SEISMIC EVALUATION & RETROFIT TECHNOLOGY FOR BRIDGES

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J. MAFFEI
Department of Civil Engineering,
University of Canterbury,
Christchurch, New Zealand

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PO Box 2331, Lambton Quay, Wellington, New Zealand
Telephone (04) 473-0220; Facsimile (04) 499-0733

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CONTENTS

LIST OF FIGURES

EXECUTIVE SUMMARY

Summary
Past earthquakes have clearly shown the seismic vulnerability of existing bridges. Despite recent progress by the structural engineering profession in addressing bridge seismic risks, several areas exist where improvements in bridge evaluation and retrofit practices are needed. This report reviews the common seismic deficiencies of bridges, procedures and criteria for the seismic evaluation of bridges, and the engineering techniques which have been used, up to 1996, for retrofitting bridge seismic deficiencies. Information on seismic deficiencies, retrofit techniques, and related research has been summarised in Table 1. The review indicates several areas where effective retrofit techniques have been established, and other areas where improved procedures or further research are needed. Seismic upgrade measures proposed for Thorndon bridge, Wellington City, New Zealand, including an innovative retrofit of superstructure linkages, illustrate the benefits of a capacity-design approach to seismic evaluation and retrofitting.

Conclusions
The review of seismic evaluation and retrofit methods and examples leads to several conclusions, as follows:

- Past earthquakes have clearly demonstrated a number of serious seismic problems in bridges including the unseating of bridge spans, and the non-ductile failure of columns, beam-column joints, and column-foundation joints.

- Improvement of bridge structural analysis methods is needed, particularly for modelling superstructure movement joints.

- Some seismic analysis assumptions may need verification.

- The development of a bridge evaluation handbook would be valuable.

- Several seismic-retrofit techniques, such as the steel jacketing of columns, are now well established in California, USA.

- Experimental testing of new retrofit designs is important. Additional research may be needed in several areas.

- A novel scheme of retrofitting the superstructure linkages of the Thorndon bridge has been developed which eliminates some uncertainties in the response of the structure.

- The proposed retrofit measures for the Thorndon bridge illustrate the application of recent bridge seismic retrofit research and the benefits of a capacity-design approach to seismic evaluation and retrofitting.

Table 1 Summary of seismic deficiencies in bridges, and recommended methods for retrofitting bridges.

Bridge Seismic Deficiencies	Observed Earthquake Damage	Recommended Retrofit Methods	Examples of Application of Retrofit Methods	Remarks on Design Criteria
1 Movement joints, seats, restrainers, bearings	Numerous earthquakes, particularly 1971 San Fernando	High-strength bar restrainers, looped cable restrainers, straight-through cable restrainers	Numerous applications in California and elsewhere	Analysis techniques and design criteria need further development
		Steel pipe seat extenders	Numerous applications in California	
		Concrete or steel bracket abutment seat extenders and stopper devices, steel-chain and steel-plate restrainers	Applications in Japan and elsewhere	
		Base isolation and energy absorbing devices	Pioneered in New Zealand; used on several bridge retrofit projects in New Zealand and US; used in Italy, Japan and Canada	Inelastic behaviour of isolators can be modelled elastically using equivalent stiffness and increased damping; generic specifications needed
		Elimination of movement joints by deck slab replacement	Used in Italy	Applicable only to shorter bridges
		Addition of high-strength slack restrainers to distribute movement to several joints	Proposed for Thorndon Bridge, New Zealand	Applicable to bridges with closely-spaced movement joints
2 Cap beams	1989 Loma Prieta	Strengthening by concrete or steel jacketing and pre-stressing, retrofit by adding a new link beam below	China Basin Viaduct, San Francisco (jacketing)	Designed to force plastic hinging into columns
		Cored internal prestressing, fibre wrapping and prestressing	Oak St and Granville St bridges, British Columbia	At Oak St designed to force hinging into columns; at Granville St designed for elastic earthquake forces
3 Outrigger beam knee joints	1989 Loma Prieta	As for cap beams, plus removal of existing knee-joint concrete and rebuilding of joint	I-980 Freeway, Oakland, California	Designed to force hinging into columns or ductile-torsional hinging in cap beam; reduces cap-beam cracking in minor earthquakes
4 Anchorage of column longitudinal bars in beam-column joints	1971 San Fernando	Addition of new link beam, prestressing of joint region, clamped steel confining jacket	Santa Monica Viaduct, Los Angeles (link beam)	Design criteria and retrofit methods need development
		Added anchorage end plates on straight bars	-	-

Bridge Seismic Deficiencies	Observed Earthquake Damage	Recommended Retrofit Methods	Examples of Application of Retrofit Methods	Remarks on Design Criteria
5 Column flexural confinement and bar-buckling restraint	1971 San Fernando	Elliptical (or circular) steel jacketing	Numerous applications in California; used for circular columns in Japan; proposed for Thorndon Bridge, New Zealand; used in British Columbia	Detailed design criteria are provided by Caltrans (1992); can be full or partial height; fibreglass method needs material certifications and durability assurances
		Active fibreglass jacketing	Trial applications in California for circular columns; used in British Columbia for square columns	
		Reinforced concrete jacketing	Applications in Japan and British Columbia	–
		Removal and replacement of cover concrete with added transverse reinforcing steel	–	–
		External steel hoops with turnbuckles	–	For circular columns only; typically more costly than other methods
		Prestress wire jacketing	–	
		Carbon fibre jacketing	–	–
6 Column lap splices	1971 San Fernando 1982 Urakawa-oki, Japan 1989 Loma Prieta	Partial confinement elliptical (or circular) steel jacketing (with polyethylene cushion)	Numerous applications in California on rectangular and circular columns	Intended to improve ductility capacity, while keeping moment capacity limited by allowing some lap-splice slippage (Caltrans 1990)
		Elliptical (or circular) steel jacketing (full confinement); active and passive fibreglass/epoxy jacketing (full confinement); reinforced concrete jacketing; added transverse reinforcing; external steel hoops; prestress wire jacketing; carbon fibre jacketing	See item **5**	See item **5**
		New foundation topping	See item **9**	Effective for thick toppings which cover lap-splice region
7 Column shear strength	1971 San Fernando 1987 Whittier 1989 Loma Prieta	Elliptical (or circular) steel jacketing (full confinement); passive fibreglass jacketing (partial confinement); reinforced concrete jacketing; added transverse reinforcing; external steel hoops; prestress wire jacketing; carbon fibre jacketing	See item **5**	See item **5**, Jackets are typically full height, except for some flared columns
8 Pier walls	1995 Hyogo-ken Nanbu, Kobe	Steel jacketing with through bolts	Used in California, with full confinement and partial confinement jackets	Design criteria need development; Caltrans does not retrofit if axial load is low and ductility demand ≤ 4.0

Bridge Seismic Deficiencies	Observed Earthquake Damage	Recommended Retrofit Methods	Examples of Application of Retrofit Methods	Remarks on Design Criteria
9 Column to foundation joints	1971 San Fernando	New foundation topping	Used in California; proposed for Thorndon Bridge, New Zealand	Effectiveness is limited; design criteria need development; Caltrans recommends connecting topping with perimeter ties to bottom reinforcing mat
		Prestressing through foundation	Proposed for Thorndon Bridge, New Zealand	Design criteria and research are needed
10 Footings and pile caps	No failures noted	Foundation strengthening with new piles or soil anchors; foundation topping and prestressing (see item 9)	Applications in California, including China Basin Viaduct; Oak St Bridge, and Queensborough Bridge, British Columbia	Further study of foundation rocking could reduce the need to retrofit
11 Abutments	Failures caused by poor soil conditions in 1990 Costa Rica, 1987 Edgecumbe, New Zealand, and other earthquakes	Anchor slab; tension tie backs; anchor piles	Applications in California and Japan	Analysis and design criteria need further development
Retrofit Methods Addressing Multiple Deficiencies				
12 Two-level bridge structures	1989 Loma Prieta	Substantial rebuild of structure with added longitudinal beams	I-280 Freeway, San Francisco, California	–
13 Columns and cap beams in multi-column bents	See items **5, 6, 7**	Infill structural wall between columns	Used in Japan, US, and British Columbia; proposed for Thorndon Bridge, New Zealand	Eliminates transverse moments in cap beams, but can increase loads to foundations
14 Column and beam deficiencies	See items **2, 5, 6, 7**	External longitudinal post-tensioning; cored internal post-tensioning	Used in Japan, US, and British Columbia	Applicable to elements protected from failure by other ductile elements
15 Column or pier wall deficiencies	See items **5, 6, 7, 8**	Added reinforced concrete outrigger frame	Used in California, US	Cap-beam retrofit is difficult
Site-Related Seismic Deficiencies				
16 Pounding of adjacent structures	1989 Loma Prieta	–	–	–
17 Soil liquefaction	1990 Costa Rica and other earthquakes	Ground improvement methods such as vibro replacement (stone columns), jet grouting, driven displacement piling, compaction grouting; and ground containment using bored piles, jet grouting, or concrete walls	Vibro replacement (stone columns) and jet grouting proposed for Thorndon Bridge, New Zealand, and used in British Columbia	–
18 Potential for landslides, surface fault rupture and tsunami	1993 Hokkaido-nansei-oki, Japan, and other earthquakes	–	–	–

ABSTRACT

Despite recent progress by the structural engineering profession in addressing bridge seismic risks, several areas exist where improvements in bridge evaluation and retrofit practices are needed. This report reviews the common seismic deficiencies of bridges, procedures and criteria for the seismic evaluation of bridges, and the engineering techniques which have been used, up to 1996, for retrofitting bridge seismic deficiencies. Information on seismic deficiencies, retrofit techniques, and related research has been summarised in tabular form.

The review indicates several areas where effective retrofit techniques have been established, and other areas where improved procedures or further research are needed. Seismic upgrade measures proposed for Thorndon bridge, Wellington City, New Zealand, including an innovative retrofit of superstructure linkages, illustrate the benefits of a capacity-design approach to seismic evaluation and retrofitting.

1. INTRODUCTION

1.1 Background

As emphasised in Kobe (1995), Japan, and Northridge (1994), California, damage to bridges can be one of the most catastrophic results of strong earthquakes. Past earthquakes have clearly shown that bridges can be seismically vulnerable, and that many types of existing bridge structures are inadequate to resist earthquake shaking.

Over the last decade substantial progress has been made to address the seismic risk posed by existing bridges. Since the 1989 Loma Prieta earthquake, in California, a major programme of seismic retrofitting for California's highway bridges has been undertaken. Other US states and other countries have also begun or intensified seismic evaluation and retrofit programmes for their bridges. Along with these programmes, research has been carried out that has resulted in greatly improved procedures for evaluating bridges and designing retrofit measures.

Despite this progress, there is still room for improvement. Because of the complexities of seismic performance and seismic risk of bridges, our knowledge of, and ability to provide, effective seismic-retrofit solutions are incomplete. The problem is compounded by the wide variety of structure types and characteristics, levels of seismicity, soil conditions, and available resources for upgrading structures to accommodate the forces exerted during seismic events, which are found in different locations or under a single jurisdiction. In some areas further research is needed; in other areas sufficient research has been carried out but the practising engineers may not be fully aware of the research results or its implications.

Summaries of bridge seismic evaluation and retrofit procedures have been published. The most current and complete of these summaries has been *Design Guidelines for Assessment, Retrofit, and Repair of Bridges for Seismic Performance*, by Priestley et al. (1992a). Some of the information presented in Chapters 2 and 3 of this report is discussed in more detail in these *Design Guidelines* of Priestley et al. Another major source of information for these chapters were communications with Ray Zelinski of the California Department of Transportation (Caltrans) Seismic Technology Section. He provided a wealth of information on the Caltrans evaluation and retrofit criteria as well as many of the retrofit details discussed in Chapter 3. Information from the seismic retrofit of buildings has also been drawn on, in those cases where it is applicable to bridge structures.

The typical bridge structures of New Zealand have some significant differences to those of California and, because this review is based largely on work done in California, not all the information is directly applicable to New Zealand bridges. For example, most bridges in New Zealand are water crossings, and few are freeway (motorway) interchange structures. Other differences such as outrigger beams are not common in New Zealand; most New Zealand bridges have only one or two lanes and carry little traffic compared to Californian bridges; span seating and column confinement details for New Zealand bridges are better than for many Californian bridges of the same age. Conversely, New Zealand bridges are more likely to suffer foundation erosion caused by river scour.

Despite such differences, most of the seismic retrofit concepts used in California can be applied to the bridges of New Zealand.

1.2 Objectives

The aim of this study was to fill some of the gaps in knowledge and communication in the structural engineering profession regarding the seismic evaluation and retrofitting of bridges. Its emphasis is on the practical aspects of the problem, for structural engineering designers as well as for researchers. The report presents a comprehensive review of seismic evaluation and retrofit technology, emphasising practical and specific examples.

The material is focused towards two main goals:

1. Select and present that information which would be of the most use to the structural designer or engineering manager who is responsible for implementing bridge seismic evaluations and retrofits, and

2. Identify areas where further study is needed and, where appropriate, to outline ideas for improved seismic evaluation, or retrofit procedures.

In New Zealand, the upgrading of seismically deficient bridges (i.e. bridges that have structural weaknesses which are liable to be damaged during seismic shaking) is lagging behind the progress in other earthquake-prone areas such as California and Japan. Fortunately, the international research results and implementation experience can be put to use in New Zealand. The information contained in this report should help in the task of implementing an effective bridge seismic-retrofit programme in New Zealand and elsewhere.

1.3 Evolution of Study

The scope of this study has changed somewhat since its inception in July 1992 when it began with the experimental study of the column-foundation region of a 1936-designed bridge. The original study also included a literature review on the seismic retrofitting of reinforced-concrete bridge columns. In exploring the literature, and considering the material that would be most helpful for engineers involved in seismic retrofitting, the review was changed so that it:

- covered all types of seismic deficiencies, not only those of concrete columns,
- emphasised seismic evaluation methods and research as well as retrofit measures,
- included the implementation of retrofit measures in addition to research results, and

- was organised according to types of seismic deficiencies and summarised in tabular form.

The amount of material available on the topic, much of which has been published since the 1989 Loma Prieta earthquake, meant that even a summary review would be lengthy. The review also indicated that improvement was needed in procedures for prioritising bridges for retrofitting.

The study was eventually split into three parts:

- The present report, covering the review of seismic evaluation and retrofit technology,
- The report, *Seismic Testing and Behaviour of a 1936-designed Reinforced-concrete Bridge* (Maffei 1997), and
- Studies of bridge-retrofit prioritisation (Maffei 1995, 1996).

The review of seismic evaluation and retrofit technology was begun in 1992 and completed in October 1993. Between then and January 1996, a number of new developments have taken place: Caltrans has substantially improved their seismic evaluation procedures; the 1994 Northridge and 1995 Kobe earthquakes have provided additional evidence of typical bridge seismic deficiencies and the need for retrofitting; and more bridges are being retrofitted. The report has been revised therefore to include the most important information from such new developments.

In 1994, a unique opportunity to report on bridge retrofit issues in more detail and from a real-world perspective was presented by Beca, Carter, Hollings and Ferner (1994b). Their project for the design of retrofit concepts for the Thorndon bridge in Wellington, New Zealand, provided several pertinent examples of state-of-the-art seismic evaluation and retrofit principles. The examples are presented in Chapter 4 of this report.

1.4 Organisation of Report

The body of this report is divided into five chapters. Chapter 2 reviews bridge seismic deficiencies, procedures and criteria for seismic evaluation, and Chapter 3 reviews seismic-retrofit techniques. The information from Chapters 2 and 3 is summarised in Table 2.1. For each type of seismic deficiency the following information is listed: observed earthquake damage; research on earthquake performance and retrofit methods; recommended retrofit methods; examples of application; and remarks on retrofit design criteria. The organisation of the table, by types of seismic deficiencies, is designed to allow a structural engineer to quickly determine the applicable information for the specific type of structure that he or she is evaluating.

Chapter 4 provides some examples in more detail of recommended seismic evaluation and retrofit practices, taken from the Thorndon bridge project, then Chapter 5 presents the conclusions and recommendations for the study.

2. SEISMIC EVALUATION OF BRIDGES

Several aspects of the problem of evaluating the ability of a bridge to withstand earthquake effects are reviewed[1]. Section 2.1 reviews the possible seismic deficiencies of bridges, and the seismic vulnerability of many common features of existing bridges is highlighted. Section 2.2 discusses the procedures and criteria which have been proposed for the seismic evaluation of bridges, including recommendations for improved procedures.

2.1 Observed Seismic Deficiencies in Bridges

Throughout the world, past earthquakes have revealed the structural deficiencies of existing bridges. Because reviews of earthquake damage to bridges have already been written (Priestley et al. 1992a), only a summary of damage is given here. Although many existing bridges were designed to allow for earthquake effects, the inadequate seismic design criteria used before the early 1970s make these bridges vulnerable to collapse. Typically, the forces and expected deflections used in the seismic design were too low, structural members that become critical in earthquakes were not designed for ductility and thus could suffer brittle failures, and the concept of capacity design (i.e. precluding the failure of brittle members by providing them with strength exceeding that of a ductile yield mechanism) was not used.

In California, the events that occurred during the February 1971 San Fernando earthquake caused a number of improvements in seismic design practice to be implemented over the next decade. Caltrans (State of the California Department of Transportation) considers that its seismic design code was not fully developed until 1980 (Zelinski pers.comm. 1994), and consequently has carried out an extensive programme of evaluation and retrofitting for structures built before 1980.

Several observed deficiencies in bridge structures are discussed in this section of this report. The deficiencies are presented in the sequence that a structural designer might consider them, roughly following the seismic load path from the roadway superstructure, through the beams, connections, columns, and foundations and abutments, to the supporting soil.

Figure 2.1 illustrates the locations in a structure of potential seismic deficiencies discussed in this Chapter 2 of the report, and Table 2.1 (i.e. Table 1 in Executive Summary) summarises this information on seismic deficiencies. Columns 1-3 (reading left to right) of the table outline the common seismic deficiencies in bridges and the structural engineering research associated with them. Columns 4-7 indicate the possible retrofit solutions, discussed in Chapter 3, for each seismic deficiency. The parts of a bridge structure that have different seismic deficiencies are considered in Sections 2.1.1-2.16.

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[1] Measurements obtained in US research are retained in US units, and have not always been converted to SI units.

Table 2.1 Summary of seismic deficiencies in bridges, recommended methods for retrofitting bridges, and related research.

Bridge Seismic Deficiencies	Observed Earthquake Damage	Research on Earthquake Performance	Recommended Retrofit Methods	Research on Retrofit Methods	Examples of Application of Retrofit Methods	Remarks on Design Criteria
1 Movement joints, seats, restrainers, bearings	Numerous earthquakes, particularly 1971 San Fernando	On restrainers: University of California (UC) Los Angeles [18,19] Babaei and Hawkins (1991a,b) Dynamic analysis of structures with movement joints: Maragakis et al. (1993), Singh and Fenves (1993), Priestley (pers.comm. 1993)	High-strength bar restrainers, looped cable restrainers, straight-through cable restrainers	UC Los Angeles [18,19]	Numerous applications in California and elsewhere	Analysis techniques and design criteria need further development. Babaei and Hawkins (1991a,b) present typical details and cost estimates.
			Steel pipe seat extenders	Cypress Viaduct tests of Dec. 1989 (Zelinski pers.comm. 1994)	Numerous applications in California	
			Concrete or steel bracket abutment seat extenders and stopper devices, steel chain and steel plate restrainers	–	Applications in Japan and elsewhere	
			Base isolation and energy absorbing devices	Various research	Pioneered in New Zealand. Used on several bridge retrofit projects in New Zealand and the United States [68], Used in Italy, Japan and Canada (Skinner et al. 1993)	Inelastic behaviour of isolators can be modelled elastically using equivalent stiffness and increased damping - Generic specifications, needed.
			Elimination of movement joints by deck slab replacement	–	Used in Italy	Applicable only to shorter bridges.
			Addition of high-strength slack restrainers to distribute movement to several joints	–	Proposed for Thorndon Bridge, New Zealand	Applicable to bridges with closely-spaced movement joints.
2 Cap beams	1989 Loma Prieta	–	Strengthening by concrete or steel jacketing and pre-stressing, retrofit by adding a new link-beam below	Proof tests at UC Berkeley and UC San Diego (Zelinski pers.comm. 1994)	China Basin Viaduct, San Francisco (jacketing)	Designed to force plastic hinging into columns.
			Cored internal prestressing, fibre wrapping and prestressing	Univ. of British Columbia, with Klohn-Crippen Assoc.(Kennedy pers.comm. 1995)	Oak St and Granville St bridges, British Columbia	Designed to force hinging into columns (Oak St); or designed for elastic earthquake forces (Granville St).

Bridge Seismic Deficiencies	Observed Earthquake Damage	Research on Earthquake Performance	Recommended Retrofit Methods	Research on Retrofit Methods	Examples of Application of Retrofit Methods	Remarks on Design Criteria
3 Outrigger beam knee joints	1989 Loma Prieta	Ingham et al. (1993), Thewalt and Stojadinovic (1993)	As for cap beams, plus removal of existing knee joint concrete and rebuilding of joint	Ingham et al. (1993), Thewalt and Stojadinovic (1993)	I-980 Freeway, Oakland California	Designed to force hinging into columns or ductile - torsional hinging in capbeam. Reduces capbeam cracking in minor earthquakes.
4 Anchorage of column longitudinal bars in beam-column joints	1971 San Fernando	Seible et al. (1993), Priestley (1993a)	Addition of new link beam, prestressing of joint region, clamped steel confining jacket	–	Santa Monica Viaduct, Los Angeles (Link beam)	Design criteria and retrofit methods need development.
			Added anchorage end plates on straight bars	Park et al. (1993)	–	–
5 Column flexural confinement and bar buckling restraint	1971 San Fernando	UC San Diego [16], UC Berkeley [22], UC Irvine [23], Washington State University Pullman [29] University of Canterbury (numerous)	Elliptical (or circular) steel jacketing	UC San Diego [15, 26] (Yuk Hon Chai et al. 1991), Japan Public Works Research Institute [29]	Numerous applications in California; used for circular columns in Japan; proposed for Thorndon Bridge, New Zealand; used in British Columbia	Detailed design criteria are provided by Caltrans (1992); can be full or partial height; fibreglass method needs material certifications and durability assurances,
			Active fibreglass jacketing	UC San Diego [27]; British Columbia (Kennedy pers.comm. 1995)	Trial applications in California for circular columns; used in British Columbia for square columns	
			Reinforced concrete jacketing	Rodriguez and Park [61]	Applications in Japan and British Columbia	–
			Removal and replacement of cover concrete with added transverse reinforcing steel	University of Canterbury (Dekker and Park 1992)	–	–
			External steel hoops with turnbuckles	University of Washington, Seattle [1]	–	For circular columns only; typically more costly than other methods.
			Prestress wire jacketing	UC San Diego	–	
			Carbon fibre jacketing	Japan Highway Public Corporation [32]	–	–

Bridge Seismic Deficiencies	Observed Earthquake Damage	Research on Earthquake Performance	Recommended Retrofit Methods	Research on Retrofit Methods	Examples of Application of Retrofit Methods	Remarks on Design Criteria
6 Column lap splices	1971 San Fernando 1982 Urakawa-oki 1989 Loma Prieta	UC San Diego [15] Yuk Hon Chai et al. (1991)	Partial confinement elliptical (or circular) steel jacketing (with polyethylene cushion)	UC San Diego (Yuk Hon Chai et al. 1991) (only circular column tested)	Numerous applications in California on rectangular and circular columns	Intended to improve ductility capacity, while keeping moment capacity limited by allowing some lap-splice slippage (Caltrans 1990)
			Elliptical (or circular) steel jacketing (full confinement); active and passive fibreglass/epoxy jacketing (full confinement); reinforced concrete jacketing; added transverse reinforcing; external steel hoops; prestress wire jacketing; carbon fibre jacketing	See item 5	See item 5	See item 5.
			New foundation topping	See item 9	See item 9	Effective for thick toppings which cover lap-splice region.
7 Column shear strength	1971 San Fernando, 1987 Whittier, 1989 Loma Prieta	UC San Diego [28], University of Canterbury (numerous) University of British Columbia with Klohn-Crippen Associates (Kennedy pers.comm. 1995).	Elliptical (or circular) steel jacketing (full confinement); passive fibreglass jacketing (partial confinement); reinforced concrete jacketing; added transverse reinforcing; external steel hoops; prestress wire jacketing; carbon fibre jacketing	See item 5 Univ. of British Columbia with Klohn-Crippen Assoc.	See item 5	See item 5, Jackets are typically full height, except for some flared columns.
8 Pier walls	1995 Hyogo-ken Nanbu (Kobe) earthquake	Haroun et al. (1993) [33]	Steel jacketing with through bolts	UC Irvine [33] (Haroun et al. 1993) Tests in weak direction	Used in California, with full confinement and partial confinement jackets	Design criteria need development. Caltrans does not retrofit if axial load is low and ductility demand ≤ 4.0.

Bridge Seismic Deficiencies	Observed Earthquake Damage	Research on Earthquake Performance	Recommended Retrofit Methods	Research on Retrofit Methods	Examples of Application of Retrofit Methods	Remarks on Design Criteria
9 Column to foundation joints	1971 San Fernando	Yan Xiao et al. (1993), Yuk Hon Chai et al. (1991)	New foundation topping	Yan Xiao et al. (1993)	Used in California. Proposed for Thorndon Bridge, New Zealand	Effectiveness is limited; design criteria need development; Caltrans recommends connecting topping with perimeter ties to bottom reinforcing mat.
			Prestressing through foundation	–	Proposed for Thorndon Bridge, New Zealand	Design criteria and research are needed.
10 Footings and pile caps	No failures noted	Yan Xiao et al. (1993)	Foundation strengthening with new piles or soil anchors; foundation topping and prestressing (see item 9)	–	Applications in California, including China Basin Viaduct; Oak St Bridge; and Queensborough Bridge, British Columbia	Further study of foundation rocking could reduce the need to retrofit.
11 Abutments	Failures due to poor soil conditions in 1990 Costa Rica, 1987 Edgecumbe New Zealand, and other earthquakes	UC Davis [20]	Anchor slab; tension tie backs; anchor piles	–	Applications in California and Japan	Analysis and design criteria need further development.
Retrofit Methods Addressing Multiple Deficiencies						
12 Two-level bridge structures	1989 Loma Prieta	Bollo et al. (1990), Moehle and Sawyer (1993)	Substantial rebuild of structure with added longitudinal beams	Priestley et al. (1992b) Moehle et al. (1993)	I-280 Freeway, San Francisco, California	–
13 Columns and cap beams in multi-column bents	See items 5, 6, 7	See items 5, 6, 7	Infill structural wall between columns	–	Used in Japan, US, and British Columbia. Proposed for Thorndon Bridge, New Zealand	Eliminates transverse moments in cap beams, but can increase loads to foundations.
14 Column and beam deficiencies	See items 2, 5, 6, 7	See items 2, 5, 6, 7	External longitudinal post-tensioning; cored internal post-tensioning	–	Used in Japan, US, and British Columbia	Applicable to elements protected from failure by other ductile elements.
15 Column or pier wall deficiencies	See items 5, 6, 7, 8	See items 5, 6, 7, 8	Added reinforced concrete outrigger frame	–	Used in California, US	Cap beam retrofit is difficult.

