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Session C2

Seismic Strengthening of Bridges: Case Studies
Renforcement parasismique des ponts: études de cas
Seismische Verstärkung von Brücken: Fallstudien

Experience with Seismic Retrofit of Major Bridges

Expérience acquise dans la consolidation parasismique de grands ponts

Erfahrungen bei der Verstärkung von grösseren Brücken gegen Erdbebeneinwirkung

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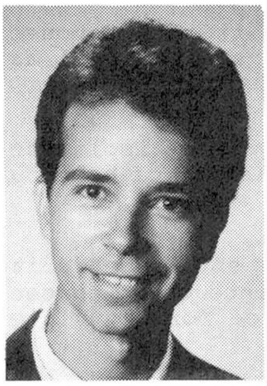

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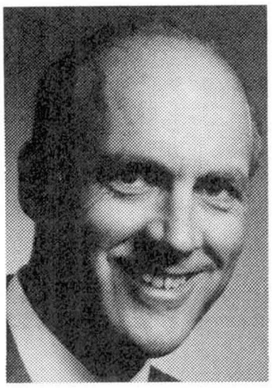

Peter Taylor, born in 1939, received his Ph.D. in civil engineering in 1965 from the University of Bristol. Since 1970 he has specialized almost exclusively in bridge engineering, and in the past ten years he has developed specific expertise in the seismic retrofitting of bridges.

SUMMARY

This paper describes aspects of seismic assessment and retrofit of four major bridges, one in California and three in Vancouver. Each bridge posed a different set of seismic problems and required a different set of retrofit solutions. Items of interest are the use of innovative analyses and their benefits, the implementation of innovative retrofit designs and their benefits, the prioritization of individual retrofit items, and the benefit of retrofitting over several years.

RÉSUMÉ

Cet article présente différents aspects de l'évaluation et de la consolidation parasismique de quatre grands ponts situés en Californie et à Vancouver. Lors de l'évaluation, chacun de ces ponts présenta des problèmes aséismiques différents et, conséquemment, requit des solutions différentes. Les principaux éléments d'intérêt présentés sont l'utilisation de méthodes d'analyse nouvelles ainsi que leurs avantages, l'emploi de concepts innovateurs pour la consolidation ainsi que leurs avantages, le traitement des éléments selon leur ordre d'importance ainsi que les avantages associés à la répartition dans le temps des travaux de réfection.

ZUSAMMENFASSUNG

Der Beitrag beschreibt einige Gesichtspunkte der Widerstandsbestimmung gegen Erdbebeneinwirkungen von vier größeren Brücken und die vorgeschlagene Verstärkung derselben. Eine der vier Brücken befindet sich in Kalifornien und drei sind in Vancouver, Kanada. Jede der vier Brücken stellte besondere seismische Probleme und benötigte eine spezifische Lösung für die Verstärkung. Speziell erwähnt werden neuartige Berechnungsmethoden für die Analyse und deren Nützlichkeit, die Ausführung von neuartigen Verstärkungsdetails und deren Vorteile, die Prioritätssetzung für verschiedene individuelle Verstärkungsdetails, und die Vorteile der Verstärkungsausführung über mehrere Jahre.

1. INTRODUCTION

This paper describes aspects of seismic assessment and retrofit of four major bridges, one in California and three in Vancouver Canada. They are:
Golden Gate Bridge South Approach, San Francisco, and
Burrard Street Bridge,
Granville Street Bridge, and
Second Narrows Bridge, Vancouver Canada.

Each bridge posed a different set of seismic problems and required a different set of retrofit solutions. Items of interest are the use of innovative analyses and their benefits, the implementation of innovative retrofit designs and their benefits, the prioritization of individual retrofit items, and the benefit of retrofitting over several years.

Analysis techniques used for the different structures included linear response spectrum analyses, both linear and non-linear time history analyses, as well as non-linear push over analyses.

Retrofits included member strengthening, installation of dynamic isolation bearings, the use of base isolation with friction dampers, installation of bearing restrainers and keepers, placement of bumpers, and bearing seat extensions.

Retrofit prioritization ranged from design and construction of the entire retrofit in one package for Golden Gate, to breaking the retrofit package into three phases and spreading the design and construction over a seven year period for Granville and Burrard.

The Golden Gate Bridge, located in San Francisco CA, is a 6-lane steel suspension bridge. The south approaches consist of a 215m long elevated steel viaduct, two 60m high concrete pylons, and a 100m long arch. The bridge is owned by the Golden Gate Bridge, Highway & Transportation District and was constructed in the mid 1930's. Figure 1 shows an elevation of the bridge.

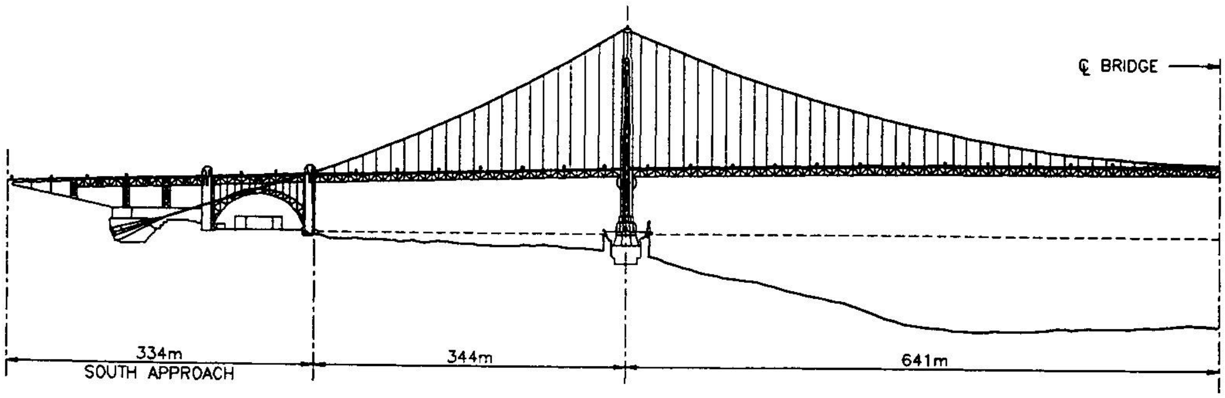

Fig.1 Golden Gate Bridge

The Burrard Street Bridge, located in Vancouver BC, is a 6-lane steel truss bridge with a main channel span of 90m. The bridge consists of 330m of steel spans and 510m of concrete approaches. The structure is owned by the City of Vancouver and was constructed in 1930. Figure 2 shows an elevation of the bridge.

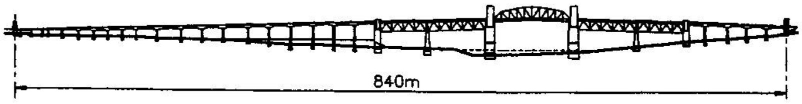

Fig.2 Burrard Bridge

The Granville Street Bridge, located in Vancouver BC, is an 8-lane steel truss bridge. The main channel span is 120m, and the bridge comprises 540m of steel spans and 630m of concrete approach spans. The structure is owned by the City of Vancouver and was constructed in 1950. Figure 3 shows an elevation of the bridge.

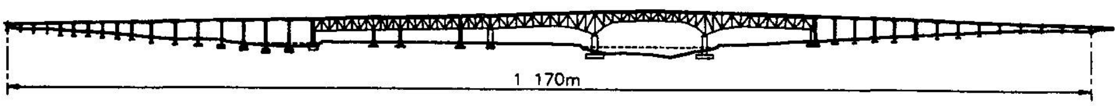

Fig.3 Granville Bridge

The Second Narrows Bridge, located in Vancouver BC, is a 6-lane steel truss bridge. The main channel span is 335m, and there is a total of 965m of steel spans and 330m of concrete stringer approaches. The structure is owned by the Province of British Columbia and was constructed in 1958. Figure 4 shows an elevation of the bridge.

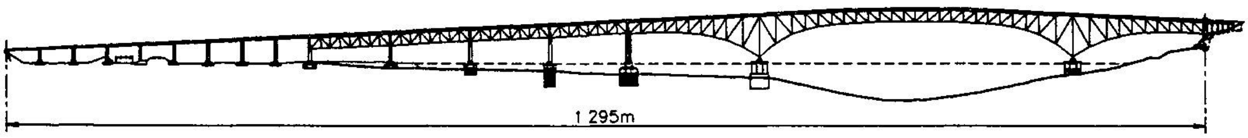

Fig.4 Second Narrows Bridge

2. SEISMIC CRITERIA

Seismic retrofit criteria varies with regional geology and with bridge authorities. Regional geology affects the size of the design earthquake, which is impacted by both the proximity to and the type of major faults in the region. Bridge authorities determine the level they wish to upgrade their structures to. A decision has to be made between retrofitting to a safety level or a functional level. A safety retrofit is meant to be the minimum upgrade required to avoid collapse of the bridge during a major earthquake, with significant repairs or possibly even replacement needed after the earthquake. A functional retrofit is a more significant upgrade that allows the bridge to remain operational after the design quake, with only minor repairs needed. Other factors affecting retrofit decisions are the importance of the bridge, the availability of alternate routes, and the amount of budget available.

The Golden Gate Bridge is a major structure in the San Francisco area, it is close to both the San Andreas and Hayward faults, and it is a major element in the regional transportation infrastructure. As such, the District required a functional retrofit to a level that would require only minor repairs following a Richter Magnitude 8.3 earthquake on the San Andreas fault, only 10km from the bridge site.

The City of Vancouver, however, decided that the Burrard and Granville Bridges would be upgraded to the safety retrofit level in stages. The stages were chosen based on priorities, and once complete, the bridges are to be re-evaluated for a functional level retrofit.

3. SEISMIC ASSESSMENT

3.3 Golden Gate Bridge

The south viaduct of the bridge was analyzed using linear response spectrum, linear time history, and fully non-linear time history analyses. Virtually all of the bracing in the steel superstructure was found to be deficient, as were many of the riveted connections. The steel supporting towers were also found to be lacking adequate strength and ductility. The existing bearings and their seat lengths were grossly deficient and were of major concern.

3.2 Burrard Street Bridge

The similar soil conditions along the length of the Burrard Bridge made the use of a linear response spectrum analysis appropriate. The lateral and longitudinal shear demands in the main piers were found to be a major problem. The lateral tension only bracing in the steel trusses was also deficient. The approach piers had a more moderate shear problem. Their lack of lateral bending capacity and ductility due to inadequate splices of undeformed reinforcing bars and inadequate transverse reinforcement was a concern. Bearing seat lengths in the approaches was also a problem.

3.3 Granville Street Bridge

As was the case with Burrard, the soil conditions made the use of a linear response spectrum analysis appropriate for the assessment of the bridge. The major deficiencies in the bridge were the short bearing seat lengths throughout the structure and the inadequate shear capacity of many of the concrete bent caps. The lateral bracing and the approach bents were a lesser problem, though still inadequate. The clearances between adjacent truss spans was inadequate, and pounding of the trusses was identified as a problem.

3.4 Second Narrows Bridge

Linear response spectrum analysis was initially used to assess the maximum member forces in the Second Narrows Bridge. However, the south end of the bridge is founded on rock while the north is on deep gravel deposits. The linear response spectrum analysis was too conservative since the spectrum, used for the entire structure, was amplified to account for the soil effects of the north end of the bridge. As a result, a linear time history analysis was used with soil amplification at the north side piers only. In addition, softening stiffness was used for overloaded bracing members from the instant of overload, allowing dynamic redistribution of the bracing load. The problems identified were steel bracing overload, inadequate bearing travel lengths, excessive shear in the bents of the approaches, and liquefaction in the deep gravels. In this case, the more sophisticated analysis was justified, and resulted in a better understanding of the bridges dynamic behaviour. The result was a retrofit whose cost was approximately 20% less than the cost of a retrofit based on a linear spectral analysis.

4. SEISMIC RETROFITS

4.2 Golden Gate Bridge

In the south viaduct, the bearings between the superstructure and the support towers are to be replaced with dynamic isolation bearings. In order to provide some redundancy and also to minimize the number of expensive deck expansion joints, all six of the viaduct spans are to be linked together axially. In addition, two of the main support towers are to be completely replaced.

By linking the spans together and installing isolation bearings, the seismic demands in the steel superstructure were reduced to low enough values that very minor additional retrofit is required.

4.2 Burrard Street Bridge

All of the truss span bearings were replaced with lead core dynamic isolation bearings to reduce the shear demands in the massive concrete piers. New lateral diaphragms were added to the approaches to provide a reliable load path for the seismic forces from the deck to the piers. The approach piers were allowed to rock at the soil level in the longitudinal direction. This resulted in large relative displacements at the level of the bearings so bearing restrainers were added throughout the approaches. In the transverse direction, a limited number of piers were strengthened to carry all of the seismic load.

Through the use of dynamic isolation bearings and pier rocking, the seismic input into the bridge was reduced to acceptable levels. In addition, these two strategies resulted in a more robust retrofit, elevating the retrofit to that of a functional retrofit for a very minor cost increase.

The retrofit chosen required only a linear response spectrum analysis, using secant stiffnesses to model the bearings and a reduced spectrum to capture the effect of the added damping.

4.3 Granville Street Bridge

In the truss spans of the bridge, the lateral bracing was strengthened and cable restrainers were added at the bearings. Large bumpers were added between adjacent trusses to reduce the impact loads associated with the trusses pounding into each other. The cap beams of the bents that support the truss spans were encased with new post-tensioned concrete.

The retrofit chosen for the Granville approaches was to isolate the approach superstructure from the supporting piers. Isolation bearings, similar to those used on Burrard and Golden Gate, could not be used here due to lack of clearance height between the substructure and the superstructure. Instead, sliding bearings free in the horizontal plane were used to replace the existing bearings and friction dampers were installed between the sub and superstructures. This, however, necessitated the use of a non-linear time history analysis for the final assessment of the approaches. The bearings and friction dampers were modelled as friction-spring-damping elements in this analysis.

The use of isolation and damping dramatically reduced the demands in the substructure, allowing for a modest retrofit in the piers. This consisted of wrapping the columns with fibreglass to increase confinement and ductility, and post-tensioning some of the bent caps.

4.4 Second Narrows Bridge

The bearings in the approach trusses are to be replaced with dynamic isolation bearings, and the main span bearings are to be constrained laterally. The bent caps of the approach piers are to be retrofit using post-tensioning for the caps and concrete encasement at the base of the columns. In addition, the bents will be allowed to rock. The bearings in the concrete girder approaches will also be retrofit to increase their performance during an earthquake.

Isolation of the main truss span was investigated, however a strength retrofit was chosen for several reasons. The large scale of the bearings did not encourage replacement. In addition, a replacement isolation system would have had to be relatively stiff to carry non-seismic loads elastically, thereby providing little benefit to the superstructure or the short supporting piers. Therefore, replacement of the bearings was not considered cost effective. However, since the main bearings were not expected to perform well in a seismic event, constraints will be added to maintain the bearings integrity. For the truss approach spans, the isolation system not only reduced the demands in the superstructure, but also in the much taller supporting piers, so it was cost effective and therefore will be installed.

Soil densification of the north piers was investigated, however the cost of densification around all of the piers was very high. A compromise was reached that accepted the risk of liquefaction around the approach piers supporting the concrete spans, but not around the main piers or the approach piers supporting the steel truss spans. The existence of very long seat lengths for the concrete approach span bearings made it unlikely that a span would be lost in the event of liquefaction, which ensured structural integrity.

5. PRIORITIZATION OF RETROFITS BASED ON RISK AND COST

During the seismic assessment stage for the Burrard, Granville and Second Narrows bridges, a semi statistical cost-risk-benefit analysis was performed on all of the retrofit items. This process resulted in each retrofit item receiving a high, medium, or low priority. As an example, a low cost item that resulted in a high benefit was given a high priority. The high priority items were packaged into phase one of the design, the medium priority items into phase two, and the low priority items into phase three.

In addition to spreading the cost of retrofitting these bridges over several years, the added time allowed the design engineers to benefit from "current thinking". Since the field of earthquake engineering is developing rapidly, the methods of assessing bridges for seismic deficiencies are continually evolving. When first assessed with linear techniques, the approach piers of Granville and Burrard bridges were highly over stressed in both bending and shear. Due to the large cost of retrofitting all of these piers, their upgrade was delayed until the final phase of the retrofit. When the final design of phase three was undertaken, the profession's understanding of pier behaviour had developed significantly. The re-evaluation of the piers used non-linear push over analyses and this indicated much less of a problem than was originally identified. In addition, recent testing of pier jacketing has shown that wrapping deficient columns is an effective and inexpensive retrofit measure, which through its use, further reduced the retrofit cost below that originally anticipated.

6. CONCLUSIONS

The engineering profession's understanding of how bridges behave in earthquakes is increasing every year. The analysis methods being used to assess existing structures are becoming more sophisticated, and the tools and remedies available to the retrofit designer are more and more detailed. With careful consideration of what is important, and a good understanding of how structures behave dynamically, designers can develop innovative retrofit methods with the use of sophisticated analysis techniques. However, as was the case with the Burrard Bridge, the use of these sophisticated analysis tools is not always warranted.

In addition, prioritization of retrofit items allows owners to spread the cost of retrofitting major bridges over several years. This also gives the designers an opportunity to gain valuable experience and avail themselves of current research and testing results for the more difficult and expensive portions of the retrofit.

Seismic Performance of Long-Span Suspension Bridges in the USA

Comportement sismique des ponts suspendus de grande portée , USA

Seismische Leistung von weitgespannten Hängebrücken, USA

Subcommittee on Seismic Performance of Bridges
ASCE Structural Division
New York, NY, USA

The subcommittee, started in 1993, reports on the performance of bridges in earthquakes. In addition to reporting on long-span bridges, reports are planned on the performance of bridges in the Northridge Earthquake of January 17, 1994, and the Kobe Earthquake of January 17, 1995.

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SUMMARY

Of the 117 long-span suspension bridges built in the USA between 1801 and 1994, 57 bridges are in service and 60 are not. The bridges have performed well in earthquakes. The 57 bridges in service, from 6 to 140 years in age, have main span lengths between 126 and 1299 meters. Seismic evaluation studies have started on some of the 57 bridges with the objective of predicting performance, assessing vulnerabilities, and recommending retrofit of deficient bridges.

RÉSUMÉ

Des 117 ponts suspendus à grande portée construits aux États-Unis depuis 1801, 57 sont actuellement en service et 60 ne le sont plus. Ces ponts suspendus se sont bien comportés sous l'action sismique. Les 57 ponts en service aujourd'hui ont de 6 à 140 ans, des portées entre 126 et 1299 mètres. Des études pour l'évaluation sismique ont déjà commencé pour quelques ponts afin de prévoir leur comportement sismique et déterminer leur vulnérabilité et pour en recommander la consolidation si nécessaire.

