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Some Thoughts on Seismic Engineering for Bridges seen from a Canadian Perspective

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ABSTRACT

Progress in Seismic engineering for bridges in Canada has to date been driven largely by earthquake events outside our borders. Some of these events and the effects they have had on Canadian codes and practice are reviewed.

Because our infrastructure is relatively mature, retrofit of existing bridges is the dominant seismic topic for Canadian bridge engineers. A logical process for developing a retrofit strategy is outlined and some Canadian retrofit installations are discussed.

Research is described into the seismic behaviour of bridge components, which has been successful in establishing structural reserves to permit safe energy dissipation without collapse.

Two interesting new bridges are described, in which seismic issues dominate the conceptual and detail design process and where some of the ideas discussed in this paper are applied.

RECENT SEISMIC EVENTS IN NORTH AMERICA AND THEIR EFFECT ON SEISMIC ENGINEERING FOR BRIDGES

Over the past three decades, several major seismic events have occurred in North America. These events include the 1964 Alaska earthquake (M8.4), the 1971 San Fernando earthquake (M6.6), the 1989 Loma Prieta earthquake (M7.0), and the 1994 Northridge earthquake (M6.7). These earthquakes provided unique opportunities for bridge engineers to observe and learn from the seismic performance of bridge structures. Observations of bridge damage caused by these seismic events have led to the identification of seismic design deficiencies and to significant improvements in the seismic provisions of bridge design codes.

Bridge Damage In Recent Earthquakes

During the 1964 Alaska earthquake, several bridges were damaged due to large ground movements caused by soil liquefaction. This observation led to research activities in evaluation of liquefaction hazards and in development of soil densification techniques.

Five major freeway bridges collapsed during the 1971 San Fernando earthquake. The observed bridge failures and damage included (a) span failures due to unseating at movement joints and bearing supports; and (b) column failures due to inadequate shear reinforcement or inadequate transverse confinement in plastic hinge regions. The 1971 San Fernando earthquake also demonstrated that the lateral seismic design forces which had been used were too low.

The 1989 Loma Prieta earthquake highlighted the effect of local soil conditions. Local soft soils resulted in significant amplification of ground motions at several bridge sites. The Cypress viaduct collapsed because of amplified ground motions and poor structural detailing. Insufficient anchorage of cap beam reinforcement into the end regions and inadequate transverse reinforcement in columns were observed to have contributed to the collapse of the upper deck of the viaduct. Joint shear failures were observed in the connections between cap beams and columns at several bridges. Bond failure of lap splices at column bases was also observed.

The 1994 Northridge earthquake resulted in partial or complete collapse of five bridges and significant damage to three bridges in the epicentral region. Ground motions recorded in the epicentral region were characterized by large pulses with high energy due to near fault effects. All collapse cases involved column flexural/shear failures due to inadequate

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transverse reinforcement or unseating at in-span or abutments hinges. Damage to steel bridges was also observed and comprised buckling of cross bracing, bending of cross-brace gussets, and damage to bearings. There were several bridges in the epicentral region which had been retrofitted with hinge restrainers and/or column jackets. Therefore, the Northridge earthquake provided a unique opportunity to examine the performance of retrofitted bridges. Column jackets, whether made of steel or fiberglass, performed well in all cases. However, hinge restrainers did not perform properly in some cases, as evidenced by one of the collapsed bridges which had been retrofitted with cable restrainers at the in-span hinges.

A summary of these recent major earthquakes in North America and their engineering consequences is given in Figure 1.

Events and Magnitude	Observed Damage	Engineering Consequences
Alaska 1964 M8.4	Liquefaction caused collapse of 9 bridges	Research on soil liquefaction susceptibility
San Fernando 1971 M6.6	Collapse of 5 major freeway bridges	Design horizontal forces and seat widths increased
Loma Prieta 1989 M7.0	Cypress Freeway collapsed	More attention to soft ground amplification
Northridge 1994 M6.7	Intense energy pulse	More attention to near field effects

Figure 1. Recent Seismic Events in North America and their Engineering Consequences

EVOLUTION OF BRIDGE SEISMIC CODES

Bridge seismic codes in North America have evolved over the years as a result of lessons learned and knowledge gained from past major earthquakes.

Prior to the 1971 San Fernando earthquake, bridges were typically designed for a lateral seismic force which was only a small fraction of the actual seismic force that could be developed, and no ductile detailing requirements were specified. The 1971 San Fernando earthquake was the major turning point in the development of seismic design criteria for bridges in North America.

Following the 1971 San Fernando earthquake, the California Department of Transportation (Caltrans) initiated revisions to its seismic design criteria. In 1973, Caltrans introduced new seismic design criteria for bridges, which included effects of fault proximity, site soil conditions, structural dynamic response characteristics, and ductile details for reinforced concrete construction. In 1975, the American Association of State Highway and Transportation Officials (AASHTO) adopted Interim Specifications which were a slightly modified version of the 1973 Caltrans provisions.