Bridge Seismic Deficiencies	Observed Earthquake Damage	Research on Earthquake Performance	Recommended Retrofit Methods	Research on Retrofit Methods	Examples of Application of Retrofit Methods	Remarks on Design Criteria
Site-Related Seismic Deficiencies						
16 Pounding of adjacent structures	1989 Loma Prieta	Kasai et al. (1990); Assessment of Auckland Harbour Bridge, New Zealand (Kennedy pers.comm. 1995).	–	–	–	–
17 Soil liquefaction	1990 Costa Rica and other earthquakes	Various research	Ground improvement methods such as vibro replacement (stone columns), jet grouting, driven displacement piling compaction grouting; and ground containment using bored piles, jet grouting, or concrete walls	–	Vibro replacement (stone columns) and jet grouting proposed for Thorndon Bridge, New Zealand, and used in British Columbia	–
18 Potential for landslides, surface fault rupture and tsunami	1993 Hokkaido-nansei-oki and other earthquakes	Various research	–	–	–	–

Note

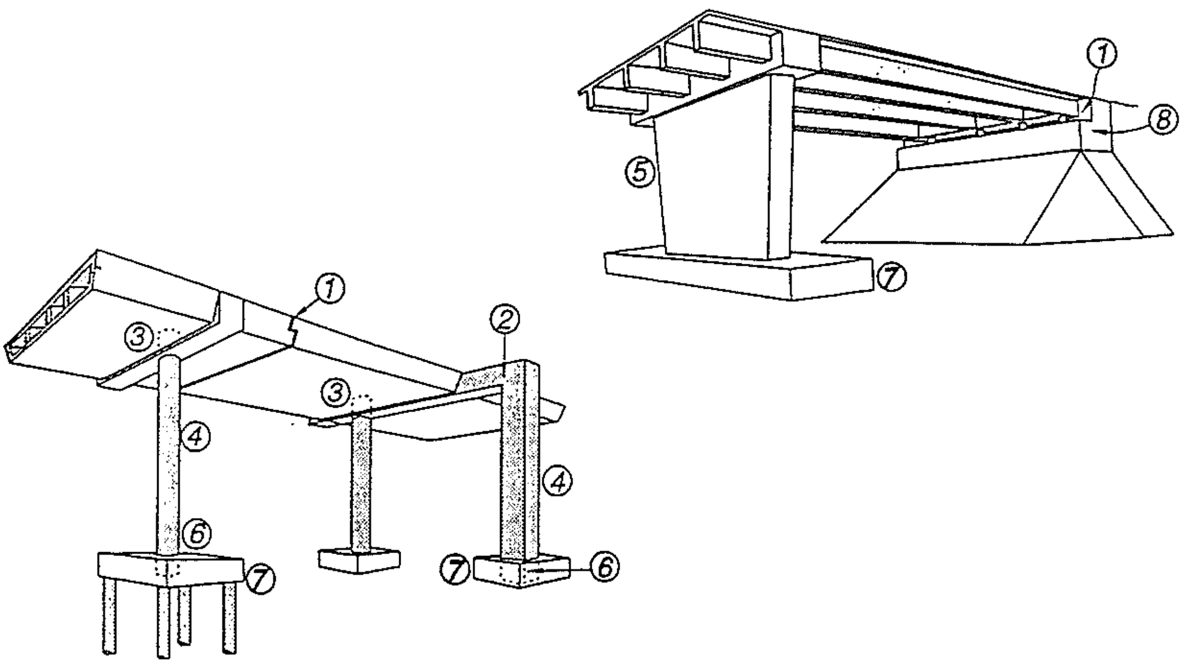
References given in ( ) are listed at the end of this report.
Items not referenced or references with [ ] are taken from Priestley et al. (1992a);
the number in [ ] corresponds to the reference listed in that report.

2.1.1 Unseating of Bridge Spans

The unseating of bridge spans is a common type of seismic failure in bridges. The bridge girders move off their supports at unrestrained movement joints because actual seismic displacements exceed the displacements provided in the design of the girder seats. Restrainer mechanisms, where provided, may have inadequate strength to keep the spans from unseating. Bridges with support lines skewed to the axis of the bridge have a greater tendency to unseat than right angle bridges; skew bridges undergo increased lateral displacements in earthquakes caused by the plan rotation of the bridge superstructure (Priestley et al. 1992a). Research on the earthquake response of bridge structures with movement joints has been carried out by Maragakis et al. (1993), Singh and Fenves (1993), and Priestley (pers.comm. 1993).

2.1.2 Beams and Beam-Column Joints

This category includes seismic deficiencies that have been identified in outrigger and non-outrigger cap beams and beam-column joints, and in two-level bridge structures.



1 Movement joints: seats, restrainers, and bearing supports
2 Outrigger piers and joints
3 Cap beam-joints and bar anchorage
4 Columns
- *(a) Concrete confinement*
- *(b) Restraint of longitudinal bar buckling*
- *(c) Shear strength*
- *(d) Lap splices*

5 Pier walls
6 Column to foundation joints and bar anchorage
7 Footings and pile foundations
8 Abutments

NOT ILLUSTRATED:
Pounding
Soil liquefaction
Surface fault displacement
Slope failure landslide
Tsunami

Figure 2.1 Potential regions that have seismic deficiencies in a typical reinforced concrete bridge structure.

2.1.2.1 Outrigger piers

Outrigger piers are commonly used in US freeway bridge structures at locations where underpassing roadways or other obstacles do not allow columns to be sited directly beneath the bridge deck. Seismic deficiencies in outrigger beams were revealed in the 1989 Loma Prieta (California) earthquake.

The principal deficiencies of these beams are:

- inadequate shear capacity, particularly when seismic shears are additive to dead load shears,
- the premature cut-off of top-beam reinforcement to resist negative moments, and
- insufficient anchorage of longitudinal beam reinforcement into the column.

Outrigger piers in San Francisco's China Basin viaduct and Oakland's Cypress structure had these deficiencies for transverse loading, and were damaged by the Loma Prieta earthquake. Outrigger piers are also potentially vulnerable to torsional failure caused by earthquake response in the longitudinal direction (Priestley et al. 1992a). Sometimes the superstructure flexural capacity to sustain longitudinal-direction column hinging or outrigger torsion is lacking. Calculations often show that the shear-friction capacity of the outrigger beam to superstructure connection is deficient (Zelinski pers.comm. 1994).

The 90° joints connecting outrigger piers to columns (knee joints) are also prone to earthquake damage. The deficiency in these joints is not limited to pre-1970 designs, because a 1984-built outrigger knee joint for the I-980 freeway structure in Oakland was damaged by the Loma Prieta earthquake. This knee joint lacked sufficient joint shear reinforcement horizontally, where 6.4 mm-diameter (W5) wire reinforcement at a 100 mm (4-inch) spacing was used, and vertically, where the 57 mm (#18[2]) column bars were not hooked and were stopped 300 mm (12 inches) below the top of the 2.4 m (8 ft) deep joint.

In addition to the lack of adequate shear reinforcement, the detailing of the pier and column longitudinal bars with 90° bends at the outside corner of the knee joint needs careful consideration. Under cyclic opening and closing moments in the plane of the knee joint, the cover concrete of the outside corner spalls off, allowing the 90° bends of the longitudinal bars to open up so that they can no longer support the compression force of the diagonal strut which is the mechanism of joint shear resistance (Priestley et al. 1992a). Research on outrigger piers and knee joints has been carried out since by Ingham et al. (1993) and Thewalt and Stojadinovic (1993).

2.1.2.2 Two-level bridge structures

A similar condition to the outrigger knee joints is found in the lower beam to column joint in the double-deck bridge structures found in San Francisco and Oakland, which were damaged by the Loma Prieta earthquake. These bridges include the I-880 Cypress structure which collapsed, the Embarcadero freeway which was damaged and

2 US terminology defining bar diameter approximately equal to number of eighths of an inch. Hence #4 or No.4 = ½ inch diameter bar; #3 or No. 3 = ⅜ inch diameter bar.

was subsequently torn down, and the I-280 freeway which was damaged and has been retrofitted. In these T-shaped joints, seismic deficiencies include poor anchorage of beam flexural reinforcement into the joint and the lack of adequate shear reinforcement (Priestley et al. 1992a). Tests on surviving portions of the Cypress elevated highway structure (Bollo et al. 1990) and on laboratory specimens (Moehle and Sawyer 1993) provided further information on these types of structures.

2.1.2.3 Beam-column joints

The exterior beam-column joint of a (non-outrigger) multi-column pier is similar to the outrigger knee joint. Non-outrigger piers are more common than outrigger piers, but less evidence of earthquake damage has shown in joints in non-outrigger piers. The interaction of the elastically responding bridge deck girders perpendicular to the pier can provide beneficial lateral confinement to the beam-column joints. Such confinement is not present in outrigger joints. Nevertheless, in the 1971 San Fernando (California) earthquake, shear failure occurred in the beam-column joints of several multi-column piers.

Inadequate anchorage of column longitudinal reinforcement into beam-column joints is a common seismic deficiency. Seible et al. (1993) have carried out full-scale tests on joints with 57 mm diameter (#18) column longitudinal bars with relatively short anchorages. The actual anchorage capacity may be better than the codes imply, because the bond failure mechanism occurs in the bearing of the steel deformations on confined concrete rather than by the splitting of the concrete assumed in design codes. Park et al. (1993) have improved the anchorage of plain-round (undeformed) bars by welding anchorage plates to the ends of the straight bars. To avoid over-conservative assessments of apparently deficient joints, mechanisms of joint shear transfer and possible bond failure patterns should be carefully investigated (Priestley 1993a, Priestley et al. 1992a). The techniques for such an assessment are as yet not well established in the structural design profession.

2.1.3 Columns and Pier Walls

Several kinds of deficiencies in bridge columns have been evident in past earthquakes. Columns can have inadequate flexural resistance because of:

- understrength,
- inadequate ductility capacity,
- insufficient lap splices, or
- the premature termination of reinforcement.

Because of the inadequate seismic criteria required by pre-1970 design codes, understrength of columns is common. This deficiency is somewhat mitigated, however, by the conservative design practice of assuming a linear axial load versus moment interaction, which was customary for bridges designed before the 1970s (Priestley et al. 1992a).

2.1.3.1 Concrete confinement and bar buckling

Few existing bridge columns have enough strength to permit them to respond elastically to major earthquakes. Thus most columns need to respond inelastically in a ductile manner. It is now well known that, to ensure ductile performance, a close spacing of transverse tie reinforcing is usually required to confine the compressed concrete of the column core in plastic hinge regions. The transverse ties are also necessary to prevent the longitudinal bars from buckling. Bridge columns designed before the early 1970s typically lack this transverse reinforcement.

Dramatic plastic hinge failures occurred in bridge columns with low levels of transverse reinforcing in the 1971 San Fernando earthquake, in the Struve Slough bridge during the 1989 Loma Prieta earthquake (Priestley et al. 1992a), and in the 1994 Northridge earthquake.

2.1.3.2 Lap splices and reinforcement cut-off

The designs of lap splices of bridge column reinforcement typically were based on gravity loads or unrealistically low service-level earthquake forces. The splices often cannot transfer the yield-level forces that will occur in the reinforcing steel under severe earthquakes when they are located in plastic-hinge regions. In the Loma Prieta earthquake, columns suffered damage at their bases attributed to lap-splice bond failure.

During the magnitude 7.1 1982 Urahawa-ohi (Japan) earthquake, columns of the Shizumai bridge suffered brittle failure at mid-height of the column. This failure was caused by the premature termination of a portion of the flexural reinforcing extending up from the foundation (Priestley et al. 1992a).

2.1.3.3 Shear strength

Inadequate column shear strength is another common deficiency in existing bridges. Current capacity-design practices (used in New Zealand since the late 1970s and in California since the early 1990s) dictate that the shear strength of a member should exceed its flexural strength, so that only a ductile flexural mechanism can occur. This was not the design practice for pre-1970 bridges.

Because of flexural overstrengths and low design shears, it is not uncommon to find bridge columns with a shear strength less than one-third of the flexural strength. This was the case for the I-5/I-605 separator, a major freeway bridge structure which suffered dramatic column shear failures in the 1987 Whittier (California) earthquake. Similar shear failures occurred in bridge columns during the 1971 San Fernando earthquake, the 1994 Northridge earthquake, and the 1995 Hyogo-ken-Nanbu, Kobe (Japan) earthquake.

In some cases, also evident in the San Fernando earthquake, shear failure can occur subsequent to flexural plastic hinging. This is a consequence of the reduction in capacity of shear mechanisms assigned to concrete (the "V_C" term) in plastic-hinge regions once flexural degradation begins. For most existing bridges it was not recognised that the concrete in the hinge regions will inevitably be damaged and therefore that these regions should be designed more conservatively for shear than for non-plastic-hinge regions (SANZ 1982, Priestley et al. 1992a).

Research on the seismic behaviour of existing bridge columns is extensive. Recent test programmes related to retrofitting have been carried out at the San Diego, Berkeley, and Irvine campuses of the University of California, and at Washington State University, Pullman (Priestley et al. 1992a).

2.1.3.4 Pier walls

Pier walls are another area of potential seismic deficiency in bridges. Before the 1995 Kobe earthquake, earthquake damage to the pier walls of bridges had not been reported. However reinforced concrete structural walls in buildings, based on similar design concepts to many bridge pier walls, have been damaged in several earthquakes including the 1964 Alaska earthquake and the 1989 Loma Prieta earthquake. As with columns, the desired failure mode for pier walls is flexural rather than shear. Park and Paulay (1975) and Paulay and Priestley (1992) described the undesirable failure modes of reinforced concrete structural walls, as follows:

- diagonal tension or compression failure caused by shear,
- sliding shear along construction joints,
- failure of lap splices or anchorage, buckling of compression reinforcement, and,
- in the case of thin walls, wall instability.

Most pier walls in existing bridges are likely to have insufficient shear strength compared to their flexural strength about the strong axis. However, the shear strength of the walls may exceed the foundation capacity so that foundation rocking or pile failure occurs, precluding damage to the wall itself. Research on the seismic behaviour of pier walls has been carried out at the University of California, Irvine (Haroun et al. 1993, Priestley et al. 1992a).

2.1.4 Pounding of Adjacent Structures

Structural pounding can cause damage to bridges during earthquakes. On the I-280 freeway in San Francisco, a separate connector roadway structure was built alongside the main freeway columns with 150 mm (6 inches) of clearance. This clearance proved inadequate during the Loma Prieta earthquake and both structures suffered pounding damage.

In many past earthquakes, building structures have provided evidence of the disastrous effects of pounding. In the 1985 Mexico City earthquake, pounding of adjacent buildings caused brittle column failures leading to severe damage or collapse of several multi-storey reinforced concrete frame structures (Aguilar et al. 1989). Recent analytical research has shown that pounding impact forces can be up to ten times the magnitude of typical seismic forces (SEAONC 1991, Kasai et al. 1990).

2.1.5 Foundations and Abutments

Foundations are another area of potential seismic deficiency in bridges. Currently (c.1996) the footings and pile caps of hundreds of bridges in California are being strengthened to resist earthquake forces. Common deficiencies include:

- lack of a top reinforcement mat for seismic flexure and uplift,
- lack of footing shear reinforcement,
- lack of joint shear reinforcement at the column-foundation joint, and

- bending of the main column reinforcement outward rather than inward at the base, aggravating joint shear problems (Priestley et al. 1992a).

Figure 2.2 shows other possible failures in bridge foundations, as identified by the Applied Technology Council (ATC 1983).

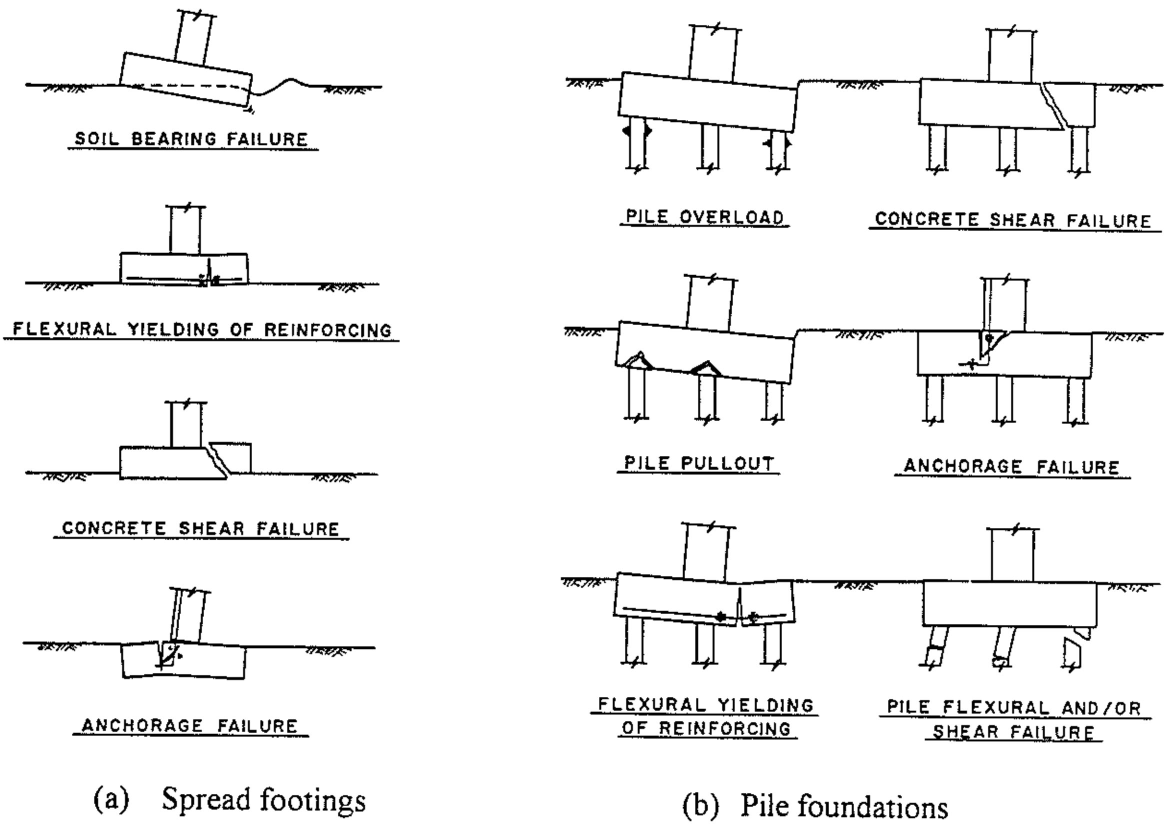


Figure 2.2 Possible seismic failure modes for bridge foundations (ATC 1983).

Despite the perceived need for bridge foundation retrofit, seismic damage to foundations has seldom been reported. Only a few of the failure modes depicted in Figure 2.2 have actually been observed in earthquakes. Several possible reasons for this include:

- Foundations are usually buried and thus not inspected following an earthquake,
- The premature failure of the structure above the foundation caused by column shear, joint failure, lap-splice failure, etc., may have prevented the full plastic column moment from being applied to the foundation, and
- Foundation rocking may have occurred, limiting the forces in footings or pile caps.

A critical seismic deficiency for bridge foundations is the potential anchorage failure of column bars. This deficiency can be closely related to insufficient joint shear capacity and the lack of a top mat of footing reinforcement. For example, inadequate anchorage of column bars into footings allowed the complete pull-out of the column bars resulting in the collapse of a freeway structure in the San Fernando earthquake (Priestley et al. 1992a).

Although not reported for bridges, pile-shear failures have been observed for buildings. In the 1993 Hokkaido-nansei-oki (Japan) earthquake, a school building of nearly new construction was displaced laterally 40 cm because of pile-shear failures. This failure was observed only because the soil had liquified and slumped away from the building exposing the piles (Tanaka pers.comm. 1993). Yan Xiao et al. (1993) studied the seismic behaviour of bridge column footings and tested the rocking response of footings.

For earthquakes acting in the longitudinal direction of a bridge, almost all of the seismic force may be transferred into the abutments (depending on the degree of contact between the end of the bridge, the abutment, and the approach fill). Abutment failures that have been evident in past earthquakes have been mainly caused by poorly compacted or liquefiable soil conditions. Damage caused by soil slumping and consequent abutment settlement or rotation occurred to the Rio Viscaye, Rio Banano, and Rio Bananito bridges during the 1990 Costa Rica earthquake (Priestley et al. 1992a). Large scale tests on the seismic resistance of bridge abutments have been conducted at the University of California, Davis (Maroney and Chai 1994).

2.1.6 Geotechnical Hazards

Soil liquefaction is a seismic hazard that should be included in the evaluation of an existing bridge. Typically liquefaction potential is highest for cohesionless soil layers which are below or near to the ground-water table. Soil liquefaction beneath footing or pile foundations can lead to the displacement of bridge supports. Unrestrained, simply supported spans have frequently collapsed from such liquefaction-induced displacements. Examples of such failures occurred in the 1964 Alaska earthquake, the 1985 Chile earthquake, and the 1990 Costa Rica earthquake.

In addition to liquefaction, other soil-related seismic hazards exist. Bridges on sloping sites can be damaged by earthquake-induced landslides or slope failures. Bridges located near or over faults can be damaged by the structural displacements induced by surface fault rupture. The potential for tsunami should also be considered when investigating the seismic hazards to a bridge near a sea coast.

2.2 Procedures and Criteria

To identify the seismic hazards of existing bridges and to assess the severity of these hazards in a consistent manner, seismic evaluation procedures and criteria are needed. A systematic procedure for seismic evaluation is useful because

- a step-by-step or checklist approach, following the seismic-force path of the structure, will ensure that no seismic deficiencies are overlooked, and
- a more uniform basis for assessment assumptions can result.

Evaluation criteria are required to define what is acceptable seismic performance for a given bridge. The evaluation criteria for existing bridges should be different from the design criteria for new bridges.

For new bridge construction, conservative seismic design and detailing requirements generally cost little compared to the improved security that they provide. However, few older bridges can meet the requirements of building codes for new bridges, and to retrofit to these standards is expensive. Some older existing bridges may be able to withstand severe earthquakes even though they do not meet all code requirements. Criteria for older existing bridges must be based on a more in-depth understanding of the available strength and ductility capacity of "substandard" structural details.

Seismic evaluations can be carried out at different levels of detail. For a first-pass identification of potentially vulnerable bridges, a brief assessment is usually appropriate. This assessment may be based on only a few risk factors such as year of construction, bridge height and skew, soil type, and seismic zone (i.e. a low level of detail). At the other end of the spectrum, a detailed evaluation (i.e. high level of detail) – involving assessment of as-built conditions, dynamic analyses, and calculation of plastic mechanisms, strengths, and ductility capacities – is appropriate once the actual retrofit of a bridge is planned. At a middle level of detail, evaluations involving, for example, quick hand calculations of seismic demands and capacities may only be required. This type of middle-level evaluation procedure is likely to be appropriate for secondary screenings of a bridge stock to identify the highest priority retrofits. With the limited availability of funds for bridge retrofitting, procedures for seismic evaluations of low to middle level detail are important.

Priestley et al. (1992a) have briefly reviewed the seismic evaluation methods used in Italy, Japan, New Zealand, and the US. Further information on these methods is contained in the proceedings of the *International Workshop on the Seismic Retrofitting of Bridges* held in Bormio, Italy (Calvi and Priestley, 1991). The most established seismic assessment methods have been two procedures from the US: the Applied Technology Council *ATC 6-2* approach (ATC 1983) and the current procedure of Caltrans (1992), which are described below.

2.2.1 *ATC 6-2* Evaluation

The *ATC 6-2* (1983) publication was a pioneering effort towards a consistent seismic assessment approach for bridge structures. Figure 2.3 shows a flow chart of the *ATC 6-2* procedure for evaluating existing bridges. The method requires the designer to follow the seismic load path for the bridge structure and, for evaluating each member of an existing bridge, e.g. cap beam, column, or footing, the seismic force demand and its capacity are calculated from analysis. The demand on a member is considered to be the forces resulting from a linear-elastic response spectrum (LERS) dynamic analysis for irregular bridges, or the equivalent static lateral loads (ESL) for regular bridges. The *ATC 6-2* procedure requires the demand to equal the sum of 100% of the response in one direction plus 30% of the response in the perpendicular direction, and vice versa.

ATC 6-2 procedure recommends that bridges with movement joints are modelled with two separate analyses. In the first analysis, a spring element is used to model the movement joint restrainer stiffness; in the second analysis the two sides of the joint are connected with a pin restraint. Column and foundation forces are taken as the worst of the two cases.

2.2.1.1 Member capacities

The capacity of a member is determined based on conventional principles for calculating shear or flexural/axial strength, and is then multiplied by a ductility "indicator" (i.e. ductility capacity) to give an equivalent capacity. Requirements for confinement and reinforcement anchorage and splices are similar to the New Zealand concrete code requirements (NZS 3101:1982, SANZ 1982). For "substandard" details in existing bridges, the ductility indicator is reduced from that assigned to code-complying details.

For columns, ductility indicators are separately calculated for longitudinal bar anchorage, column-lap splices, column shear, and confinement.

For confinement, the column-ductility capacity (μ) depends on the amount of confining-steel area, the spacing of confining ties, and the degree of anchoring of ties to the column core. The assigned column-ductility capacity ranges from μ=2 to 6. For columns which fail in shear before flexural hinging, the ductility capacity is taken as 1. For columns which begin to yield in flexure, the ductility indicator for shear ranges from 2 to 5.

The ductility-capacity calculation for column-lap splices considers the transverse reinforcement that is clamping the splices, and the splice length. The calculation for the capacity of bars anchored into foundations considers bar-anchorage length, transverse clamping-steel area, the presence of bar end hooks and their orientation inwards or outwards, and if the foundation has a top mat of steel reinforcement.

For footings, ductility indicators (μ) are those shown in Table 2.2. The ductility indicators correspond to the failure modes shown in Figure 2.2 of this report.

At movement joints, support length demand is taken as the larger of

- that resulting from the elastic analysis, or of
- the length N calculated by the following formula:

$$N_{\text{(mm)}} = 305 + 2.5L_{\text{(metres)}} + 10H_{\text{(metres)}},$$

where L is the length of the adjoining bridge segments,
and H is the average height of supporting columns or wall piers adjacent to the movement joint.

