear test

»Distinctive characteristics

Flexibility

KNRH is installed in combination with energy dissipation devices, which increases its design flexibility.

Durability

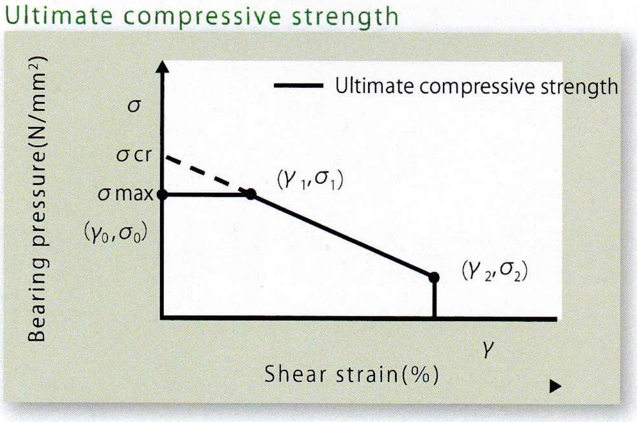
The capability of KNRH to maintain its high-performance over time was evaluated trough aging accelerated tests equivalent to 60 years of service. The outstanding performance of KNRH in the durability test (less than 10% degradation of the shear deformation characteristics, and 6% creep rate) demonstrates the reliability of the device.

Stability

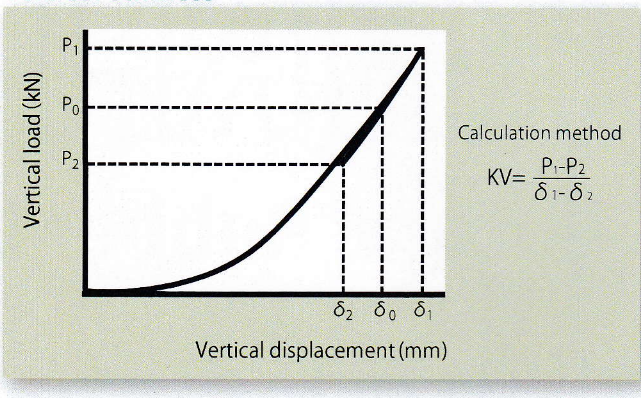
KNRH shows a stable deformation under high vertical compressive force.

»Performance

Ultimate compressive strength



Vertical stiffness



»Corrosion protection

Galvanizing

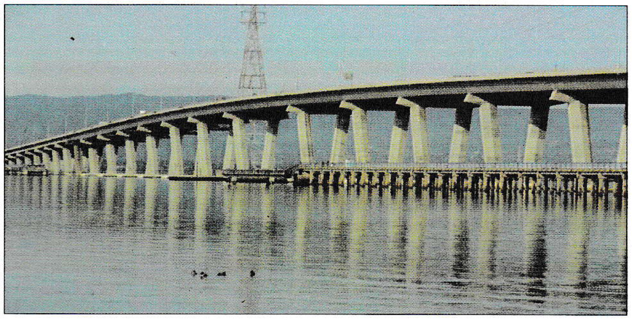
Part	Specification
Steel material	Hot-dip galvanizing (HDZ55)
Bolt	Galvanized bolt

Paint coating

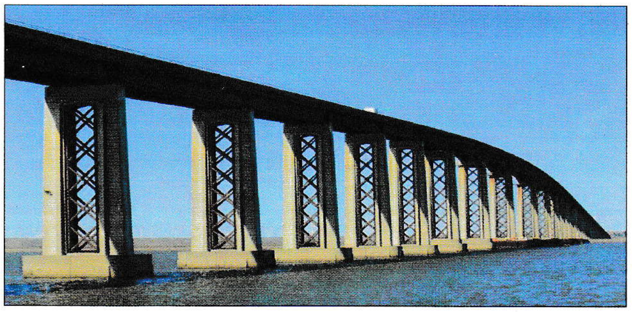
Part	Specification	
Steel material	Surface preparation	Blasting SSPC-SP-10(SIS Sa-2 1/2)
	Undercoating	Zinc-rich primer over 75μ
	Second coating	Epoxy resin paint over 60μ
	Final coating	Epoxy resin paint over 35μ

Dumbarton and Antioch Toll Bridges
San Francisco Bay

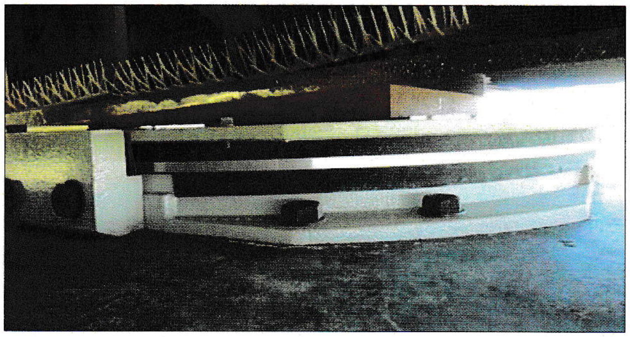
Seismic retrofit design of major toll bridges relies on Friction Pendulum™ bearings



Dumbarton Bridge



Antioch Bridge



Antioch Bridge Installed Bearing

Caltrans and the Bay Area Toll Authority (BATA) evaluated the Dumbarton and Antioch Toll Bridges for seismic vulnerability and concluded that they required seismic retrofit work to make them safe during a major earthquake. Together these two bridges carry 70,000 vehicles a day and are important transportation links in the San Francisco Bay Area.

Friction Pendulum™ Bearings are used to retrofit the elevated spans of the main channel crossing. Low profile, Single Pendulum Bearings were designed to best fit into the existing spaces and still allow 26 inches and 42 inches of displacement capacity for the Antioch and Dumbarton Bridges, respectively. Prototype bearings were extensively tested at EPS to verify their properties and these properties were confirmed by independent testing at the University of California, San Diego SMRD test facility. Installation of the bearings on top of the bent caps of these steel girder bridges minimizes the required strengthening of the columns and eliminates any retrofitting of the foundations beneath the water level.

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May 2013