In 1978, commissioned by the US Federal Highway Administration, the Applied Technology Council (ATC) developed new and improved seismic design guidelines for highway bridges. These guidelines known as ATC-6 increased the seismic design force to a more realistic level, introduced minimum seat width requirements, and provided detailing requirements for reinforced concrete construction. The ATC-6 guidelines were first adopted by AASHTO as a set of Guide Specifications in 1983. They were later adopted as seismic provisions in the Standard Specifications in 1990.

After the 1989 Loma Prieta earthquake, funded through the National Cooperative Highway Research Program (NCHRP), AASHTO revised its seismic design provisions. The improved seismic provisions were adopted in the sixteenth (1996) edition of the Standard Specifications.

The AASHTO Load and Resistance Factor Design (LRFD) Code first published in 1994 has seismic provisions similar to those of the AASHTO Standard Specifications.

After the 1989 Loma Prieta earthquake, Caltrans engaged the Applied Technological Council to review and revise seismic design standards for new bridges. The ATC-32 project was still in progress at the time of the 1994 Northridge earthquake. The ATC-32 report "Improved Seismic Design Criteria for California Bridges: Provisional Recommendations" was issued in 1996.

In Canada, the CSA Standard CAN/CSA-S6 has been the national design code for highway bridges. However, this code has now become inadequate for addressing the seismic design issue. In recognition of this, the Province of British Columbia, a high seismicity region in Canada, has used the AASHTO seismic provisions for seismic design of new bridges with some modifications. Such modifications are contained in the BC Ministry of Transportation and Highways (MoTH)'s Appendix C "Bridge Engineering Design Memorandum - Seismic Design Requirements for New Bridges in British Columbia".

The new Canadian Highway Bridge Design Code (CHBDC) is due for publication later this year, 1999. This is the first Canadian bridge code which has comprehensive seismic provisions. The CHBDC has retained the basic framework of the AASHTO seismic provisions. However, some significant changes were made, which are described later in this paper.

Rules For Bridge Seismic Retrofit

After the 1971 San Fernando earthquake, seismic assessment and retrofit design of older bridges became a matter of considerable urgency. While the underlying philosophy for retrofit design should be the same as that for design of new bridges, a less conservative approach would be justifiable for assessment of existing bridges. In 1983, funded by the Federal Highway Administration, Applied Technological Council developed guidelines for seismic retrofit of highway bridges. These guidelines known as "ATC-6-2" provided procedures for seismic assessment of existing bridges and seismic retrofitting concepts. These guidelines were revised and updated to incorporate new knowledge acquired through research and earthquake reconnaissance studies in the 1993 FHWA report "Seismic Retrofitting Manual for Highway Bridges".

Following the 1971 San Fernando earthquake, Caltrans initiated a three-phase bridge seismic retrofit program. The first phase of the program involved installation of hinge and joint restrainers intended to prevent deck joints from unseating. The second phase focused on retrofit of columns in single-column bridges. The 1989 Loma Prieta earthquake led to acceleration of the bridge seismic retrofit program, and initiation of Phase 3 which focused on the retrofit of multi-column bridges.

Similar to Caltrans, MoTH has established a seismic retrofit program for highway bridges in British Columbia. The first stage of this retrofit program involves safety retrofit of important bridges in the Lower Mainland, such as the 2nd Narrows Bridge and the Port Mann Bridge. A later stage of the retrofit program may involve functional retrofit of these bridges. MoTH's Appendix C provides design criteria for seismic assessment and retrofit of highway bridges in BC. Earlier versions of the Appendix C were mainly based on ATC-6-2 and Caltrans' retrofit criteria. The Appendix C has been revised recently to incorporate new knowledge acquired through research and earthquake damage observations. The City of Vancouver also developed a phased retrofit program for the major city bridges, such as the Granville Bridge and the Burrard Bridge.

The new CHBDC provides provisions for seismic evaluation of existing bridges. It also provides guidance on seismic rehabilitation of existing bridges.

Current Bridge Seismic Codes

The AASHTO LRFD is the national bridge design code in the United States, and it contains seismic provisions applicable to all regions of the United States. CAN/CSA-S6 is the current national bridge code in use in Canada. However, it does not contain adequate seismic provisions. The upcoming CHBDC will replace CAN/CSA-S6 and will be the first Canadian bridge code containing comprehensive seismic provisions. This section highlights and compares seismic provisions in AASHTO LRFD and CHBDC.

Ground Motion Characterization

Both AASHTO LRFD and CHBDC use seismic zoning maps of "firm ground" horizontal acceleration having a 10% probability of exceedance in 50 years (or a return period of approximately 475 years). Both codes recognize the importance of site specific studies for bridge sites close to active faults.