Figure 2.3 *ATC 6-2* procedure for seismic evaluation and retrofit procedure (from ATC 1983). (References are to ATC 6-2 report)

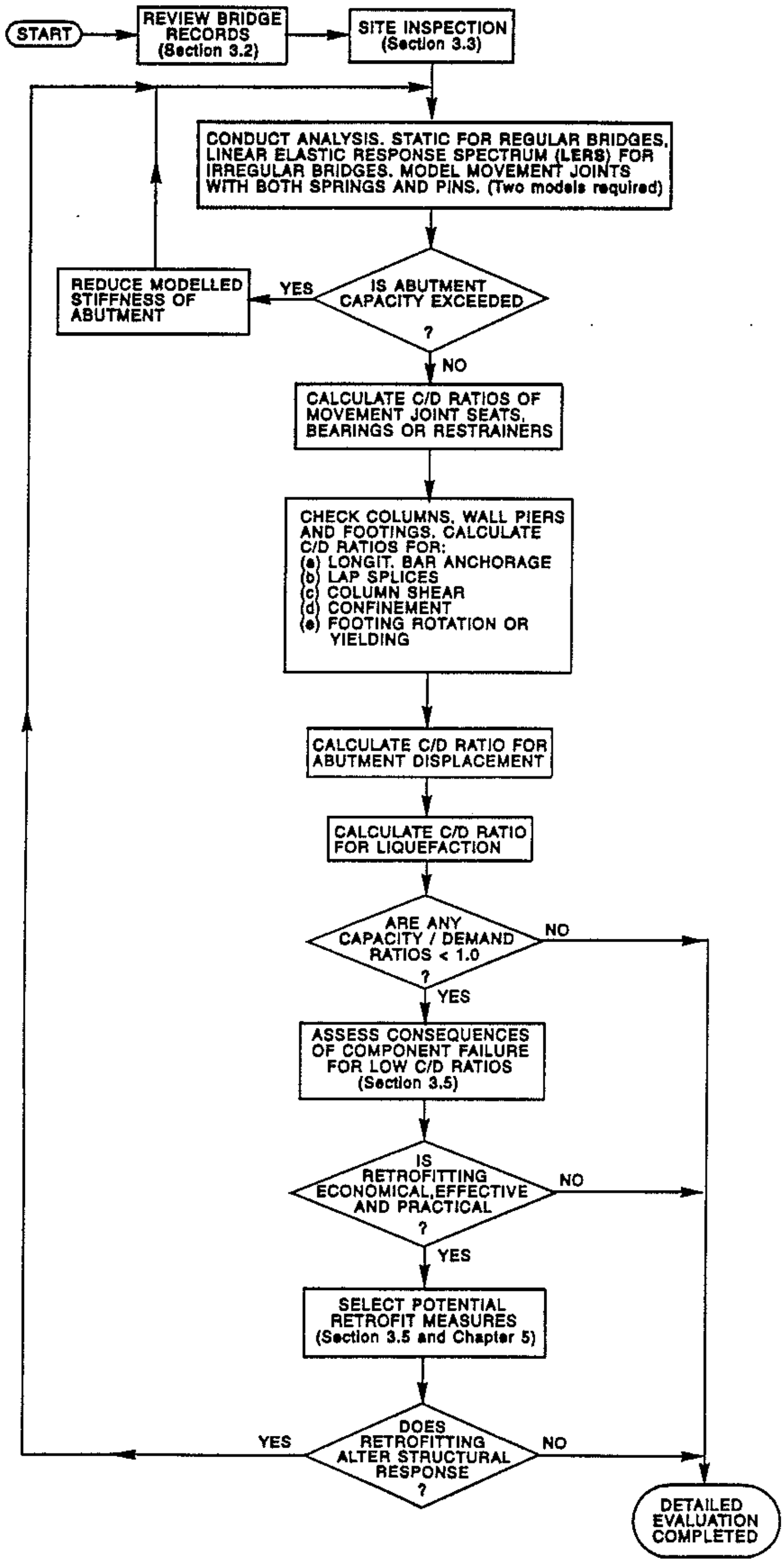


Table 2.2 Footing ductility indicators from *ATC 6-2* (ATC 1983, p. 36).

Type of Footing	Factor Limiting the Capacity	Ductility Capacity μ
Spread Footing	Soil bearing failure	4
	Reinforcing steel yielding in the footing	4
	Concrete shear or tension in the footing	1
Pile Footing	Pile overload, compression	2
	Pile overload, tension	3
	Reinforcing yielding in the footing	4
	Pile pullout at footing	2
	Concrete shear or tension in the footing	1
	Flexural failure of piles	4
	Shear failure of piles	1

This empirical formula is used because elastic analyses may under-estimate movement demands. If movement joint restrainers are used, the support length demand can be computed by considering only the elastic analysis method, but a minimum restrainer force of 0.20 times the deadload reaction must be used because *a linear analysis of a bridge often results in relatively low bearing or restrainer forces* (ATC 1983).

Seismic demand for abutment displacements is taken as the computed abutment deflection in both the longitudinal and transverse directions. Displacement capacity of the abutments is taken as 76 mm (3 inches) in the transverse direction and 152 mm (6 inches) in the longitudinal direction. This seemingly arbitrary criterion is applied regardless of the abutment type or its construction details. The criterion is based on engineering judgement, experience from past earthquakes, and the consideration that large abutment displacements can prevent vehicles from using the bridge. For liquefaction, the capacity to demand (C/D) ratio is taken as the effective peak ground acceleration at which damaging liquefaction is expected to occur, divided by the design acceleration coefficient.

2.2.1.2 Capacity/Demand ratios

When all the demands and equivalent capacities have been determined, the engineer computes a capacity to demand (C/D) ratio for each item checked. Structural members with C/D ratios less than 1 are susceptible to failure. The smaller the C/D ratio, the more vulnerable the member is. An advantage of this approach is that the engineer, by scanning the C/D ratios, can quickly identify the vulnerable members of the structure. This process can readily indicate those members which are likely to require retrofitting.

The general approach of *ATC 6-2* has been applied to buildings in the subsequent publications, *ATC 14* and *NEHRP Handbook* (formerly *ATC 22*) (BSSC 1991). As the ATC evaluation approach has evolved, the procedure has become more thorough and more user-friendly. The *NEHRP Handbook*, now widely used in the US by structural engineers to evaluate buildings, provides checklists designed to identify seismic deficiencies in 15 common building structure types (plus a *none-of-the-above* category). The text of the handbook provides background information for assessing the seismic behaviour of particular types of structure members. The handbook could be considered a "middle-level" evaluation procedure as previously described.

2.2.1.3 Disadvantages of the *ATC 6-2* method

Priestley et al. (1992a) identify the following four drawbacks with *ATC 6-2*:

1. Some requirements such as reinforcement-anchorage provisions are too conservative based as they are on the assumption that full compliance with the current code is necessary for ideal performance.
2. The method may be unconservative because demands are based on the elastic distribution of forces, whereas the local ductility demand may be much higher than global ductility.
3. Some critical areas of bridge structures such as beam to column connections and column to footing connections are not covered.
4. The calculation of demand, based on an elastic distribution of forces, ignores the capacity-design principle that the demand on non-yielding members depends on the capacity of the yielding members.

The first three of these faults can be corrected with little change to the basic *ATC 6-2* procedure. The last point requires a more fundamental (but beneficial) change in the seismic evaluation procedure. To properly understand the expected seismic performance of a structure, the engineer should determine the critical inelastic mechanism for the structure. Once the mechanism is identified, the associated strength and ductility capacities of the yielding members should be calculated. The demand on the other members in the structure is then based on this mechanism strength. Members whose capacity exceeds the demand based on the mechanism strength are protected from failure and need not be ductile. This is the basic procedure of the New Zealand-born concept called "capacity design" (SANZ 1982). The concept is used by engineers in New Zealand and several other countries, but it is not consistently used in the US. One advantage of the capacity-design procedure is that the engineer develops a feel for the actual seismic behaviour of the structure. A capacity-design procedure proposed by Priestley et al. (1992a) for the seismic evaluation of bridges is outlined later in this Section 2.2 of this report.

2.2.2 Caltrans Evaluation Procedure

The seismic evaluation procedure which was used by Caltrans (1992) is a revised and simplified version of the *ATC 6-2* provisions, although in some areas much more specific procedures are provided. The Caltrans approach is outlined in Figure 2.4, which has been simplified (by the author of this report) from a procedural flow chart in *Interim Memo to Designers 20-4* (Caltrans 1992) which describes the then current Caltrans seismic evaluation procedure. The memo notes that *the designer must be cautioned to follow all load path demands and assure that no portion of the resisting structural frame is deficient. Seismic evaluation must not be limited to column or* *pier ductility capacities.* However, unlike *ATC 6-2* or the *NEHRP Handbook*, the Caltrans procedure is not strictly organised around a load-path approach.

2.2.2.1 Bridge analysis

Interim Memo to Designers 20-4 (Caltrans 1992) alludes to the advantages and drawbacks of a collapse-mechanism type of analysis:

> *Structural evaluation at ultimate conditions (i.e. failure analysis) is an extreme challenge to an engineer. Cookbook or prefabricated processes do not lend themselves well to such a situation. Yielding of a single element in a particular mode may not cause collapse.*
>
> *A potential failure mechanism must be achieved before collapse can take place. The distribution, or redistribution, of additional load in a structural system after incremental yielding will be different for each structure. Therefore, each structure must be thoroughly evaluated.*

However, as Figure 2.4 indicates, the Caltrans method relies upon computer LERS analyses, rather than on hand-calculations or collapse-mechanism calculations, for its first-stage evaluations. Recently the Caltrans evaluation procedure has been revised as shown in Figure 2.5 (Zelinski pers.comm. 1994).

Similar to the *ATC 6-2* recommendations, the Caltrans elastic analysis procedure requires that the stiffnesses assumed of abutments and movement joints be iteratively determined. *Bridge Design Aids*, Chapter 14 in *Seismic Design References* (Caltrans 1990), provide guidance on dynamic analysis modelling assumptions. For longitudinal earthquake response, abutments provide horizontal restraint to a bridge principally in compression but not in tension. To account for this, Caltrans recommends allocating one half of the compressive stiffness to each abutment in the analysis model. This gives the correct total longitudinal stiffness to the bridge, and the correct output for abutment displacement, but the resulting compressive reactions to the abutments must be doubled.

Caltrans *Memo to Designers 5-1/5-2* (Caltrans 1988) discusses the expected seismic behaviour of various types of abutments. Caltrans policy is *to accept abutment damage caused by earthquake action provided the damage does not result in collapse of the bridge.* Design loads and typical abutment details are provided in *Bridge Design Aids*, Chapter 1 (Caltrans 1990). *Interim Memo to Designers 20-4* (Caltrans 1992) notes that *field inspections after the 1971 San Fernando earthquake suggest that abutments which moved up to 60 mm (0.2 ft) in the longitudinal direction into backfill soil appeared to survive with little need for repair.*

Figure 2.4 Caltrans procedure for seismic evaluation and retrofit. Adapted from Caltrans *Memo 20-4* (1992), and subsequently revised (see Figure 2.5).

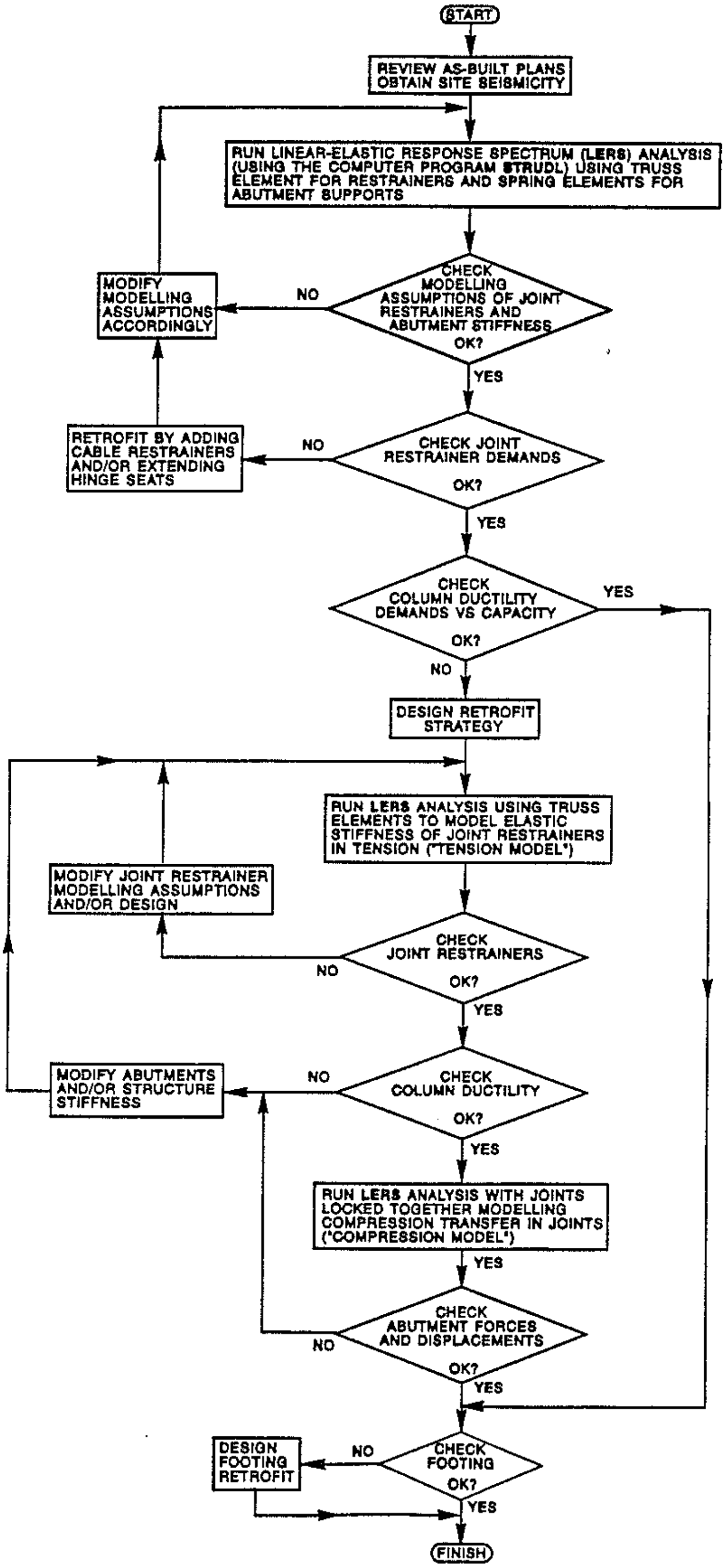
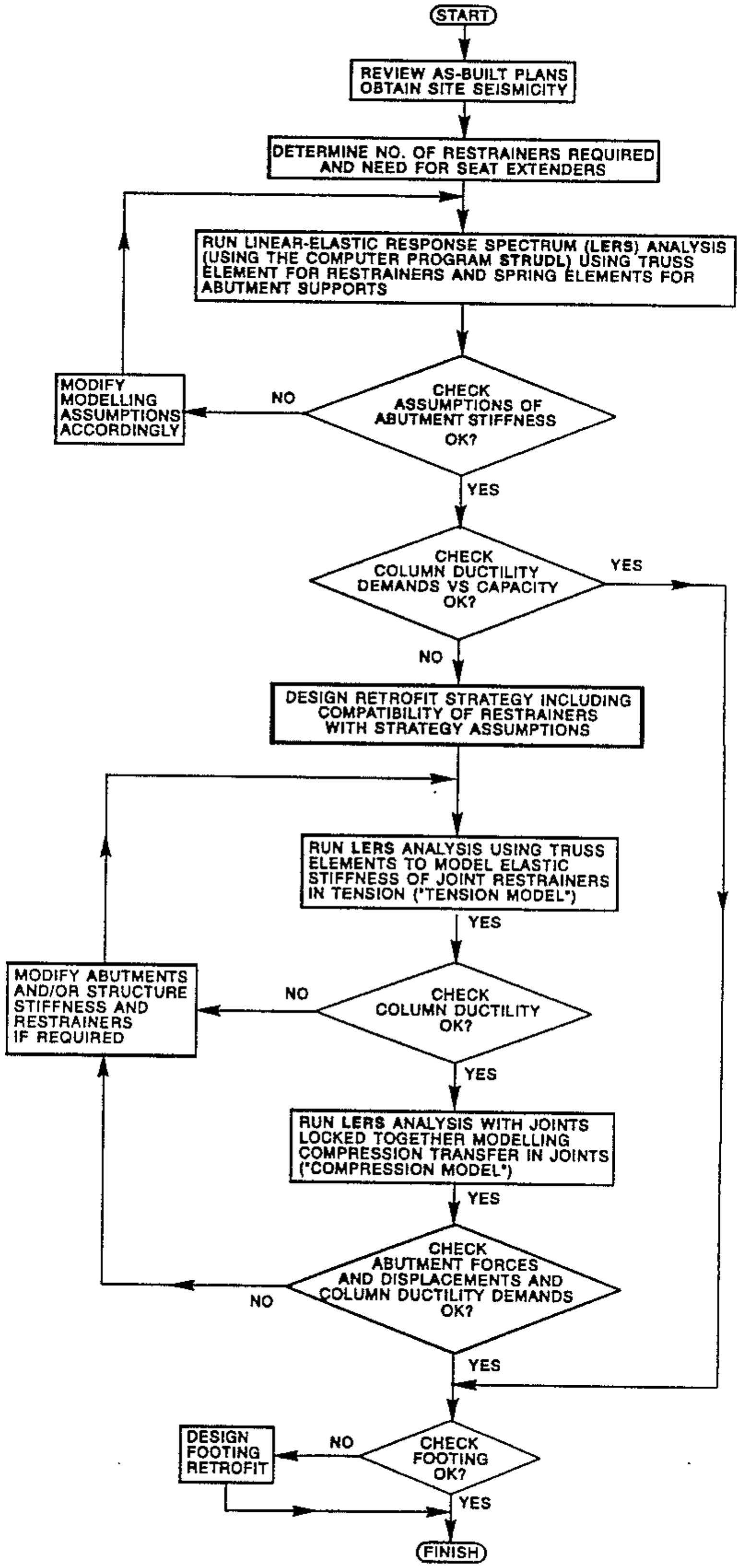


Figure 2.5 Revised Caltrans (1992) procedure for seismic evaluation and substructure retrofit (Zelinski pers.comm. 1994).



Attachment A to *Memo 20-4* gives the bridge retrofit designer additional guidelines for linear elastic computer modelling, using the STRUDL computer program. Curved bridges are modelled with several computer runs in an attempt to represent the different tension and compression stiffnesses of the abutments. Long bridges are modelled in units of up to five spans. The number of vibration modes included in the analysis is typically taken as three times the number of spans. As with *ATC 6-2*, orthogonal direction effects are accounted for by combining 100% of the response in one direction with 30% of the response in the perpendicular direction, and vice-versa. Live loads are not combined with seismic forces, except for outriggers and C-bents (i.e. single-column piers with large gravity-load eccentricities) where the effects of live loads and vertical accelerations can be significant. Special response spectrum curves are used in cases of deep and very soft soil such as the bay mud prevalent in the San Francisco area.

Caltrans recommends including foundation springs in the computer model when such soft soil is present. Bridges less than 91 m (300 ft) long, with no movement joints and with little or no skew, are allowed higher damping values of 10 to 15%, compared to the 5% otherwise used (Caltrans 1992).

An equivalent static analysis is used to design restrainers at movement joints. In the Caltrans approach, *results obtained from [linear-elastic computer] analyses for the design of restrainer units have been proven to be inappropriate because of the demand to resist extremely large elastic column forces which are not actually attained* (Caltrans 1992). The equivalent static method, described in Chapter 14 of *Bridge Design Aids* (Caltrans 1990), recommends that the longitudinal stiffness of bridge segments are calculated assuming that a majority of columns develop plastic hinges before the restrainer capacity is reached. In the example analysis given by Caltrans, the longitudinal stiffness is calculated assuming that plastic hinges with zero stiffness (i.e. pinned conditions) form in 75% of the column ends, resulting in a stiffness which is one-eighth of the elastic value. The restrainer forces are calculated based on this reduced longitudinal stiffness. Clearly, this approach involves gross approximations and assumptions.

However, a study using non-linear models of movement joints indicated that the equivalent static approach may provide conservative results for the design of restrainers (Priestley pers.comm. 1993). More recent Caltrans policy is to model column-end conditions according to expected behaviour or retrofit type. Well-confined column ends are assumed to develop a plastic hinge at full yield forces, while those retrofitted with partial-confinement jackets (allowing lap-splice slip) are assumed to develop into a pin condition (Zelinski pers.comm. 1994). Caltrans *Memo to Designers 20-3* (Caltrans 1989) gives a useful discussion of design issues and practical considerations for the use of movement joint restrainers, considering typical bridge types and configurations.

2.2.2.2 Column capacities

The Caltrans assessment of column capacities has been much more simplistic than the *ATC 6-2* procedure. Rather than calculating separate capacities for anchorage, lap splices, confinement, and shear, Caltrans uses a single ductility capacity depending on the column and pier type. The specified ductility capacities are shown in Table 2.3. In some cases the column connections are simulated with a moment-released pin at one or both ends, thus reducing or eliminating the calculated contribution of the column to the required lateral strength of the structure.

The pin connection is assumed to be located at the base of a column if it has inadequate lap-splice capacity at the base (i.e. the lap length or amount of transverse "clamping" steel is deficient so that lap splices will slip before the column reaches its capacity). A pin condition at the base is also assumed if the column footing fails before the column capacity can be reached.

A pin connection is assumed to be at the top of the column if bar development into the superstructure is judged deficient, or if the superstructure fails before column capacity is reached (Zelinski pers.comm. 1994).

Table 2.3 Column-ductility capacities assigned by Caltrans for assessment purposes (Caltrans 1992).

Column Type	Single Column Pier	Multi-Column Pier
Round columns	1.5 - 2.0	2.0 - 3.0
Rectangular columns	1.0	1.5 - 2.0
Round column pile shafts (in-ground hinge only)	2.0 - 3.0	3.0 - 4.0

Notes:

For multi-column pier bridges with larger amounts of redundancy, such as several sets of three (or more) column piers, the maximum of the allowable ductility range may be used on columns.

For single-column pier bridges, the maximum of the allowable ductility range should not predominate (i.e. not be more than 33% of the fixed column ends) the range of ductility demands for the total bridge (Caltrans 1992).

This procedure for assigning ductility capacities has been questioned (Chapman pers.comm. 1994) because it is made without evaluating the likely failure mode of the column. The procedure also ignores the strength and flexibility of the foundation, which can affect the failure of the columns and the displacement capacity of the structure.

Attachment A to *Memo 20-4* (Caltrans 1992) provides background discussion on using the equal energy assumption to relate inelastic force reductions to ductility capacities. The assumption is thought to be applicable for shorter period structures. Despite this commentary, the Caltrans procedure seems to be based only on the equal displacement principle: ductility capacities are used directly as force reduction factors. This approach may be unconservative for shorter period structures. Zelinski (pers.comm.1994) notes, however, that *for short bridges and short-period structures, general retrofit practice includes a rotational displacement capacity check.*

Caltrans computes column-moment capacities based on probable material strengths. If the actual steel yield strength is not known, a strength of 1.1 times the specified yield is used. Column shear strength is compared to the demand corresponding to full flexural plastic hinging. *Attachment B* to *Memo 20-4* describes how column shear strength is assumed to vary with ductility.

In potential plastic-hinge regions the shear stress carried by the concrete, v_c, is assumed to degrade linearly from 0.58 $\sqrt{f_c}$ MPa (3.5 $\sqrt{f_c}$ psi) prior to ductility 2, to zero at ductility 4. For columns with "moderate" levels of confinement, v_c is assumed to degrade from 0.58 $\sqrt{f_c}$ MPa at ductility 2, to 0.20 $\sqrt{f_c}$ MPa (1.2 $\sqrt{f_c}$ psi) at ductility 4.

If the shear strength is less than the shear force corresponding to a flexural response at overstrength, then shear retrofit is required. Note that flexural overstrength is taken to be 1.5 times the nominal flexural strength. If lap splices occur in a potential plastic-hinge region, retrofit is required. As discussed in Chapter 3 of this report, lap-splice regions can be retrofitted with either a full-confinement or partial-confinement column jacket depending on retrofit design assumptions. As a final evaluation step, foundations are checked. Compared to new designs, more liberal soil and pile capacities are sometimes allowed for retrofit design.

2.2.2.3 Pier-wall capacities

Until recently little guidance has been given for the evaluation of pier walls about their strong axes. Pier walls are assumed to have weak-axis ductility capacities of up to 4.0. In many cases, however, the pier walls are assumed pinned at the base because the foundation fails before the wall's weak-axis moment capacity can be developed.

Current Caltrans policy assigns a ductility capacity of 2.0 to pier walls in the strong-axis direction. The massive size and ample horizontal reinforcing used in most pier walls is considered to allow them to respond elastically to major earthquakes. In the strong direction of the walls, the foundation piles or pile connections are usually determined to fail (or "fuse") long before the wall would reach its shear or flexural capacity. Thus the designer would assume that either foundation sliding or rocking (whichever is more likely) will govern the earthquake response.

The Caltrans approach (1992) has been continually evolving in recent years. Some bridges are evaluated using a capacity-design approach and an incremental plastic-mechanism (push-over) analysis. Until recently, these were usually applied to borderline cases where a more detailed analysis can show that retrofit is not required, and more than 90% of bridge retrofits were based on an elastic structural analysis only.

Since 1993 however, the capacity-design and push-over analysis approaches have been used on almost all major bridge evaluations. Caltrans engineers had felt that the elastic approach was easier to put into a step-by-step procedure, that the results were conservative, and that some added protection of serviceability in smaller earthquakes was provided as a side benefit (Zelinski pers.comm. 1993, 1994). However, the Caltrans criteria are intended *to prevent collapse. ... Where structure serviceability is defined as a design requirement, a more conservative design approach than that outlined in this Memo 20-4 must be followed.*

2.2.3 Improved Evaluation Approaches

Researchers have shown that elastic analysis methods for bridges can be inadequate. Priestley et al. (1992a) have developed an improved evaluation approach.

2.2.3.1 Inadequacies of elastic analyses

Priestley et al. (1992a) provide a good discussion of the inadequacies of elastic dynamic analyses, noting that there *is a tendency to think, as a result of advances in computational models and the widespread availability in design offices of computers with considerable power, that our ability to predict the seismic demand on a bridge in terms of required moments, shears, displacements, etc, is at a much higher level of precision than our ability to predict capacities. This is a delusion, ...* .
The following inadequacies of elastic analysis are identified:

1. The basic parameters of elastic analysis and modal combination – mode shapes, periods and effective damping – are all altered by inelastic action.
2. Predictions of site seismicity and elastic response spectra involve "uncertainty" factors in the order of 2.
3. Modelling of movement joints involves significant and compounded approximations. Initial gaps, frictional resistance, three dimensional effects, and the effects of bridge curvature are usually neglected.
4. Abutments and foundations are poorly modelled especially in the usual case when the compression and tension stiffnesses of an abutment are different.
5. The effect of non-coherent (out-of-phase) seismic input is ignored. In softer soils seismic wavelengths may only be 150 to 450 m (500 to 1500 ft) causing out-of-phase effects in medium to long bridges.
6. Pre-yield member stiffnesses are often poorly modelled.

2.2.3.2 Recommended approach

Instead of an elastic analysis and a check of C/D ratios, Priestley et al. (1992a) proposed an approach that is more representative of actual structural seismic behaviour. The approach is more general than the Caltrans method and has not been developed into a "procedure". However, Figure 2.6 shows a flowchart (prepared for the present report from Priestley et al.) which outlines the basic steps of the method. As shown in the figure, the method requires

- explicit consideration of the serviceability, damage-control, and survival limit states, and
- a diagnosis of the critical plastic mechanism for the structure.

Rather than using an elastic dynamic analysis, the approach recommends a frame-by -frame incremental plastic mechanism, or "push-over", analysis which will identify critical structural members and define a force-versus-deformation curve for each frame in the transverse direction. Such analyses have been used by bridge designers in New Zealand for many years (Chapman pers.comm. 1993) but their use in California is new. Until recently, plastic analyses were not conducted there for most bridge evaluations. For longitudinal response the same push-over analysis is recommended, with the consideration that the moment transfer between a column and its superstructure may be limited by the torsion capacity of the cap beams. Although the above calculations can be done by hand, a computer program for plastic analysis would however simplify the designer's job. Producing this type of program is straightforward, but no such software has been generally available.