ZUSAMMENFASSUNG

Von 117 weitgespannten Hängebrücken, die in den U.S.A. zwischen 1801 und 1994 gebaut wurden, sind 57 noch in Betrieb und 60 nicht mehr. Die Brücken haben sich bei Erdbeben gut bewährt. Die 57 Brücken, die zwischen 6 und 140 Jahren alt sind, haben eine Spannweite des Hauptteils zwischen 126 und 1299 Metern. Seismische Beurteilungsstudien an einigen der 57 Brücken wurden begonnen mit dem Ziel, Leistungen zu prognostizieren, Anfälligkeiten zu beurteilen und Empfehlungen in Bezug auf eine Wiederherstellung von defekten Brücken anzubieten.

1. INTRODUCTION

This paper summarizes a work-in-progress report on the seismic performance of 117 long-span highway, railroad, and pedestrian suspension bridges built in the United States between 1801 and 1994. The report, based on available information, is under review by the subcommittee. A long-span suspension bridge is defined as one where the length of the main span is ≥ 122 meters. Fig. 1 shows the components of a suspension bridge. Of the 117 bridges, 57 bridges are in service and 60 are not. Of those not in service, 22 bridges were closed, replaced, or destroyed for reasons other than damage in earthquakes such as -- corrosion of the suspension cables, snow/ice overloads, wind storms and floods; inadequate load carrying capacity or general deterioration. At present, no information is available as to why the remaining 38 bridges are no longer in operation. However, a survey of available literature indicates no mention of poor performance in earthquakes. Table 1 lists data on the 57 bridges, in service, and Fig. 2 shows the number of such bridges in each state.

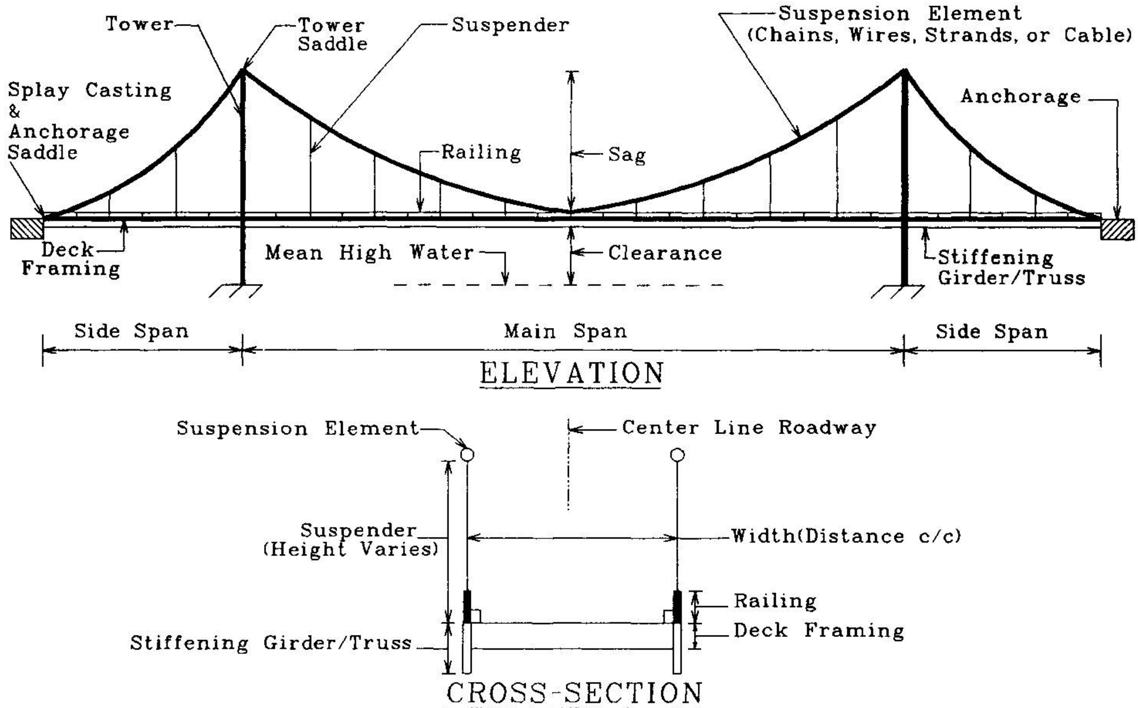


Fig. 1 Components of a Suspension Bridge

2. PERFORMANCE

The 57 bridges, in service, have performed well in earthquakes -- there are no reported failures. The majority of these bridges were neither designed to resist seismic forces nor subjected to strong (magnitude 6.0-6.9) or major (magnitude 7.0-7.9) earthquakes. Unless a bridge is shown to be safe, based on a seismic evaluation study, past performance should not be assumed to insure similar performance in the future. The subcommittee report documents: recorded performance of instrumented bridges, observed performance of non-instrumented bridges, predicted performance based on computer models, and simulated performance of laboratory models. The 31-year-old Vincent Thomas Bridge, located in Los Angeles, California, and instrumented with 26 strong-motion sensors, has performed well in earthquakes. Tiltmeters installed on the piers of the Golden Gate Bridge, in San Francisco, California, show that this 57-year-old bridge has not displaced as a result of settlement, scour, or earthquakes. The bridge performed well in the 1989 Loma Prieta earthquake, moment magnitude M_w=7.0, since the bridge site experienced a peak ground acceleration of only 8% of gravity. However, computer models have predicted that a major earthquake on a nearby segment of the San Andreas or Hayward faults, with a peak ground acceleration of 65% of gravity, would cause severe damage to the Golden Gate Bridge.

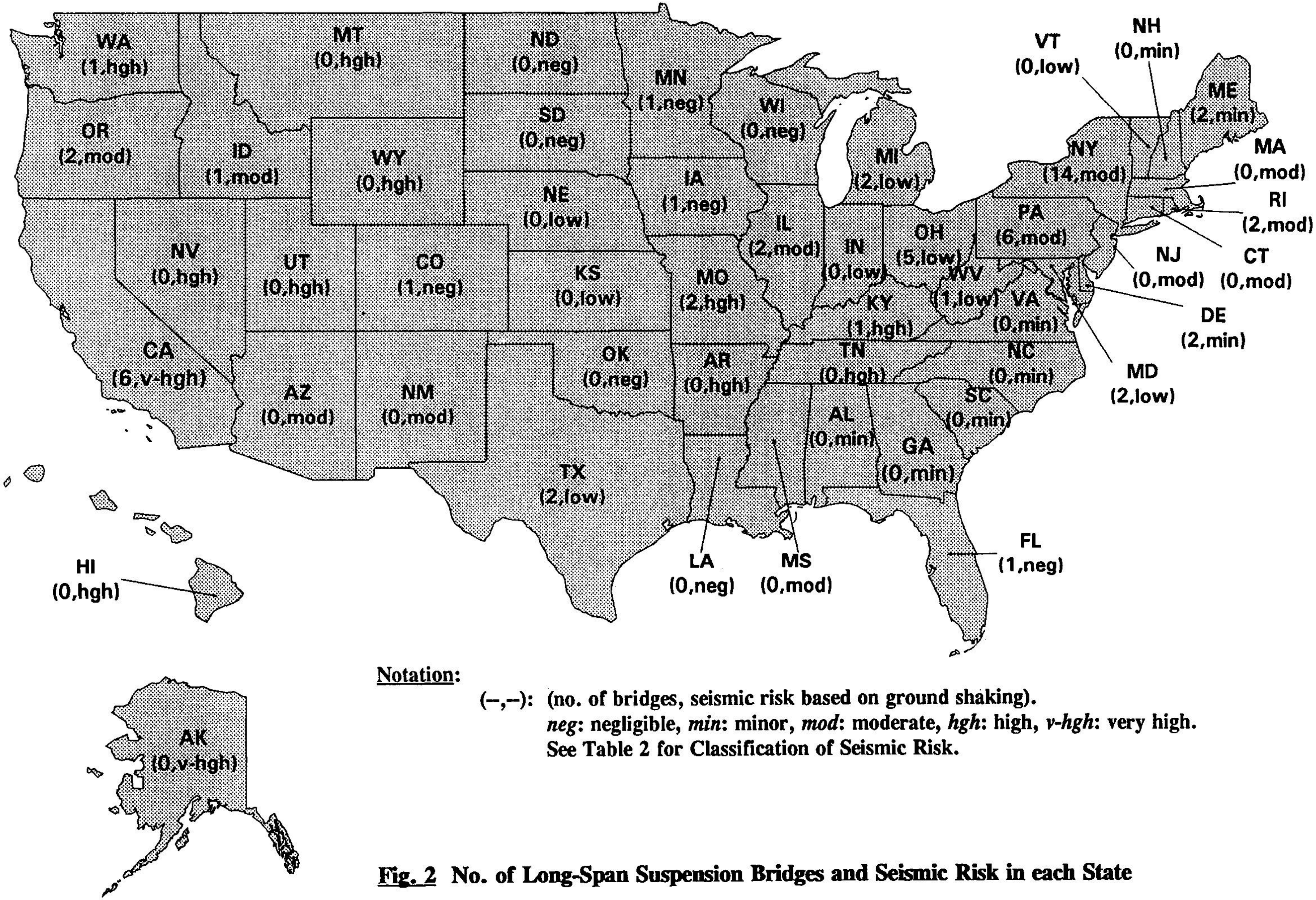
Name of Bridge	Location (state)	Opened to Traffic in	Main Span (meters)	Age as of 1994 (years)	Seismic Risk
San Francisco-Oakland Bay	California *(6)*	1936	704	58	Very High *(6)*
Golden Gate		1937	1280	57	
Vincent Thomas		1963	457	31	
Bidwell Bar		1965	338	29	
Klamath River II		1967	131	27	
Guy A. West		1968	183	26	
Maysville	Kentucky	1931	323	63	High *(4)*
Grand Auglauze	Missouri *(2)*	1920	126	74	
Missouri River		1954	188	40	
Tacoma Narrows II	Washington	1950	854	44	
Dent	Idaho	1972	320	22	Moderate *(27)*
Davenport	Illinois *(2)*	1935	226	59	
Missouri River		1956	196	38	
Brooklyn	New York *(14)*	1883	486	111	
Williamsburg		1903	488	91	
Manhattan		1909	448	85	
Kingston-Poughkeepsie		1922	215	72	
Bear Mountain		1924	498	70	
Mid-Hudson		1930	457	64	
George Washington		1931	1067	63	
Triborough		1936	421	58	
Thousand Islands-USA		1938	244	56	
Bronx-Whitestone		1939	701	55	
South Channel		1958	273	36	
Ogdensburg-Prescott		1960	351	34	
Throgs Neck		1961	549	33	
Verrazano Narrows		1964	1299	30	
St. Johns	Oregon *(2)*	1932	368	62	

Note: ( ) – refers to no. of bridges. Seismic risk is based on ground shaking – see Table 2.

Table 1 Data on 57 Long-Span Suspension Bridges

Name of Bridge	Location (state)	Opened to Traffic in	Main Span (meters)	Age as of 1994 (years)	Seismic Risk
Crooked River	Oregon *(cont)*	1963	179	31	Moderate *(cont.)*
Benjamin Franklin	Pennsyl-vania *(6)*	1926	534	68	
Seventh Street		1926	135	68	
Ninth Street		1927	131	67	
Sixth Street		1928	213	66	
South Tenth Street		1933	221	61	
Walt Whitman		1957	610	37	
Mount Hope	Rhode Island *(2)*	1929	366	65	
Newport		1969	488	25	
Delaware Memorial I	Delaware *(2)*	1951	655	43	Minor *(4)*
Delaware Memorial II		1968	655	26	
Waldo-Hancock	Maine *(2)*	1931	244	63	
Deer Isle		1939	329	55	
Chesapeake Bay I	Maryland *(2)*	1952	488	42	Low *(12)*
Chesapeake Bay II		1972	488	22	
Ambassador	Michigan (2)	1929	564	65	
Mackinac		1956	1159	38	
John A. Roebling	Ohio *(5)*	1867	322	127	
Steubenville		1904	213	90	
U.S. Grant		1927	229	67	
Fort Steuben		1928	210	66	
Anthony Wayne		1930	239	64	
Waco	Texas *(2)*	1870	145	124	
Red River		1927	213	67	
Wheeling II	West Virginia	1854	308	140	
Royal Gorge	Colorado	1929	268	65	Negligible *(4)*
Mayo	Florida	1947	129	47	
Missouri River	Iowa	1960	226	34	
Father Louis Hennepin	Minnesota	1988	191	6	

Table 1 Data on 57 Long-Span Suspension Bridges *(cont.)*



Notation:

(–,–): (no. of bridges, seismic risk based on ground shaking).
neg: negligible, *min*: minor, *mod*: moderate, *hgh*: high, *v-hgh*: very high.
See Table 2 for Classification of Seismic Risk.

Fig. 2 No. of Long-Span Suspension Bridges and Seismic Risk in each State

3. SEISMIC RISK

Since the seismicity and geology in the United States varies, bridges are subject to different seismic risks. Table 2 shows classification of seismic risk based on ground shaking using the maximum value of the AASHTO (American Association of State Highway and Transportation Officials Standard Specifications for Highway Bridges, 1992) acceleration coefficients specified for each state. This is only a preliminary guide. As more information becomes available, the subcommittee will document seismic risk based on site-specific conditions, probability of future earthquakes, and other factors. Fig. 2 also shows the seismic risk for each state -- varying from negligible to very high.

Seismic Risk	Value of AASHTO Acceleration* Coefficients	No. of States Subject to Seismic Risk	No. of Long-Span Suspension Bridges (Total = 57)
Very High	≥ 0.50	2	6
High	≥ 0.29 and < 0.50	10	4
Moderate	≥ 0.15 and < 0.29	12	27
Minor	≥ 0.10 and < 0.15	8	4
Low	≥ 0.05 and < 0.10	9	12
Negligible	< 0.05	9	4

** Bedrock acceleration with a 90% probability of not being exceeded in 50 years*

Table 2 Classification of Seismic Risk based on Ground Shaking

4. SEISMIC EVALUATION STUDIES

The objective of an evaluation study is to predict how an existing bridge will perform in a design earthquake, assess vulnerabilities, and recommend retrofit of deficient bridges. In general, an evaluation study includes the following stages: (i) assessing seismic risk and developing input ground motions, (ii) establishing seismic performance criteria, (iii) developing and validating computer models based on soil-structure interaction, (iv) conducting linear and non-linear seismic analysis, (v) laboratory testing, and (vi) developing conceptual retrofit schemes and estimating their costs for deficient bridges. The most extensive study so far has been on the Golden Gate Bridge. Seismic retrofit on the bridge is scheduled to start in the spring of 1995. The retrofit is estimated to cost $155 million, including studies, design and construction, and is scheduled for completion in 1998. Studies have also been completed on the 140-year-old Wheeling II Bridge (West Virginia), the 58-year-old San Francisco-Oakland Bay Bridge (California), and the 44-year-old Tacoma Narrows II Bridge (Washington). Studies are in progress on the the 62-year-old St. John's Bridge (Oregon) and the 55-year-old Bronx-Whitestone Bridge (New York). The subcommittee will survey the owners of the remaining bridges regarding the status of evaluation studies and proposed retrofits, if any.

5. DISCLAIMER

Views expressed are those of the members and not those of the ASCE or their employers.

6. REFERENCES

Are too numerous to list and are included in the report which is under review by the subcommittee.

Seismic Strengthening of an Aqueduct in New Zealand

Renforcement parasismique d'un aqueduc en Nouvelle Zélande

Seismische Verstärkung eines Aquädukts in Neuseeland

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SUMMARY

The Kaitoke Flume Bridge near Wellington, New Zealand, is a reinforced concrete aqueduct built in the early 1950s which carries 50% of Wellington's water supply. The 51 m long bridge spans a 16 m deep ravine and is supported by two high reinforced concrete piers. Water is carried inside a 1.1 m square in-situ concrete flume. The bridge site is close to the Wellington Fault and the structure lacked the ductile detailing necessary to survive the expected M7.5 earthquake. The paper describes the various methods considered for improving seismic performance. Design for elastic response was favoured.

RÉSUMÉ

Construit au début des années 1950, l'aqueduc en béton armé de Kaitoke près de Wellington, Nouvelle-Zélande, assure 50% de l'approvisionnement en eau de Wellington. Le pont d'une longueur de 51 mètres enjambe un ravin d'une profondeur de 16 mètres et est soutenu par deux piliers en béton armé. L'eau est acheminée à l'intérieur d'une section carrée de 1,1m en béton coulé sur place. Le pont est situé à proximité de la faille de Wellington, mais la construction ne présente pas la ductilité nécessaire pour résister au tremblement de terre attendu d'une magnitude de 7,5. Le document décrit les différentes méthodes envisagées pour améliorer la performance sismique. Le projet de dimensionnement élastique a été retenu.

ZUSAMMENFASSUNG

Bei der Kaitoke Flume Bridge in der Nähe von Wellington, Neuseeland, handelt es sich um ein Stahlbetonaquädukt, das am Anfang der 50er Jahre gebaut wurde und über welches 50% der Wasserversorgung Wellingtons geleitet wird. Die 51 Meter lange Brücke überspannt eine 16 Meter tiefe Schlucht und wird von zwei Stahlbetonpfeilern getragen. Das Wasser wird in einem 1,1 Meter quadratischen Betongerinne geführt. Die Brücke liegt nahe der Verwerfung Wellington und die Bauart der Konstruktion verfügte nicht über die nötige Duktilität, um ein zu erwartendes Erdbeben der Stärke 7,5 zu überstehen. Diese Arbeit beschreibt verschiedene Methoden zur Verbesserung der seismischen Leistung. Das Projekt mit elastischer Berechnung wurde bevorzugt.

1. Introduction

Wellington, New Zealand's capital city, is in one of the country's most seismically active regions and is built astride the Wellington Fault. This fault is capable of producing a M7.5 earthquake with a probability of occurrence of more than 10% in 50 years. (500 year return period approximately). The City's engineering lifelines are all under threat from strong earthquake shaking and/or fault displacement, and their vulnerability to earthquake prompted a region-wide study in 1990 and 1991. (Reference 1).

Wellington's water supply was shown to be particularly vulnerable in places and has been the subject of closer examination by the controlling authority, the Wellington Regional Council (WRC). In 1993, a major inspection of all bulk water lines was made in order to identify vulnerabilities more detail and to estimate the costs and programming of mitigation measures. This has been the subject of a paper by the author. (2)

The Kaitoke source is one of four serving Wellington and supplies 50% of the region's needs. A 30km long, 900mm diameter water main from Kaitoke to Wellington City crosses the Wellington Fault twice and delivers water to a natural reservoir at Karori and to prestressed concrete reservoirs located throughout the region.

The Kaitoke Flume Bridge was recognised as being a critical element in the bulk water network and was known to lack ductile detailing required by current earthquake standards. In mid-1991, the Wellington Regional Council commissioned a structural assessment from the author's firm in order to address the unacceptably high earthquake risk to this bridge.

2. General Description

The bridge was built in the 1950's to a design prepared by the New Zealand Ministry of Works to unknown seismic design standards. It is likely that it was designed for a static load of 10% of gravity. The bridge is approximately 45m long over 3 spans with a 23m central span. The flume cross-section is a concrete box 1140mm x 1060mm inside with 150mm to 200mm thick walls. The four reinforced concrete piers vary markedly in length, the two central ones being 11.2m high. The short south pier is hinged top and bottom in the longitudinal direction to allow temperature movement. The flume is free to move independently of the abutment. At the north end the flume is built integrally into the intake spillway structure which is keyed into the solid greywacke rock which forms the foundations for all the piers.