Importance Categories

Both codes classify bridges into three importance categories. The AASHTO LRFD uses designations of "Critical Bridges", "Essential Bridges" and "Other Bridges" whereas CHBDC uses terms of "Lifeline Bridges", "Emergency-Route Bridges" and "Other Bridges".

Elastic Seismic Response Coefficient

The elastic seismic response coefficient represents the elastic design response spectrum. The elastic seismic response coefficient in AASHTO LRFD is a function of structural period, T, and is related to "firm-ground" acceleration coefficient, A, and site coefficient, S. The site coefficient, S, depends on the soil profile type and ranges from 1.0 for Soil Profile Type I to 2.0 for Soil Profile Type IV. Seismic response coefficients for various soil profiles, normalized with respect to acceleration coefficient A, are shown in Figure 2.

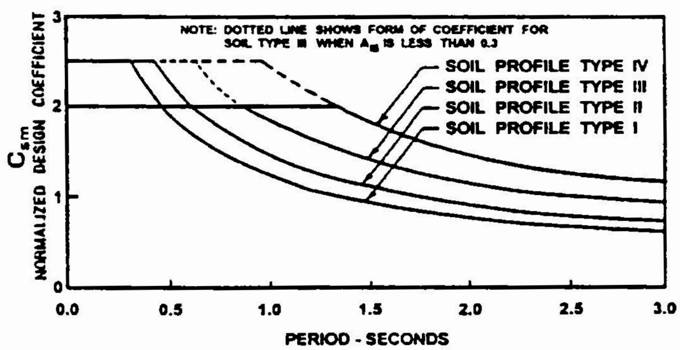

Figure 2 - Normalized Seismic Response Coefficients for Various Soil Profiles in AASHTO LRFD

CHBDC uses the same elastic seismic response coefficient as AASHTO LRFD except that an explicit Importance Factor, I, is added. The Importance Factor, I, depends on the bridge importance category and is taken as 3.0 for lifeline bridges, 1.5 for emergency-route bridges, and 1.0 for other bridges. Therefore, the elastic design response spectrum in AASHTO LRFD remains the same regardless of bridge importance category whereas the design response spectrum in CHBDC varies with the bridge importance category.

Importance Factors and Response Modification Factors

AASHTO specifies the following response modification factors, R, for different substructure systems:

Substructure	Importance Category		
	Critical	Essential	Other
Wall-type piers - larger dimension	1.5	1.5	2.0
Reinforced concrete pile bents			
• Vertical piles only	1.5	2.0	3.0
• With batter piles	1.5	1.5	2.0
Single columns	1.5	2.0	3.0
Steel or composite steel and concrete pile bents			
• Vertical piles only	1.5	3.5	5.0
• With batter piles	1.5	2.0	3.0
Multiple column bents	1.5	3.5	5.0

Table 1 - Response Modification Factors, R, in AASHTO LRFD

It can be seen in Table 1 that for the same substructure system, the R factor is a function of importance category. Therefore, the R factor in AASHTO LRFD reflects both the ability of the substructure system to dissipate seismic energy through inelastic deformation and the importance of the bridge.

The CHBDC distinguishes between the structural ductility factor and the importance factor. The importance of the bridge is accounted for by using a separate importance factor, I, in the elastic seismic response coefficient, as discussed previously. The response modification factor, R, is used to account for the ability of the substructure to dissipate seismic energy through inelastic deformation only and is independent of the importance category. The R factors for various substructure systems in CHBDC are given in Table 2.

Ductile Substructure Element	Clause for Design and Detailing Requirements	R
Wall-type piers - in direction of larger dimension	4.7	2.0
Reinforced concrete pile bents	4.7	
• vertical piles only		3.0
• with batter piles		2.0
Single columns		
• ductile reinforced concrete	4.7	3.0
• ductile steel	4.8	3.0
Steel or composite steel and concrete pile bents		
• vertical piles only	4.7 or 4.8	5.0
• with batter piles		3.0
Multiple column bents		
• ductile reinforced concrete	4.7	5.0
• ductile steel columns or frames	4.8	5.0
Braced Frames	4.8	
• ductile steel braces		4.0
• nominally ductile steel braces		2.5

Table 2 - Response Modification Factors, R, in CHBDC

Note: The lateral load resisting substructure elements must be designed and detailed to be ductile, that is, having a minimum Response Modification Factor, R, of 2.0.

Despite the different approaches in addressing the issues of importance and ductility between AASHTO LRFD and CHBDC, both codes lead to similar final seismic design forces for the same substructure system.

The categories of substructure systems covered in AASHTO LRFD and CHBDC are similar except that steel braced frame systems are added in CHBDC.