Along the same line of thinking, Moehle and Aschheim (1993) note that, for the evaluation of bridge structures, *linear analysis methods fail to capture essential response characteristics of structures for which inelastic response is expected.* In agreement with Priestley et al, they recommend the use of non-linear "push-over" analyses for evaluating bridges. Elastic response spectrum analyses are still recommended for use in tandem with the "push-over" analysis, because they *are easily done and provide a useful benchmark for the estimation of inelastic displacement response.*

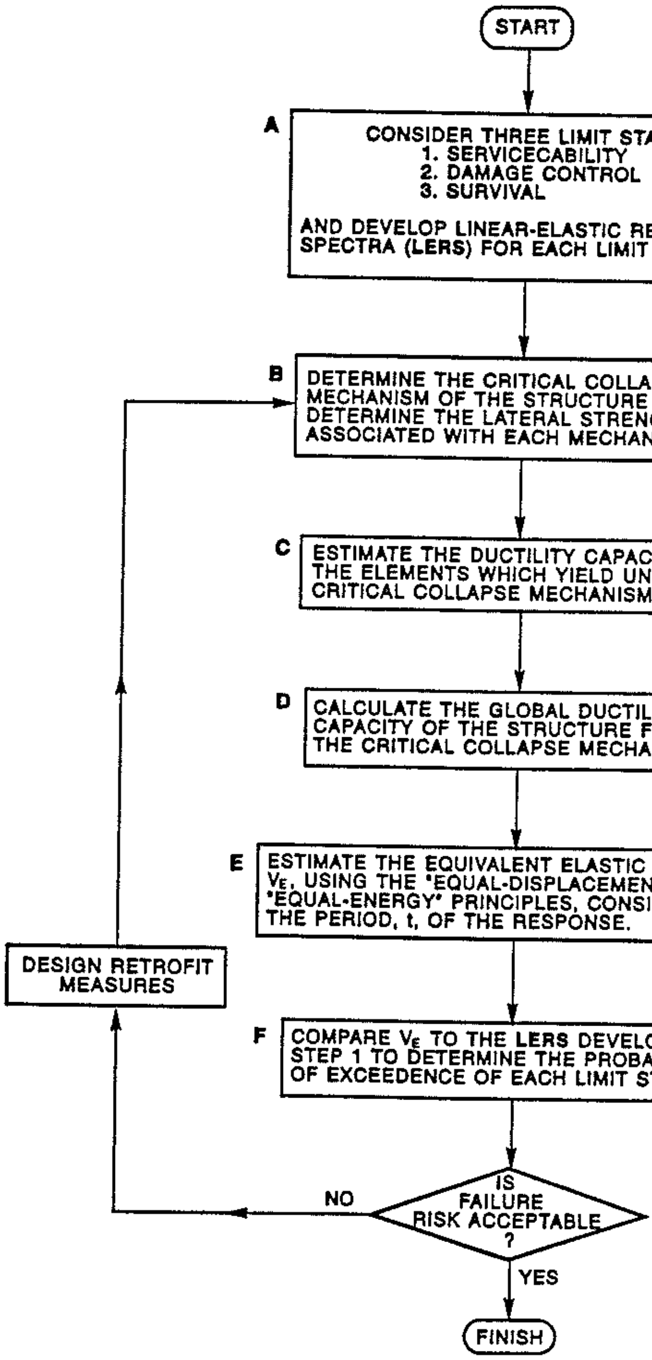
The recommendation is made to base the relationship of strength and ductility capacity to equivalent elastic response on the equal displacement principle for longer period structures, with linear interpolation to the zero period case for shorter period structures. This is the same recommendation of Priestley et al., except that some refinements are suggested in the determination of the period beyond which the equal displacement principle applies.

2.2.3.3 Evaluation of member strengths and ductility capacities

In the Priestley et al. evaluation approach, strengths and ductility capacities are to be calculated as realistically as possible. It provides several guidelines, and indicates areas where further research is needed. The guidelines recommend that flexural strengths are based on probable material strengths. An approach for assessing the flexural strength and ductility of sections with inadequately anchored or spliced longitudinal bars is presented. More test results are needed to further quantify this approach for the range of anchorage or splice lengths and amount of transverse steel encountered in existing bridges.

Flexural strength versus ductility-capacity relationships are suggested for columns with poor confinement or lap splices in the plastic hinge region, as shown in Figure 2.7. Note that such relationships could be used in an incremental collapse mechanism or "push-over" computer analysis program. Equations for the prediction of shear strength are given which have been shown to be more accurate than current code equations. Further refinements to the proposed equations have subsequently been made (Priestley et al. 1994).

Figure 2.6 Seismic evaluation and retrofit approach proposed by and adapted from Priestley et al. (1992a).



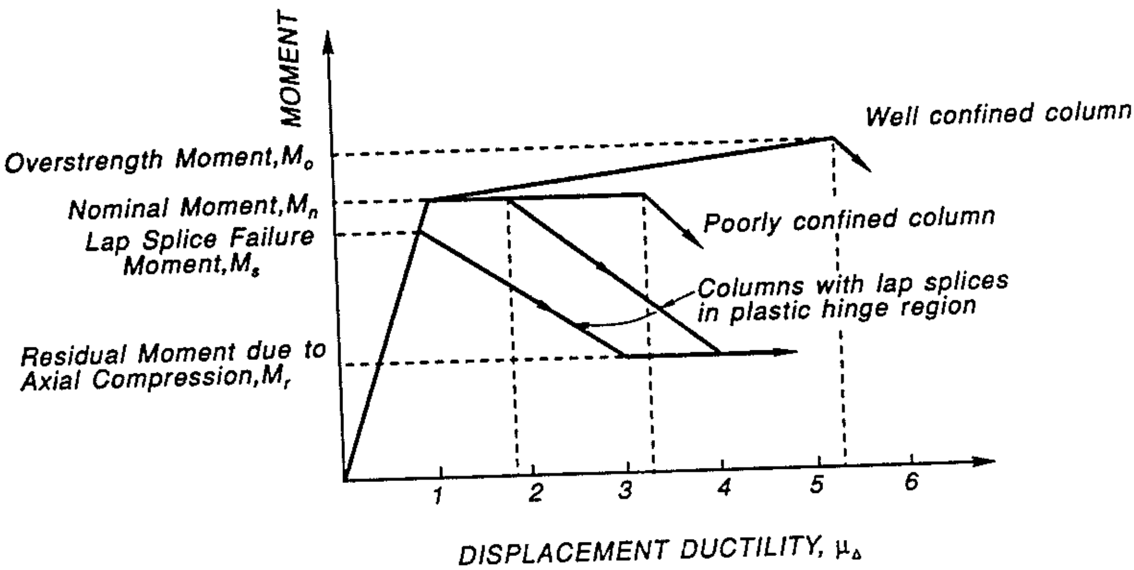


Figure 2.7 Flexural strength versus ductility capacity relationships (Priestley et al. 1992a).

The *Design Guidelines* report by Priestley et al. (1992a) also gives guidance on the design of beam to column knee joints, beam to column T-joints, and column to footing joints. Research is underway which should provide more information on the evaluation of these connecting joints. General guidance on the evaluation of footings is provided, and it is postulated that the rocking response of foundations is permissible and perhaps desirable. An iterative method for predicting the rocking response of structures, based on a displacement response spectrum analysis, is presented. In the plastic-mechanism type of analysis, foundation rocking can be considered in the same way as plastic hinging, i.e. with an assumed strength versus ductility relationship. More research is needed on the seismic performance of footings, including the behaviour of footings which yield in flexure.

Determination of local ductility demands also depends on assumptions of elastic displacements. Priestley et al. recommend that some attempt is made to assess which members will be essentially uncracked and which will be cracked when computing elastic displacements. Foundation flexibility effects should also be considered. As discussed in Section 2.2.3.4, there is some question as to what stiffness assumptions are appropriate for elastic systems.

After individual section strengths, stiffnesses, and ductility capacities have been determined, a global ductility capacity should be calculated. The global ductility depends on the local section ductilities, the geometry of the structure, and the correct identification of the inelastic deformation mechanism. Priestley et al. provide some examples of how to calculate global ductility. In a computer "push-over" analysis program, the global ductility could be automatically calculated.

Once the plastic mechanism strength and associated global ductility capacity are known they can be compared with the elastic response spectrum demand of the specified intensity of earthquake to be accommodated by the design. Maximum ductility capacities would be used for comparison against the survival-limit state, while lower ductilities, corresponding to less allowable damage, would be used for comparison against the damage-control or serviceability-limit states.

In the Priestley et al. approach, the structure capacity V_f is increased by a factor to give equivalent elastic capacity V_E. The factor V_E/V_f is equal to the global displacement-ductility capacity for longer period structures (the equal displacement assumption). For shorter period structures, V_E/V_f is less than the global ductility capacity, and is calculated according to an interpolation formula. The resulting value of equivalent elastic capacity, V_E, is compared with the elastic response spectrum demand to give an "annual probability of exceedance", that is to say the likelihood of the capacity being exceeded, based on the return period of the specified limit-state earthquake.

2.2.3.4 Inelastic response assumptions

One problem with the ductility-based approach to design is that stiffnesses of the elastic structure must be appropriately determined. Moehle and Aschheim (1993) discuss the problem and its effect on ductility-based design assumptions. Figure 2.8(a) shows the moment versus displacement response of a vertical cantilever column. For this structure, elastic stiffness could be considered as the initial stiffness, slope A, based on gross-section properties, or alternatively an effective stiffness, slope B, calculated from cracked-section properties.

Two problems are evident:

1. If slope A is used in the evaluation procedure as the basis for computing ductility demands, much higher demands would be predicted than if slope B were chosen.

2. If slope A is considered as the elastic stiffness, a shorter elastic period will be assumed, and from the displacement response spectrum, smaller displacements will be predicted than if slope B is considered. This is shown as the difference between Δ_{EA} and Δ_{EB} in Figure 2.8(b). But for a given moment-displacement relationship and earthquake input only one inelastic displacement will result. Under the equal displacement assumption, which elastic displacement is it *equal* to? It cannot be equal to both values. Thus the validity of the equal displacement assumption depends on what elastic stiffness assumptions are made.

Preliminary research results (Moehle and Aschheim 1993) indicate that if Caltrans response spectra are used as earthquake input, the equal displacement assumption is more accurate when gross-section stiffness (slope A) is assumed. If effective-section stiffness (slope B) is assumed, displacement predictions using the assumption are conservative. However, Priestley (pers.comm. 1993) has suggested that the Caltrans

ARS spectra are in themselves conservative in the longer period range. Although analytical studies have been done, more research is needed to validate the equal displacement assumption and to give guidance on selecting elastic stiffnesses for analysis. The research should compare inelastic analysis results with appropriate and corresponding elastic spectra.

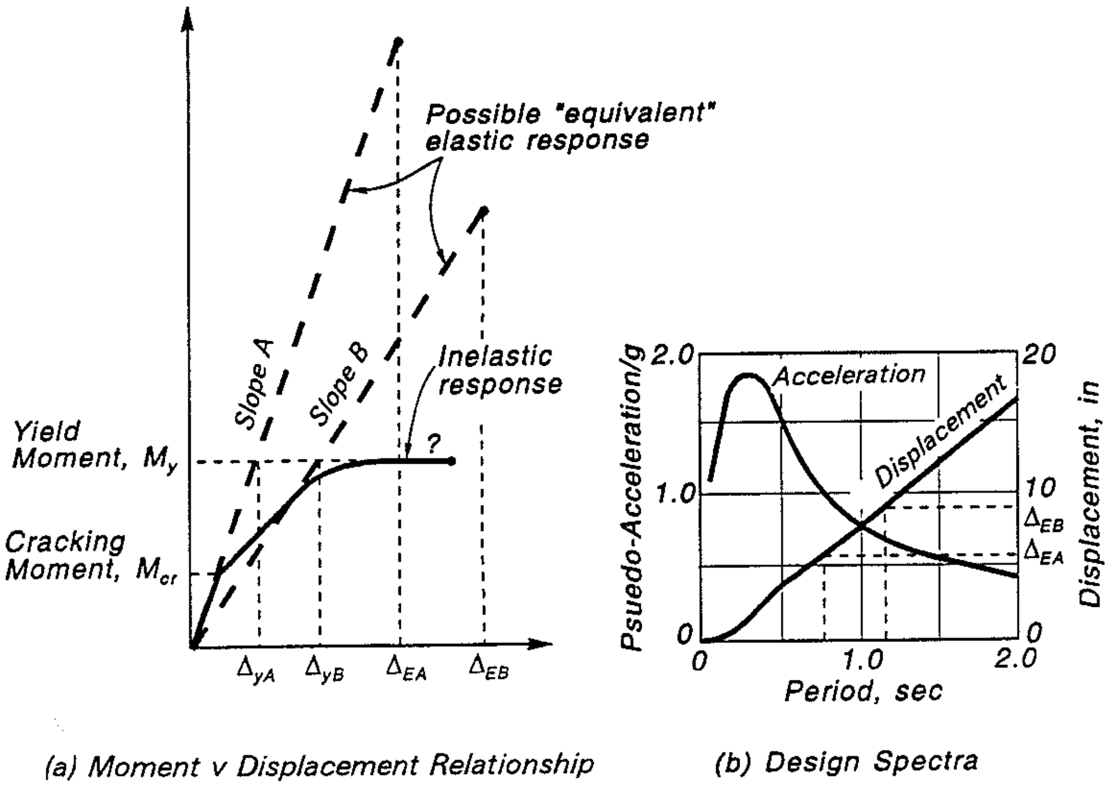


Figure 2.8 Effect of assumed elastic stiffness on the validity of the equal displacement principle. Adapted from Moehle and Aschheim (1993).

The idea of using a displacement approach to design rather than a force approach is promising, and could correct some of the problems with current analysis approaches (Priestley 1993b, Moehle 1992). A displacement-based design procedure would require, however, a fundamental change in design codes and engineering practice.

2.3 Analysis of Movement Joint Effects

One evaluation area which still presents problems is the assessment of movement joint and restrainer displacements and forces. Priestley et al. (1992) discuss five different procedures which all seem unsatisfactory.

The first two are the empirical *ATC 6-2* (1983) approach based on span length and bridge height, and the Caltrans equivalent static procedure. The third procedure, also used by Caltrans, is an elastic dynamic analysis with restrainers modelled as elastic springs. This approach cannot account for the non-linear behaviour of the movement joint as restrainers slacken or stiffen, or as the joint closes and the compression stiffness suddenly and dramatically increases. Also, the inelastic behaviour of the adjacent bridge segments is not accounted for.

A fourth procedure, the relative ground motion approach, is based on assuming relative ground motions at the foundations as a result of non-coherent seismic input. The worst case input results from the largest anticipated ground displacements coupled with the shortest anticipated seismic wavelengths. This approach is appealing for its simplicity, but it ignores the fact that structure displacements may be larger than peak ground displacements, depending on the dynamic properties of the structure. Also, the approach implies that if ground motion is coherent there is no movement or force demand at movement joints, which is not true.

The fifth procedure to evaluate movement joints is by using inelastic dynamic time–history analyses. This more complicated procedure is potentially the most rational and accurate method for determining the critical forces and displacements at movement joints. Efforts at this type of analysis are currently underway (Priestley pers.comm. 1993), including the development of appropriate hysteresis models for the movement joints.

3. SEISMIC RETROFIT TECHNIQUES

Structural engineering research plays an important role in validating proposed retrofit techniques. As a result, various seismic retrofit techniques have been proposed, tested, and/or used for bridges in California, Japan, and elsewhere. Table 2.1 (in Chapter 2 of this report) outlines seismic retrofit techniques recommended for several common bridge seismic deficiencies, and a number of effective seismic retrofit techniques for movement joints, columns, and other potential seismic deficiencies are discussed in this Chapter 3.

Table 2.1 and this chapter are organised to consider the deficiencies of a bridge along a seismic-force path from the superstructure through to the foundations and abutments. However, in seismic retrofit design the bridge must be considered as a whole, as well as member by member. The retrofitting of one bridge member can affect the seismic response, and the need to retrofit other members of the bridge.

The primary aim of this chapter is to summarise the information that would be most useful to the bridge retrofit designer. Thus, emphasis is placed on design criteria, actual applications of retrofit designs, and on those techniques which seem to be the most effective structurally. For further details of retrofit research, the reader is referred to the *Design Guidelines* report by Priestley et al. (1992a), and to the proceedings of the Caltrans *Second Annual Seismic Research Workshop* (1993).

3.1 Movement Joints, Seats, Restrainers, Bearings, and Base Isolation

Seismically deficient movement joints can be retrofitted by adding restrainer (or linkage) rods or cables, seat extensions, or base isolation devices.

3.1.1 Restrainers or Linkages

The most common way of retrofitting deficient movement joints is by adding restrainers. The restrainer ties the two ends of the movement joint together, typically using steel cables or high-strength rods.

Research on three types of restrainers used by Caltrans has been carried out at the University of California, Los Angeles, as described in 1989 publications by Selna, Malvar, and Zelinski (referred to in Priestley et al. 1992). The restrainer types tested include high strength bars, looped cable restrainers attached to box girder diaphragms, and straight-through cable restrainers attached to the box girder deck flanges. Cable restrainers can also be attached to box girder webs.

Figure 3.1 shows a retrofit plan of a movement joint using added cable restrainers and pipe seat extenders. Figure 3.2 shows the details of the cable restrainers. The restrainers are attached with brackets to the box girder webs at one side of the joint, and looped around the strengthened end diaphragm of the box girder at the other side of the joint. Restrainer cables for precast girders are often wrapped around the supporting cross beam as shown in Figure 3.3 (Zelinski pers.comm. 1993).

In addition to steel bars and cables, Japanese engineers have used steel chains and hinged steel plates to restrain movement joints. Design of restrainers should specify the initial tension or slackness of the restrainers, considering the expected temperature movement of the joint and the temperature at the time of the restrainer installation. In New Zealand, bar-restrainers and linkage bolts for bridges usually have thick rubber pads under the plate washers at each end of the bar. The rubber pads allow temperature movements to occur, and they reduce impact forces on the restrainer bar.

As discussed in Section 2.3 of this report, the Caltrans design of movement joint restrainers is based on an equivalent static analysis which involves gross approximations and assumptions. Although the method may be satisfactory for the time being, more accurate design methods should be developed.

3.1.2 Seat Extensions

Movement joint restrainers are intended to limit the movement of spans so that the spans do not fall off their supporting seats. Another solution to the problem, often used in conjunction with restrainers, is to extend the seat length. For box-girder bridges, Caltrans commonly uses steel-pipe seat extenders. The pipes are placed through core-drilled holes in the end diaphragms of the spans on either side of the movement joint as shown in Figure 3.2. Typically the steel pipes are 220 mm outside diameter with a 22 mm wall thickness (8 inch nominal diameter, double-extra strong).

Other types of seat extensions, which have been used in Japan, include added concrete corbels or added steel brackets anchored to the lip of the existing concrete seat. "Stopper" devices have also been used. These stoppers consist of steel or reinforced concrete brackets which restrain the movement of the end diaphragms of the spans. These seat extension and stopper devices can be used at abutment movement joints but may not be applicable at intermediate movement joints.

3.1.3 Base Isolation

Vulnerable types of support bearings such as steel rocker bearings can be replaced by base-isolation devices. For the Sierra Point overbridge on Route US101 near San Francisco, California, the existing spherical steel bearings were replaced with elastomeric bearings containing lead plugs. In addition, movement joints were restrained using 22 mm (7/8 inch) diameter high-strength steel rods.

As well as addressing the problem of vulnerable movement joint details, the retrofit reduced the seismic forces in the bridge columns because of the increased damping and the shift in fundamental period provided by the base isolators. For example, because of the reduced lateral forces obtained by the isolation of the superstructure of the Sierra Point bridge, column retrofitting was unnecessary (Priestley et al. 1992a).

Figure 3.1 Retrofit plan for the movement joints of a concrete box girder bridge using cable restrainers and steel pipe seat extenders (Zelinski pers.comm. 1993).

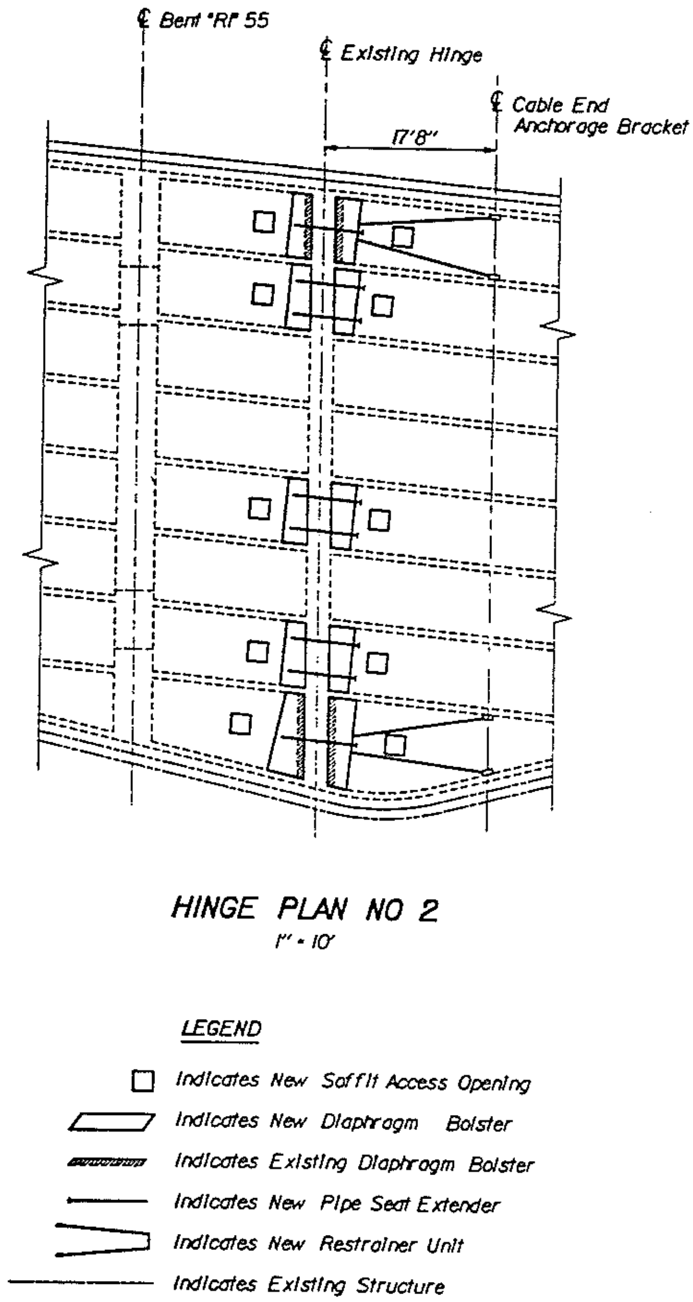


Figure 3.2 Details of cable restrainers and steel pipe seat extender (Zelinski pers.comm. 1993).

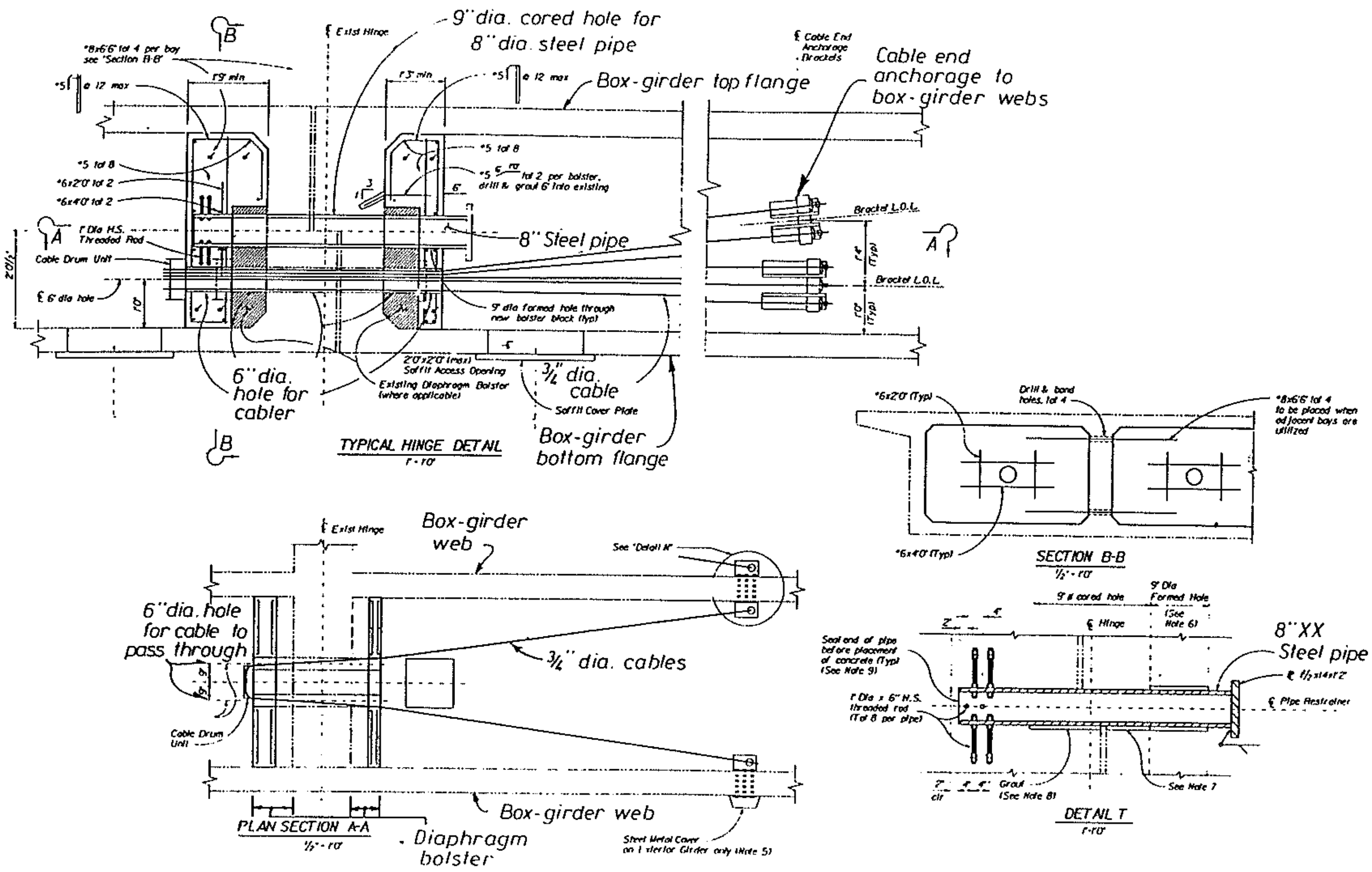
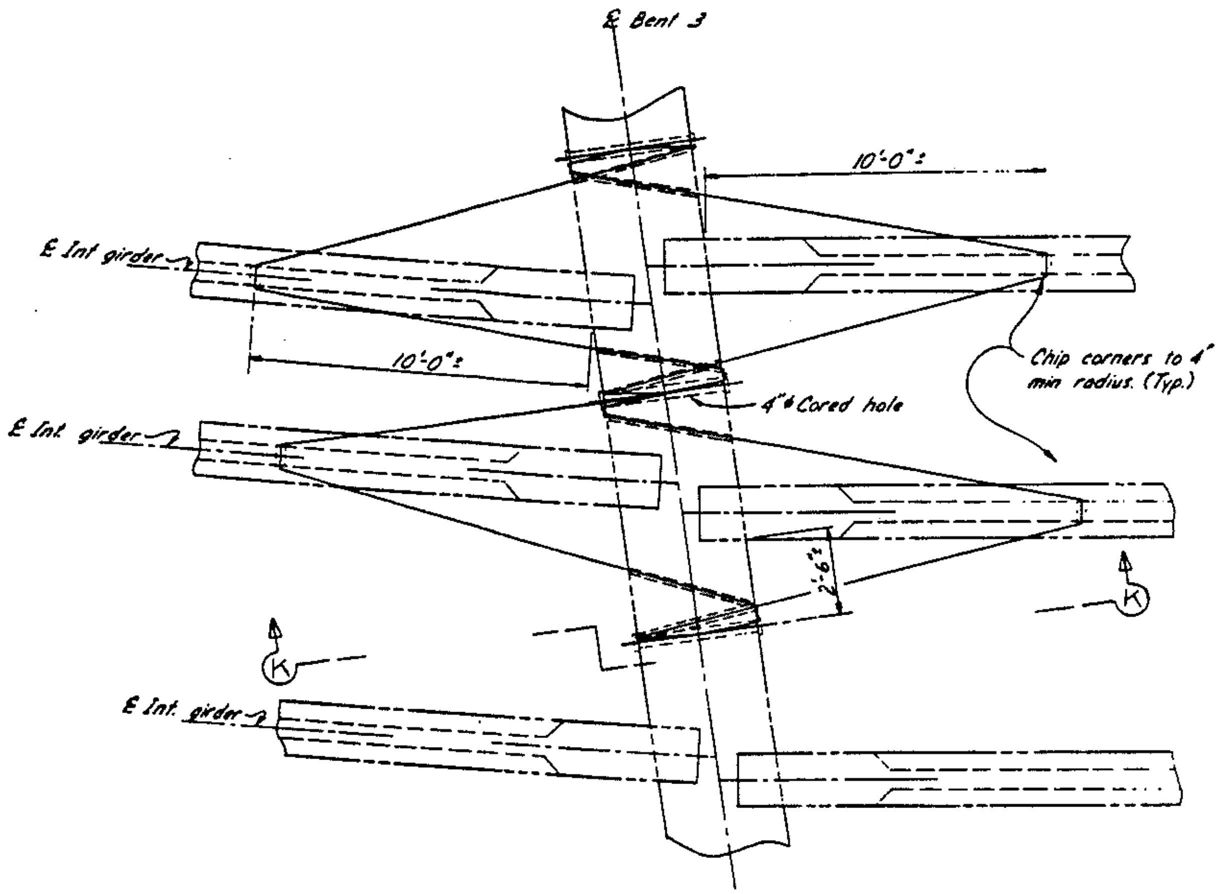
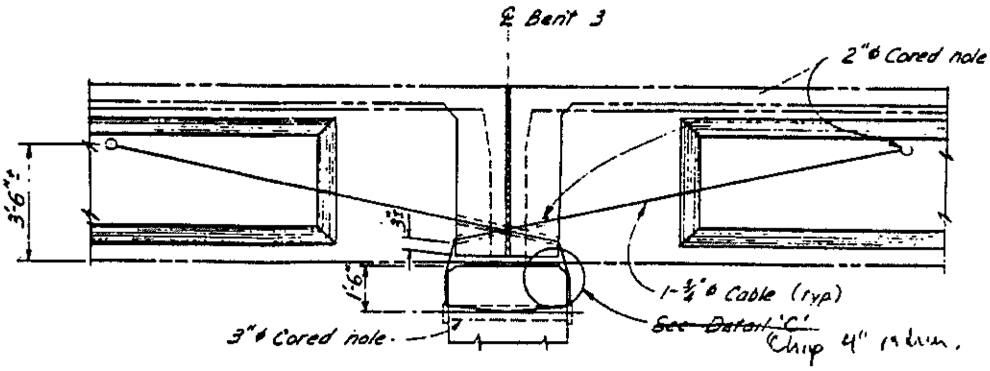


Figure 3.3 Cable restrainers for a bridge with precast girders (Zelinski pers.comm. 1993).