An unusual feature of the flume section is that the base slab has been cast in separate units each approximately 5.7m long and with no positive connection shown to the sides. Its ability to behave as a box girder was therefore questionable.

3. Initial Structural Assessment

Analysis

The initial structural assessment was based on an equivalent static analysis to evaluate the bridge's performance in earthquake. A three dimensional static analysis was carried out in which the flume was assumed to be an inverted U section. The strength of critical sections was determined in accordance with normally accepted concrete theory taking account of details on the drawings and assuming the concrete was 3000 psi (20 MPa) and the reinforcing steel 40,000 psi (275 MPa).

The vertical steel at the inside face of the flume walls was not adequately anchored into the base and it was therefore neglected. The horizontal steel in the central piers was also not adequately anchored at the ends and was neglected in strength calculations.

Earthquake Performance
Three references (3, 4, 5) were consulted to determine an appropriate design coefficient, resulting in base shear coefficients ranging from 0.4 to 1.0g, corresponding to peak ground accelerations of between 0.5 and 0.7g.

Table 1 summarises the assessed ultimate strengths of bridge elements in terms of the equivalent static base shear coefficient. A wide variation can be seen, with notable inadequacies in the flume and in the piers for overturning. In order to set these figures in context, the probabilities of earthquakes exceeding the various levels of shaking intensity are given..

Table 1 : Comparative Strengths of Bridge Elements

Element	Structural Action	Base Shear (g) at Ultimate Strength	50-yr Probability
Flume	Flexure at midspan	0.16	90%
	Flexure at pier	3.43	1%
	Side wall flexure at pier	0.04	100%
	Shear at pier	0.48	30%
	Torsion at pier	0.04	100%
Central Piers	Shear at top	0.37	50%
	Flexure at base	0.81	10%
	Overturning at base	0.22	80%

Conclusions and Recommendations
The initial structural assessment concluded that the bridge was in reasonable physical condition but that there were some unusual detailing and design characteristics which were unacceptable in a structure of this importance. Some basic strengthening measures to the flume itself were identified, but further analysis was recommended in order to provide greater insights into the likely earthquake performance of the bridge and the costs of remedial work.

4. Follow-up Evaluation

In November 1991, a closer study was made using response spectrum dynamic analyses. The analyses carried out were based on two separate structural models, the first representing the structure as it stood at the time and the second in its partially strengthened and stiffened state. A site-specific earthquake response spectrum was used, scaled to correspond to a 475-year return period, the results of which confirmed the basic findings of the initial structural assessment. However it was found that even with the basic strengthening measures initially recommended, the bridge would be capable of withstanding only about 12% of the seismic load recommended for the design of modern structures of similar importance. This raised the prospect of more extensive strengthening, full replacement or base isolation to improve the performance of the bridge.

5. Strengthening Options

Table 2 summarises the strengthening options considered, and estimated costs.

Table 2 : Summary of Strengthening Options, Strengths and Costs

Strengthening Option	Strength/Current Code (Ratio)	Cost Estimate ($NZ)
1. Leave as existing	0.04	0
2. Strengthen flume only	0.12	60,000
3. Strengthen flume, piers (partial)	0.39	175,000
4. As for Option 3 plus base isolation	1.00	260,000
5. Strengthen flume and piers fully	1.00	200,000
6. Construct replacement bridge	1.00	700,000

Base isolation (Option 4) was to be achieved by either controlled rocking of the piers or controlled sliding of the flume at the top of the piers. A particular difficulty with this concept was the relatively large horizontal movements of the flume which would need to be accommodated, and the necessity to decommission the aqueduct. There were no cost advantages.

6. Bridge Strengthening

The strengthening scheme selected (Option 5) involved the following main items:

- Construction of steel frames bolted to each pier.

- Construction of a foundation beam, extending beyond the original one and anchored to the rock foundation material.

- Installation of horizontal trusses underneath the flume base.

- Installation of yokes above each pier to reduce the vertical bending in the walls of the flume under lateral loads.

Details are shown in Figure 1.

A design base shear of 0.96g was used corresponding to elastic response of the structure.

7. Tendering and Construction

The strengthening work was tendered in early 1993 on a selected competitive basis. Interestingly, one bidder offered an alternative involving base isolation, but this confirmed the earlier costings and was not competitive with design for elastic response.

Construction commenced in August 1993 and was completed in January 1994. During construction, the generally good condition of the bridge was verified, and although awkward in places, the installation proceeded according to plan. The bridge, which is situated in a regional

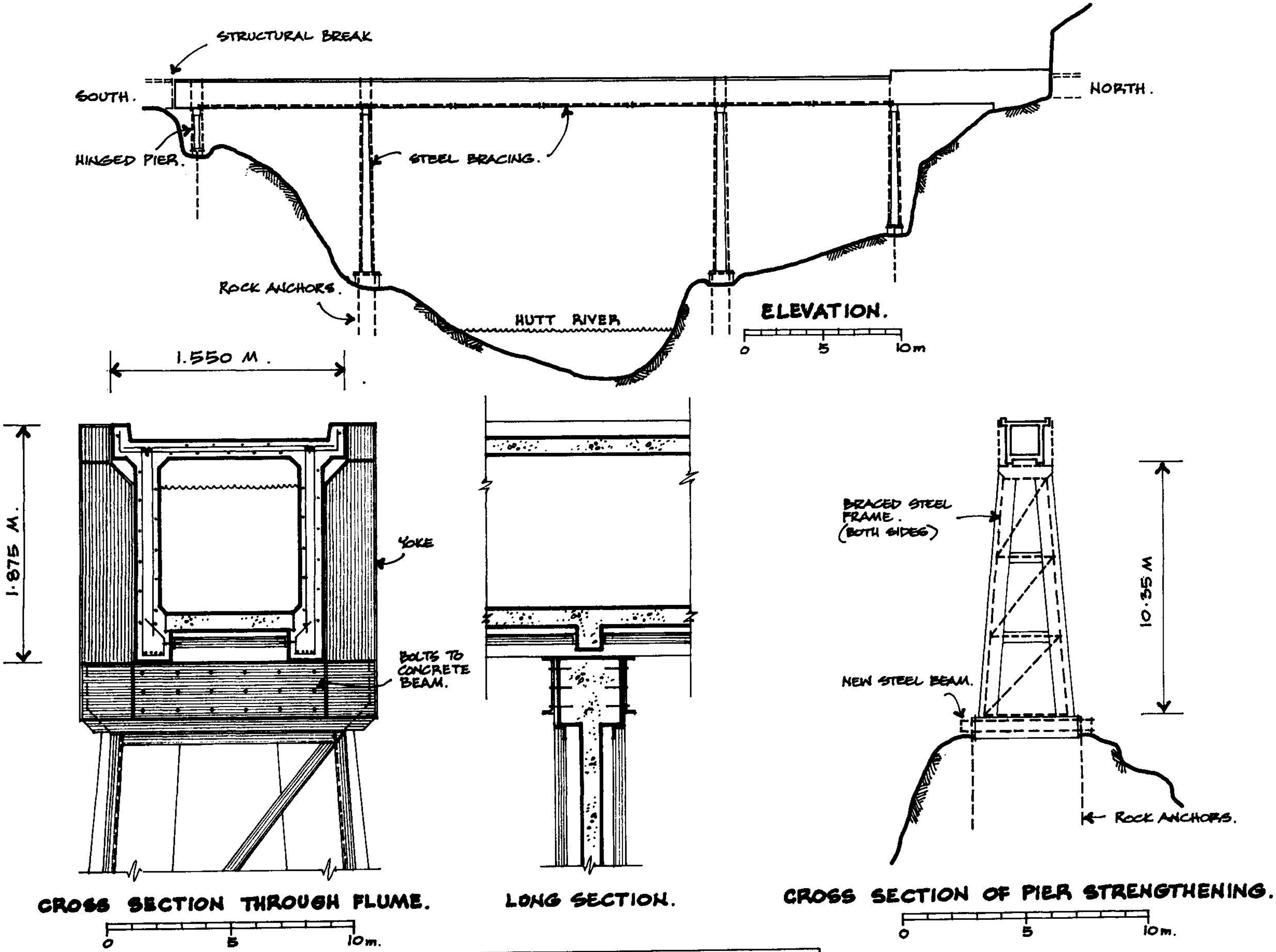

FIGURE 1 : KAITOKE FLUME BRIDGE - STRENGTHENING DETAILS.

recreational park, was strengthened without being decommissioned at any stage and without significant restriction to public access.

8. Conclusions

Initial structural assessments followed by more detailed dynamic analyses indicated that the Kaitoke Flume Bridge had an unacceptably high risk of failure in earthquake at several critical locations.

Examination of design options, which included consideration of base isolation, led to the adoption of design for elastic response to control deflections and reduce ductility demand.

The strengthening work, which brought the bridge to a condition similar to that for a new structure, was achieved at a cost of 30% of a replacement structure.

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Acknowledgements

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Seismic Retrofit of the Lake Washington Ship Canal Bridge

Consolidation parasismique du pont-canal du Lac Washington

Seismische Verstärkung der Schiffskanalbrücke des Lake Washington

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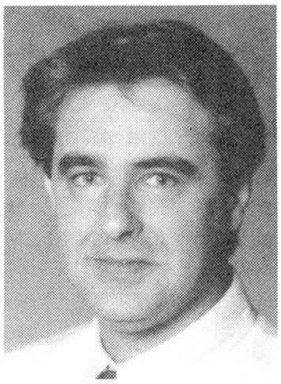

Dimitrios P. Koutsoukos holds a civil engineering degree from the Aristotle University of Thessaloniki, Greece, a MSCE from the University of Washington, Seattle, and a Ph.D. from the University of Texas at Austin. He specializes in the seismic strengthening of highway bridges and in the design of seismically isolated structures.

SUMMARY

This paper describes the seismic vulnerability assessment and retrofit design of the Lake Washington Ship Canal Bridge in Seattle, one of the most important bridges in the State of Washington. A short description of the structural configuration of the bridge, as well as information on the site seismicity and local geological hazards are included in the paper. The assessment methodology and the technical criteria used for evaluating the seismic capacity of the bridge are briefly explained. Finally, the structure's seismically deficient elements are listed and the retrofit schemes that have been selected for the immediate seismic rehabilitation of the bridge's most vulnerable steel truss units are described.

RÉSUMÉ

Cet article décrit l'évaluation de la vulnérabilité vis-à-vis de séismes et le projet de consolidation du pont-canal du Lac de Washington à Seattle, un des plus importants de l'État de Washington. Une description de la structure du pont, ainsi qu'une information sur la séismicité du site et des dangers géologiques locaux sont également inclus. La méthodologie d'évaluation et les critères techniques utilisés pour estimer la capacité sismique du pont sont expliqués. Les éléments déficients de la structure sont énumérés et les projets retenus pour la réhabilitation d'un point de vue sismique des éléments de treillis métallique les plus faibles sont décrits.

ZUSAMMENFASSUNG

Dieses Referat beschreibt die Einschätzung der seismischen Verwundbarkeit und den verstärkten Entwurf der Lake-Washington-Schiffskanalbrücke in Seattle, eine der wichtigsten Brücken im Staat Washington. Eine kurze Beschreibung der Konstruktion der Brükke wie auch Informationen über das seismographische Gebiet und der lokalen geologischen Risiken sind in diesem Referat aufgenommen. Die Methodik der Einschätzung und die technischen Kriterien, die für die Evaluation der seismischen Kapazität benutzt worden sind, sind in Kürze erklärt. Die seismisch ungenügenden Bauteile sind aufgelotet. Schliesslich wird die seismische Verstärkung beschrieben.

1. INTRODUCTION

The SR5 Lake Washington Ship Canal Bridge was constructed between 1959 and 1961 on the principal north-south route of Interstate 5 in Seattle, Washington (Fig. 1). The bridge is a complex, multi-level structure, consisting of 17 individual structural units that are separated by expansion joints. It carries twelve lanes of highway traffic with an annual average traffic in excess of 200,000 vehicles per day. It is documented to be one of the most important highway bridges in the State of Washington and as such its seismic resistance upgrading was sought by the Washington State Department of Transportation. Phase I of the structure's seismic rehabilitation was initiated in 1993 and it included an extensive vulnerability assessment of the bridge. Since then, the second phase (Phase II) of the bridge's rehabilitation program has been also initiated. Phase II included the design of the required retrofit schemes to protect the most vulnerable structural units: the steel trusses crossing the Ship Canal waterway. Results from Phase I were extensively used for the design of the most effective and cost competitive retrofit schemes for the steel truss units. Construction of the Phase II retrofit items is expected to be complete during the second semester of 1995. A final retrofit phase (Phase III) is scheduled and it will cover the seismic upgrade of the bridge's north and south concrete approaches.

A brief description of the bridge configuration along with the methodology used for its seismic vulnerability assessment and major results of the Phase I study are discussed in the paper. The main seismic retrofit schemes that have been designed for the seismic protection of the structure's truss units are also described.

2. STRUCTURE AND SITE DESCRIPTION

The total bridge length of 1,350 m is divided into the south approach, the steel trusses and the north approach with lengths of 352 m, 699 m and 299 m, respectively. The south and north approaches consist of multi-span reinforced concrete frame structures. At the concrete approaches, expansion joints consist of split columns between frames. The six spans of the steel trusses are supported by rocker and pinned bearings (Fig. 2). Reinforced concrete portal bents support the truss bearings. The foundations are spread footings, except for a few pile foundations located at the south approach.

The site soils are generally good, comprised of glacially consolidated till.

3. SITE SEISMICITY AND GEOLOGICAL HAZARDS

A site specific seismicity study confirmed that the response spectrum specified by AASHTO [1] with $a_{max} = 0.29g$, soil profile II and 5% damping is appropriate for the bridge site. Three regional zones responsible for earthquake generation were considered: a shallow crustal zone, a deep subcrustal zone and the Cascadia Subduction Zone. It is considered that the above spectrum represents motions resulting from a deep focus, near source earthquake of magnitude 7.5 with a 10% probability of being exceeded during a 50-year interval.

The geotechnical study concluded that liquefaction and instability of the foundation soils are not likely and that there is a low potential for ground surface rupture as related to faulting in the bedrock underlying the bridge alignment.

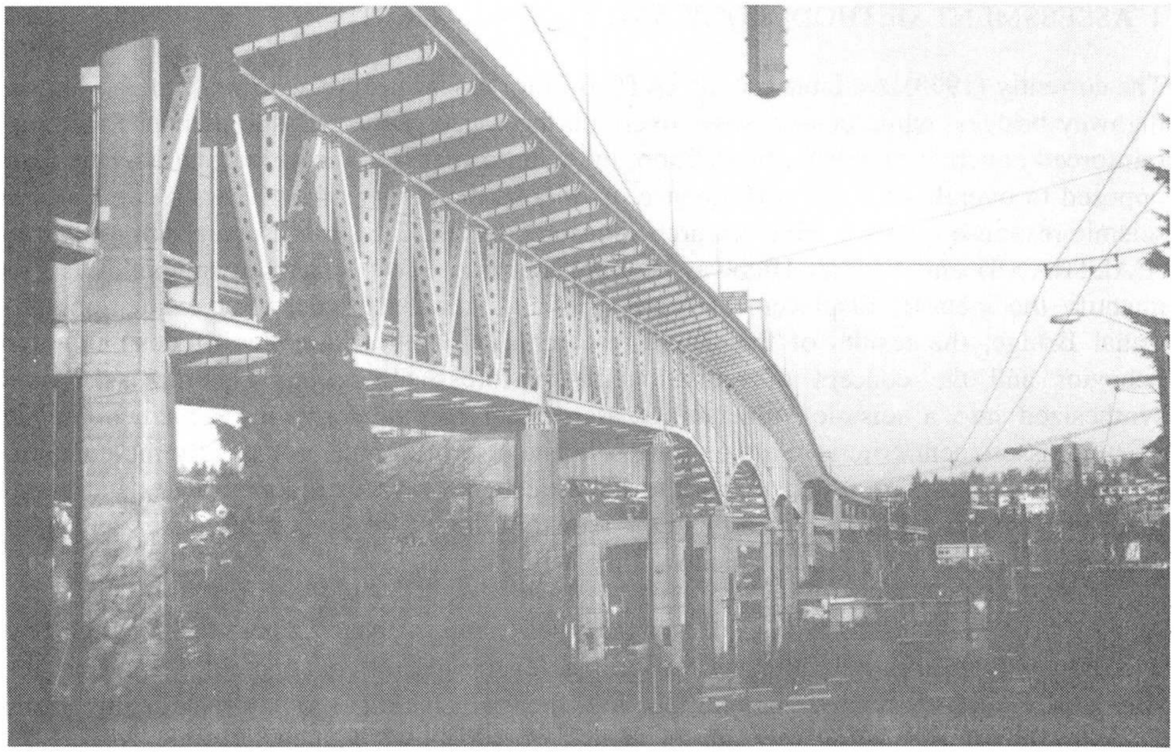

Fig. 1 The Lake Washington Ship Canal Bridge along Interstate 5. Steel truss units across the Ship Canal Waterway.

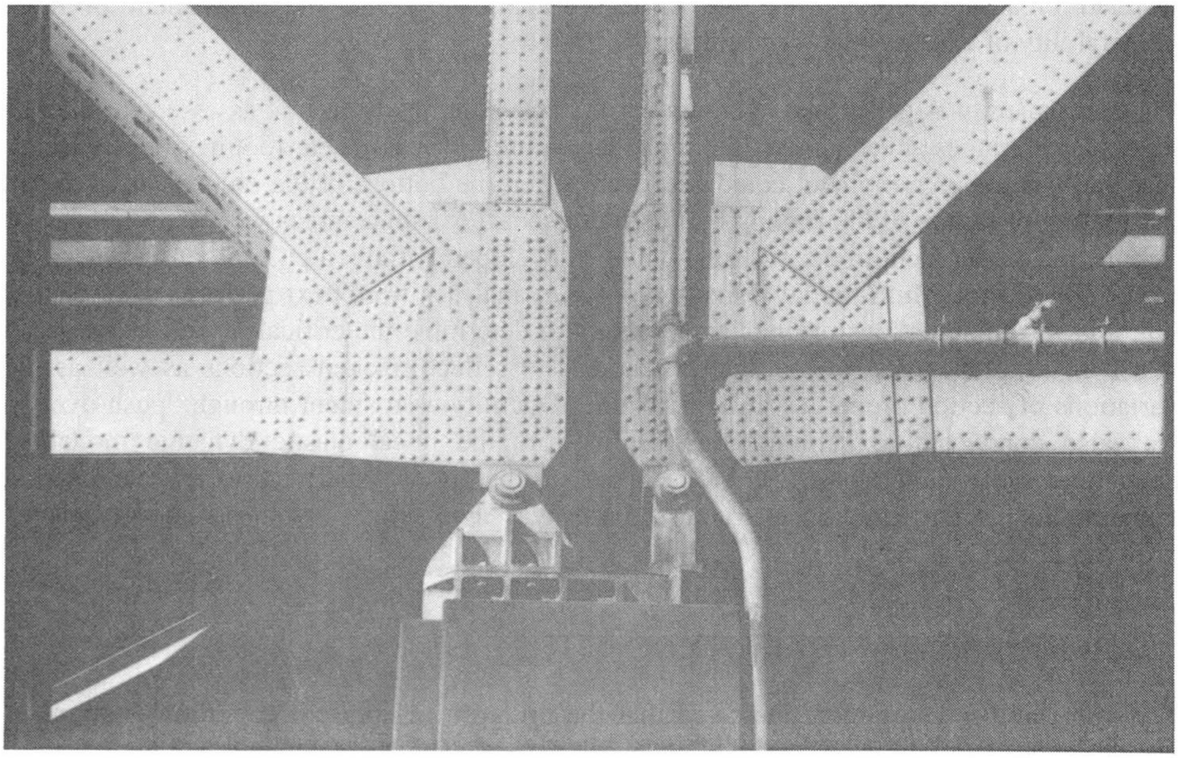

Fig. 2 A typical rocker bearing subject to loss of vertical support due to earthquake motions. Bearing collars, longitudinal restrainers and compression bumpers are used to mitigate the catastrophic consequences of support loss and toppling of the bridge's superstructure.