Design Methods and Detailing Requirements

The design methods in both AASHTO LRFD and CHBDC are based on the general philosophy of capacity design. The ductile substructure elements are designed and detailed to form plastic hinges at predetermined locations whereas other undesirable inelastic deformation mechanisms, such as shear, are inhibited.

The design and detailing requirements for concrete substructure systems are similar between AASHTO LRFD and CHBDC. Both codes provide specific seismic detailing requirements, such as column shear reinforcement and transverse reinforcement for confinement at plastic hinge regions.

The CHBDC provides specific seismic detailing requirements for ductile steel substructure systems, such as ductile moment frames or ductile braced frames. These requirements are comparable to those for ductile steel building structures in the National Building Code of Canada.

Seismic Evaluation and Rehabilitation of Existing Bridges

The CHBDC provides general provisions for the seismic assessment of existing emergency-route and other bridges, while requiring special studies for lifeline bridges. It also provides general guidance on the seismic rehabilitation of existing bridges. The CHBDC recognizes the important role of the regulatory authority in setting appropriate criteria for evaluating and retrofitting existing bridges. Therefore, adjustments to the assessment procedures are permitted if approved by the regulatory authority.

CHALLENGES IN RETROFITTING EXISTING BRIDGES

Because our infrastructure is relatively mature, the seismic challenge of bridge retrofit is a much more common occurrence than seismic design of a new bridge.

If a bridge seismic retrofit is to be successfully achieved, it must be approached in a systematic way. Key activities include:

- establish seismic hazard and retrofit level
- derive seismic input consistent with hazard
- establish characteristic dynamic behaviour of the structure
- detect all of the seismic vulnerabilities
- define a seismic retrofit strategy
- establish costs for each retrofit activity
- prioritize the retrofit activities

These steps are examined in turn.

Seismic Hazard and Retrofit Level

Seismic hazards are defined in general terms in the CHBDC by a seismic zoning map similar to that in the National Building Code of Canada. Site specific predicted acceleration and velocities versus return period derived from interpolation of the seismic hazard map are also available from the Geological Survey of Canada. For important bridge structures, seismic hazard analyses are sometimes made based on attenuation relationships for distant major subduction events and nearby smaller earthquakes. In Canada we have only just begun to retrofit our large inventory of bridges and therefore practically all current retrofits are to the safety level only.

Derive Seismic Input Consistent With Hazard

Inputs in the form of seismic spectra or time histories are necessary to simulate the anticipated seismic event. Care is necessary in the choice of input to ensure appropriate energy and frequency content in the input records and also appropriate causative mechanisms and soil characteristics at the recording station.

The practice of adjusting earthquake time histories to make them conform to a certain target response spectrum can result in unrealistic distortions. The "adjusted" accelerations and velocity time histories should always be examined after adjustment to see if any critical characteristics have been affected.

Bridges whose length exceeds 500m will see incoherent support excitations which may increase displacment demands.

Discrete multiple support excitation may be necessary in the analysis of long bridges where geotechnical variations result in significant lengthwise changes in seismic input.

Establish Characteristic Dynamic Behaviour of the Structure

Judgment is necessary here in selecting an appropriate analytical model and seismic inputs which are capable of capturing the dynamic behaviour of the bridge during an earthquake. All significant degrees of freedom must be modelled as well as all boundary conclusions.

For small conventional bridges, a simple structural model with a few dozen degrees of freedom and uncoupled X and Y direction analyses may suffice, but additional complexities in the structure demand more sophisticated analysis. Highly skewed bridges usually exhibit coupled motions in the two horizontal degrees of freedom. Similar behaviour was exhibited by Port Mann Bridge, whose 29 spans in three span continuous units, form an S shape in plan.