EXAMPLE BRIDGE D: PART PLAN

¾"=1'-0"



SECTION K - K

Skinner et al. (1993) cover the principles of seismic isolation and give a comprehensive list of projects where isolation has been implemented. Base isolation and energy dissipation devices have been used for bridge retrofits in New Zealand, the US, Japan, Italy and Canada. In Italy, large elasto-plastic absorbers are used at abutments to provide longitudinal restraint and damping. For relatively short bridges, the Italians have retrofitted movement joints by eliminating them, and the bridge deck slab in the vicinity of the movement joint is replaced with a new slab made continuous over the joint.

Babaei and Hawkins (1991a,b) reviewed seismic retrofit techniques for bridge superstructures. Their report discussed the seismic performance of movement joint retrofit measures, and provides design information and cost estimates for retrofitting with restrainers, seat extenders, and base isolators.

3.2 Beams, Beam-Column Joints, and Anchorage of Longitudinal Reinforcement

If the beams of piers are identified as seismically deficient, they are typically difficult to retrofit because of interferences with the superstructure girders. Usually adding strength to the beam by prestressing and concrete jacketing is easier than adding ductility capacity by improving confinement, anchorage, or shear capacity. Thus the best approach to retrofitting the beams of piers can be to strengthen the beam to force plastic hinging into the columns. The columns must then be checked for ductility capacity, and retrofitted if necessary.

If clearances allow, beam deficiencies can also be remedied by adding a new beam between the columns, below the existing beam. This "link" beam can preclude failure in the original beam or beam-to-column joints as well as increasing the transverse strength and stiffness of the pier. Column shear demands will be increased however, so shear retrofit of columns may be necessary. Also, this retrofit does not mitigate moment demands in the bridge's longitudinal direction.

An outrigger-beam knee joint for the I-980 freeway in Oakland, which was damaged in the Loma Prieta earthquake, has been repaired and retrofitted by Caltrans. The joint was retrofitted by completely removing all the concrete in the joint region (by jackhammer) while leaving the existing reinforcement in place. New interlocking spiral joint reinforcement, at close spacing, was added around the column longitudinal bars. An exterior cage of reinforcing consisting of 10 mm bars at a 115 mm spacing each way (#3 at 4½ inches), tied together through the joint, was added. Concrete was then placed, to the slightly increased dimensions of the new joint (Priestley et al. 1992a). This retrofit detail was tested at the University of California, San Diego, under cyclic in-plane loading and showed greatly improved response compared to the original detail (Ingham et al. 1993). Other types of retrofits for knee joints and external T-joints have been studied at the University of California, Berkeley (Thewalt and Stojadinovic 1993, Moehle and Sawyer 1993).

The beam to column joints of non-outrigger multi-column piers are often deficient because of an inadequate anchorage length of the longitudinal column bars into the joint. Retrofit of such a deficiency is difficult. The addition of a new "link" beam below the existing pier beam (as discussed in paragraph 1 of this section) may be an effective solution in some cases. Prestressing of the joint region longitudinally along the beam, and transversely through the beam, may be an effective retrofit method but testing of the detail has been limited (Priestley 1993).

3.3 Elliptical and Circular Steel Column Jacketing

The typical deficiencies of bridge columns, i.e. inadequate lap splice length, shear strength, anti-bar-buckling reinforcing, and concrete confinement, can all be mitigated by adding an external jacket to the column. Although several jacketing techniques have been studied, not all are fully effective or widely applicable.

The success of a column-jacketing retrofit upgrade depends on the stiffness of the jacket to resist the tendency, caused by concrete cracking and spalling, of the column to expand laterally. If this lateral dilation is effectively restrained by a column jacket then four potential seismic deficiencies can be corrected:

1. Existing longitudinal bar-lap splices become more effective because a transverse clamping force across the splice is created.
2. Column shear strength is increased because the jacket acts in the same way as hoop reinforcing, allowing a truss mechanism for shear resistance to develop. As well, diagonal shear cracks are restrained from opening by the jacket, preventing the degradation of aggregate-interlock shear resistance.
3. Bar buckling is restrained because the concrete cover does not spall.
4. The lateral confining pressure on the concrete increases its ultimate strain.

Most jacketing methods are applicable mainly to circular columns. Rectangular columns are more difficult to retrofit, because providing adequate jacket stiffness to confine the long flat sides of the column is a problem. For columns with high plan aspect ratios, through-bolts may be necessary to hold the long sides of the column jacket together.

Elliptical or circular steel jacketing is the most common column retrofit method, and has been shown to be fully effective in increasing the inelastic displacement capacity and strength of seismically deficient columns.

3.3.1 Retrofit Details

3.3.1.1 Complete jacketing

Circular columns are retrofitted using a complete circular steel jacket. In the typical Caltrans details, the jacket is fabricated in two semi-circular halves from steel plate of 10 mm to 25 mm (3/8 to 1 inch) thickness. The two halves are placed around the column and field-welded together using complete penetration groove welds continuous along the length of the column.

The space between the jacket and the existing column, specified to be 38 mm (1½ inches) minimum, is then injected with grout. For full height jackets a 50 mm to 100 mm (2 to 4 inch) gap is provided between the top of the steel jacket and the beam or superstructure, and between the bottom of the steel jacket and the foundation surface. The gaps at each end of the jacket are necessary to prevent an inadvertent increase in moment capacity caused by the jacket bearing against the foundation or superstructure (Caltrans 1992).

Rectangular columns are retrofitted using elliptical steel jackets, constructed in a similar manner to the circular jackets. Details of the jacketing of a single-column pier with a rectangular column are shown in Figure 3.4.

Elliptical or circular steel jackets are provided over the full height of the column if the retrofit is intended to improve inadequate shear capacity. Jackets may also be provided full-height for aesthetic reasons. If the retrofit is intended to improve inadequate concrete confinement, the jacket may be partial height, covering only the potential plastic-hinge region. For partial height jackets, Caltrans specifies a minimum jacket length of 1.5 times the largest dimension of the original column. (For example, a 1.2 m x 1.8 m rectangular column could be retrofitted with an elliptical steel jacket 2.7 m tall covering the plastic-hinge region.)

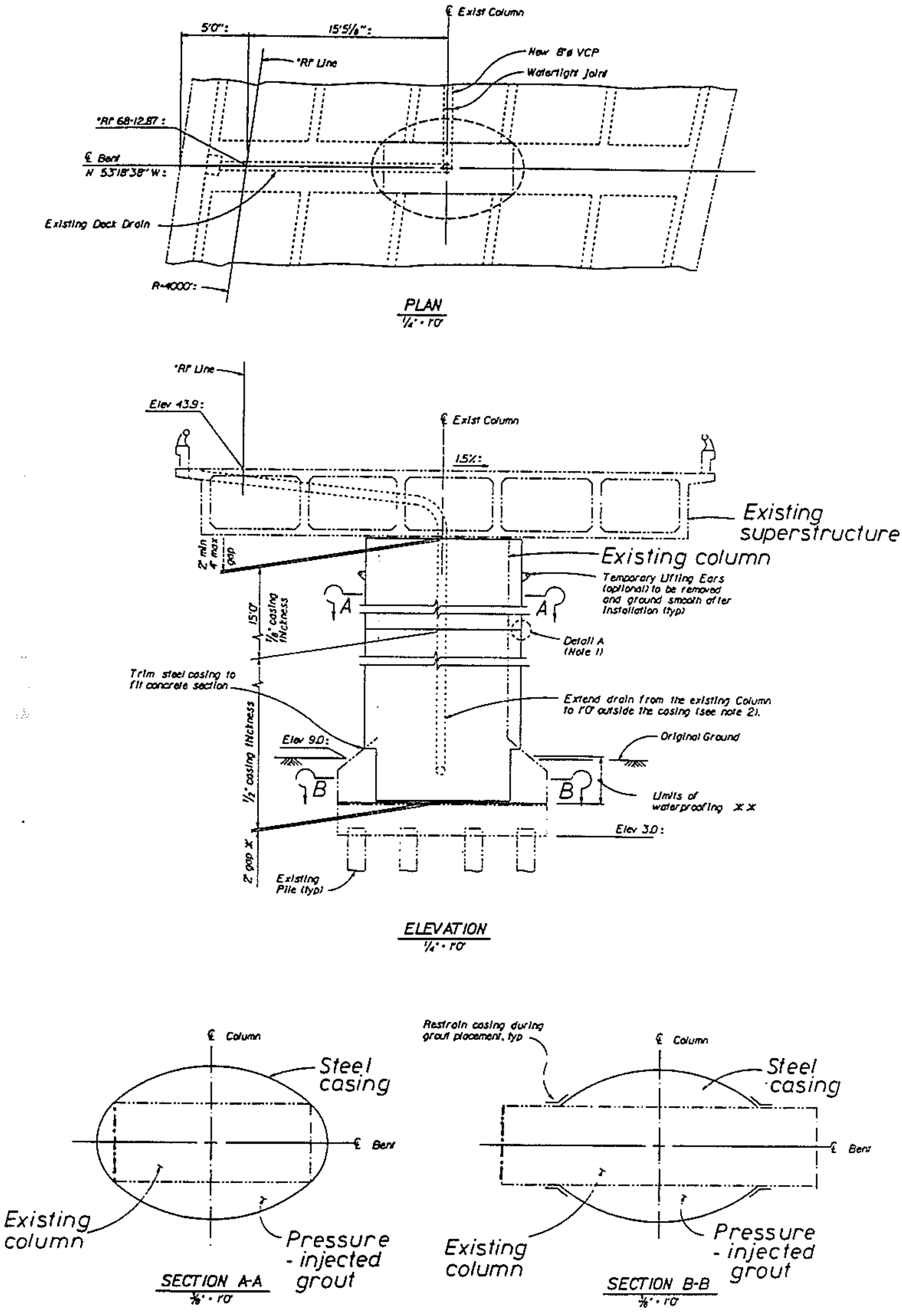
If lap splices are deficient, partial height jackets can also be used. Caltrans specifies the same minimum length for such jackets. In older California bridges, lap splices were commonly placed at the bases of columns to facilitate construction. Since these locations on bridges are typically potential plastic-hinge regions, jacketing retrofit for lap splices can be the same as that for inadequate confinement.

3.3.1.2 Partial-confinement jacketing

For many bridge retrofit designs, it is desirable to improve the inelastic displacement capacity of columns without increasing column moment capacity. If a column has deficient lap splices, a "partial confinement" retrofit may achieve this goal. The partial confinement retrofit used by Caltrans (called a "Class P" retrofit) has a 13 mm (½ inch) thickness of compressible plastic (polyethylene) placed around the column before the installation and grouting of the elliptical or circular steel jacket. The presence of this compressible layer allows cracking and dilation of the concrete at the column base, and consequently allows some slip of the column bar-lap splices.

However, the jacket prevents major spalling and deterioration of the concrete and buckling of the column bars. Therefore, the column can maintain its vertical-load-carrying capacity even when subjected to large inelastic displacement demands.

Figure 3.4 Elliptical steel jacketing retrofit of a single-column pier having a rectangular column section (Zelinski pers.comm. 1993).



In Caltrans bridge retrofits, a partial confinement steel jacket may be placed at the bottom plastic-hinge region of a column with inadequate lap splices. The retrofit allows enough column rotation for the structure to meet its expected ductility demands, but, by allowing lap-splice slip, the retrofit limits the level of force which is delivered to the foundation. Thus the need to retrofit the foundation is avoided. With this approach, of course, adequate lateral capacity must still be provided somewhere in the structure.

For many structures the Caltrans approach is to fully retrofit one bent (i.e. pier) per frame. That is, if a bridge structure between adjacent movement joints (i.e. a frame) is supported on three piers, the columns and foundations of one pier are fully strengthened for the design lateral forces. The base regions of the columns in the other two piers are retrofitted to maintain their vertical-load-carrying capacities at the expected inelastic displacements, but are assumed for design purposes to have zero moment capacity at the base (Caltrans 1992).

Although a pin connection may be used to model the base of a column with a partial-confinement jacket, the foundation capacities and column-shear strength should be based on the peak moment-capacity of the column base (e.g. that at μ=1.5 in Figure 3.6(c)).

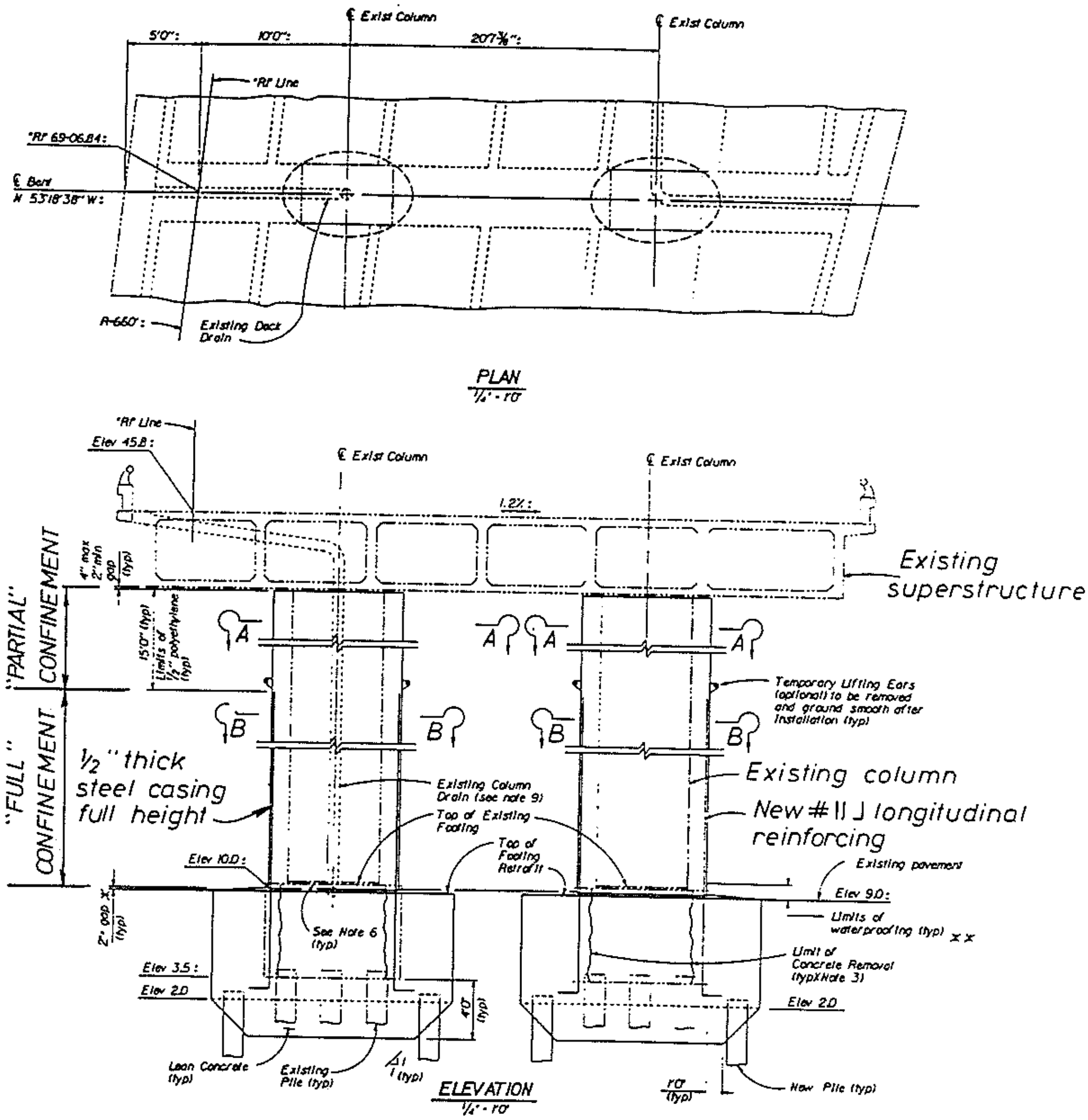
Figure 3.5 shows the column and foundation retrofit and strengthening design of a two-column pier with rectangular columns. The columns are retrofitted with full-height elliptical steel jackets. For the top 4.6 m (15 ft) of the columns, a partial confinement retrofit, with a 13 mm thick polyethylene layer, is used. This unusual detail is intended to allow greater strains in the continuous longitudinal bars in this region of the bridge and thus limit the moment demand transferred into the beam, which is not retrofitted (Zelinski pers.comm. 1994). The bottom to mid-height portion of the columns receive a full-confinement retrofit, plus an increase in the moment capacity at the bottom of the column with the addition of 25 reinforcing bars, 36 mm diameter (#11), placed around the rectangular column inside the elliptical steel jacket. In conjunction with this column strengthening, the foundations are extensively retrofitted, new piles and pile caps are used, and temporary support of the superstructure is required (Figure 3.5).

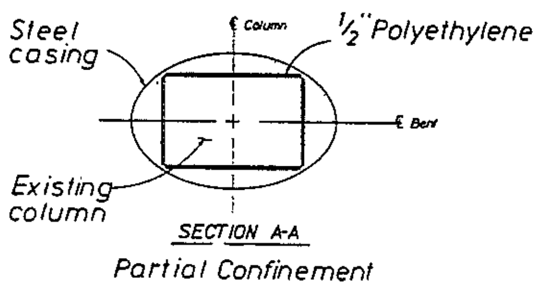
3.3.2 Research Results

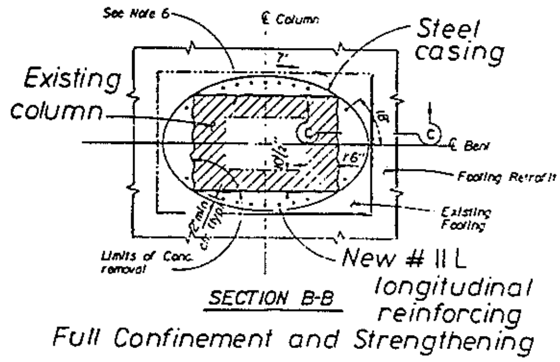
3.3.2.1 Performance of complete jacketing

Research on elliptical and circular steel jacketing of circular bridge columns has convincingly shown the effectiveness of such jackets. Figure 3.6(a) shows the lateral-load versus displacement-hysteresis loops for a circular column with 20-bar-diameter lap splices in the plastic-hinge zone. The strength of the column degrades rapidly after the first cycle to displacement ductility, μ, of 1.5, and the hysteresis loops are pinched showing poor energy absorption capacity. An identical column with a grouted circular steel jacket over the lap-splice region responds as shown in Figure 3.6(b).

Figure 3.5 Column and foundation retrofit and strengthening, using elliptical steel jacketing, for a two-column pier having rectangular columns (Zelinski pers.comm. 1993).







The hysteretic-load versus displacement-response is stable up to μ=7 with good energy dissipation capacity and no strength degradation until μ=8 when column longitudinal reinforcement is fractured.

3.3.2.2 Performance of partial-confinement jackets

Figure 3.6(c) shows the hysteresis loops for a third circular column which was retrofitted using a partial confinement steel jacket. As with the original column (Figure 3.6(a)), bond failure occurred at the lap splice at μ=1.5. However, strength degradation was less rapid and it was possible to displace the column to μ=7 without losing the vertical load carrying capacity (Yuk Hon Chai et al. 1991, Priestley et al. 1992a). Thus it appears that the concept of a partial confinement retrofit is valid. However, to conduct further research on partial confinement, retrofit designs may be prudent for the following reasons:

1. Partial confinement jacketing of rectangular columns is now common in California, but the detail has only been tested on circular, not on rectangular, columns.

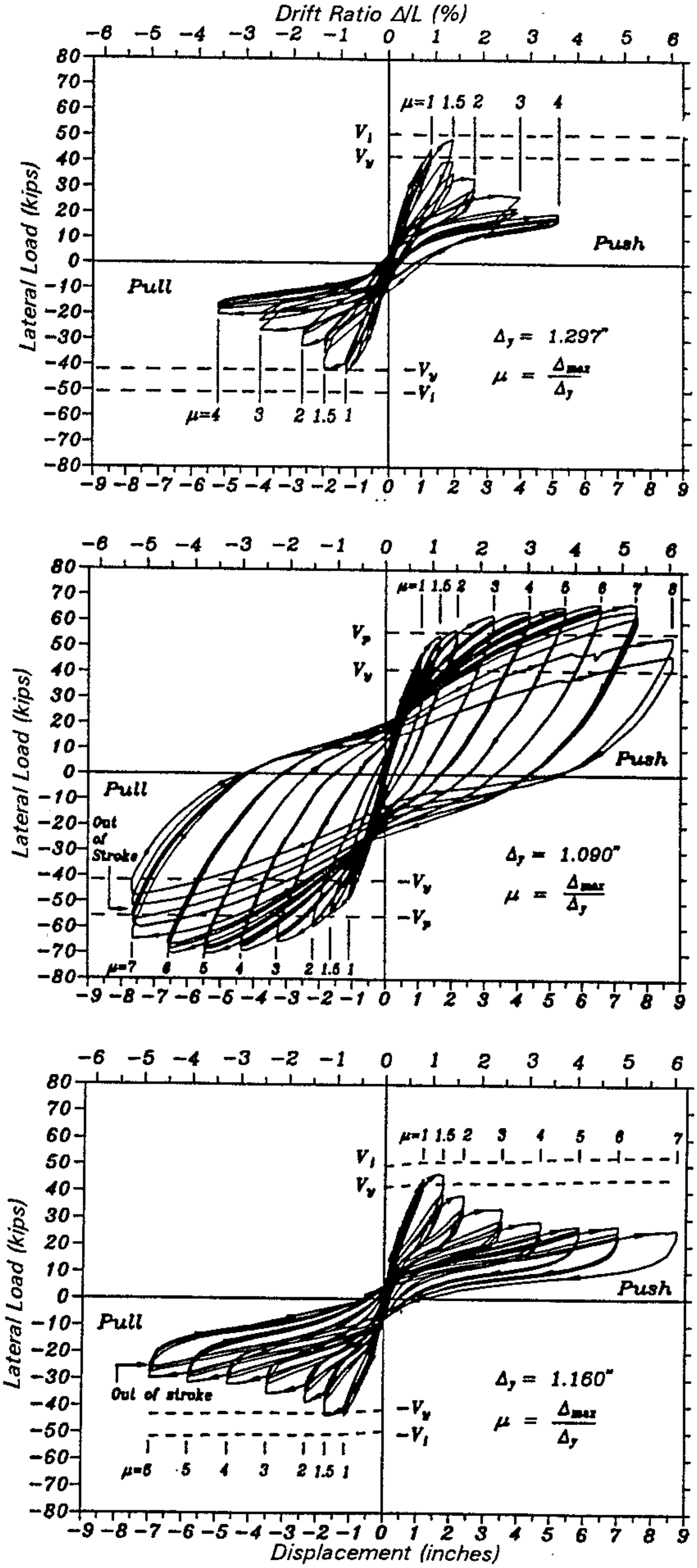
2. The hysteresis loops for the partial confinement retrofit of circular columns (Figure 3.6(c)) show a reasonable ductility capacity, but also show strength degradation and pinching. This degradation and pinching may or may not be a problem for the bridge structure, depending on the overall retrofit design, the actual hysteretic response characteristics, and on the earthquake demand.

   Inelastic, dynamic, time–history analyses of structures with partial-confinement jacketed columns could help verify the adequacy of such retrofits and to evaluate the overall retrofit strategies which use partial confinement jacketing.

   Consideration of the residual moment capacity of a column with a partial-confinement retrofit may lead to more accurate seismic evaluations and more efficient retrofit designs than the currently used assumption of a pin connection.

3. A thinner or less compressible layer of polyethylene could be used to determine if strength degradation can be reduced, without increasing peak strength. This could further improve the seismic performance of the retrofitted bridge, without increasing peak shear or foundation demands.

4. Columns with deficient shear capacity are sometimes retrofitted using a full height steel jacket which provides full confinement over most of the column height, and using only partial confinement at the lap-splice region where a layer of polyethylene is used around the column. Figure 3.8 is an example of this type of jacketing detail. Such retrofit methods for shear-deficient columns have not been laboratory tested for either circular or rectangular columns. It may be that the partial confinement detail is not as effective in improving shear capacity as a jacket providing full confinement over the full height of the column.