4. ASSESSMENT METHODOLOGY AND TECHNICAL CRITERIA

The currently (1993) available criteria (ATC 6-2) for assessment of seismic vulnerability of existing highway bridges, while conservative, overlook the capacity for inelastic displacements in existing reinforced concrete members. In addition, the criteria focus on individual member performance as opposed to overall structure performance, overlooks the ability of a given structure to redistribute seismic response forces. Recent research sponsored by the California Department of Transportation (CALTRANS) and Federal Highway Administration (FHWA) has shown that it is possible to quantify the inelastic displacement capacity of individual members. For evaluation of the Ship Canal Bridge, the results of the above research [2, 3], the principles of designing for ductile behavior and the concept of redistribution of forces throughout a complex structure were synthesized into a seismic vulnerability assessment methodology that remedied the defects in existing assessment criteria. Furthermore, selective use of nonlinear statically incremental analysis ("push-over" analysis) which tracks the inelastic behavior of structures under increasing lateral loads was performed to help determine the redistribution of lateral seismic forces as the structure's properties transition from the elastic to the inelastic range.

A set of criteria was developed as part of the assessment procedure. These technical criteria cover a wide range of factors pertaining to the bridge seismic performance such as appropriate material strengths, guidelines for foundation modeling, flexural, shear and joint strength, and seismic input. The developed techniques provide a more efficient tool than previously available for the vulnerability assessment of existing highway bridges to seismically induced damage. The analysis tool was the outcome of an extensive literature survey that resulted in an effective synthesis of existing research into a lucid, conservative methodology. The continuous input from bridge engineering experts and researchers specializing in seismic evaluation and retrofit of bridges resulted in a methodology that has universal applicability for the assessment of the seismic vulnerability of existing highway bridges.

The assessment was performed in a number of steps: columns were first assessed for their ability to form plastic hinges at each end without a prior failure in shear. Then, footings were checked to see that the plastic hinge moment could be developed at the bottom of the column prior to rocking of the footing or failure of the footing by flexure or shear. All splice areas were then checked to verify that the plastic moment in the column could be developed without a pull-out or an otherwise failure of the splice bars. Multi-modal response spectrum analyses were next performed for each separate unit and for combinations of units as necessary to determine the critical responses. Great care was taken in the preparation of the analysis models to reflect cracked sections as well as the actual variations of section properties in the structure. Ductility assessment through "push-over" analysis was also conducted to calibrate the results from the lineal elastic dynamic models. Finally, demand/capacity ratios based on the forces of the modal analyses were calculated and the vulnerability of the individual structural units was assessed by synthesizing the results of the previous phases.

5. STRUCTURAL VULNERABILITY RESULTS

The vulnerability assessment indicated that the bridge in its present condition contains elements which would compromise its survival under the design level seismic event. Noting that the adopted assessment criteria and design seismic event represent conservative estimates of the expected conditions, the vulnerability assessment shows that certain structural elements have a high probability of failure during the design event. These failures indicate that most of the structure will

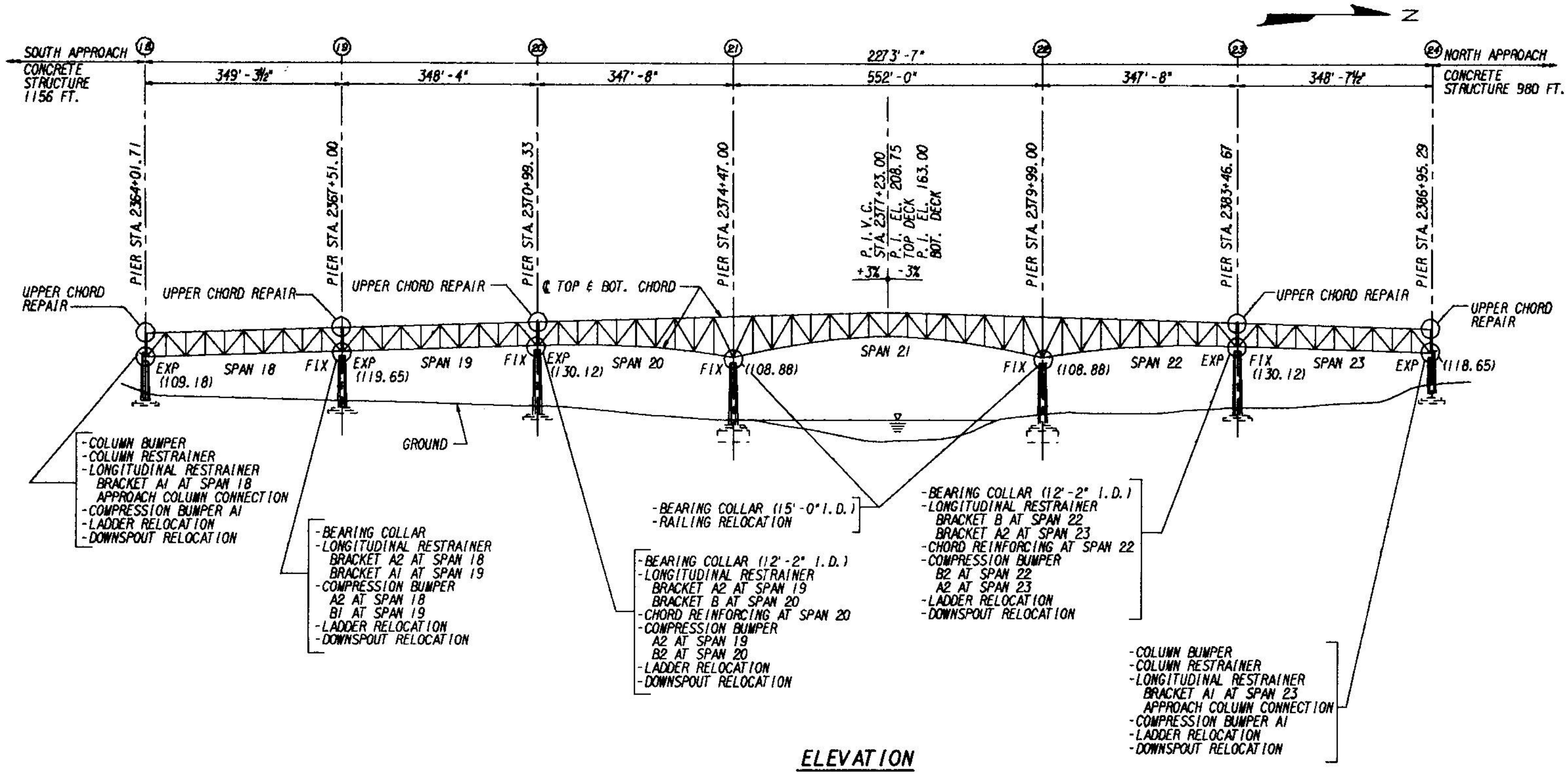

Fig.3 SR5 Lake Washington Ship Canal Bridge
General Layout of Seismic Retrofit items.

experience severe damage, with several portions of the structure likely to collapse. Specifically, the most vulnerable elements are (Fig. 3):

- Truss bearings.
- Truss lateral bracing members.
- Double-deck box girder crossbeams.
- Columns.

6. RETROFIT SCHEMES

A set of retrofit measures was designed during Phase II for the seismic strengthening of the steel truss units. The proposed retrofit measures include (Fig. 3):

- Concrete-filled steel collars at truss bearing bolsters (at Bents 19 through 23).
- Compression "bumpers" between the concrete approach structures and the truss chords, and between truss chords at the expansion joints (at Bents 18, 19, 20, 23 and 24).
- Longitudinal restrainers at the bottom chords of the trusses (at Bents 18, 19, 20, 23 and 24).

After the completion of the retrofit design, a new series of analyses was performed to ensure that the retrofit schemes will behave as intended, i.e., by markedly improving the structural seismic response. Both multi-modal response spectrum analyses and nonlinear time history analysis with structural models incorporating gap elements and nonlinear springs were performed. The results indicated that these retrofit schemes will greatly improve the bridge seismic behavior providing at least the level of seismic performance specified in the vulnerability criteria. The total estimated construction cost of the items cited is $5,700,000 in present day US dollars.

ACKNOWLEDGMENTS

The author wishes to thank Dr. John H. Clark, Vice President and Chief Bridge Engineer at ABKJ, and Ms. Joyce M. Lem, Project Manager at ABKJ, for their support during all phases of this undertaking. The help of the following individuals is also gratefully acknowledged: Mr. Gary F. Conner and Mr. Eysteinn Einarsson for their valuable comments, Ms. Betty J. Birk and Ms. Kathy L. Hitchcock for preparing the figures of this paper, Mr. Paul W. Grant, Vice President at Shannon & Wilson, Inc., for providing the geotechnical review, and Mr. Robert S. Currie for the preliminary estimates of construction costs. Finally, the support of this work by ABKJ is greatly appreciated.

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Presidio Viaduct Seismic Retrofitting
Consolidation parasismique du viaduc Presidio
Erdbebenverstärkung des Presidio Viaduct

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SUMMARY

Following the 1989 sizeable earthquake, for the first time affecting the San Francisco Bay Area's major bridge structures, a high priority seismic retrofit program was initiated by the California Department of Transportation. This contribution reports on the engineering design services for seismic upgrading of the Presidio Viaduct, close to the Golden Gate Bridge.

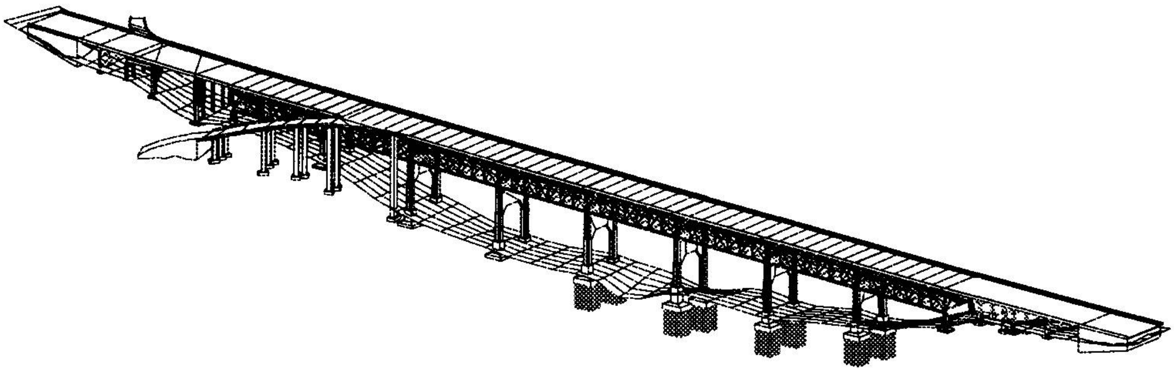
RÉSUMÉ

A la suite du violent tremblement de terre de 1989 qui, pour la première fois, affecta dangereusement les structures porteuses des grands ponts de la baie de San Francisco, le Ministère des transports de l'État de Californie ordonna un programme prioritaire de consolidation parasismique de ces ouvrages. L'article traite des études menées par les ingénieurs responsables du renforcement du viaduc Presidio, une importante voie de communication à proximité du pont de Golden Gate.

ZUSAMMENFASSUNG

Als Folge des beträchtlichen Erdbebens von 1989, das erstmals die grösseren Brückentragwerke in der Bay-Area von San Francisco traf, veranlasste das Verkehrsdepartement von Kalifornien ein dringliches Erdbebenverstärkungsprogramm. Der vorliegende Beitrag berichtet von der Ingenieurtätigkeit für die Verstärkung des Presidio Viaduct, einer Hochstrasse nahe der Golden-Gate-Brücke.

Presidio Viaduct carries US 101 over a valley in the northern part of the "Presidio of San Francisco", approximately 1 mile south of the Golden Gate Bridge. The viaduct accomodates 6 narrow driving lanes that allow a 4-2 and 2-4 lane split during morning and evening commute hours to and from San Francisco. The viaduct has a total length of approximately 1,500' and comprise the following elements:



- East Approach: Four simply supported structures with steel stringers supporting reinforced concrete decks. Substructures comprise reinforced concrete bents on spread footings.

- Viaduct : Eight, 135' span, simply supported steel truss frames supporting reinforced concrete decks. Substructures comprise reinforced concrete columns and bents, partly on spread footings and partly on piled foundations.

- West Approach: A six span structure with combinations of steel and concrete stringers supporting reinforced concrete decks. Substructures comprise reinforced concrete bents on spread footings.

The structures were built 1935-39 and were retrofitted with restrainers at all simply supported superstructure supports in 1983. The as-built structure has been analyzed and evaluated with global, elastic stick models, detailed elastic frame models and push-over/displacement ductility analysis to expose potentially weak elements under heavy seismic loads.

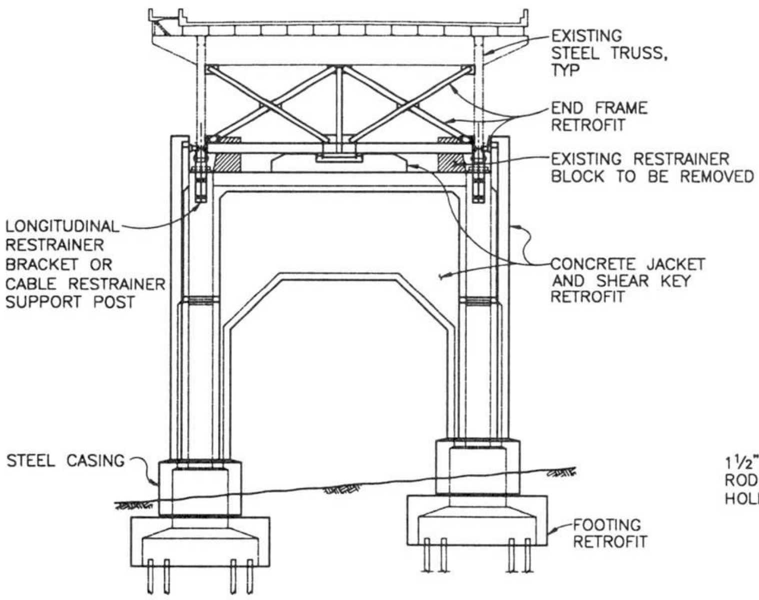
An important assumption in all our analysis is the behavior of the transverse joints in the reinforced concrete deck slabs of the main viaduct. These joints separate the deck above the floor beams at a spacing of about 20'. Therefore, the deck does not act as a continuous compression flange under gravity loads. On the other hand, a careful study of the structural details of the bridge deck system reveals that it may safely be assumed that horizontal, transverse, seismic shear forces in the deck slab can be fully transferred across the joints through stringers and floor beams to the end frames.

Our analysis has revealed the following critical problems in the main viaduct, the approaches and the ramp:

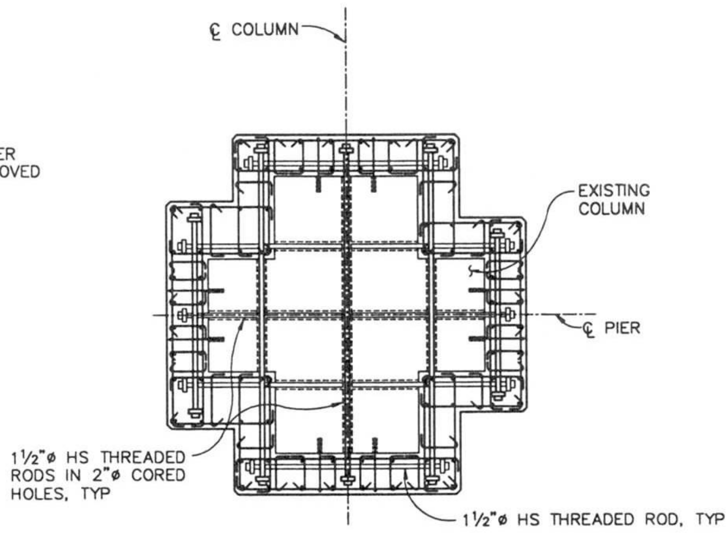
LOCATION	PROBLEM	RETROFIT ALTERNATIVE
VIADUCT SUPER - STRUCTURE	Buckling of Slender Steel Truss Members in End Frames	End Frame Retrofit Base Isolation Deck Replacement
VIADUCT BEARINGS	Insufficient Shear Capacity in Restrainers/ Concentration of long. loads on short columns	New Steel Collars & Restrainers/ Removal of Existing Restrainers
VIADUCT SUB - STRUCTURE	High Moment D/C Ratios (4 to 8+ Range) & High Shear forces in Plastic Zones Insufficient Confinement Steel in Columns	Steel Jackets Concrete Jackets Prestressing Shear Walls Combinations
VIADUCT FOUNDATIONS	No Top Mat in Foundations	Foundation Top Mats
APPROACHES	High Moment D/C Ratios (4 to 8+ Range) & High Shear forces in Plastic Zones Insufficient Confinement Steel in Columns Plastic hinging in cap beams Instability of spread footings	Steel Jackets Cap beam strengthening Foundation enlargement
RAMP	High Moment D/C Ratios (4 to 8+ Range) & High Shear forces in Plastic Zones Insufficient Confinement Steel in Columns Large deflections relative to viaduct	Steel Jackets Expansion joint upgrading

TABLE 1: POTENTIALLY WEAK ZONES OF EXISTING STRUCTURE

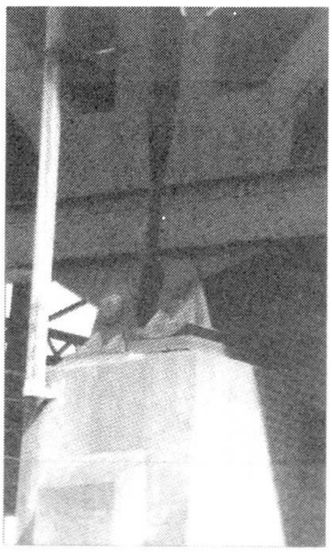
Several alternative methods of retrofitting the viaduct were considered. These were all analyzed and verified by appropriately modified global, elastic stick models and detailed elastic frame models. Careful evaluation of traffic and construction issues were made in addition to possibilities for phasing of the work, temporary shoring and construction risk.