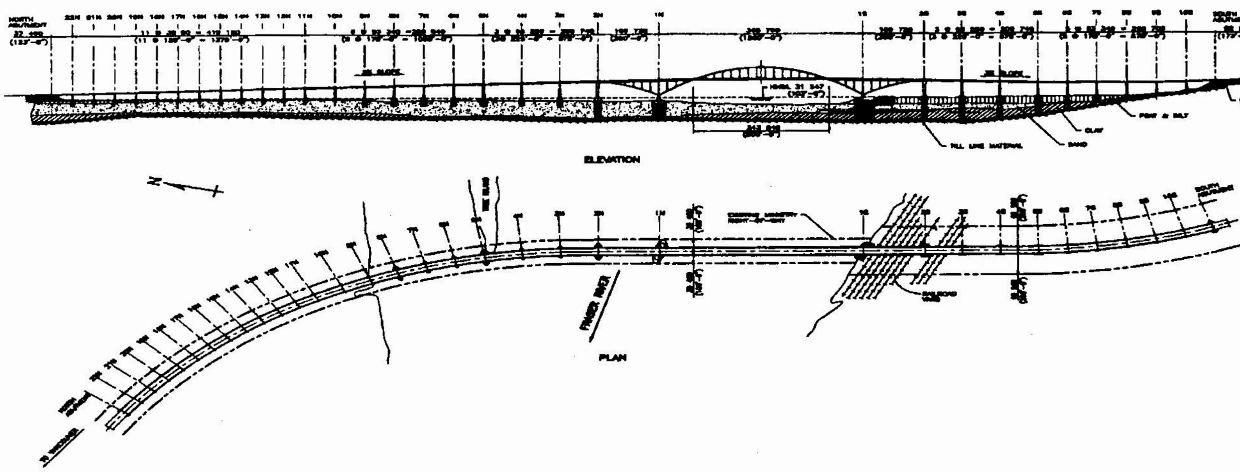

Figure 3: Port Mann Bridge

In these cases simultaneous X and Y time history inputs may be necessary to capture the coupled dynamic response. Other features which may require to be addressed in a dynamic model include closing gaps, with or without bumpers, and discrete dampers or other seismic devices.

Seismic inputs are normally delivered to the base of the structure. However, when the soil stiffness is of the same order as that of the structure, inclusion of the soil compliance springs is necessary to capture the dynamic response. The resulting increase of the periods of vibration usually reduces the system acceleration but increases the displacement response.

Reliance on further attenuation of the system response due to hysteretic damping in the soil requires very careful cyclic testing of soil samples to derive appropriate soil load displacement curves through several loading and unloading cycles and very careful soil-structure interaction modelling to confirm that the energy dissipation actually occurs.

Detect all the Seismic Vulnerabilities

Given an accurate dynamic model and appropriate seismic inputs, it will be possible to identify all of the seismic vulnerabilities by careful examination of the demands (D) generated in each element of the dynamic model and comparing them with the capacity (C) of each element. These D/C ratios may be for force, displacements or occasionally for velocity in seismic hardware devices.

Once the D/C ratios are known for all of the elements in the bridge, then this information forms the basis for an initial assessment of possible seismic strategies.

Define a Seismic Retrofit Strategy

Choice of an appropriate retrofit strategy depends upon a broad appreciation of the opportunities available to the bridge engineer to modify a particular seismic response. This is best done by examining the seismic energy balance. (Medeot R., 1998)

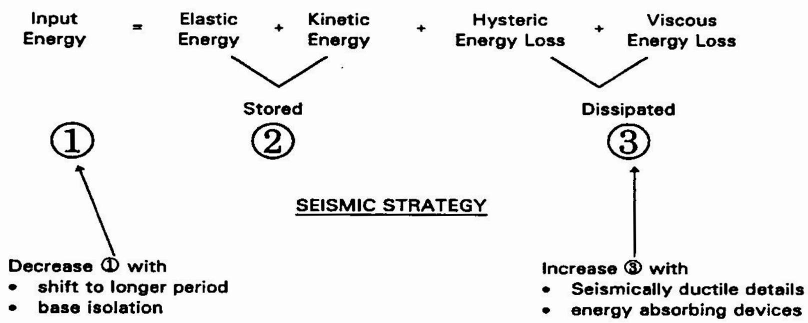

Figure 4: Seismic Energy Balance for a Bridge Structure

Figure 4 equates the seismic energy input into the bridge to four energy packages within the bridge, elastic and kinetic energy which are stored and hysteretic and viscous energy which are dissipated.

At the seismic strategy stage we have opportunities to influence all five of these energy quanta.

The input energy to the system can be attenuated by the use of base isolation techniques, which have the added attraction of tending to shift dominant response modes to longer periods with less energy input.

The elastic energy stored can be increased by stiffening the structure.

The hysteretic energy loss can be increased by engaging more seismically ductile details and the viscous energy loss can be increased by the use of seismic energy absorbing devices.

Thus it can be seen that at the seismic strategy stage the bridge engineer has a broad range of options and combination of options at his disposal to reduce excesses of demand over capacity.

Judgment, experience and further dynamic analyses are then necessary to select the appropriate strategy or strategies to deal with the particular demand of a given bridge.

Canadian examples of some of these strategies include:

1. Decrease Input Energy

    a) Lead filled seismic Isolation bearings - Queensborough Bridge, 1994, Burrard Bridge, 1997, Second Narrows Bridge 1999
    b) Friction pendulum base isolation bearings - White River Bridge, Yukon, 1997