(a) As-built circular column

(b) Full confinement circular steel jacket retrofit

(c) Partial confinement circular steel jacket retrofit

Figure 3.6 Steel jacket retrofit test results for circular columns with 20-bar-diameter lap splices (Yuk Hon Chai et al. 1991, Priestley et al. 1992a).

(kips = kilopounds)

3.3.2.3 Performance of rectangular columns with lap splices

Figure 3.7(a) shows the lateral-load versus displacement-hysteresis loops for a rectangular column with inadequate lap splices. Similar to the circular column, bond failure occurred in the lap splices at μ=1, after which the strength degraded rapidly. The hysteresis loops were pinched and the ideal moment capacity was not reached.

Figure 3.7(b) shows the response of an identical column retrofitted using an elliptical steel jacket. The hysteresis loops are stable up to μ=7, when lap-splice failure eventually occurred. The energy dissipation (equal to the area inside the hysteresis loops) is good up until the end of the test at μ=9.

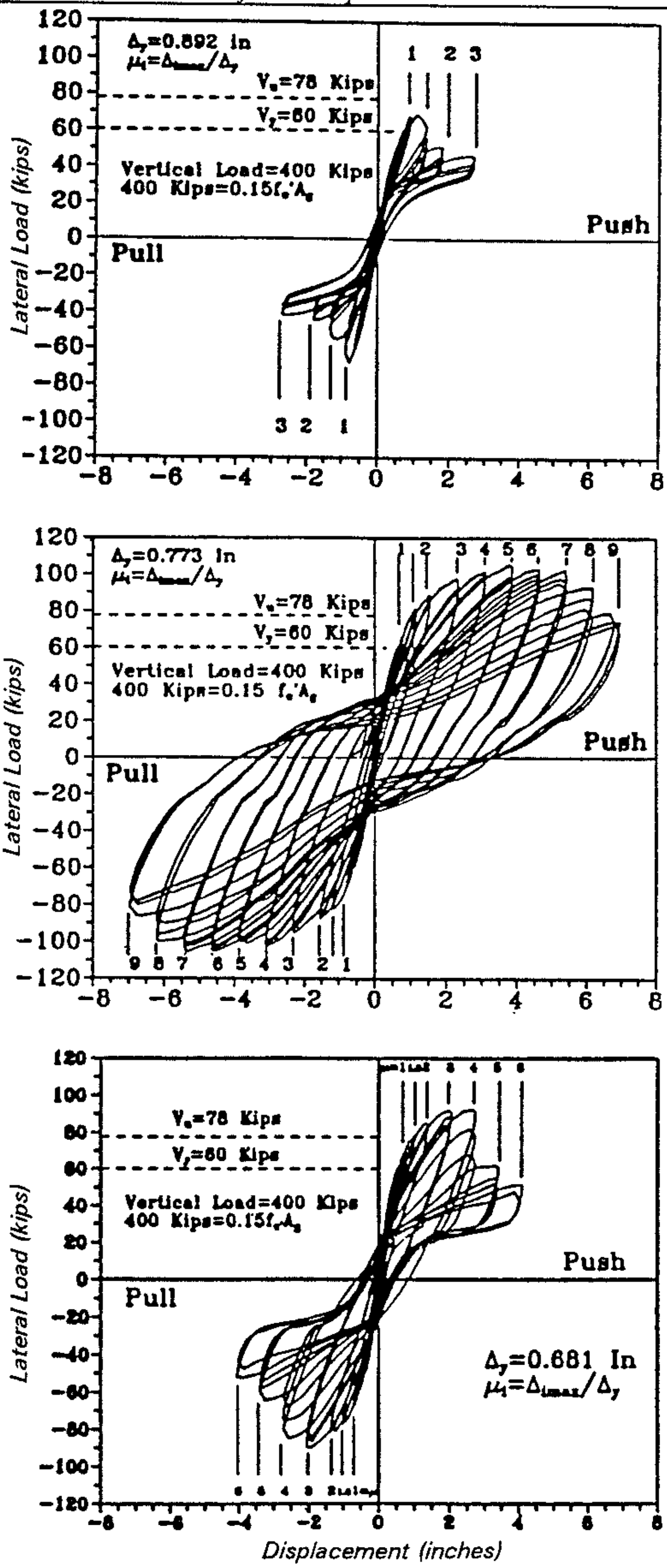
Not all jacketing methods are so successful. Figure 3.7(c) shows the lateral-load versus displacement-response of a column retrofitted with a stiffened steel-plate rectangular jacket. As shown by the limited ductility capacity and poor energy dissipation, this retrofit method is only marginally effective (Priestley et al. 1992a). Consequently, this method was not used for permanent retrofit work in California.

3.3.2.4 Retrofit of shear strength, confinement, and bar-buckling deficiencies

Figure 3.8 shows the effectiveness of elliptical steel jacketing for rectangular columns with deficient shear strength. The hysteresis loops in Figure 3.8(a) are for a rectangular column that suffered a brittle shear failure at displacement ductility, μ, of 1.5. An identical column which was retrofitted with an elliptical steel jacket showed excellent seismic performance with hysteresis loops stable up to μ=8 (Figure 3.8(b)). Similar results have been obtained for circular columns deficient in shear strength (Priestley et al. 1992a).

The improvement in seismic response provided by jacketing is less dramatic for columns whose only deficiency is insufficient transverse reinforcement for flexural confinement and bar-buckling restraint. Figure 3.9 shows the original and retrofit hysteresis loops for circular columns without lap splices. The original column showed good hysteretic response up to μ=5 when the longitudinal bars in the compression zone of the plastic hinge buckled. (Note that with higher axial loads the original column performance would not be as good.) The column retrofitted with a circular jacket (Figure 3.9(b)) showed improved behaviour with good hysteretic response up to μ=8.

Research on steel jacketing has also been carried out in Japan with emphasis on retrofitting regions where reinforcing is prematurely terminated. For the Japanese jacketing designs, epoxy is sometimes used between the jacket and column instead of cementitious grout (Priestley et al. 1992a).



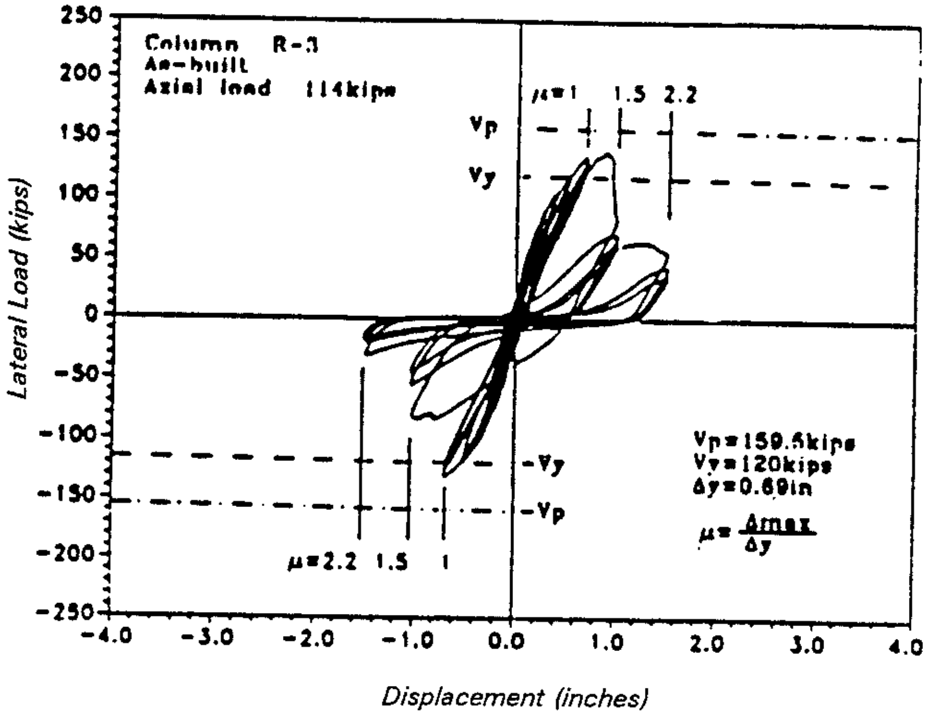
(a) As-built rectangular column

(b) Elliptical steel jacket retrofit

(c) Stiffened steel-plate rectangular jacket retrofit

Figure 3.7 Steel jacket retrofit test results for rectangular columns with 20-bar-diameter lap splices (Priestley et al. 1992a).

(a) As-built rectangular column



(b) Elliptical steel jacket retrofit

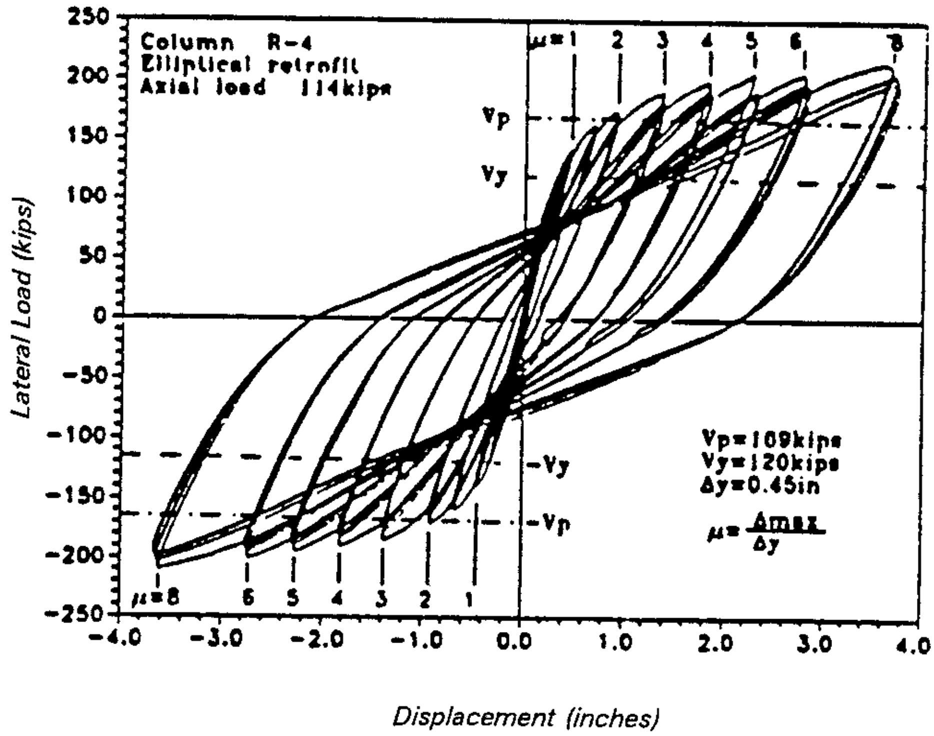
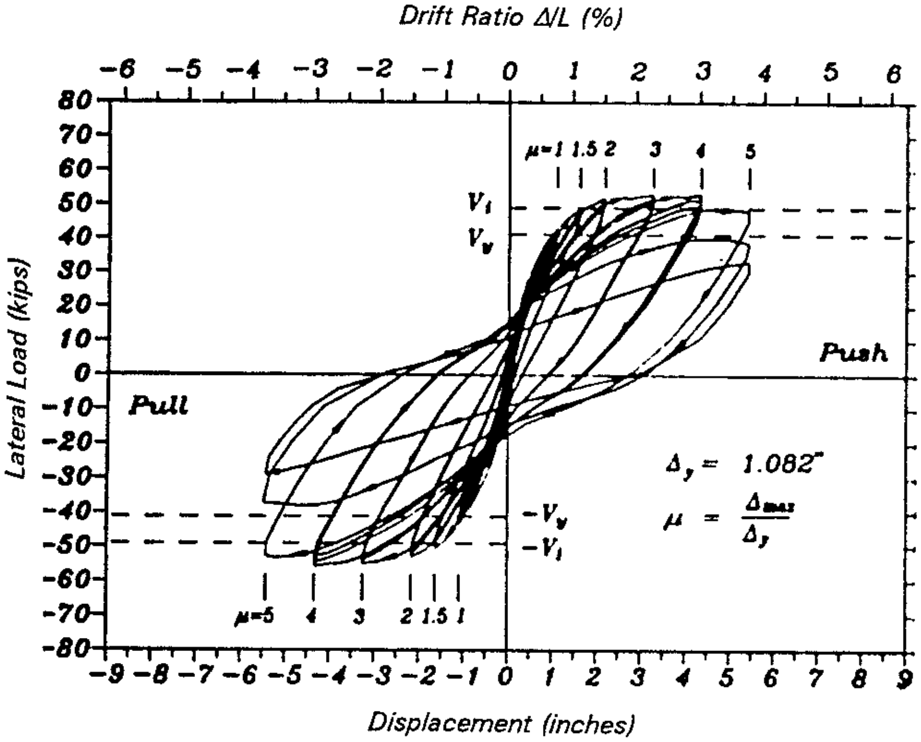


Figure 3.8 Steel jacket retrofit test results for rectangular columns with insufficient shear capacity (Priestley et al. 1992a).

(a) As-built circular column



(b) Circular steel jacket retrofit

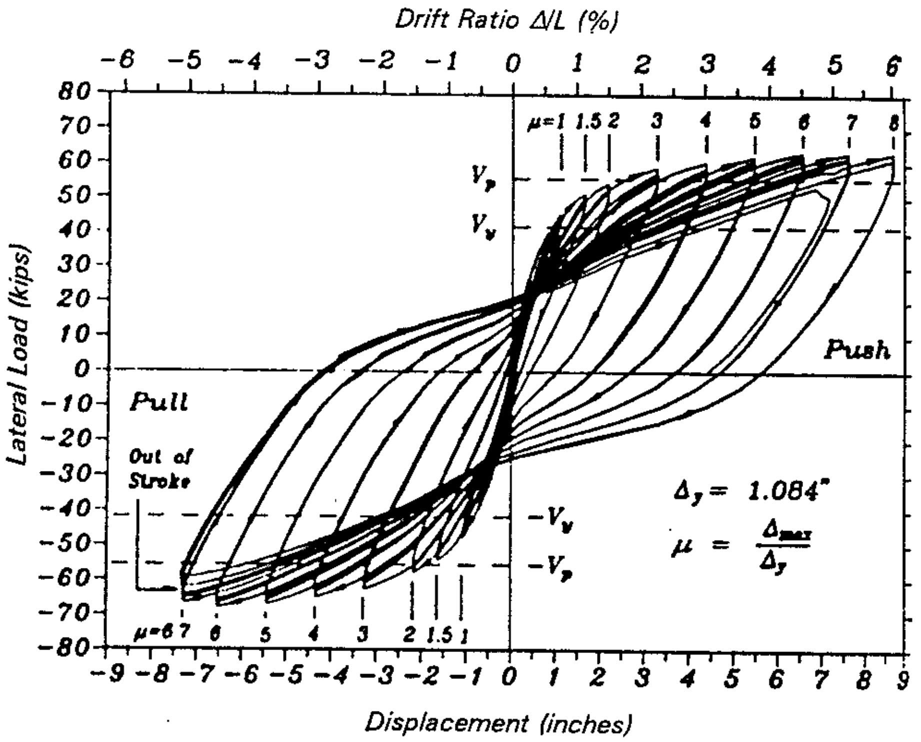


Figure 3.9 Steel jacket retrofit test results for circular column with continuous longitudinal bars, but with insufficient ties for flexural confinement and bar-buckling restraint (Yuk Hon Chai et al. 1991).

3.3.3 Dimensions of Steel Jackets

Caltrans (1992) has developed tables of recommended steel jacket dimensions and thicknesses for different column sizes. The tables are based on providing a specified amount of confining pressure at a specified radial dilation strain. This design approach was developed at the University of California, San Diego (Priestley et al. 1992a). For columns with lap splices, the critical radial dilation strain is taken as 0.001. For concrete confinement where no lap splices are present a larger radial dilation strain, 0.004, is permissible. The contribution of the steel jacket to shear strength is considered in the same manner as for hoop or spiral reinforcing, based on a truss analogy with diagonal compression struts between inclined cracks in the concrete section. Specific design equations for elliptical and circular steel jackets are given by Priestley et al. (1992a, 1994).

For a given rectangular column, different elliptical jacket dimensions can be used as shown in Figure 3.10. More oblong-shaped elliptical jackets (Figure 3.10(a)) will provide better shear strength and confinement for strong axis behaviour, while more circular-shaped jackets (Figure 3.10(b)) will provide better shear strength and confinement for weak axis behaviour. Caltrans (1992) design criteria define the overall dimensions of the jacket so that the aspect ratio A/B of the ellipse is equal to the aspect ratio h/b of the column. The jacket shapes and dimensions are different from those of a true ellipse, because their elliptical shape is approximated as four circular segments. In practice the steel jackets are typically fabricated with just two different radii, r_1 for the long sides of the column and r_2 for the short sides of the column, as shown in Figure 3.10. The points of tangency of the two radii are usually taken at the corners of the rectangular column.

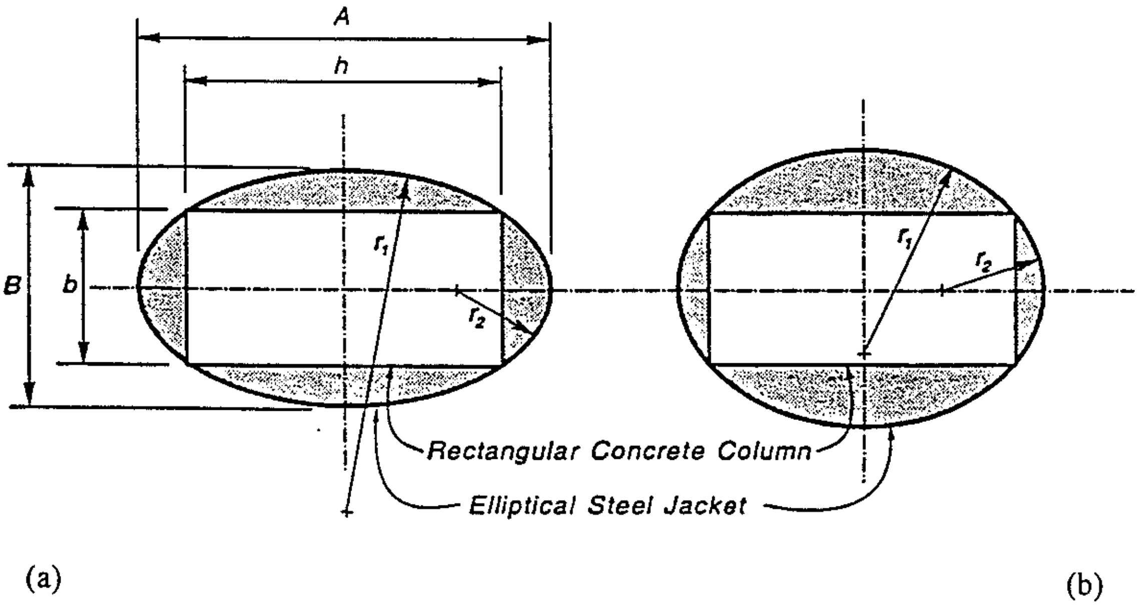


Figure 3.10 Geometry of elliptical column jacketing.

3.4 Other Column Jacketing Methods

Besides elliptical steel jacketing, other materials and methods can be used to add confining jackets to bridge columns. Most of these other methods are applicable mainly to circular columns. Although some methods may be applied to rectangular columns, the designer must ensure that the jacket provides sufficient confining stiffness and strength on all sides of the column where it is needed. In general, jackets of rectangular geometry, even if they are stiffened, are not as effective as elliptical jackets or jackets with through bolts. It is important that unproven techniques for column jacketing are laboratory tested. Several retrofit methods designed on engineering judgement have been shown by laboratory tests not to be effective.

3.4.1 Fibreglass/Epoxy Column Jackets

Fibreglass/epoxy column jackets may be preferable for circular columns near salt-water environments, where steel would be likely to corrode. In some cases the final column diameters for fibreglass/epoxy jackets are 50 mm to 100 mm (2 to 4 inches) less than for steel jackets, so the fibreglass/epoxy retrofit may also be preferred in situations of tight clearances. Approval of fibreglass/epoxy jackets by Caltrans has been delayed, however, by questions of durability (Zelinski pers.comm. 1994).

Research on fibreglass/epoxy retrofitting has been carried out at the University of California, San Diego (Priestley et al. 1992a). Their tests have shown that columns with inadequate lap-splice lengths, jacketed with fibreglass and epoxy and pressure grouted to 1700 kPa (250 psi), show excellent seismic response, similar to that of steel-jacketed columns.

Caltrans specifies three levels of confinement with fibreglass/epoxy jacketing:

- active confinement at 1700 kPa (250 psi),
- active confinement at 690 kPa (100 psi), and
- passive confinement.

The active confinement is accomplished by installing an elastomeric bladder around the column before wrapping with fibreglass and epoxy. After wrapping, cement grout is pumped into the bladder up to the specified pressure. For passive confinement, the fibreglass/epoxy is wrapped directly around the column without grout. For both active and passive jacketing, multiple layers of the fibreglass/epoxy are used to achieve the desired total thickness.

Active confinement is used only at potential plastic hinge regions in columns. The lower confining pressure, 690 MPa, is used where the required displacement ductility capacities are less than 4.5 or where there are no lap splices. For lap-splice regions, and for higher ductility demands, a confinement pressure of 1700 MPa is used. The passive fibreglass/epoxy jacketing is used outside the plastic-hinge regions where column shear strength is inadequate.

For columns where shear strength is the only seismic deficiency, a full-height passive jacket is used (Caltrans 1992). The passive jacket can also be used for plastic-hinge zones without lap splices. The thicker jacket required for active confinement may not be cost-competitive with steel jackets (Zelinski pers.comm. 1994).

Passive fibreglass/epoxy jacketing is also used at lap splice locations where a partial confinement retrofit is desired. As discussed in Section 3.3 on steel jacketing, the intent of partial confinement is to allow some lap-splice slip (thus limiting force demands in the foundations) but to prevent serious degradation of the plastic-hinge region and the consequent loss of the column's vertical load carrying capacity.

Caltrans had developed a standard specification for the fibreglass/epoxy jacketing materials, i.e. glass and polyaramid fibres and high-elongation epoxy, and has tabulated the required thicknesses of the jacketing. For a 1.8 m (6 ft) diameter column, for example, the total fibreglass/epoxy jacket thickness was specified as 6.1 mm (0.24 inches) for passive confinement, 13.8 mm (0.54 inches) for low pressure active confinement and 17.5 mm (0.69 inches) for the higher pressure active confinement (Caltrans 1992). However, after two trial installations, use of the fibreglass/epoxy jacketing is on hold until issues of durability are resolved (Zelinski pers.comm. 1994). Priestley et al. (1992a) present the basis for some of the design criteria used by Caltrans.

3.4.2 Other Methods

Active confinement of circular columns can also be provided using prestressed wires wrapped around the column. The effectiveness of such jacketing has been demonstrated in tests of the University of California, San Diego, and general design criteria have been developed. However, cost-effective installation methods have not yet been developed (Zelinski 1994). Carbon fibre wrapping can also be used to jacket both circular and rectangular columns. Tests conducted in Japan indicate that the jacketing improves the ductility and strength of columns which have prematurely terminated reinforcement and are prone to shear failure (Priestley et al. 1992a).

Column confinement can also be added to bridge columns by removing the cover concrete, adding additional reinforcing steel ties, and then placing new cover concrete. This retrofit method was investigated by Dekker and Park (1992) on a full scale specimen representative of a 1936-designed New Zealand bridge. The test is discussed in Chapter 2 of Maffei (1997). Reinforced concrete jackets with substantial transverse reinforcing can be used to retrofit bridge columns, although the approach is generally more expensive than using a steel or fibreglass/epoxy jacket.

3.5 Pier Walls

Bridge pier walls have been retrofitted in California using jacketing methods which are similar to those used for columns. In fact, for bridge structures the distinction between a pier wall and a column is not always clear because rectangular "columns" may have plan aspect ratios of 3 or more. As mentioned in Section 3.4, for such columns the jacket may need through-bolts to provide the necessary confinement to the long sides of the column. The situation is the same with pier walls.

Steel jackets for pier walls have been designed by Caltrans with flat steel plates along the long sides of the wall, joined around the ends of the wall with flat or circular steel-plate segments. The long sides of the jacket are tied together with through-bolts.

Typically these are 32 mm (1¼ inch) diameter high-strength bolts, in holes drilled through the existing wall on a grid spacing of 1.2 to 2.4 m (4 to 8 ft) in each direction. Cementitious grout is injected between the steel jacket and the concrete column.

As with steel column jackets, the pier wall retrofit can be full or partial height, and full or partial confinement. Full height jackets are used if the strong-direction shear strength of the wall is deficient. The partial confinement detail at lap splices uses a 13 mm (½ inch) layer of compressible polyethylene wrapped around the wall before jacketing. For wide columns with flared bases, which are similar to pier walls, Caltrans has used through-bolted plates in an attempt to confine lap splices. This detail is shown in Figure 3.11.

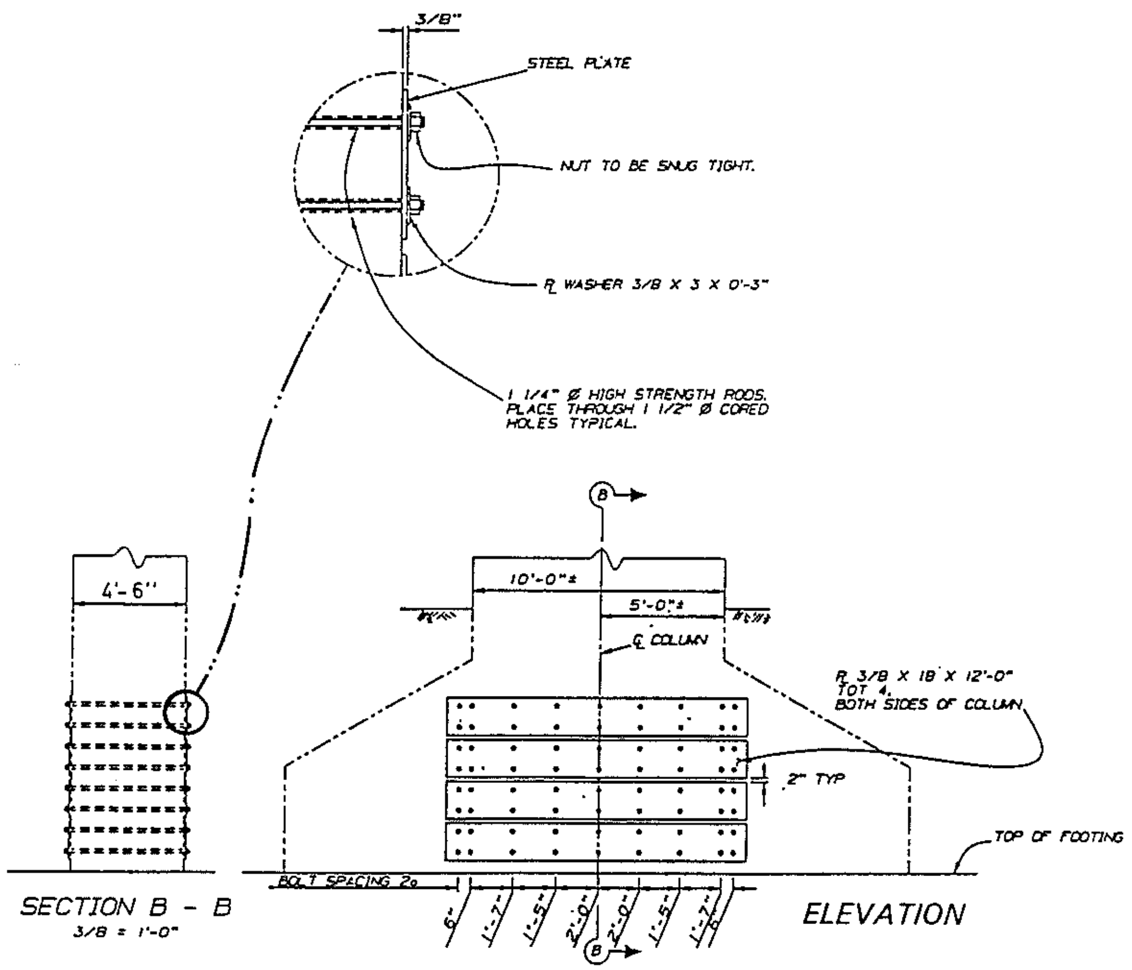


Figure 3.11 Column or pier wall retrofit using confinement through-bolted plates at the lap splices (Zelinski pers.comm. 1993).