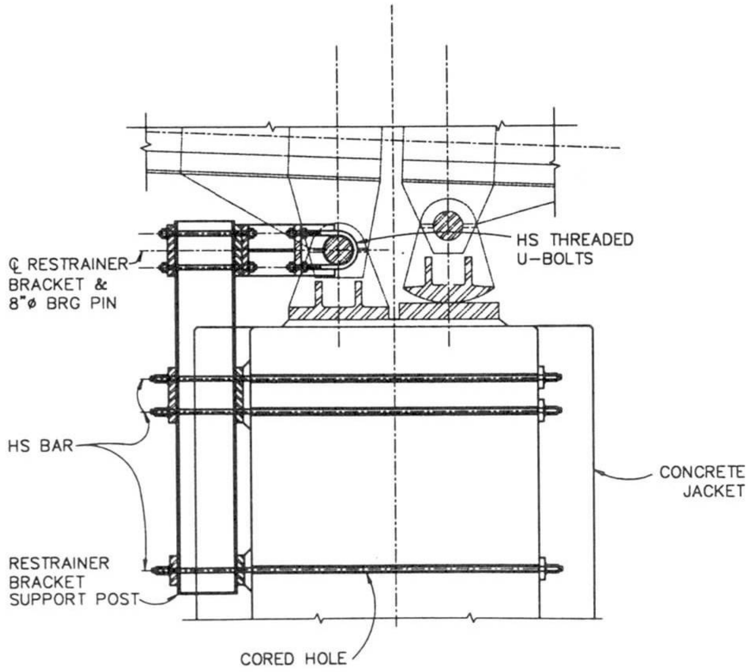
TYPICAL SECTION PIERS 3–8



PIER SECTION



DETAIL AT TRUSS SUPPORTS



LONGITUDINAL RESTRAINER BRACKET

Finally, an attempt was made to develop preliminary construction costs for the various retrofit alternatives. A "fatal flaw" analysis was then carried out to eliminate some of the marginally acceptable solutions. The main result of the fatal flaw analysis is shown below:

VIADUCT	RETROFIT ALTERNATIVE	FATAL FLAWS
SUPER - STRUCTURE	End Frame Retrofit	-
	Base Isolation	Deflections Installation
	Deck Replacement	Costs
BEARINGS	New Steel Collars & Restrainers/ Removal of Existing Restrainers	-
SUB - STRUCTURE	Steel Jackets	Complex detailing at haunched sections
	Concrete Jackets	Insufficient confinement
	Prestressing	Installation Insufficient confinement
	Shear Walls	Weight Aesthetics Causes Foundation Problems Poor Torsion Characteristics
	Combination steel/ concrete jackets	-
FOUNDATIONS	Foundation Top Mats	-

TABLE 2: FATAL FLAW ANALYSIS OF RETROFIT ALTERNATIVES

As an alternative to the maximum credible earthquake approach in the standard Caltrans procedures (safety evaluation), a design peak bedrock acceleration level for the next 10-20 years of exposure has been evaluated by our Geotechnical Consultant (functional evaluation). The bedrock acceleration will of course be smaller than the 0.6 g used for the analysis of the Presidio Viaduct. In view of future decision making, we have obtained the D/C ratio's of the structure for smaller levels of bedrock acceleration.

A 0.2 g design level would result in a potentially cheaper retrofit alternative, essentially avoiding the jacketing of the substructure bents and replacement of restrainers of the main viaduct. In addition, no retrofit would be required for the ramp structure.

A 0.4 g design level appears equivalent to a 0.6 g design level in terms of extent and costs of seismic retrofit.

Based on our analysis and the developed construction costs for each of the retrofit alternatives for the viaduct and the proposed retrofit for the approaches and ramp, our recommended retrofit solutions were developed.

LOCATION	ELEMENT	ALTERNATIVE	ESTIMATED COST	RECOMMENDATION
VIADUCT	SUPER - STRUCTURE	End Frame Retrofit	$1,000,000	$1,000,000
		Base Isolation	$2,800,000	
		Deck Replacement	$8,600,000	
	BEARINGS	New Steel Collars & Restrainers/ Removal of existing Restrainers	$ 300,000	$ 300,000
	SUB - STRUCTURE	Steel Jackets	$1,600,000	
		Concrete Jackets	$1,500,000	
		Combination steel/ concrete Jackets	$1,600,000	$1,600,000
		Prestressing	N.A.	
		Shear Walls	$1,300,000	
	FOUNDATIONS	Foundation Top Mats	$ 500,000	$ 500,000
APPROACHES		Steel Jackets, Cap Beam & Foundation Retrofit	$ 900,000	$ 900,000
RAMP		Steel Jackets/Exp. Joint Upgrade	$ 700,000	$ 700,000
TOTALS	Subtotal Mobilization (10%)			$5,000,000 $ 500,000
	Subtotal Contingency (25%)			$5,500,000 $1,400,000
	TOTAL			$6,900,000

TABLE 3: RECOMMENDED SEISMIC RETROFIT ALTERNATIVE

The adopted retrofit strategy includes strengthening the truss end frames for lateral stability, strengthening of the truss rocker bearing pin plates, longitudinal and transverse restraining elements at the rocker bearing level to prevent instability, combination of steel and concrete confinement jackets for increased ductility and shear resistance, and reinforced concrete overlays at spread footings and pilecaps.

Carquinez Bridges' Seismic Hazard Assessment and Conceptual Retrofit

Évaluation du risque sismique et renforcement des Ponts de Carquinez

Abschätzung von seismischen Gefahren und der konzeptionellen Verstärkung der Carquinez-Brücken

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SUMMARY

This paper summarises the results of a study performed to evaluate the seismic vulnerabilities and develop a conceptual retrofit for the existing Carquinez Bridges near San Francisco. These two steel truss bridges are classified as important bridges by the state. The study involved evaluating the dynamic behavior of the bridges with three-dimensional computer models using various analytical methods, identifying vulnerabilities in the structures based on calculated structural responses, and developing a conceptual retrofit strategy to address the vulnerabilities.

RÉSUMÉ

L'article résume les résultats d'une étude ayant pour objectif l'évaluation du risque sismique et le développement d'un modèle de renforcement pour les ponts de Carquinez, près de San Francisco. Ces deux ponts à treillis sont considérés comme très importants par l'État. L'étude a consisté en l'évaluation du comportement dynamique des ponts à l'aide de modèles tridimensionnels générés par ordinateur et analysés en utilisant diverses méthodes, permettant d'identifier les faiblesses des structures et préparer un plan de renforcement.

ZUSAMMENFASSUNG

Dieser Artikel fasst die Resultate einer Studie zusammen, die die seismische Anfälligkeit der Carquinez-Brücken in der Nähe von San Francisco evaluiert und eine konzeptionelle Verstärkung entwickelt. Diese zwei Stahlbrücken sind vom Staat als wichtig klassiert worden. Die Studie beinhaltet eine Evaluation des dynamischen Verhaltens der Brücken, welche aus einem dreidimensionalen Computermodell gewonnen wurde. Dazu wurden, gestützt auf berechnetem Bauwerksverhalten, verschiedene analytische Methoden verwendet, um Schwachstellen in Konstruktionen zu identifizieren. Diese Studie erlaubt einen Verstärkungsentwurf.

1. INTRODUCTION

1.1 Scope of Study

There are two bridges across the western end of the Carquinez Strait about 40 km north of San Francisco. The first bridge was erected in 1927, the second in 1958. Both are part of Interstate 80, which is a major interstate highway connecting California with states to the east. Since I-80 is a vital link for interstate commerce and important to the regional economy of the San Francisco Bay area, the Carquinez Strait Bridges are classified as important bridges by the state.

This paper summarizes the results of a study [2] performed for the California Department of Transportation (Caltrans) to assess the seismic vulnerabilities of the bridges when subjected to two intensities of ground motion. The highest intensity ground motion was based on a deterministic assessment of the maximum credible earthquake, and is called the "safety level" event. The lowest intensity ground motion was based on a probabilistic assessment of ground motions which have a 40% chance of occurring during the useful life of the bridge, and is called the "functional level" event. Caltrans performance objectives for important bridges include no structural damage or loss of function for the functional level event, and only limited structural damage without loss of function for the safety level event. Upon determination of the seismic vulnerabilities, a conceptual retrofit strategy was developed to meet these objectives.

1.2 Bridge Description

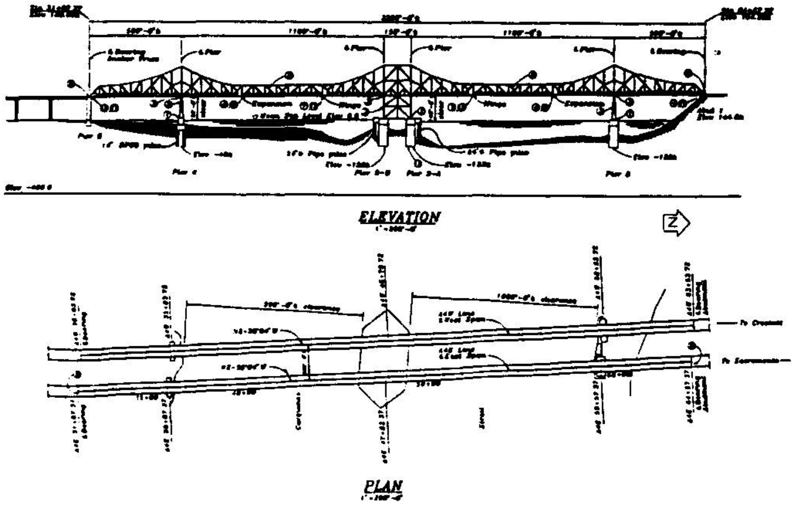

Fig. 1 General Bridge Plan

The two bridges are parallel four-span steel truss structures each with a total length of 1021 m (Fig. 1). Each bridge has two long spans of 335 m composed of suspended spans interconnected to cantilever spans at each end with hinge and expansion joints. The four interior supports for each bridge are tapered steel towers with heights varying from 36 to 41 m. These towers are typically supported by large concrete caissons located in as much as 30 m of water and penetrating 15 m into the soil. The roadway for each bridge is a concrete slab supported by steel floor beams and stringers. Southern approach viaducts for each bridge are approximately 335 m long, and consist of steel girders supported on reinforced concrete piers.

The west bridge was built in 1927 by the American Toll Bridge Company for a total construction cost of $4.6 million. It was the world's longest highway bridge at the time, and was unique since, for the first time, hydraulic buffers were used to provide bottom chord continuity across the two expansion joints during dynamic loading. In addition, the foundation caissons were the deepest-water caissons in the United States when they were constructed. The bridge was sold to the state of California in 1939. The main truss members of the 1927 bridge are built-up laced/riveted steel members. Tension members are typically steel eyebars.

The east bridge was built in 1958 by the state of California. The global geometry of the bridge is almost identical to the 1927 bridge except that it is wider and uses different steel types and member shapes and sizes. Several of the main truss members of the 1958 bridge were constructed using high strength T1 steel to reduce the cost and to minimize secondary stresses. High strength bolted field connections were also used, which was a first for bridge construction.

1.3 Site Soils

The subsurface conditions along the east and west Carquinez Bridge sites consist of a 10 m thick layer of soft to very soft bay mud typically underlain by a 23 m thick layer of stiff alluvium over bedrock. At the north and south ends of the bridges, bedrock is at or near the ground surface.

1.4 Site Seismicity

Two sets of site specific motions were developed by Geomatrix Consultants [3] for the Carquinez bridge site. The first set corresponded to probabilistic equal-hazard spectra for return periods ranging from 100 years to 2000 years. The second set corresponded to the maximum credible earthquakes deterministically defined for the San Andreas (M_w 8), Hayward (M_w 7.3), and Franklin (M_w 6.5) faults, which are as close as 41 km, 13 km, and 1 km, respectively, of the bridge site. For the maximum credible earthquake, free-field motions at each bridge support location were generated at the rock outcrop and at the mudline. The different motions at the support locations account for the variation of the seismic waves (phasing, frequency content, and amplitude) due to the characteristics of the soil profiles and the distance between the supports. The event associated with the Franklin fault dominates the seismic response due to its proximity to the site.

2. ANALYTICAL METHODOLOGY

2.1 Analysis Summary

In order to evaluate seismic vulnerabilities of the as-built bridges, as well as to evaluate the effects of various bridge retrofit strategies, complete three-dimensional computer models were developed for each bridge (Fig. 2). Numerous analyses with varying levels of complexity were performed to study the dynamic behavior of the bridges. Soil modeling was also performed to study the effects of soil-structure interaction. All of the different analytical tools used for this study are described in the sections which follow.

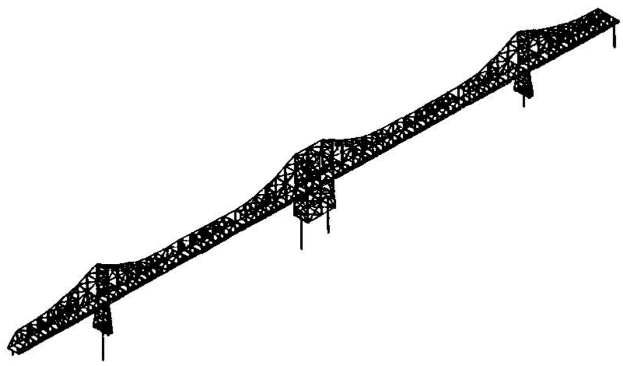

Fig. 2 Finite Element Computer Model

2.2 Soil Modeling

The properties of soil during an earthquake vary with the level of shear strain induced by the earthquake. With increasing shear strain, the shear modulus of the soil decreases and material damping increases. This nonlinear behavior of the soil was simulated by performing a linear unidimensional wave analysis using a computer model of the soil column with the ground motion defined at the rock outcrop. An effective strain equal to 65% of the maximum strain was calculated for each soil layer. New soil properties consistent with this effective strain were then calculated, and the iterative process repeated. The procedure was halted when the soil properties converged to a stable result. The computer program SHAKE [5] was used for the analysis, and the resulting properties included the strain compatible shear modulus, bulk modulus, and damping values for the soil. In addition, free-field time histories compatible with the strain-compatible soil profile were calculated at the mudline at each support location.

The soil-structure interface has frequency dependent characteristics which are represented by complex valued impedance and scattering functions. Impedances describe the load-displacement characteristics of the soil-foundation system, and depend on the soil configuration, material behavior, excitation frequency, and foundation geometry. Scattering functions modify the free-field seismic motion due to the presence of the foundation. The SASSI [4] and CLASSI [6] families of programs were used to calculate these functions at each support location. These functions were used for the soil-structure interaction analyses performed for the bridges.

2.3 Response Spectra Analyses

Response spectra analyses were performed for the safety and functional level events using the computer program SAP90 [1]. This was the most widely used method of analysis in this study because it provided a simple and efficient way to assess the basic dynamic behavior of the structures and consider the relative effects of certain variables on the overall system response.

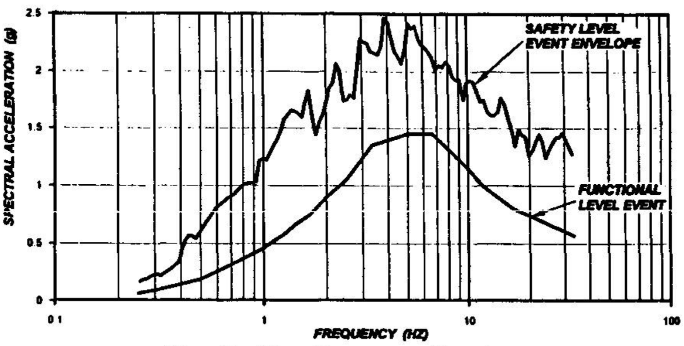

Fig. 3 Site Response Spectra

For the safety level earthquake evaluation, the ground motion time histories were used to calculate longitudinal, transverse, and vertical response spectra for each support location. These spectra for each support were then enveloped to develop a single set of three orthogonal response spectra. For the functional level analysis, the equal-hazard response spectrum for a 300-year event was used. This is consistent with the selected occurrence probability stated in Section 1.1 for a useful bridge life of 150-years. (Fig. 3).

Although all of the response spectra analyses performed were linear elastic, non-linear behavior was simulated in some instances by iteratively reducing member stiffnesses. Response spectra analyses were essential to develop a basic understanding of the bridge behavior and to provide a benchmark to assess the effects of time history loading, soil-structure interaction, and multiple-support excitation.

2.4 Single Input Time History Analysis

Single input time history analyses were performed to study the response of the bridges to time history loading. The safety level ground motion time histories at one support location were applied simultaneously to all of the supports. The structural responses calculated in this analysis were generally within 5 to 20% of the responses calculated in the response spectra analyses. Since the multiple-support excitation analysis described in the next section provided a more accurate representation of ground motion, the single input time history analyses were not used for final vulnerability and conceptual retrofit assessment. The primary benefit in performing this analysis was to obtain a better understanding of the bridge behavior under time history loading.

2.5 Soil-Structure Interaction Analysis Considering Multiple-Support Excitation

Due to the distance between supports and the variation of the soil profile along the bridge alignment, the seismic motion at each support differs in phasing, frequency content, and amplitude. In addition, the interaction between the soil and the structure can have a large impact on the dynamic response of the bridges. Therefore, a soil-structure interaction analysis considering multiple-support excitation was performed to provide the most accurate representation of the dynamic response of the bridges during the seismic event.

The bridge response to multiple-support excitation contains two components. A dynamic component represents the bridge inertial response to the seismic input. A pseudo-static component represents the bridge response to relative foundation motions. The total response is the sum of these components. This analysis was performed in the following manner:

- A modal analysis was performed assuming fixity at the soil-foundation interface. The mode shapes were used to determine the inertial response of the superstructure and foundation elements to the input ground motion.
- Pseudo-static mode shapes were calculated by applying a unit displacement sequentially at each degree of freedom at the soil-foundation interface (6 supports x 6 DOF/support = 36 total shapes). These shapes relate the bridge response to differential foundation motion.

- Frequency dependent impedance matrices were calculated for the soil region at each support.
- Incident wave scattering functions were calculated to consider the effect of motion deconvolution and foundation geometry on the free-field ground motion input.

This information was combined to calculate the response of the coupled soil-structure system using the CLASSI computer programs. Free-field acceleration time-histories were applied to each support at the soil-structure interface and the responses were calculated in the frequency domain using Fourier analysis methods. This was the most advanced analysis performed for this study.