2. Increased Elastic Energy

    a) Stiffen bracing system - Granville Bridge, 1993, Second Narrows Bridge, 1999

3. Hysteretic Energy Loss

    a) Steel hysteretic dampers - Granville Bridge, 1997

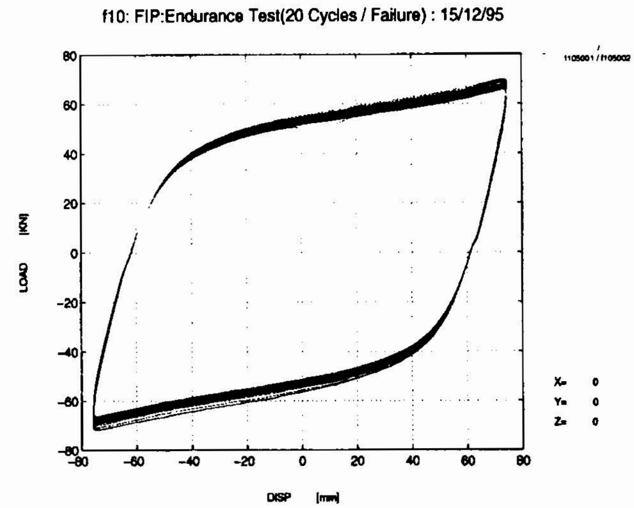

Figure 5: Hysterectic Energy Loss of Steel Dampers used on Granville Bridge

b) Increased ductility in existing concrete details - Queensborough Bridge, 1996, Oak Street Bridge, 1997, Granville Bridge, 1996
c) Viscous energy absorption devices - Arthur Laing Bridge, Vancouver, 1996

Establish the Cost and Benefit of Each Retrofit Item

Once a preferred retrofit strategy has been established, then it is necessary to estimate the costs of each retrofit activity, based on estimating from first principles or, if available, from previous project unit costs.

Prioritization of retrofit activities can then be based upon the benefit/cost ratios. (Sexsmith, R.G. 1994)

RESEARCH AND DEVELOPMENT IN SEISMIC ENGINEERING

Research in seismic engineering for bridges is one area of civil engineering research which has made great contibutions to design practice the last 10 years. The work has been extremely cost effective in reducing the seismic risk to bridges and bridge users. Having initially established effective retrofits for seismic deficiencies, some of the current research is focusing on improving the cost benefit ratio of the more expensive retrofit components.

Research on Component Seismic Behaviour

Basis Research on the seismic behaviour of concrete bridge columns and joints at the University of California at San Diego (UCSD), and elsewhere, has been useful both for the design of new bridges and for assessment of existing bridge retrofits. In both cases, designers need to know what level of seismic ductility can be reliably assigned to a given detail.

Thanks to Caltrans funding, UCSD has been able to test a large number of types of columns and joints and to make the results available to the profession. Similar tests on laced steel compression members in bridges have been carried out at the University of California at Berkeley, as well as tests on seismic hardware such as dampers. Cost effective details have resulted from this research.

Some of these results have now found their way into seismic codes, such as the new AASHTO LRFD code, in the form of ductility factors or force reduction factors R.

Other research, at UBC and elsewhere, has focused on the ductility factors achievable with project specific retrofits, such as in the support bents for Oak Street and Queensborough bridges.

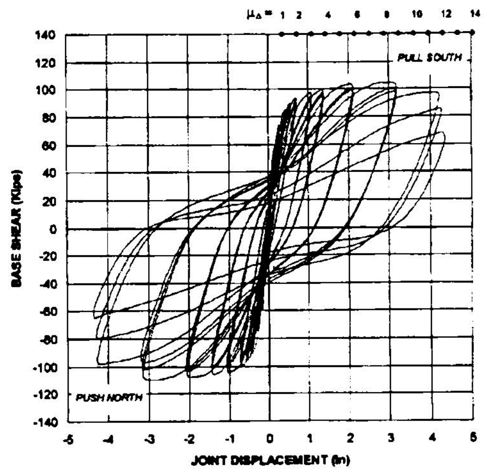

Figure 6: Oak Street Bridge - Retrofit Bent Test Results (Anderson et al 1995)

Seismic isolation bearings and damping devices are now commercially available for bridges. The claimed capacities of these devices can only be confirmed by full scale testing <u>at realistic seismic load application rates</u>.

UCSD is presently commissioning one of the few facilities in the world which is capable of achieving these loading rates for bridge size devices.

<u>Future Research and Post Graduate Education</u>

To date University seismic research has provided an excellent environment for training a generation of post graduate engineers, who have then brought acquired seismic engineering skills into our design offices.

Looking back to the University research laboratory from the design office, what do we need less of, and more of, in future seismic research?

We need less of:

- theoretical analyses of the seismic responses of cable-stayed and other exotic bridges (such bridges form a dynamically well behaved small minority of the bridge population);
- yet more complex ways of ascertaining that soils are prone to liquefaction (let us focus on the cure, not further diagnosis techniques);
- "smart" seismic devices for bridges (can we rely on such devices after decades of neglect?).

We need more of:

- seismic demands upon, and retrofit of, foundations, particularly piled foundations;
- seismic ductility of wall type bridge foundations;
- dynamic testing, at realistic loading rates, of seismic hardware;
- research on more cost effective methods for reducing liquefaction hazards.