Research on retrofit methods for pier walls has been carried out at the University of California, Irvine (Haroun et al. 1993). Half-scale wall specimens were retrofitted with steel jackets, or plates, of various heights and with various arrangements of through-bolts. The wall specimens were then tested in the weak direction. The specimens had either 16-bar-diameter-long or 28-bar-diameter-long lap splices in the vertical reinforcing at the base of the wall. Specific design recommendations based on the research have yet to be developed. Continued testing may be necessary to:

- assess the effectiveness of the through bolts in providing weak direction confinement,
- develop retrofit methods for strong direction shear strength deficiencies, and
- investigate the performance of partial confinement retrofits.

3.6 Foundations

For bridge retrofits in California, footings are often substantially strengthened using added piles and an expanded pile cap. Figure 3.12 shows the details of the strengthening of the pile foundation for the two-column pier retrofit of Figure 3.5. The existing foundation for each column consisted of six steel H-piles and a 2.7 m x 1.8 m (9 ft x 6 ft) plan pile cap. The strengthened foundation contains twenty-four added steel pipe piles with a pile cap expanded to 4.9 m x 5.5 m (16 ft x 18 ft). For this strengthening, the superstructure is temporarily supported on shoring, while the existing pile cap is undermined so that new bottom mat reinforcing can be placed underneath it. A new top mat of reinforcing is added above the existing pile cap to provide the negative flexural capacity needed when piles are in tension caused by overturning. The top mat of reinforcement can also improve the anchorage capacity of the column bars into the foundation, because the reinforcement restrains flexural-tension cracks from opening at the top of the foundation, near the anchored bars.

Such methods as shown in Figure 3.12 are effective in strengthening the bridge foundation. The problem is that such foundation retrofits are expensive, several times more expensive than column-jacket retrofitting. Paradoxically, while a large portion of retrofit costs are spent on foundations, few foundation failures have been noted in past earthquakes. However, this observation is likely to change as more of the once fragile columns are strengthened. Tests at the University of California, San Diego, have highlighted the lack of toughness in foundations once columns are retrofitted (Zelinski pers.comm. 1994).

A less expensive foundation strengthening scheme, using soil tension anchors, is also used by Caltrans. This retrofit, shown in Figure 3.13, is most efficient when the compressive capacity of the soil is not being fully utilised. Otherwise the full tension cannot be applied to the soil anchors without overloading the soil. If the anchors are not tensioned the rotational stiffness of the foundation is greatly reduced. As shown in Figure 3.13, the soil anchors are often placed between the existing piles, and a reinforcing concrete topping is added to the existing footing or pile cap.

More important than strengthening foundations to increase their capacity in the soil may be the retrofit of foundation joint shear and anchorage deficiencies. Yan Xiao et al. (1993) tested a footing retrofit with an added reinforced concrete topping which had two layers of reinforcing and was attached to the existing footing with drilled dowels.

Figure 3.12 Details of foundation strengthening using new pipe piles and a new pile cap (Zelinski pers.comm. 1993).

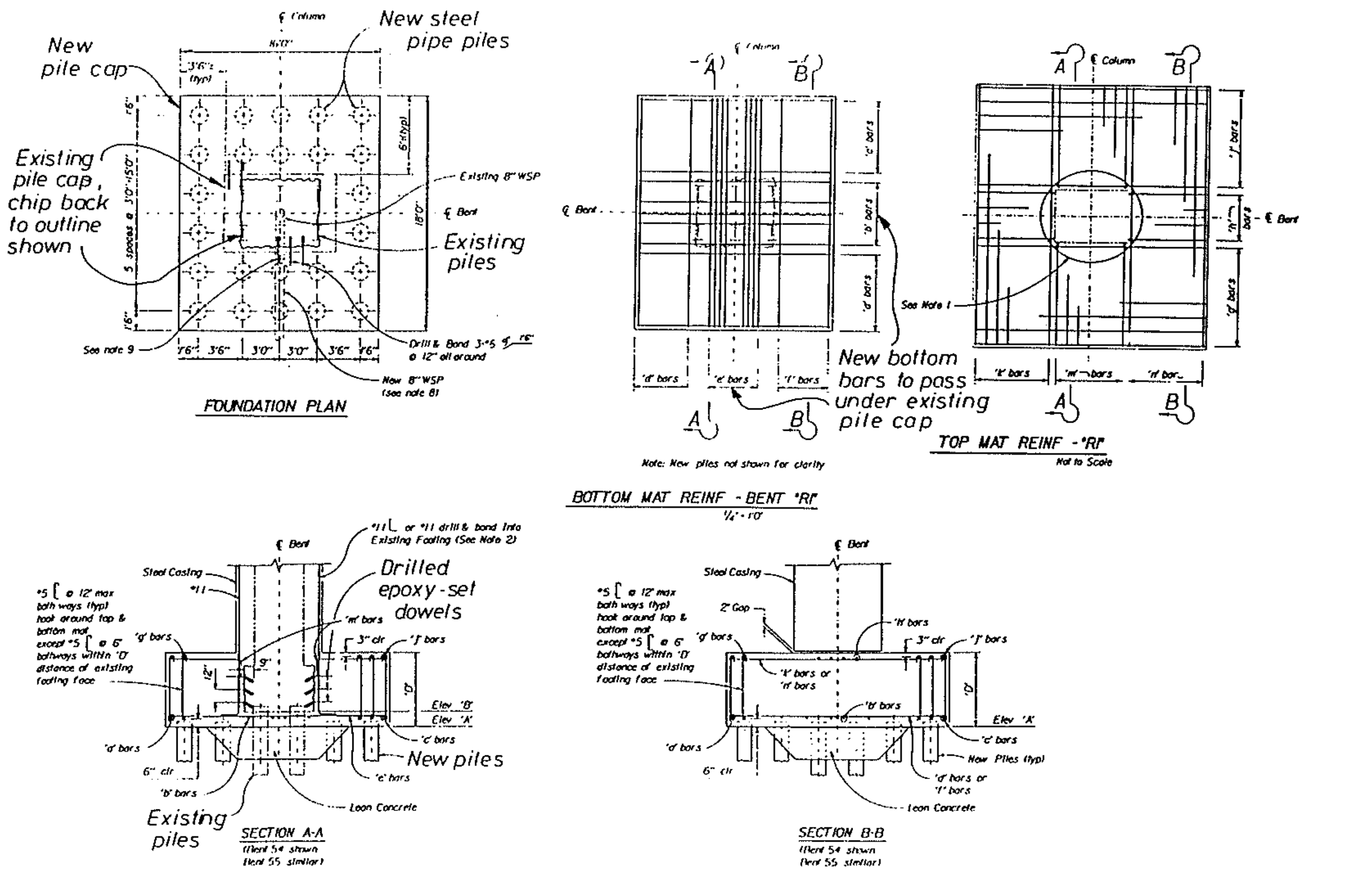
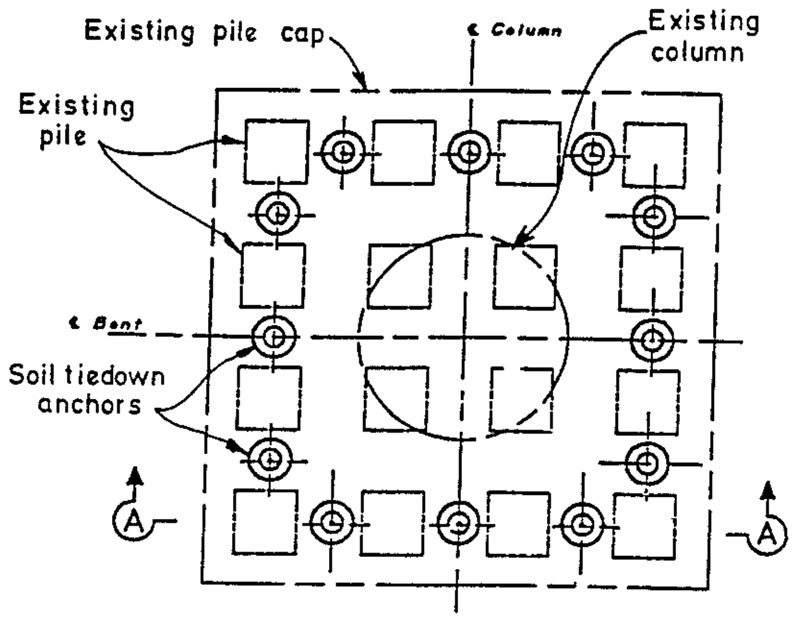
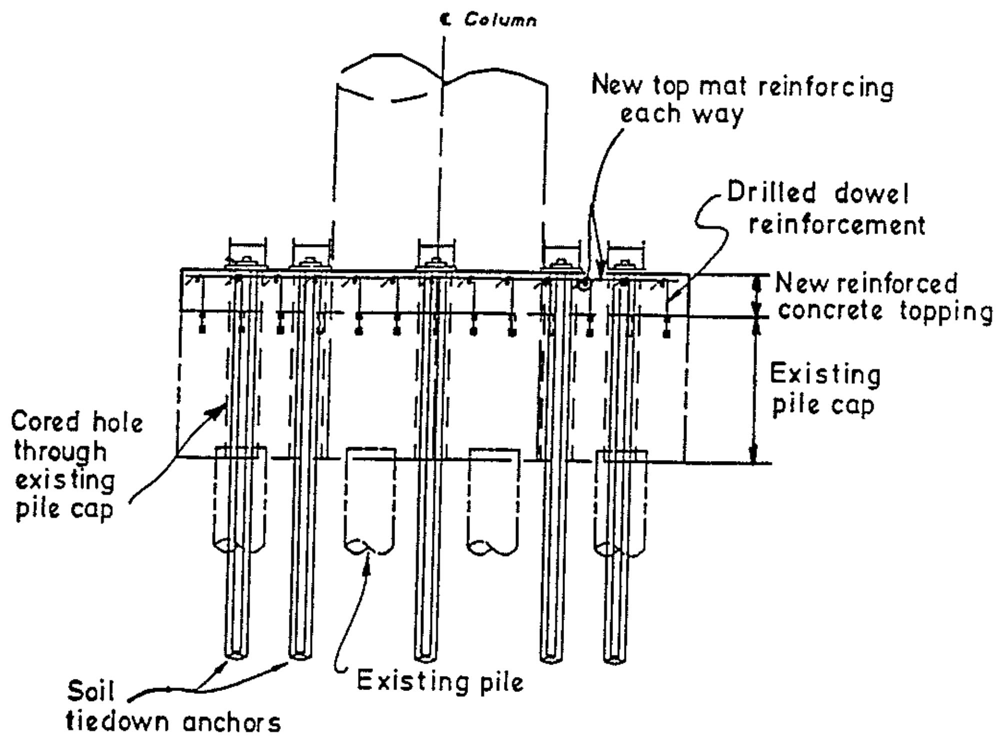


Figure 3.13 Foundation strengthening using soil tension anchors (Caltrans 1992).



FOOTING PLAN



SECTION A - A

The as-built specimen had failed in shear at the column-footing joint at a load of less than 60% of the column capacity. The retrofit footing of a second test specimen allowed the column to reach its flexural strength, and the ultimate displacement of the column was 1.5 times greater than that for the as-built specimen. The final failure was caused by a degradation of the footing overlay at a ductility of μ=4. Thus the retrofit appears to be effective to limited ductility levels.

Adding external prestressing to existing footings and pile caps may be an effective retrofit technique for column-to-foundation joint shear and anchorage deficiencies. However, the development of feasible design details and experimental research is needed to validate this retrofit concept. Research is also needed on establishing requirements for tying the top reinforcing mat of a foundation down to the bottom reinforcing mat (Zelinski pers.comm. 1994). Priestley et al. (1992a) provide some general design criteria for bridge foundation retrofits.

3.7 Abutments

Caltrans has occasionally used a seismic anchor slab to strengthen and stiffen the abutments of existing bridges. As shown in Figure 3.14, the waffle-shaped slab connects the existing abutment to new piles or drilled piers. The goal of this type of retrofit is to *attract larger seismic forces to the abutments and [thereby] reduce the amount of column, footing, or other retrofit which may be required in adjacent bents (piers). The seismic anchor slab is more effective on shorter bridges with no hinges. However, it has been proposed for use on larger structures with expansion hinges* (Caltrans 1992). In New Zealand, below-grade friction slabs have been used to restrain abutments.

If the abutments of a straight bridge are being retrofitted primarily for longitudinal direction earthquakes, then soil tieback anchors can be used instead of an anchor slab. For transverse direction earthquakes, large anchor piles can be installed and monolithically tied to the existing abutments. Figure 3.15 shows the schematic seismic retrofit of a curved bridge using tension tie backs for north-south direction earthquakes, and cast-in-drilled-hole anchor piles for restraint in the east-west direction (Caltrans 1992). Similar retrofits using anchor piles were used on the Kunou viaduct for the Tomei expressway in Japan (Priestley 1992a).

3.8 Other Retrofit Techniques

Multi-column piers can sometimes be strengthened with the addition of an infill structural wall (or "shear" wall) between two of the columns. This method was used for the retrofit of two-column piers on the Nishiokazu viaduct on the Tomei expressway in Japan (Priestley 1992a). The designer of such a retrofit must consider the effects of the retrofit on increasing the seismic forces in the foundations of the pier.

Figure 3.14 Abutment retrofit using new anchor slab and piles (Caltrans 1992).

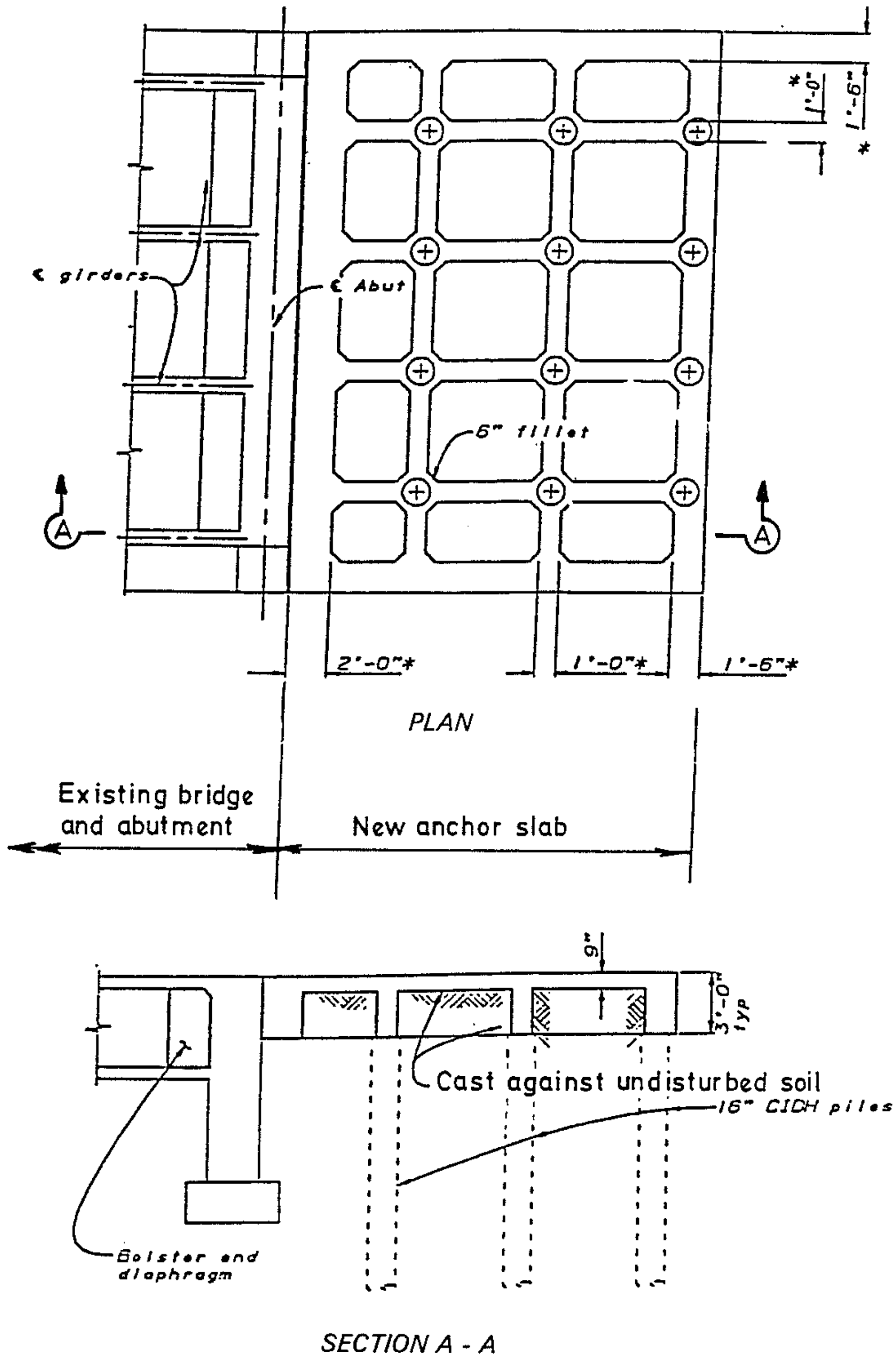
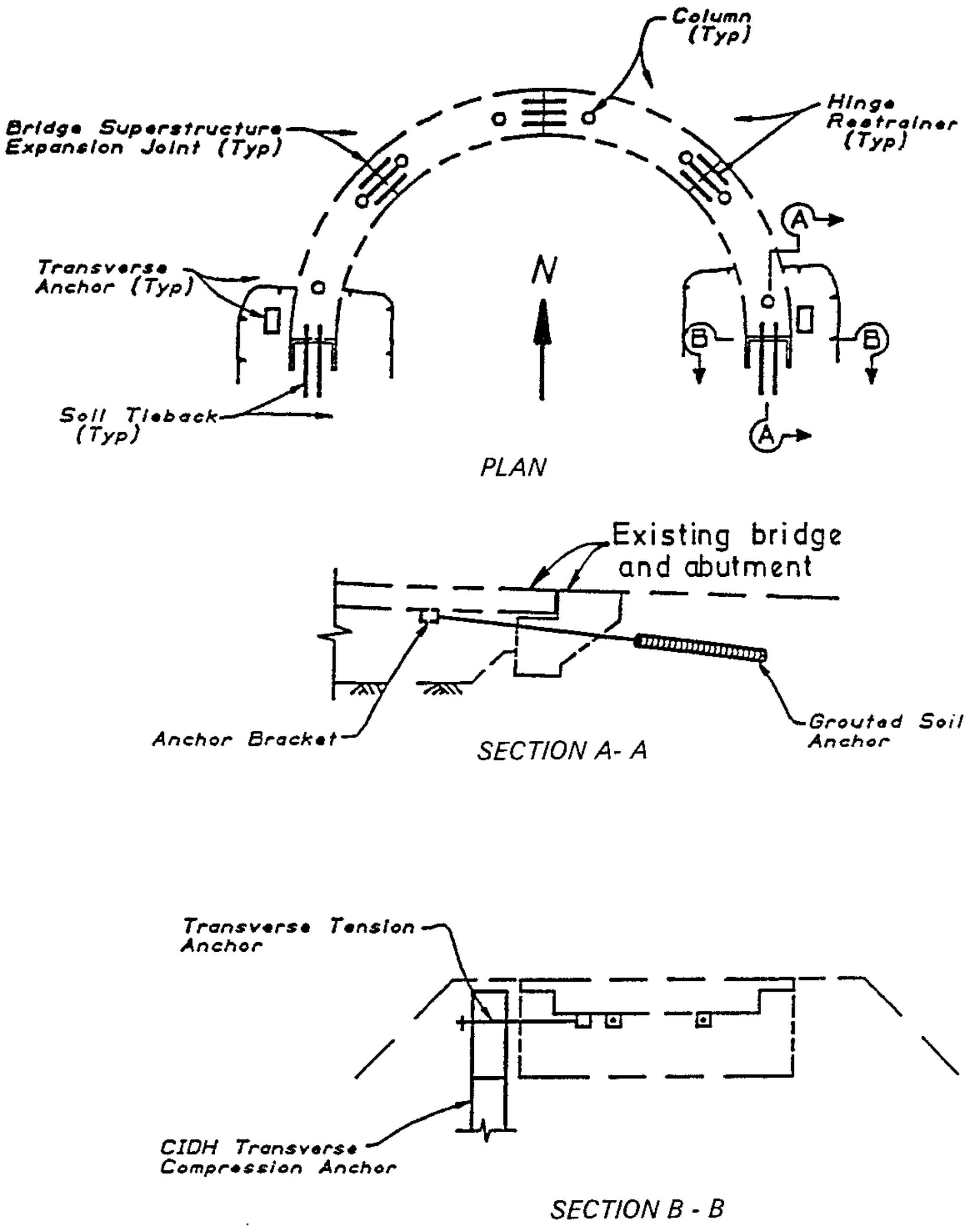


Figure 3.15 Retrofit of a curved bridge using movement joint restrainers, abutment tension tiebacks, and concrete piles for the transverse anchoring of the abutment (Caltrans 1992).



For the Asada ramp of the Metropolitan expressway in Kawasaki, Japan, columns and cantilever beams were strengthened using added external post-tensioning (Priestley 1992a). Again, for such a retrofit the designer must be careful to determine whether the potential seismic failure has been redirected into other members, and to assess the strength and ductility capacity of the new mechanism.

In San Bernadino County, California, on the Colton I-10/I-215 interchange, seismically deficient wall-type piers were strengthened with the addition of a new reinforced concrete outrigger frame, as shown in Figure 3.16. The outrigger frame is used in conjunction with a partial confinement steel jacket at the base of the pier walls. For this design, the designer must take care to consider the high shear forces and flexural rotations which can develop in the short cap-beam spans between the existing pier walls and the new columns (Priestley 1992a, Zelinski pers.comm. 1994).

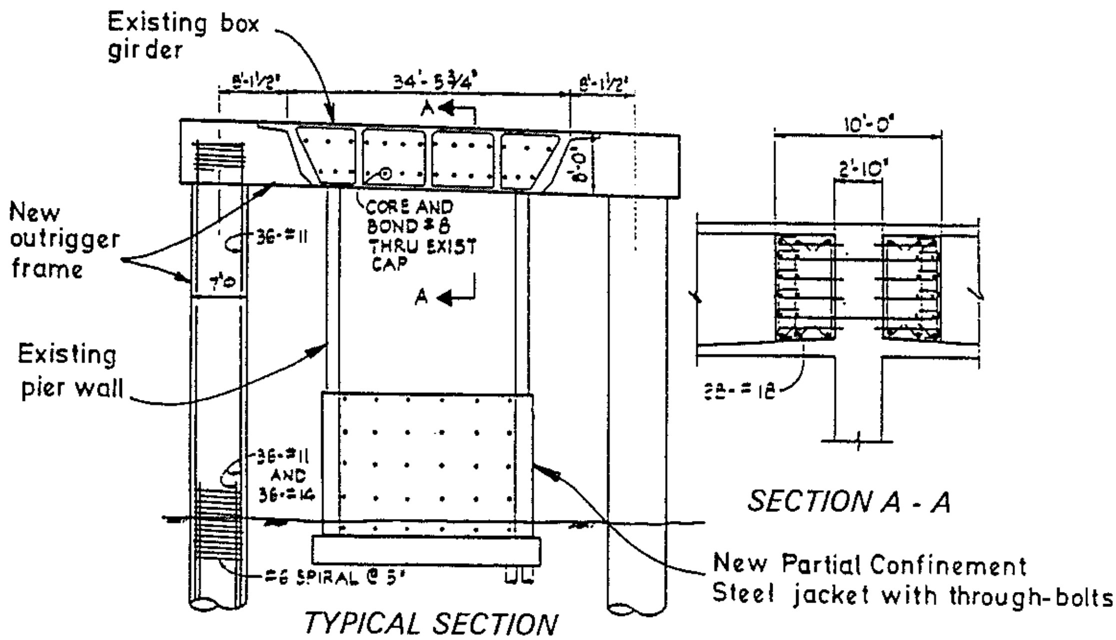


Figure 3.16 Retrofit of a wall-type pier using a new outrigger frame and a partial confinement steel jacket made by Caltrans in 1988 (Priestley et al. 1992a).

For the retrofit of San Francisco's double deck viaducts, an extensive strengthening design which included replacing columns and adding new longitudinal edge beams has been employed. This retrofit concept, using independent edge beams, was proof tested with a large scale model at the University of California, San Diego. A similar retrofit scheme using integral edge beams was tested at the University of California, Berkeley. Both retrofit schemes were shown to offer excellent seismic performance (Priestley 1992a).

4. EVALUATION & PROPOSED RETROFIT MEASURES FOR THORNDON BRIDGE

The seismic evaluation and retrofit designs for the Thorndon bridge in Wellington, New Zealand, illustrate the practical application of some of the evaluation and retrofit methods discussed in Chapters 2 and 3 of this report. The project made use of the most recent research results available, and followed a capacity-design approach. Although the capacity-design approach has been used in New Zealand for over 15 years, it is only in the last decade that it has been used for bridge retrofit work in California and elsewhere. The example of the Thorndon bridge shows how the capacity-design method can avoid the problems, discussed in Chapter 2, of the great uncertainty in modelling complex structures and predicting earthquake demands.

Three main examples from the Thorndon bridge project are presented here: the retrofit of superstructure linkages described in Section 4.2, of single-column piers described in Section 4.3, and of multi-column piers described in Section 4.4 of this report. Some additional proposed retrofit measures for the bridge, including a comprehensive investigation of possible ground-improvement methods, are reviewed in Section 4.5. The superstructure linkage retrofit is a unique application of the capacity-design principle.

The design of retrofit concepts for the bridge was carried out by BCHF consulting engineers (see acknowledgments) in June to October, 1994.

4.1 Background

The Thorndon bridge and its expected seismic performance have been described by Chapman and Kirkcaldie (1992) and BCHF consulting engineers (BCHF 1994a, Billings and Powell 1994). Retrofit concepts have also been proposed by BCHF (1994b). Information from these papers and reports is summarised below.

4.1.1 Description of the Bridge and Site

The Thorndon bridge carries the Wellington Urban Motorway over railway and harbour facilities for a length of 1.34 km (0.85 miles). Ramps at the halfway point provide access to and from a major local street, Aotea Quay. The bridge comprises two parallel structures, each with three lanes, carrying traffic volumes of 71,000 vpd (vehicles per day) on the section north of the ramps, and 53,000 vpd on the section south of the ramps. In 1992 the replacement value of the bridge was estimated to be NZ$50 million (Chapman and Kirkcaldie 1992).