3. AS-BUILT BRIDGE EVALUATION

3.1 Dynamic Behavior

The analyses performed for the as-built bridges described in Section 2 provided valuable information on the behavior of the bridges for the safety and functional seismic events. The following is a partial summary of the expected bridge behavior:

- The natural frequency of each bridge superstructure is 0.3 Hz in the transverse direction and 0.6 Hz in the longitudinal direction.
- Displacements and forces obtained from the soil-structure interaction multiple-support excitation analysis averaged 25% less than response spectra results. Primary reasons for this reduction are the effects of motion phasing and soil radiation damping, and the fact that the response spectra were developed by enveloping the ground motion at the different support locations, resulting in a more conservative definition of motion.
- The concrete caissons which support the steel truss towers are extremely large, and make up almost 85% of the total system mass. Since the majority of the mass is located in the foundation, and the superstructure is relatively flexible, there is a distinct separation between the foundation response and the superstructure response to the ground motion. While the superstructure responds to low frequency excitation due to its flexibility, the foundation responds to high frequency excitation near the peak of the response spectra. The majority of the seismic forces acting on the foundation are generated by inertia of the foundation itself.

3.2 Seismic Vulnerability Assessment

Based on Caltrans' performance objectives for important bridges, a set of assessment criteria was developed to allow a quantitative evaluation of demand to capacity ratios to identify vulnerabilities. Refer to Reference 2 for a description of the criteria used for this study. The following is a partial list of vulnerabilities identified for the bridges:

- Certain primary truss members and critical components and connections, including tower columns and diagonal members, suspended span bottom chord members, and hydraulic buffers, have calculated forces which exceed their capacities. In the 1927 bridge, seismic compression demands during the safety level event are likely to overcome tensile gravity loads in eyebar members, resulting in member buckling.
- At the ends of each bridge, there is an existing 15 cm separation between the steel truss and the concrete abutment. Longitudinal displacements of each bridge are expected to exceed this gap, resulting in large impact forces between the truss and the abutments.
- Due to the large inertial forces which generate in the caissons, caisson rocking and sliding may occur during the safety and functional level events.

4. CONCEPTUAL RETROFIT

4.1 Global Retrofit Strategy

Several global retrofit strategies to address bridge vulnerabilities were studied. The computer models were modified and analyzed to assess the effects of retrofit on the bridge response. The final strategy recommended for each bridge included the following:

- Allow unrestricted longitudinal movement of the bridges by increasing the abutment gap and retrofitting the rocker bearings at each abutment.
- Strengthen selected steel members and connections, and replace hydraulic buffers.
- Interconnect the foundation caissons to control rocking response.

This retrofit strategy strengthens but does not otherwise alter the existing lateral load path. It is important to note that further verification of the foundation retrofit is necessary. Further analyses based on detailed soil borings at the site are recommended for the final design stage to more accurately assess the effect of non-linear foundation response.

4.2 Conceptual Retrofit Details

Conceptual retrofit details were developed for various structural elements and components in accordance with the recommended retrofit strategy. These details include the following:

- As described above, seismic compression demands could overcome gravity tensile loads in eyebar members. To prevent member buckling, new plates connected by high-strength bolts would be added at the top and bottom of the eyebars (Fig. 4) to create a box-shaped section capable of resisting compression.
- To strengthen typical steel truss members, new cover plates would be bolted to the members. Bolted connections were recommended due to potential difficulties in field welding to existing steel.

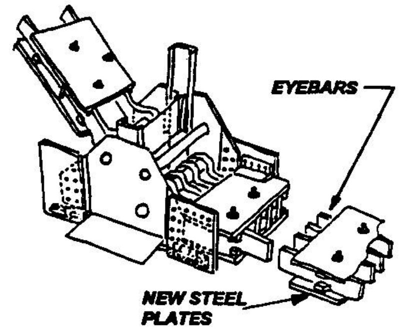

Fig. 4 Conceptual Eyebar Retrofit

4.3 Retrofit Cost Estimate

The total cost to retrofit the 1927 and 1958 Bridges for the safety level event was estimated to be $29 million and $24 million, respectively. This cost includes the superstructure, foundation, and approach structure retrofit for each bridge.

5. ACKNOWLEDGMENTS

The authors wish to acknowledge Mr. Stan Larsen, Mr. Jim Gates, Mr. Mark Yashinsky, Mr. Bob Bridwell, Mr. Brian Maroney, and Mr. Todd Day of the Caltrans Division of Structures, and Mr. Jim Roberts, Deputy Director for Transportation Engineering, for their assistance during this project.

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Seismic Response of the Tagus River Bridge, Portugal

Comportement du Pont sur le Tage, Portugal, vis-à-vis des séismes

Seismisches Verhalten der Tagus-Brücke in Portugal

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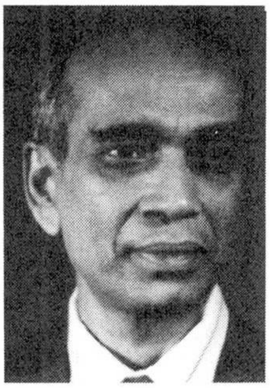

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SUMMARY

To carry the railroad over the Tagus River Bridge in Portugal, the design strengthens the existing bridge with a second cable system. In the original design, the bridge was evaluated for seismic design criteria significantly different from those used today. This paper outlines the present analysis procedure and the seismic response of the modified Tagus River Bridge.

RÉSUMÉ

Afin de pouvoir supporter le trafic ferroviaire, le projet prévoit le renforcement du pont sur le Tage avec un deuxième système de câbles. Dans le projet original, le pont avait été calculé pour des forces sismiques assez différentes des valeurs retenues actuellement. L'article présente la méthode actuelle de calcul et le comportement du pont modifié vis-à-vis des séismes.

ZUSAMMENFASSUNG

Zur Überführung einer neuen Eisenbahnlinie über die Tagus-Brücke verstärkte der neue Entwurf die damalige Brücke mit einem zweiten Kabelsystem. Die seismischen Kriterien des neuen Entwurfs unterscheiden sich stark von den alten Anforderungen. Der Artikel fasst die neuen Rechnungsverfahren und das seismische Verhalten der modifizierten Tagus-Brücke zusammen.

1. INTRODUCTION

In 1992 Junta Autonoma de Estradas (JAE) of Portugal awarded a design contract to Steinman, Inc, New York for the installation of a new railroad deck on the suspension bridge over the Tagus River in Lisbon. The contract also calls for the widening of the existing upper roadway deck from five to six lanes.

Tagus River Bridge, designed by Steinman for the US Steel Export Company, was opened to vehicular traffic in 1966. The suspended spans of the Tagus River Bridge are 483 m, 1013 m and 483 m. The stiffening truss is 2271 m long and continuous throughout the suspended spans and three backstay spans.

To carry the railroad, Steinman's design strengthens the existing bridge with a second cable system supported at new anchorages and extensions to the existing towers and bents. The new lower deck, designed to carry the railroad, will be an orthotropic system of new floorbeams, laterals and railway stringers, all of which will participate with the existing truss bottom chords. Construction is scheduled for 1995-1998. Fig.1 shows the proposed bridge elevation and cross section.

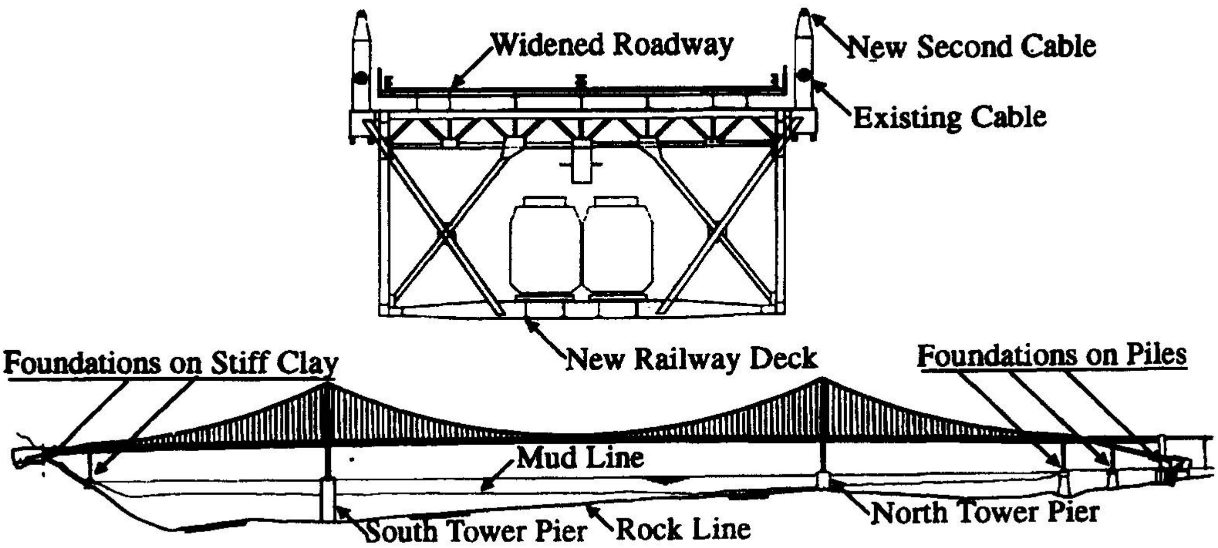

Fig. 1: Tagus River Bridge Elevation and Cross Section

2. SEISMIC ACTIONS

In 1961, seismic action was quantified as 10% of the dead load acting either in the longitudinal or transverse direction. The present Portuguese Code, RSA, characterizes Seismic actions by Response Spectrum with 5% damping (Fig. 2). Due to the importance of the structure, Steinman engaged Prof. George Gazetas, a soil-structure-dynamics expert, to provide realistic support excitation data and support spring constants for the Tagus River Bridge. Prof. Gazetas in association with a team of experts (Prof. M.K. Yegian, Prof. P. Dakoulas, Dr. V.G. Ghahraman, Dr. H. Abou-Seed, Mr. G. Mylonakis and Ms. A. Nikolau) studied the site seismicity and soil data. They recommended the following:

1. A 15 second seismic action with a peak acceleration (PGA) of 0.25g at the rock level. This seismic action (NEAR EQ) is assumed to originate at a nearby source and has an estimated return period of 200 years. NEAR EQ must not damage the structure.
2. A 55 second 0.15g PGA seismic activity originating at a distant source (DISTANT EQ) with an estimated return period of 2000 years. The structure may undergo repairable damage during DISTANT EQ.

A thorough soil-structure interaction study was undertaken. Using a strain-compatible soil profile, a free-field soil seismic response analysis of a multi-layered soil column was performed. The seismic responses of the tower caissons (partly embedded in soil) were obtained from finite element models of the soil-structure system. The caissons were modeled as three dimensional solid elements. The surrounding soil was modeled without artificial lateral boundaries. Hydrodynamic masses and the flexibility of the bedrock were considered. The soil-structure interaction study provided the following information for NEAR EQ and DISTANT EQ in the longitudinal and transverse directions:

1. Time histories of accelerations at the base of the towers, bents and anchorages (See Fig. 3 for typical examples).
2. Translational and rotational spring constants for the base of the towers, bents and anchorages.

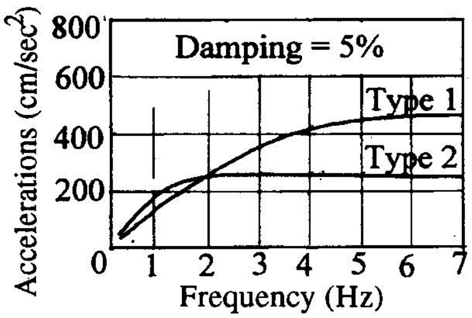

Fig.2: RSA Response Spectra

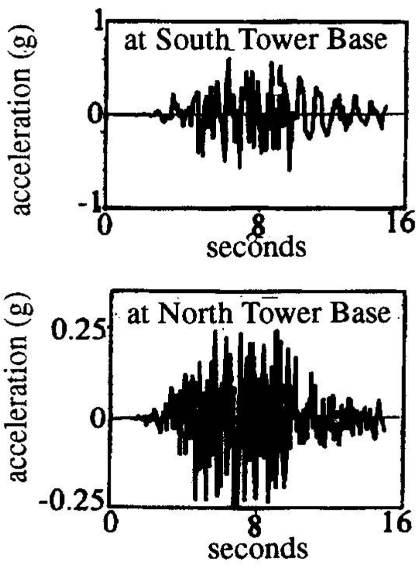

Fig.3: Long. Motion (NEAR EQ)

3. ANALYSIS PROCEDURE

Determination of the seismic response corresponding to the original 1961 design criteria is a simple static analysis problem. Seismic actions characterized by RSA spectra require a standard spectral analysis. Steinman's seismic action criteria using different support excitations demand an elaborate and time consuming time-history integration (See Table 1).

Analysis	Seismic Action	Mass Excitation	Duration	Integration Time Step
Time-history:	NEAR EQ	Longitudinal	15 sec	0.125 sec
	NEAR EQ	Transverse	15 sec	0.125 sec
	DISTANT EQ	Longitudinal	55 sec	0.250 sec
	DISTANT EQ	Transverse	55 sec	0.250 sec
Spectral	RSA: Type 1	Longitudinal		
	RSA: Type 1	Transverse		
	RSA: Type 2	Longitudinal		
	RSA: Type 2	Transverse		
1961 Style (Static)	10% of DL	Longitudinal		
	10% of DL	Transverse		

Table 1: Seismic Analysis of the Tagus River Bridge

An in-house computer program, developed by the primary author, was used for all of the design and analysis phases. The program uses standard matrix methods for non-linear structural analysis. The DYNAMICS module of the program computes mode shapes and frequencies, and performs standard spectral analysis. For the time-history analysis, the program module uses Newmark's step-by-step direct integration method and computes the response of the structure due to a given time-dependent load $\{F_t\}$ by solving the basic dynamic equilibrium equation:

$$[M]\{U''\} + [C]\{U'\} + [K]\{U\} = F_t$$

Before proceeding with time-history integration, the following items needed clarification:

1. Integration Time Step
2. Different Support Excitations
3. Quasi-Static Stresses due to Support Displacements
4. Computer Model

3.1 Integration Time Step

Ordinarily, one tenth of the period of the significant lowest mode of the structure is taken as the integration time step. The fundamental periods of the structure in the longitudinal and transverse directions are 5.76 seconds and 15.03 seconds respectively. The integration time steps indicated in Table 1 are much less than one twentieth of the fundamental period. To establish integration time steps, plots of spectral accelerations were generated by integrating the support motions (Fig. 3) at 0.02, 0.04, 0.125, 0.25 and 0.5 second time steps. The chosen integration time steps yielded response spectra almost identical to those with much smaller time steps. This was particularly true within the range of the important lowest few frequencies of the structure. The maximum stress obtained with the integration time steps in Table 1 is 88% of the yield stress. A smaller time step at the cost of substantial computational effort may have produced slightly higher stresses without significantly impacting the results.

3.2 Different Support Base Excitations

The in-house computer program uses time-dependent load vector F_t to account for the different support base excitations. For each earthquake, time-dependent load vectors were generated for every joint in the computer model. In other words, the structure was analyzed for time-dependent joint loads.

At any time, t, load F_j at joint j was taken as

$F_j = M_j \sum \delta_{kj} U''_k$ where

M_j = mass at joint j

δ_{kj} = displacement of joint j due to a unit displacement of support k while all the other supports are immovable.

δ is assumed to vary linearly between support k and the adjacent support.

U''_k = acceleration of support k.

The summation applies to all supports.

3.3 Quasi-Static Stresses due to Support Displacements

The quasi-static stresses in the structure are the stresses due to the support displacements corresponding to time t. If all of the support displacements at any time, t, are the same, the structure experiences only a rigid body motion and there are no quasi-static stresses. From the soil-structure interaction study, the maximum quasi-static displacements of 0.15 m and 0.05 m occur at the base of south and north towers respectively. Member stresses due to these maximum displacements were found to be less than 5% of the member yield strength. Compared to the tower heights, the quasi-static displacements are very small, and thus were ignored from further consideration.

3.4 Computer Model

In time-history integration, the structure is analyzed independently for every time step. For 55 seconds of seismic action with an integration time step of 0.25 second, 220 independent analyses are required. To minimize the computational effort, a simplified 3D model of the Tagus River Bridge was used for the seismic analysis. The bridge was idealized as a space frame with 482 joints and 1000 members. The stiffening trusses were modeled with equivalent beams supported by half the actual number of suspenders. The bents and towers in the model were supported on springs. The lowest significant mode shapes and frequencies of the seismic model were compared with the corresponding values obtained from a more elaborate 3D model of the bridge composed of 3062 joints and 7044 members. The mode shapes from the two models were identical, while the difference in the corresponding frequencies was less than 2 percent.

4. RESULTS

The time-history analysis was carried out for NEAR EQ and DISTANT EQ, with separate analyses for longitudinal and transverse directions (See Table 1). Typical examples of deformations and member stresses are given in Fig.4 and Fig. 5.

The stresses in the bents and towers due to the seismic excitations included in this study did not exceed the yield stress. The south tower was found to experience tension at the base during DISTANT EQ acting in the longitudinal direction (Fig. 5). The corresponding south tower anchor bolt tension is approximately 69 N/mm^2. The south tower has a long caisson through mud and thus reacts unfavorably to seismic actions. However, considering the return period of DISTANT EQ and the low magnitude of the south tower tensions the situation was not considered of great concern. No such tension in the towers occurred due to the NEAR EQ.

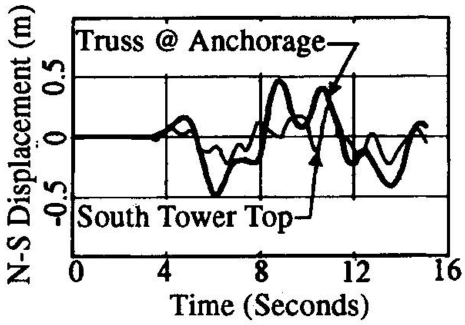

Fig. 4: System Response (NEAR EQ @ N-S)

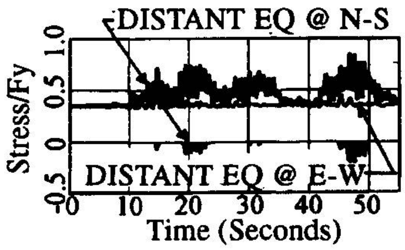

Fig. 5:South Tower Bottom Stresses

Many bridge failures are due to spans slipping from their supports during intense seismic actions. In the Tagus River Bridge, the truss roller movements at the anchorages due to seismic actions were found to be less than the movements due to live load and temperature. Furthermore, the existing bridge is equipped with restraints to limit the roller movements at the anchorages.

A comparison of the results from the various analyses identified in Table 1 found that the RSA Spectral Analysis under-estimated both the deformations and member stresses obtained from the time-history analysis, while the 1961 style equivalent static analysis grossly overestimated the deformations and underestimated the tower and bent stresses. The reconstructed Tagus River Bridge is capable of resisting stresses obtained from all the three types of seismic analysis.

5. ACKNOWLEDGMENTS

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Research on Seismic Damage of Reinforced Concrete Bridge Piers

Recherche sur les dommages sismiques des piles de pont en béton armé

Forschung über Erdbebenschäden an Stahlbetonbrückenpfeilern

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SUMMARY

In this paper, seismic damage of reinforced concrete bridge piers is discussed. The paper presents experimental research on the damage to reinforced concrete bridge piers subjected to reversed cyclic lateral loading, as well as on the damage mechanism of reinforced concrete bridge piers. A modified low-cycle fatigue damage model combined excessive deformation and low-cycle fatigue influence. Variables included confinement reinforcement and shear span ratio. Test results indicate that failure modes mainly depend upon shear span ratio, and increasing confinement reinforcement can improve the ductility of reinforced concrete bridge piers. Besides, crack propagation is related to the deterioration of strength and stiffness.