SEISMIC DESIGN OF SOME INTERESTING NEW BRIDGES

The state of the art in current seismic design of major new bridges is represented by East Bay Bridge in San Francisco and Rion Antirion Bridge in Greece.

<u>East Bay Bridge - San Francisco</u>

The current 3500 m long East Bay Bridge is a 10 lane double deck cantilever truss with trussed approach spans. The masonry piers are supported by timber piles founded in deep marine deposits. Because the anticipated $700 million retrofit cost for this bridge was considered excessive, a new bridge is to be constructed alongside and the existing bridge demolished.

The new twin 5 lane bridge has a "signature span" at its West end, adjacent to Yerba Buena Island, leading into a long viaduct sloping down to the Oakland toll plaza. The "signature span" comprises a single tower self anchored suspension bridge founded on rock, see Figure 7. The viaduct spans are supported in the deep marine deposits.

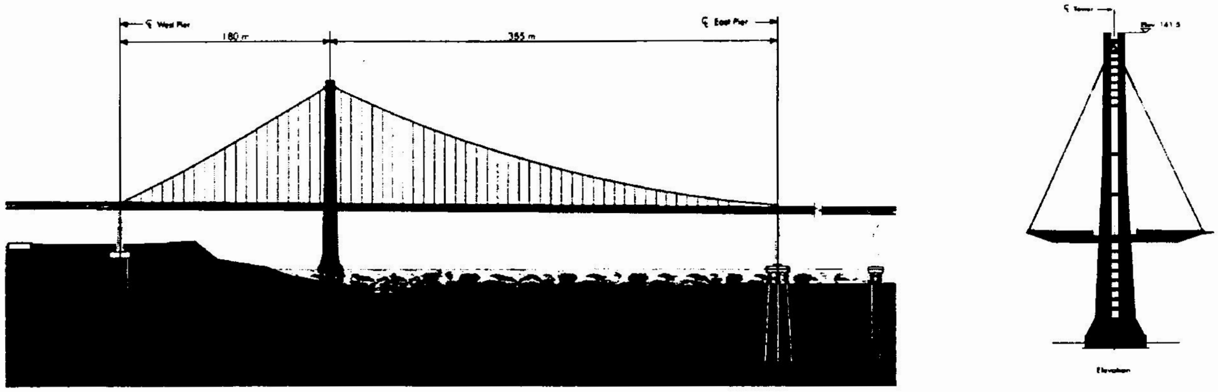

Figure 7: East Bay Bridge

For this bridge, situated midway between the Hayward and San Andreas faults, the seismic hazard is high and site specific design motions were derived for two levels of earthquake; a safety evaluation earthquake and a more likely to occur Functional Evaluation Earthquake. The zero period rock acceleration associated with the Safety Evaluation Earthquake is 0.6g. Mudline accelerations in the marine deposits exceed 1.5g in some locations.

Because the site has significant variations in rock elevation and depth of marine deposits, discrete support excitation inputs are specified at each pier. Displacement demands are significant at the joint between the rock supported "Signature Structure" and the approach structure supported on piles in the deep marine deposits. This joint is located within an approach span and is designed to handle the large displacment demands.

Assuring ductile behaviour in lateral bending for the tower of the self anchored suspension bridge was a significant challenge, elegantly solved by the designers T.Y. Lin International.

The single tower actually comprises two close spaced slender towers, separated by ductile steel link beams, which show good ductility in lateral pushover response. This can be seen from Figure 8 where yielding of the steel links permits a ductility factor in excess of four.

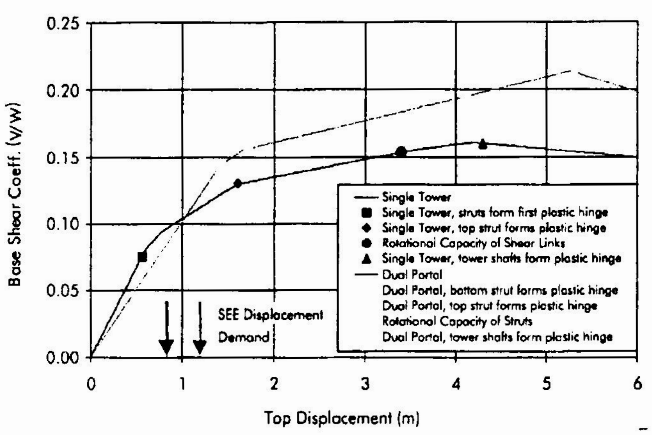

Figure 8: East Bay Bridge - Lateral Load Response of Single Tower and Dual towers of Suspension Bridge

The typical piers for this bridge will comprise large diameter drilled shafts founded in the marine deposits and supporting pilecaps at water line. Two legged ductile concrete bents will support each of the twin superstructures. Most elements of the bridge remain elastic under the safety evaluation earthquake.

Rion-Antirion Bridge - Greece

This is a five span design-build cable stayed bridge of length 2100 m crossing the Gulf of Corinth. The site features water depths of 60m over deep soft marine deposits. Tectonic movements of 2 m in any direction must be accommodated by the design and mudline seismic accelerations of 0.8g are to be handled essentially elastically.

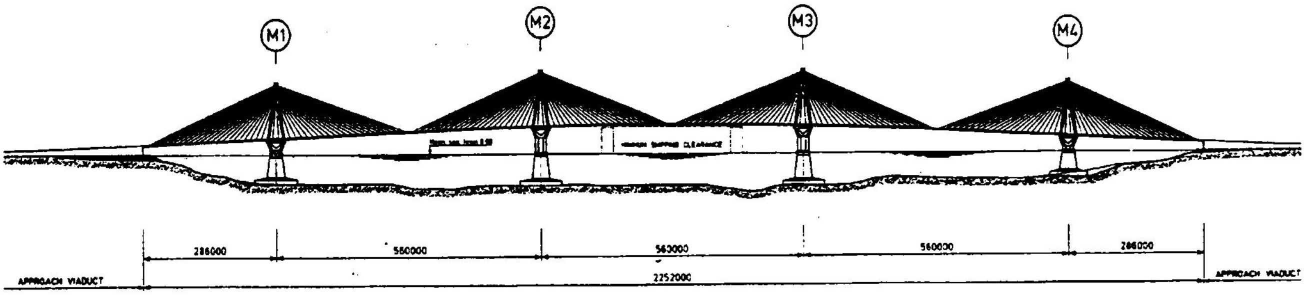

Figure 9: Elevation Rion Antirion Bridge, Greece

The slender composite cable-stayed superstructure floats continuously through the five spans and is supported only by the cables. As has been shown in previous cable-stayed structures, this concept is capable of handling very large seismic demands without attracting large forces. Longitudinally it is the ultimate base isolation system, however it pays a price in displacement demands at each end.

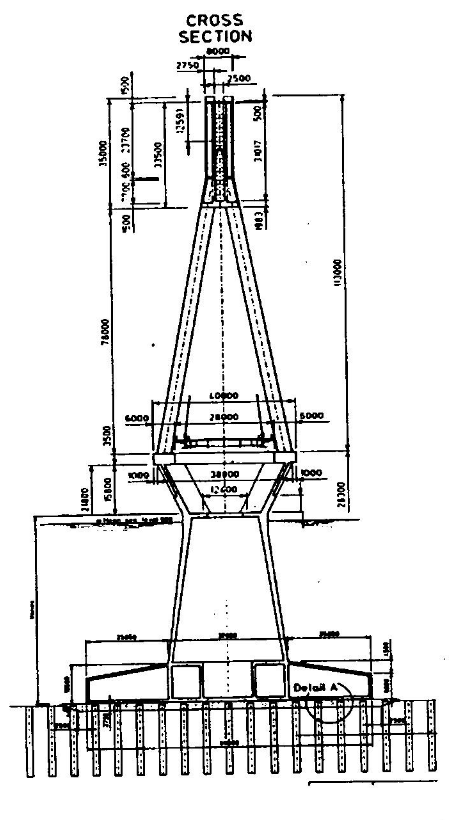

Figure 10: Typical Pier - Rion Antirion Bridge

The piers are very large (90m diameter) precast elements surmounted by four legged A-frame towers. Seismic accelerations at 45° to the bridge centreline result in very large compressions in one tower leg. It is difficult to achieve ductile behaviour in axially loaded concrete box members of such a triangulated structure under an earthquake which exceeds the design event.

The mass of the bridge is dominated by the precast piers which rest on a gravel bed over soft marine deposits reinforced by steel pipe inclusions. Rocking and sliding displacements are anticipated at the gravel layer under the design earthquake. The steel inclusions prevent a slip circle failure beneath the footing under the combined vertical and lateral seismic loading.

CONCLUSIONS

The design of these two modern long span bridges incorporates many of the points made in this paper, such as the use of modern design codes, site specific inputs, the use of laboratory test results to confirm the ductile behaviour of major structural elements and the choice of inherently base isolated bridge types to minimize the capital costs premium to be paid for excellent seismic performance.

REFERENCES

Medeot, R. 1998 Sesimic Design Approach and Related Hardware using Energy Concepts, 5th International Conference Short and Medium Span Bridges, Calgary

Sexsmith, R.G. 1994 Seismic Risk Management for Existing Structures, Canadian Journal of Civil Engineering, Vol.21, No.2, pp 180-185

Anderson, D.L. et al 1995 Oak Street and Queensborough Bridges Two Column Bent Tests, University of British Columbia, Department of Civil Engineering Technical Report 95-02