RÉSUMÉ

L'auteur présente une recherche expérimentale menée sur les dommages subis par les piles de pont en béton armé, soumises à des efforts horizontaux cycliques et alternés; il fournit aussi des informations sur les mécanismes significatifs de dégradation. Il expose un modèle pour étudier la fatigue sous faible cycles de charges combinée aux effets d'une déformation extrême. Ce modèle prend en compte deux variables, les armatures de frettage et le rapport portée-cisaillement. Les résultats montrent que le mode de rupture dépend essentiellement du rapport portée-cisaillement, et que l'augmentation du pourcentage d'armatures de frettage améliore la ductilité des piles en béton armé.

ZUSAMMENFASSUNG

Der Beitrag berichtet über experimentelle Forschung an Stahlbetonbrückenpfeilern unter zyklischer Horizontalbelastung und über die relevanten Schädigungsmechanismen. Ein modifiziertes Schädigungsmodell für Ermüdung unter wenigen Zyklen mit extremer Deformation wurde entwickelt, in das als Variablen die Umschnürungsbewehrung und das Schubspannweitenverhältnis eingehen. Die Versuchsergebnisse zeigen, dass die Versagensform vor allem vom Schlussspannweitenverhältnis abhängt, während die Duktilität mit dem Gehalt der Umschnürungsbewehrung zunimmt.

1. RESEARCH SIGNIFICANCE

The experimental research presented in this paper provides design information for earthquake-resistant RC bridge piers. The aim of theoretical work is to study the method which can evaluate the behaviour of RC bridge piers in earthquake . Improvements for seismic performance of RC bridge piers are also discussed.

2. EXPERIMENTAL PROGRAM AND RESULTS

2.1 Description of Test Piers

The test bridge piers were about one-third scale model of practical bridge piers designed in accordance with China Railway Bridge Design Code ,TBJ2-85, and constructed with ready-mixed concrete using pea gravel in practical environment. Test piers were representative of a column between the foundation and the lateral loading point. Fig.1 illustrates the pier geometry. The strength of longitudinal and transverse steel are 395.9 and 235MPa, respectively. A summary of test piers properties is presented in Table 1.

Test piers	Concrete strength, (MPa)	Transverse steel Percent (%)	Transverse steel Spacing (mm)	Height (m)
P-1	27.9	0.26	80	1.5
P-2	30.9	0.21	100	2.0
P-3	30.3	0.21	100	2.5
P-4	32.9	0.26	80	2.0
P-5	39.4	0.33	50	2.0

Table 1 Properties of test piers

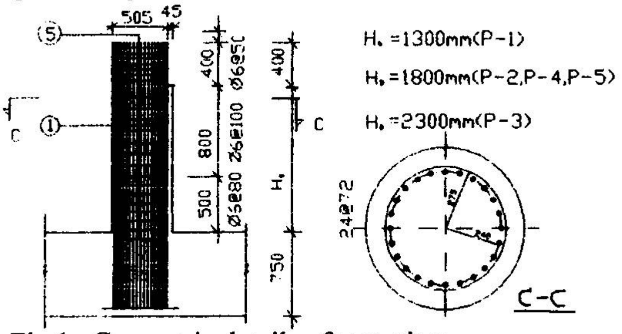

Fig.1 Geometric details of test piers

Test variables included confinement reinforcement and shear span ratio. According to the two variables, the piers were divided into two groups. The first group contained piers labeled as P-1, P-2 and P-3, respectively. Their effective heights were 1.5, 2.0 and 2.5 meter,respectively. The second group contained piers labeled as P-2, P-4 and P-5, respectively. They had same effective height 2.0 meter, but the spacings of confinement reinforcements were 100, 80 and 50 millimeter, respectively. All test piers had a 550 millimeter diameter and 1.14% longitudinal steel ratio .

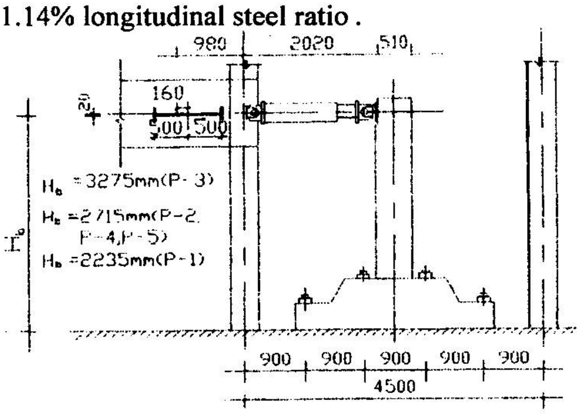

Fig.2 Test setup

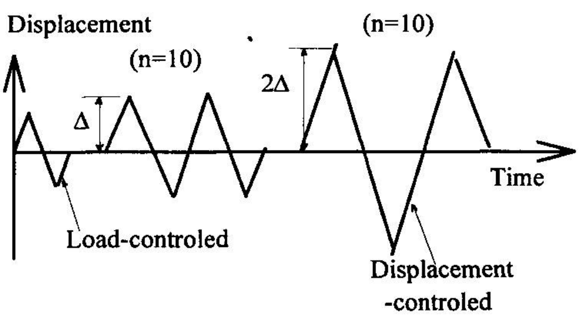

Fig.3 Load history

2.2 Test Setup and Testing Procedure

In this test program, the test setup was used to apply the lateral load by a 200KN capacity actuator. Fig.2 illustrates the test setup. No axial load was applied in this test program . The foundations of piers were fixed on the laboratory floor. The foundations were constructed having very high stiffness to assure no rotation occurrence during loading.

The piers were instrumented for deflection and steel strain measurements. The steel strains were measured by using dynamic electrical resistance strain gages. The top deflection and applied load can be measured by automatic data acquisition system. The load history adopted in the test is shown in Fig.3, it consists of two stages. In the first cycle, the piers were subjected to load-controlled lateral load reversals. It is used to determine the yield displacements (referred to as Δ_y throughout this paper) experimentally. In the following cycles, cyclic lateral displacements were increased by multiples of Δ_y. The lateral load was applied at the top of the pier, and the cycling was performed under displacement control. Pier failure was defined as the point at which the extreme tensile longitudinal bar fractured or the extreme compressed longitudinal bar buckled.

2.3 Test Results and Discussion

As expected, the failure mode for Model P-3 (shear span ratio 4.8) was a typical flexural failure. Horizontal flexural cracks were formed in the plastic hinge region, followed by gradual extension of the cracks around the circumference of the model pier. The increased lateral displacement resulted in spalling of concrete at the base of the column to a height of approximately one pier diameter. At the end, external longitudinal reinforcement fractured in the plastic hinge region.

The failure mode for Models P-2, P-4 and P-5 (shear span ratio 3.9) was similar to that for the flexural pier (P-3), except that the extensive diagonal cracks formed on the sides of piers in the plastic hinge region prior to spalling. Despite the presence of diagonal cracking, the pier shear span ratio of 3.9 was not sufficiently low to permit a true shear failure. The effection of transver bar spacing to ductility was remarkable. Ductility of P-5 (spaing of transver bar was 50mm) was 1.4 times of P-2 (spaing of transver bar was 50mm).

The failure mode for Model P-1 (shear span ratio 2.9) was a typical shear failure. Following side concrete spalling in plastic hinge region, the pier broke along a main diagonal crack, very large shear deformation was observed. The final damage states of model piers are illustrated in Fig.4.

The overall performance of each pier was measured by plotting the lateral displacement at the top of the pier as a function of the lateral load,as shown in Fig.5. The load-deflection curves exhibited stable behavior until the strength and stiffness deterioration were serious. Seismic properties of test piers are given in table 2.

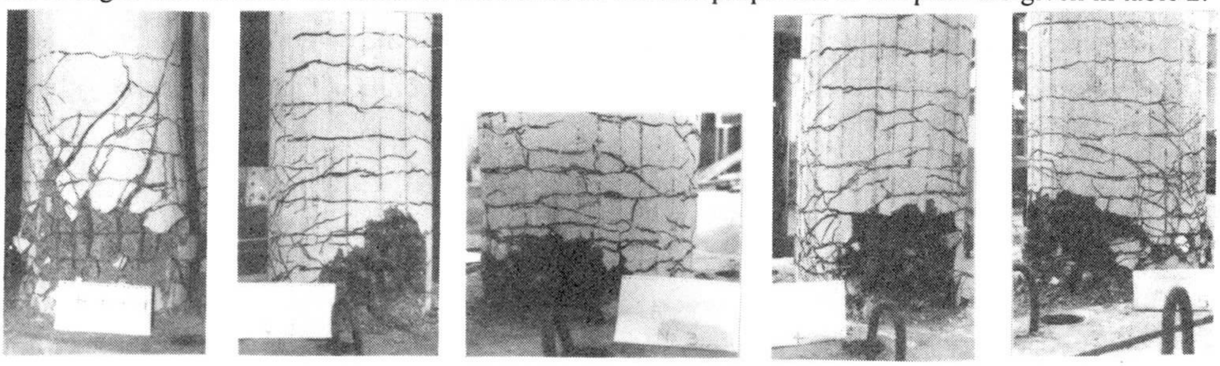

Fig.4 Final damage states of test piers .

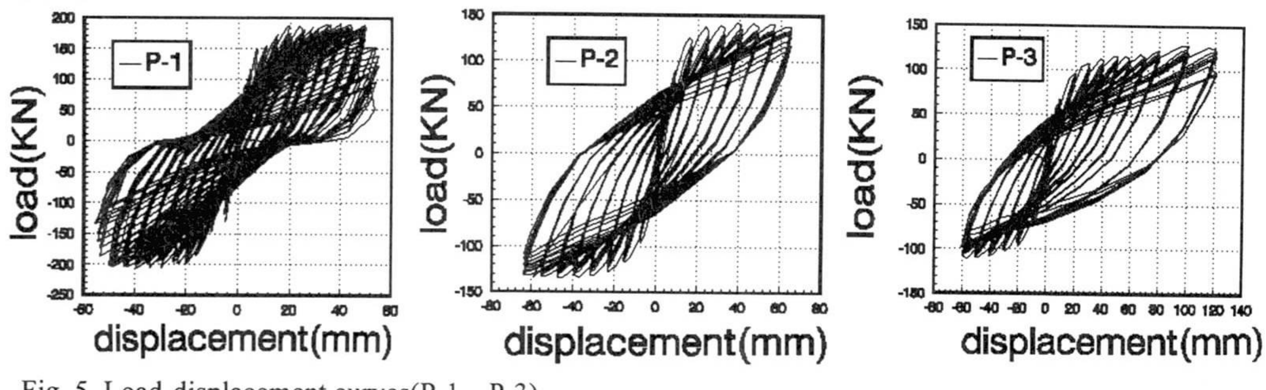

Fig. 5 Load-displacement curves(P-1 ~ P-3)

Test piers	Δ_y mm	P_y KN	Δ_u mm	P_u KN	Δ_u / Δ_y	P_u / P_y	Absorbed energy(KN-M)	s, mm	height m
P-1	9	135.87	52.6	187.63	5.85	1.38	513.294	80	1.5
P-2	13.7	118.37	64.7	140.48	4.72	1.18	393.102	100	2.0
P-3	19.4	96.48	120.2	128.55	6.2	1.33	572.31	100	2.5
P-4	13.7	118.4	*	*	*	*	*	80	2.0
P-5	13.3	110.46	81.3	132.9	6.11	1.20	583.159	50	2.0

Table 2 Seismic properties of test piers(* represents that no data was recorded.)

3. DAMAGE MECHANISM OF RC BRIDGE PIERS

3.1 General Discussion

Conventionally, the energy-absorption capacity for RC member is measured by ultimate ductility, but the index of ductility is not sufficient as a damage indicator for evaluating the damage sustained by RC members in earthquake. The main reason is that ductility index only reflects the influence of the amplitude of earthquake acceleration, the effect of earthquake duration is not considered. Hence, many damage models were proposed , some of them are tied to the dissipated energy during cyclic loading[1], others are based on the stiffness deterioration or accumulation of plastic deformation[3], and again others employ a linear combination of dissipated energy and normalized displacement[4],etc. The effect of duration to damage sustained by RC structure is commonly considered as "low-cycle fatigue". Because of the hysteretic behaviour of RC material, there is even less justification to apply Miner's hypothesis to reinforced concrete. The displacement at which energy dissipation reaches a maximum value is referred to as Displacement Barrier of low-cycle fatigue.

3.2 A New Damage Definition

The damage index is defined as the ratio of strength drop Δf_i at a specific deformation level to the elastic strength f_i at same deformation level (see Fig. 6). According to this definition, the damage index D_s can be expressed in the form

$$D_s = 1 - \frac{k_i}{k_y} \tag{1}$$

where $k_i = P_i/\delta_i$ is equivalent stiffness and $k_y = P_y/\Delta_y$ is elastic stiffness.

Defining $D_s = 1$ is corresponding failure, some normalized factor had to be introduced. Let $k_i = k_f$ (see Fig.6) be corresponding $D_s = 1$ in equation 1, the normalized factor λ is obtained in the form

$$\lambda = \frac{k_y}{k_y - k_f} \tag{2}$$

The stiffness-based damage index is defined as

$$D_s = \lambda(1 - \frac{k_i}{k_y}) \tag{3}$$

3.3 A Modified Low-Cycle Fatigue Damage Model

The damage index D_e proposed by Chung, Meyer and Shinozuka [2] is defined as

$$D_e = \sum_i \sum_j (\alpha_{ij}^{+} \frac{n_{ij}^{+}}{N_i^{+}} + \alpha_{ij}^{-} \frac{n_{ij}^{-}}{N_i^{-}}) \tag{4}$$

where i: indicator of different displacement or curvature levels, j: indicator of cycle number for a given load i, N_i : number of cycles (with curvature level i) to cause failure, n_{ij} : number of cycles (with curvature level i) actually applied, α_{ij}: damage accelerator, and +, - : indicator of loading sense.

The main problem of damage index in Eq.4 is that assuming the beginning-point of low-cycle fatigue is at yield point. This assumption is not corresponding to the experimental results of model piers. In fact, the damage sustained by model piers under cyclic reversed load mainly depends upon the maximum displacement experienced by model piers in large range of displacement (generally three or four times Δ_y). When the displacement exceeds a specified value called the barrier of low-cycle fatigue, the cyclic effect become remarkable. The influence of load history is considered by introducing a so-called damage-factor α. The damage-factor α is defined as

$$\alpha = 1 - \frac{|\delta_{\max}|}{\delta_f} \tag{5}$$

If ΔP_i denotes the strength drop in one load cycle for given displacement level, and it can be expressed as

$$\Delta P_i = pk_y(\delta_f - \delta_y)\left(\frac{\delta_i - \delta_y}{\delta_f - \delta_y}\right)^{\omega} \tag{6}$$

ω is a constant number, P_{fi} denotes the failure strength for different displacement levels , and it is defined as(Fig.7)

$$P_{fi} = P_f \frac{2\Delta_i}{\Delta_i - 1.0} \tag{7}$$

where P_i: failure strength for monotonic loading, $\Delta_i = \delta_i / \delta_f$: displacement ratio; δ_f : failure displacement for monotonic loading. Finally, the damage index of modified low-cycle fatigue is defined as

$$D_m = \frac{|\delta_{\max}|}{\delta_f} + \sum_i \sum_j \alpha \frac{n_{ij}}{N_i} \tag{8}$$

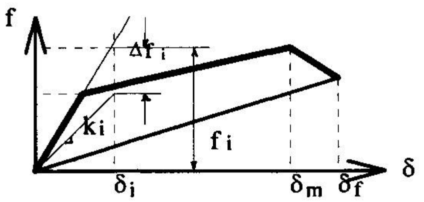

Fig. 6. Stiffness-based damage index

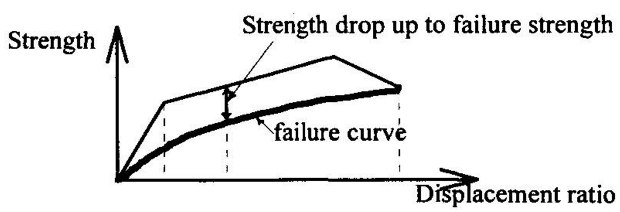

Fig.7 Definition of failure curve

We use Eq.3, Eq.4 and Eq.8 to calculate the damage index of model pier P-2 under cyclic reversed lateral load, the results are summarized in Table 3

	Amplitude	Maximum displacement (m m)	Minimum displacement (mm)	Maximum load (KN)	Minimum load (KN)	D_e(Eq.3)	D_e(Eq.4)	D_e(Eq.8)
$2\delta_0$	n=1	16.6	-15.2	122.493	-116.37	0.157	0.0000	0.2024
	n=10	16.9	-15.2	111.050	-108.89	0.248	0.0000	0.2061
$3\delta_0$	n=1	24.6	-23.5	132.813	-127.14	0.450	0.0004	0.3002
	n=10	24.8	-23.3	121.229	-117.93	0.510	0.0037	0.3076
$4\delta_0$	n=1	32.7	-31.5	136.948	-131.39	0.620	0.0076	0.4013
	n=10	32.7	-31.4	123.583	-121.14	0.670	0.0435	0.4129
$5\delta_0$	n=1	40.6	-39.3	140.478	-131.78	0.727	0.0603	0.5120
	n=10	40.7	-39.1	127.17	-122.79	0.765	0.2185	0.5525
$6\delta_0$	n=1	49.1	-47.6	137.917	-133.86	0.810	0.2728	0.6598
	n=10	48.6	-47.2	127.463	-124.92	0.835	0.7493	0.7372
$7\delta_0$	n=1	57.0	-55.5	141.155	-132.60	0.863	0.8946	0.8497
	n=10	56.7	-55.0	129.661	-124.18	0.885	2.1849	1.0135
$8\delta_0$	n=1	64.7	-63.7	137.747	-131.77	0.909	2.5373	1.1065
	n=10	64.4	-63.8	122.599	-109.39	0.960	2.2432	1.3139

Table 3 Test results and damage index of model pier P-2

As shown in Table 3, the results given by the damage index of modified low-cycle fatigue coincide with the results given by the stiffness-based damage index very well.

4. CONCLUSIONS

According to experimental results and theoretical analysis, the following conclusions are obtained:

(1) The failure modes mainly depend upon the shear span ratio ;

(2) Increasing amount of confinement reinforcement in plastic hinge region can improve the ductility of RC bridge piers;

(3) The damage sustained by RC bridge piers under cyclic reversed load is related to the stiffness deterioration;

(4) Modified low-cycle fatigue damage is suited for quantifying the damage sustained by RC bridge piers in earthquake.

ACKNOWLEDGMENT

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Improving Seismic Performance of Outrigger Knee Joints

Amélioration de la résistance sismique des portiques de support d'autoroutes surélevées

Verbesserung des seismischen Verhaltens von Brückenauslegerverbindungen

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SUMMARY

Improving the seismic performance of bridge outrigger knee joint systems became imperative after the 1989 Loma Prieta earthquake. The existing outrigger knee joints were experimentally evaluated on half-scale specimens. Two upgrade strategies were proposed and tested on prototype specimens. The "strong" strategy was chosen for the final upgrade design, tested on three specimens and recommended in the form of upgrade design guidelines.

RÉSUMÉ

A la suite du séisme de Loma Prieta en 1989, il a fallu revoir la conception des portiques de support des autoroutes surélevées. Les connections entre la colonne et la poutre de support ont particulièrement souffert. Des connections de conception traditionnelle ont été testées en laboratoire a l'échelle 1:2. Deux stratégies de renforcement ont été proposées et testées sur différents prototypes. La stratégie "forte" a été adoptée comme solution finale. Cette stratégie a été testée sur trois spécimens additionnels et recommande un renforcement des directives de projet.

ZUSAMMENFASSUNG

Nach dem Loma Prieta Erdbeben im Jahre 1989 wurde ersichtlich, dass Verbesserungen des seismischen Verhaltens an Brückenauslegerverbindungen unbedingt erforderlich sind. Bis zu dem Zeitpunkt übliche Brückenauslegerverbindungen wurden experimentell an Modellen im Maßstab 1:2 getestet. Zwei Verbesserungsstrategien wurden vorgeschlagen und in Modellversuchen getestet. Die "starke" Strategie wurde schließlich für den endgültigen Verbesserungsentwurf gewählt. Nach dem Testen dieser Strategie an drei Modellen wurden Richtlinien zum Verbesserungsentwurf empfohlen.

1 INTRODUCTION

The catastrophic collapse of the Cypress Viaduct during the Loma Prieta 1989 earthquake emphasized the vulnerability of elevated freeway bridge structures. One track of the joint California Department of Transportation and University of California at Berkeley research project is the investigation of outrigger beam and knee joint systems found in elevated freeway bents (Figure 1). This project has two principal goals: to evaluate the behavior of the existing outrigger and knee joint systems under a combined transverse and longitudinal loading and to devise and experimentally verify upgrading strategies and repair techniques suitable for improving the seismic performance of outrigger knee joints [1].

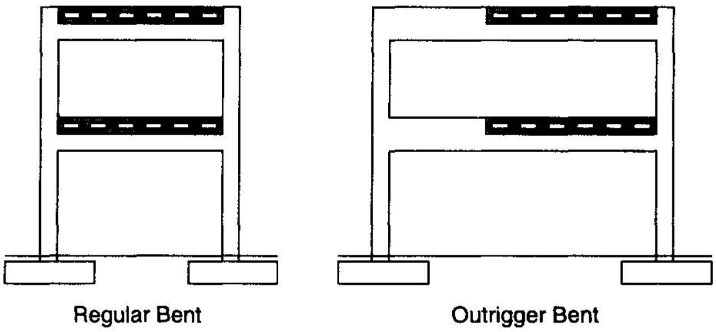

Figure 1: Elevated freeway bents.

2 EXPERIMENT SETUP

The outrigger knee joint specimens are scaled models of the existing outrigger knee joint systems. The length scale factor of 2 and a model/prototype stress identity similitude requirements governed the specimen design process. The choice of materials and the specimen details reflect the features of the elevated freeway structures designed in the San Francisco bay area during 1950's and 1960's.

The specimens were placed in the loading frame in up-side-down position to facilitate loading and anchoring (Figure 2). The actuator displacements were computer controlled to apply the loading in a quasi-static manner. The loading pattern, designed to simulate the outrigger knee joint earthquake loading, models the simultaneous horizontal motion in both directions, the effect of the frame action and the dead load of the bridge. Two types of horizontal displacement patterns were used in the experiments (Figure 2): the clover-leaf pattern and the cross-and-circle pattern. A test was made up of repeated application of the chosen load pattern, with the magnitude of the horizontal displacement increasing in multiples of the yield displacement value until failure.

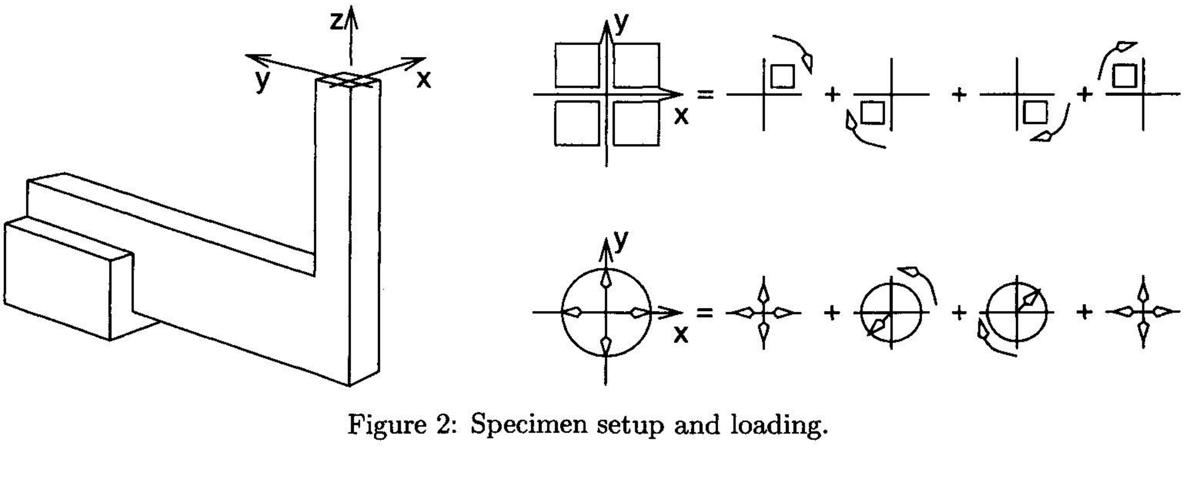

Figure 2: Specimen setup and loading.

3 AS-BUILT SPECIMENS

Performance of exiting outrigger knee joints was evaluated using two half-scale specimens, one with a long outrigger beam the other with a short outrigger beam. Compared to the current practice, confinement and detailing of the outriggers and knee joints is unsatisfactory. The outrigger beam shear reinforcement consists of hoops closed with a U-cap, placed approximately one quarter of the depth of the beam apart. The development length of the bottom beam bars into the joint is approximately 20 bar diameters. The joint contains no confining steel.

The deficient details of the existing outriggers contribute to the poor behavior of both as-built specimens [2]. The specimens developed diagonal cracks in the joint area during the pre-yield cycles. The combination of shear and torsion produced a set of inclined diagonal cracks to form on the sides of the long beam. The failure of both specimens occurred slightly after yielding of the column reinforcement and beam hoops. The sides of the joint dilated and then the layer of column bars on the outside face of the joint split away from the joint core.

The failure was sudden and brittle, as seen from the force/displacement response graphs for the short-span outrigger specimen (Figure 3). As expected, the unconfined joints were unable to transfer the cyclic joint shears, and the outrigger torsion capacity was inadequate due to the lack of closed stirrups.

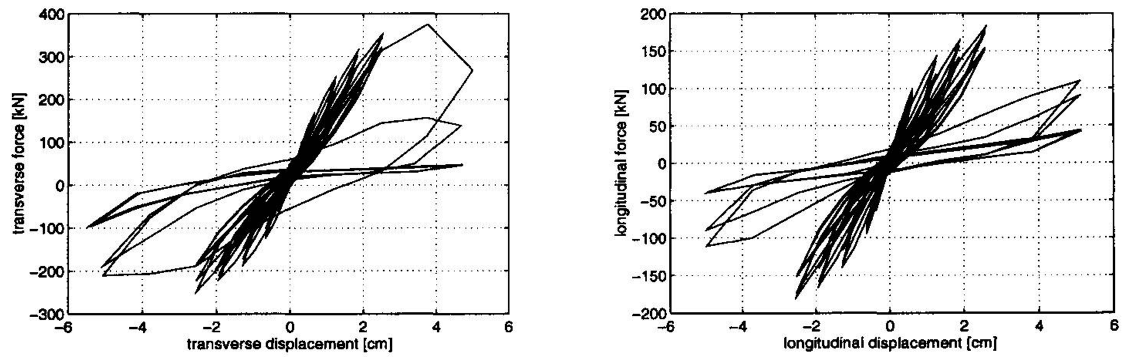

Figure 3: Force/displacement behavior of the short as-built specimen. Displacement of 3.6 centimeters corresponds to a drift ratio of 1%.

4 PROTOTYPE UPGRADES

Systematic upgrading of the outrigger knee joint system is necessary to elevate the performance of the system to a level implied by the current earthquake-resistant design practice. The goal of upgrading is to prevent catastrophic failure and to enhance the deformation and energy dissipation capacity of the outrigger knee joint system so that it behaves as well as the rest of the upgraded bridge structure [3]. The elements of the system should be strengthened to form a stable, ductile, energy dissipating system with a well-controlled failure mechanism. Furthermore, the strengthened outrigger knee joint system should be easy to inspect and repair after an earthquake.

The experiments on the as-built specimens show that the capacity of the joint region must be increased to limit damage there. The strength and stiffness of the outrigger beam for both bending and torsion must be improved. Jacketing the system elements was chosen to accomplish both of these goals. Starting from these common points, two upgrade strategies were proposed: The ductile upgrade strategy; And the strong upgrade strategy.

4.1 Ductile Upgrade Strategy

Ductile upgrade strategy is designed to produce multiple plastic hinges in the outrigger and knee joint system. In transverse direction, for both joint opening and joint closing, a plastic hinge is expected remaining elements of the outrigger and knee joint system are strengthened to the capacity required to sustain the designed plastic hinges.

The prototype ductile upgrade was made using a concrete jacket. Two 15 cm thick side bolsters connected with closed stirrups and T-headed through-bars strengthened the beam. The bolster horizontal reinforcement was wrapped around the outside face of the joint to increase confinement, arrest joint dilation and prevent the column bar bond-splitting failure.

The ductile upgrade prototype behaved according to expectations. The two distinct plastic hinge zones provided a high level of ductility and energy dissipation. In addition, the forces transferred to the bridge deck were minimized. However, the distributed hinging produced a comparatively large amount of damage, suggesting that an upgraded outrigger and knee joint system may be hard to inspect and repair after a strong earthquake.

4.2 Strong Upgrade Strategy

The fundamental premise of the strong upgrade strategy is to form a single plastic hinge in the column for both the transverse and the longitudinal loading directions. The knee joint, the beam and the beam/bridge deck interface of the outrigger and knee joint system are strengthened to the level required to sustain the forces transferred through the column plastic hinge.

A steel jacket made of 12.5 mm A36 steel plate was used in the strong upgrade prototype. The jacket was welded together around the beam and the knee joint, strengthened with post-tensioned through-bars and injected with epoxy. The column of the outrigger knee joint system was not altered.

As expected, the prototype strong upgrade developed a plastic hinge in the column. The hinge insured ductile behavior of the upgraded system, with the damage concentrated in the column base. The forces generated in the column hinge under longitudinal loading caused failure of the jacket anchors at the beam/bridge deck interface, suggesting the final upgrade design must take into account the possibility of significant beam weak axis bending.

5 FINAL UPGRADE DESIGN

Despite the good energy dissipation behavior and the ability to sustain the dead load after severe deformations, the extent of damage caused by multiple plastic hinge zones makes ductile upgrade strategy less suitable for fulfilling the upgrade design goals. The strong upgrade strategy offers a sufficiently ductile solution applicable to outrigger systems with both short and long beams. Therefore, the strong upgrade strategy was chosen for the final design of the outrigger knee joint system upgrades.

Two versions of the strong upgrade strategy were tested. A strong upgrade employing a post-tensioned reinforced concrete jacket was tested on one long outrigger specimen. The concrete jacket was made up of two 22.5 centimeter thick bolsters resembling the ductile upgrade prototype. The post-tensioning was designed to secure the beam/bridge deck connection and increase the torsional resistance of the beam.

Another strong upgrade design, using a 6 mm A572-50 steel plate jacket, was tested on two specimens, one with a long and the other with a short outrigger beam (Figure 4). The jackets were welded together, tied with post-tensioned through bars and injected with epoxy. The anchorage of the beam jacket to the bridge deck was carefully detailed without the use of post-tensioning.

The columns of all three final design specimens were enhanced to achieve large curvature ductility with the smallest possible increase in the strength of the column plastic hinge. A grouted cylindrical steel casing was placed around the column. The casing is closed from above and below by four pairs of end-plates. The anticipated rotation of the plastic hinge dictated a 2.5 centimeter clearance between the beam jacket and the column casing. The shear capacity of the as-built column was determined to be sufficient to carry the shear force required to hinge the column in bending. Therefore, the steel casing was extended only part-way down the column. The length of the steel casing was determined by considering the ultimate moment capacity of the column hinge and the yield moment strength of the as-built column.

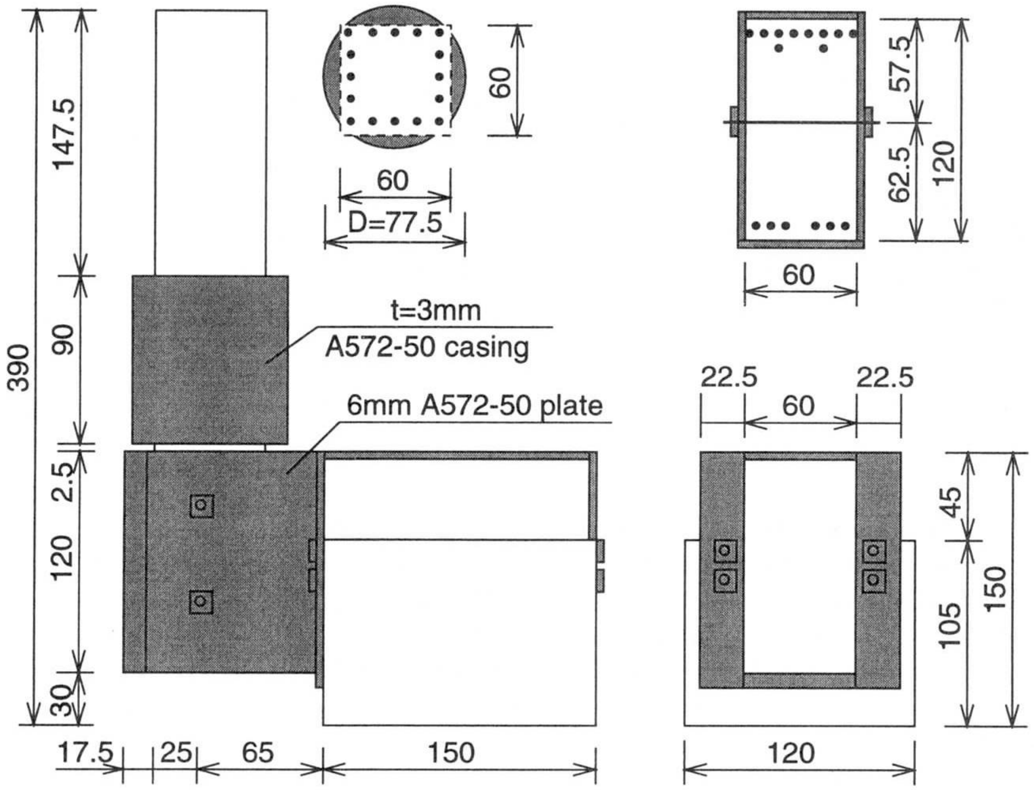

Figure 4: Final design of the strong steel jacket upgrade.

The behavior of all three specimens complied fully with the upgrade design goals. The strengthened beam and knee joint were sufficiently stiff to make the bi-directional plastic hinge form at the base of the column. The curvature ductility achieved in the hinges was 21. The damage in all three specimens was concentrated in the column hinge. The improvement in the behavior achieved by the strong upgrade is evident from the force/displacement response of the upgraded short outrigger specimen (Figure 5) and the comparison of tip displacement ductility and drift measures for all seven specimens (Table 1).

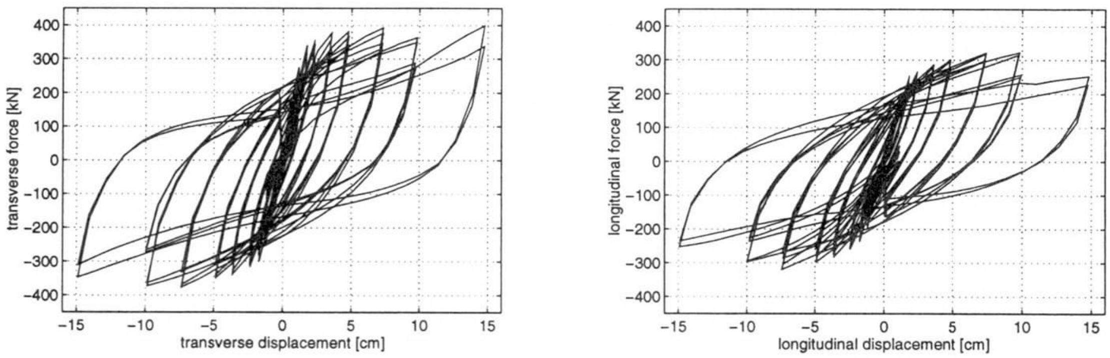

Figure 5: Force/displacement response of the upgraded short outrigger.

6 CONCLUSIONS

The results of the investigation on improving the seismic performance of outrigger knee joint systems were summarized in the form of seismic upgrade design guidelines. The guidelines define the necessary steps to implement the strong upgrade strategy using a steel plate jacket on existing outrigger knee

specimen	transverse		longitudinal	
	displ. ductility	drift [%]	displ. ductility	drift [%]
as-built long	1.5	1.04 (2.78)	5.33	1.39 (2.78)
as-built short	2.0	1.04 (1.39)	1.33	0.69 (1.39)
ductile prototype	5.5	5.56 (5.56)	6.0	2.78 (5.56)
strong prototype	6.0	2.78 (4.16)	4.0	2.78 (4.16)
strong concrete	12.0	2.78 (4.16)	12.0	2.78 (4.16)
strong steel long	12.0	2.78 (5.56)	12.0	2.78 (4.16)
strong steel short	12.0	2.78 (4.16)	12.0	2.78 (4.16)

Table 1: Specimen ductility and drift measures. The bracketed drift values are computed at the largest displacement level achieved during the test.

Particular attention is directed to detailing the column plastic hinge zone. The thickness of the cylindrical column casing is determined to provide the effective confinement pressure needed to achieve the desired drift of the outrigger knee joint system. The necessary column hinge curvature ductility is calculated first, using the desired drift and assuming all of the deformation is generated in the column plastic hinge. Given the curvature demand, a program for non-linear analysis of reinforced concrete cross-sections, AFcS [4], is used to design the confinement of the hinge cross-section. Using Mander's model for confined concrete [5], the level of confinement is adjusted to enable the hinge cross section to achieve the necessary curvature ductility while keeping the largest strain in the concrete core below the crushing strain level.

In addition to providing design tools and detailing recommendations, the guidelines emphasize the fundamental principles of capacity design in the seismic upgrade setting. The guidelines complete the research loop by providing results in a form useful to practicing engineers.

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