4.1.1.1 Structural features

Planning for the bridge began in the mid 1950s, design took place between 1963 and 1967, and construction was finally completed in October 1969. Design was undertaken in three stages, with each stage tending to reflect the current developments in seismic design, which were undergoing major evolution in New Zealand during this time.

Figures 4.1 to 4.4 show some typical portions of the structure (also showing retrofit measures which are discussed in Section 4.2.2. The bridge superstructure consists of precast, prestressed I-girders, simply supported with spans ranging from 20 to 42 m (66 to 136 ft). At the northern end (the first construction stage), these are supported on 9 multi-column piers founded on groups of 600 mm-diameter steel-encased concrete piles. For the remainder of the bridge the I-girders are typically supported on single-column piers with cast-in-place box girder umbrellas[2].

North of the ramps (the second stage of construction), these single-column piers are founded on groups of 600 mm-diameter steel-encased concrete piles. South of the ramps (the third construction stage), 1.5 m-diameter cast-in-place drilled piers are typically used at the foundations (Chapman and Kirkcaldie 1992).

The original structure was designed for a lateral seismic acceleration of 0.3g and typically the seismic detailing improved with each stage of construction. The structure was not designed using the "capacity design" concept and consequently the inelastic mechanisms are not well defined (Billings and Powell 1994).

4.1.1.2 Site conditions

The bridge is sited on the shore of the Wellington Harbour on reclamations which were constructed in stages between 1882 and 1970, and which contain potentially liquefiable hydraulic fills. The reclamations were placed over sediments which are also susceptible to liquefaction. The fill typically consists of silts and sands ranging in depth from 4.5 m to a maximum of 17 m. Considerable variability is exhibited by the fill, reflecting several stages of deposition. For a short length along the bridge, the fill is retained by a mass concrete seawall, up to 17 m high, which runs adjacent to the seaward side of the bridge.

The active Wellington fault trace runs adjacent to the bridge over much of its length and passes beneath the bridge approximately midway between the ramps and the south abutment. An earthquake on this fault could result in permanent ground movements of up to 5 m horizontally and 1 m vertically. Permanent ground displacements of up to 1 m can also occur in a "fault-disturbed" area which covers the southern half of the bridge. Movement on the fault has been estimated to recur on average every 480 to 780 years, with the last rupture estimated to have occurred between 300 and 450 years ago. Seismologists estimate that there is an 11% probability of the Wellington fault rupturing over the next 50 years (Chapman and Kirkcaldie 1992, BCHF 1994b).

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2 Umbrella - an in situ concrete pier cap that is integral with the pier stem, and extends over the width of a bridge into a significant part of the span. It supports the span beams.

The bridge spans numerous railway tracks, two major city streets, and a number of utilities and services. Several buildings and parking areas are located near and underneath the bridge, and include a busy passenger ferry terminal.

4.1.2 Assessed Seismic Performance

A detailed seismic assessment of the Thorndon bridge (BCHF 1994a) revealed several potential seismic deficiencies, some of which are discussed in Sections 4.2 to 4.5. The study indicated that the bridge:

> *was vulnerable to major damage and collapse at relatively low levels of seismic ground shaking. Vulnerable items included collapse of the off-ramp due to liquefaction of an underlying sand layer, collapse of spans due to either insufficient seating length or due to failure of the bridge pile caps, and collapse of spans onto the ferry terminal due to failure of the seawall and retained ground in this area. In an earthquake caused by the Wellington fault, which runs under the bridge, collapse of the main bridge and the off-ramp where they cross the fault can be expected* (BCHF 1994b).

4.1.3 Approach to Seismic Retrofitting

The weak link in many of the piers of the bridge is yielding of the pile-cap reinforcement. Accordingly, most of the proposed retrofit work strengthens the pile caps. In the assessment of the unretrofitted bridge, column performance was typically not identified as critical. However, after retrofitting the pile caps, the force and displacement demands on the columns are increased and in many cases necessitate column retrofitting.

The retrofit measures proposed to address the column and pile-cap deficiencies are based on the extensive programmes of bridge retrofit research and implementation carried out in California and elsewhere, and they particularly rely on the recommendations of Priestley et al. (1992a, 1994) and Yan Xiao et al. (1994).

The retrofit measures proposed for the columns and foundations do not include adding any new piles. For most piers the capacity of the existing piles is adequate to resist the overstrength of the column or pile-cap mechanism above. For some piers compression or uplift demand on the piles exceeds nominal capacity; and foundation rocking could occur in these cases. The research results of Yan Xiao et al. (1994) show that such rocking is not detrimental to seismic response, and that in fact limited rocking of foundations can be beneficial in dissipating earthquake energy and isolating the structure above. Shear or flexural failures in the piles do not occur, except for the ductile flexural mechanisms for the multi-column piers (described in Section 4.4) and for ground-block sliding movements caused by liquefaction (described in Section 4.5). Accordingly no retrofit is needed for any of the existing bridge piles.

An explicit capacity-design approach was used for both the seismic assessment of the unretrofitted bridge and the design of proposed retrofit measures. The essential first step in this approach is the identification of the governing mechanism for the inelastic lateral displacement of the structure. In many cases the proposed retrofit measures change the governing mechanism. Retrofitting is designed to eliminate undesirable mechanisms such as pile-cap yielding, cap-beam flexure-shear failures, column-shear failures, or beam-column joint failures. Instead it supports the development of more ductile mechanisms such as the flexural hinging of steel-jacketed columns, existing steel-encased concrete piles, or in some cases foundation rocking. These effects of retrofitting are described in Sections 4.3 and 4.4 of this report. The retrofitting of the superstructure linkage bolts, described in Section 4.2, is a unique application of the capacity-design principle, where linkage capacities are designed to exceed the overstrength linkage force demands which can come from the adjacent movement joints.

4.2 Evaluation and Retrofit of Superstructure Linkages

Because of the use of precast simply supported girders, the Thorndon bridge has numerous superstructure movement joints, which typically are two joints at each pier. Such movement joints are commonly a source of seismic vulnerability in bridges. A unique retrofit scheme of providing a few high-strength slack linkage bolts at each movement joint has been proposed for the Thorndon bridge.

4.2.1 Seismic Assessment

By the standards of the day, the Thorndon bridge was constructed with good seismic detailing at the superstructure movement joints. Seating lengths at the ends of the superstructure girders range from approximately 450 mm (18 inches) for construction stages 1 and 2, to 760 mm (30 inches) for construction stage 3. In California, bridges of the same era can have seating lengths of only 150 to 200 mm (6 to 8 inches).

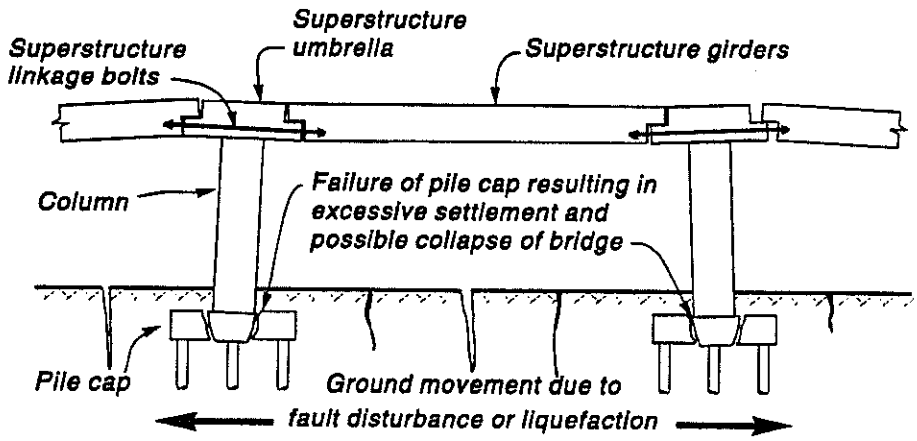
On the Thorndon bridge, substantial linkage bolts are also provided to tie adjacent spans together. Thick rubber pads are used under the ends of the linkage bolts to reduce earthquake impact forces. For construction stages 2 and 3, the linkage bolts are 8 to 12 m long (25 to 40 ft), extending across the umbrellas, and a welded end detail is used rather than threaded bolt ends. This allows the entire length of the bolt to yield, giving excellent elongation capacity. At the multi-column piers of stage 1 however, much shorter linkage bolts, with threaded ends, were used. Little elongation capacity could be expected in these bolts because failure would occur at the threads.

Despite the reasonable seating lengths and restrainer capacities for the Thorndon bridge, span unseating could still occur caused by permanent ground deformations. The ground deformations can result from liquefaction, or from movements of up to 1 metre in the Wellington fault-disturbed area. (The main 5-m offset of the Wellington fault is addressed separately.)

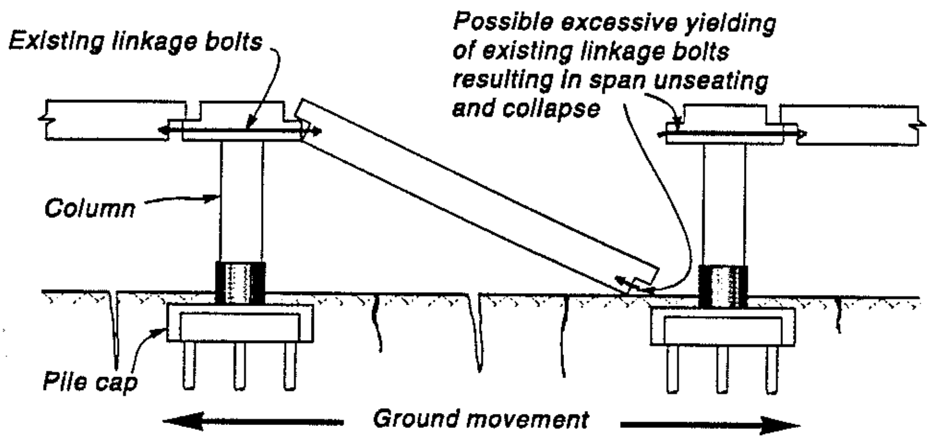
4.2.1.1 Stage 3 part of the bridge

In the stage 3 part of the bridge, ground movements from fault disturbance are anticipated, but the linkage bolts typically have a greater capacity than that corresponding to the failure of the pile caps. Thus, for this part of the bridge, the linkage bolts will not yield and span unseating would not occur from movement-joint deficiencies. However, as discussed in Section 4.4, the failure of the pile caps in this part of the bridge could lead to excessive settlement of the supporting piers and possibly to span collapses. This situation is shown in Figure 4.1(a).

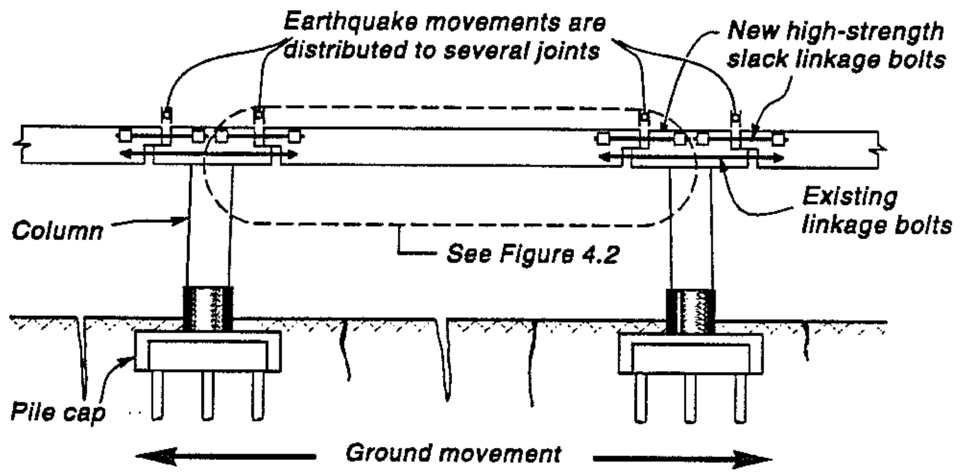
Figure 4.1 Bridge performance under earthquake-induced permanent ground movements, before (a) (b), and after (c) retrofitting superstructure linkages with high-strength slack[3] linkage bolts.



(a) Unretrofitted bridge, where linkage system strength exceeds substructure strength



(b) Unretrofitted bridge where substructure strength exceeds linkage strength



(c) Superstructure retrofit using high-strength slack linkage bolts

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[3] US terms 'slack' and 'snug' are equivalent to NZ terms 'loose' and 'tight' respectively.

To address the seismic deficiency of the pile caps, retrofitting would be implemented to strengthen the substructure, as discussed in Section 4.4 of this report. Once the substructure is strengthened, the columns and foundations would have enough capacity to cause the yielding of the linkage bolts under induced ground movements. This yielding could tend to concentrate at a single movement joint. (Typically this yielding would be at whichever movement joint the vertical hold-down bolts first failed.) With only one movement joint to take up the major portion of induced ground movements, span unseating could occur. This situation is shown in Figure 4.1(b).

4.2.1.2 Stage 2 part of the bridge

In the stage 2 part of the bridge, permanent ground movements caused directly by disturbance along the Wellington fault are not expected. However, this part of the bridge site is susceptible to liquefaction which could result in permanent sliding movements of the blocks of ground on which the bridge is founded. The failure of the mass concrete seawall is also possible, which would increase ground-block sliding movements.

In the stage 2 part of the bridge, the existing linkage bolts are of a smaller diameter than those used in the stage 3 part. These linkage bolts typically can yield under longitudinal ground displacements before the columns or pile caps reach their capacities. The yielding could tend to concentrate at one movement joint resulting in span unseating as shown in Figure 4.1(b). In the stage 2 part this type of failure is possible even if the pile caps are not strengthened.

4.2.2 Proposed Retrofit Measures

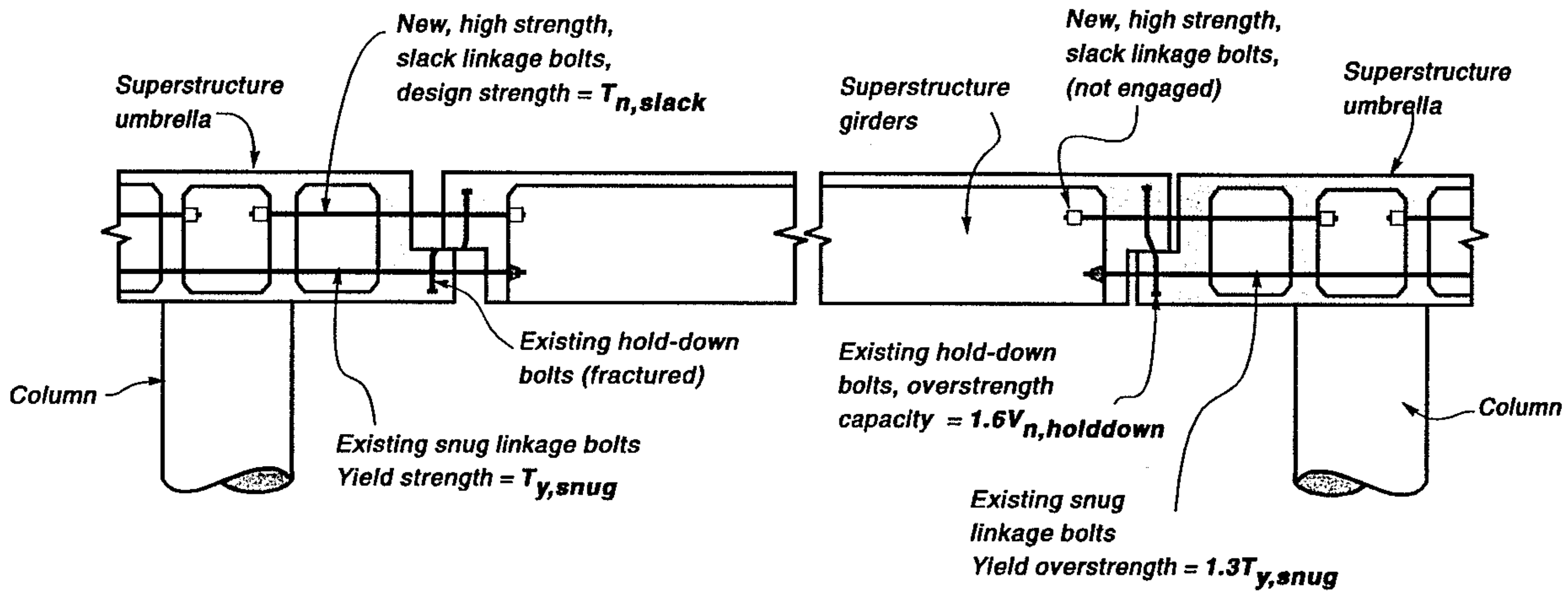
A scheme of linkage-bolt retrofitting has been devised to prevent unseating collapses caused by permanent ground movements. The retrofit requires replacing typically 3 or 4 of the existing linkage bolts at each superstructure umbrella with high strength Dywidag or Macalloy bolts. The retrofit is shown schematically in Figure 4.1(c) and Figure 4.2. Figure 4.4 shows the locations of the existing and replacement linkage bolts on a cross-section of the bridge.

The new Dywidag or Macalloy bolts are installed with slack linkage at each end of the umbrella. The linkage bolts are anchored within the umbrella to allow equal displacement at each end of the umbrella. If the fault disturbance tends to pull the bridge apart, the existing linkage bolts will yield, but the high-strength linkage bolts will engage before unseating occurs. Thus ground displacement demands can be distributed to several adjacent movement joints without any span collapses.

Typically, the initial linkage-bolt yield strength after retrofitting is less than the column lateral strength. Thus for longitudinal-direction displacements the linkage-bolt yielding can preclude serious damage to the substructure, assuming that the weak pile caps in the area have been retrofitted.

The slack bolts are designed so that the ultimate strength of linkages at a movement joint before unseating exceeds the overstrength of the linkage bolts in the adjacent joints, including the hold-down bolt overstrength. This is illustrated in Figure 4.2,

Figure 4.2 Retrofitting of superstructure linkages using high-strength bolts installed with slack linkage..



To prevent unseating, want:

$$\left(T_{n,slack} + T_{y,snug}\right)_{\substack{\text{Movement}\\ \text{Joint } i}} \geq \left(1.6V_{n,hold\text{-}down} + 1.3T_{y,snug}\right)_{\substack{\text{Movement}\\ \text{Joint } i\pm1,\, i\pm2}}$$

which shows that the design strength of the initially slack bolts plus the yield strength of the snug bolts at the left movement joint ($T_{n,SLACK} + T_{y,SNUG}$) should exceed the hold-down bolt overstrength plus the snug bolt overstrength at the right movement joint ($1.6\ V_{n,HOLD\text{-}DOWN} + 1.3\ T_{y,SNUG}$). The overstrength factors of 1.6 and 1.3 are chosen by judgement.

For the hold-down bolts a higher overstrength factor of 1.6 was chosen because strain hardening, bolt kinking, and shear-friction type mechanisms could increase strengths.

For the existing snug linkage bolts, strain hardening will be limited because of the long bolt lengths. Laboratory testing has been recommended to verify the bolt strengths, overstrengths, and displacement capacities (BCHF 1994b).

The linkage-bolt retrofitting is complicated by variations, between different bridge piers, in the number and size of the existing linkage bolts. A general aim of the retrofit is to protect against abrupt changes in the linkage capacity along the length of the bridge. The snug and slack linkage capacities are designed to exceed the snug linkage plus hold-down overstrength at the adjacent joints, considering at least two movement joints on either side. This is illustrated in Figure 4.3.

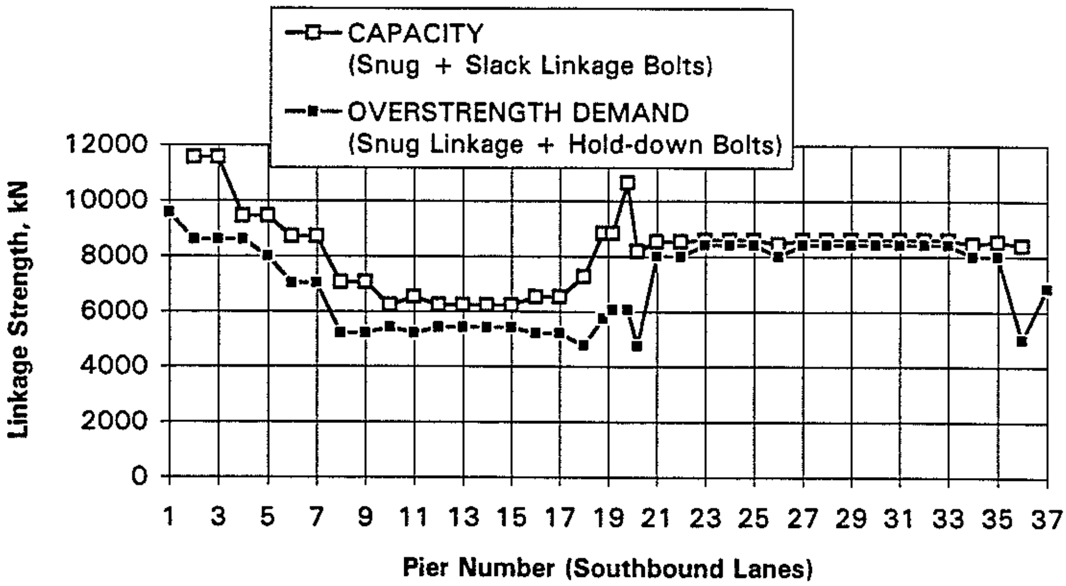


Figure 4.3 Strength of superstructure linkages along the bridge length after retrofitting.

At the umbrellas, replacement linkage bolts are anchored where possible in umbrella box-girder cells which have existing access serviceholes. Otherwise replacement linkage bolts are grouted into the existing steel pipe ducts, or new access serviceholes are cut. High-strength linkage bolts are not placed in the bottom locations of the umbrellas because girder end-diaphragms are weaker in these locations. To provide the maximum plan rotation capacity at the movement joints, the new slack linkage bolts are located near the centre of the superstructure width at the umbrellas (BCHF 1994b).

4.3 Evaluation and Retrofit of Single-Column Piers

Most of the piers on the Thorndon bridge are supported on single columns. Seismic evaluations indicate deficiencies in the performance of these piers, which can be addressed by foundation strengthening and column jacketing retrofit measures.

4.3.1 Seismic Assessment

Structural calculations reveal that the pile caps typically represent the weak link in the seismic resistance of the single-column piers. During the assessment of pile-cap strength, likely crack patterns, yield-line mechanisms, and strut and tie modelling were considered.

The reinforcing of the existing pile caps was apparently designed primarily for gravity loads, because the amount of reinforcing in the top mat of the pile caps is typically very small compared to that in the bottom mat. Consequently a ratcheting type of permanent displacement can occur when the pile cap is subjected to cyclic flexural yielding caused by earthquake actions. The ratcheting results from the following sequence of effects:

- For earthquake actions in one direction, flexural cracks open from the bottom of the pile cap and the bottom reinforcing steel yields and elongates.
- Then the bottom reinforcing steel is subjected to a matching compression force. However, this compression force is not enough to reverse the permanent tensile strain present in the bottom reinforcement, or to close the cracks at the bottom of the pile cap.
- Therefore on successive cycles, the inelastic tensile strain in the bottom reinforcement accumulates, and the pile-cap hinge develops a cumulative rotation in one direction.

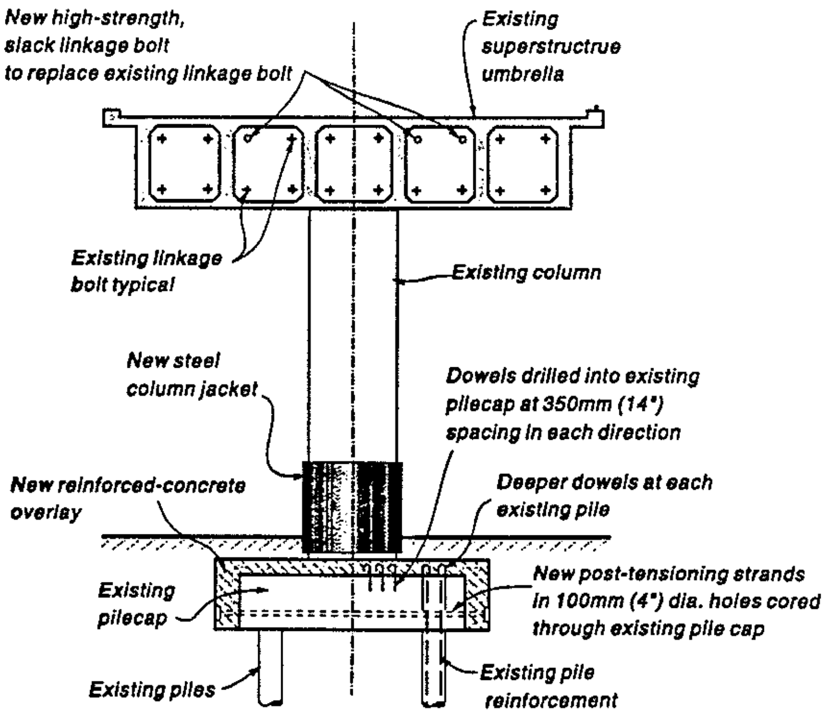
According to Dr Richard Fenwick of the University of Auckland (pers.comm. 1994), the strain-ductility demands on such "one-way" plastic hinges, for a given plastic rotation, can be three times those for a conventional plastic hinge (Billings and Powell 1994). This type of pile-cap failure is schematically illustrated in Figures 4.1(a) and 4.5(b).

For the single-column piers of the unretrofitted bridge, failure of the columns is typically precluded by yielding of the pile caps. After pile-cap retrofitting is implemented, however, column yielding becomes a likely inelastic mechanism for many of the single-column piers. The original designers of the Thorndon bridge made an effort to increase the amount of transverse reinforcement at the column ends above what was used in construction of that era. However, the amount of transverse reinforcement is still typically inadequate to provide the desired column ductility capacities, as judged by current evaluation methods. For the seismic assessment, column strengths and ductility capacities were calculated using a moment-curvature analysis based on the Mander stress-strain model for confined concrete (Billings and Powell 1994). Column-shear capacities were assessed according to the recommendations of Priestley et al. (1994).

4.3.2 Proposed Retrofit Measures

The proposed retrofit measures for a typical single-column pier of the Thorndon bridge are shown in Figure 4.4. The figure shows that the pile cap is strengthened using a new reinforced-concrete overlay, and post-tensioned reinforcement which is cored through the existing pile cap. The overlay is tied to the existing pile cap with drilled, grouted dowels, so that the overlay and pile cap will behave compositely. The overlay covers the sides of the existing pile cap with end-blocks which protect the post-tensioning anchors. The dowels across the overlay-pile cap interface are calculated to act in shear friction, and the end-blocks of the overlay, which contain stirrups as vertical reinforcement, are assumed to contribute to the shear capacity at the interface. Drilled dowels are also provided to lap with the reinforcement of the existing piles, to develop the potential pile-tension force through the full depth of the pile cap plus overlay.

Figure 4.4 Proposed retrofit measures for a typical single-column pier.



Considering a strut-and-tie model of the forces in the retrofitted pile cap, the additional dowels at the existing piles allow a steeper compression-strut mechanism to develop on the uplift side of the pile cap. These additional drilled dowels at the existing piles are also assumed to contribute to the horizontal shear-friction capacity at the overlay-pile cap interface.

Among the many single-column piers of the Thorndon bridge there is considerable variation in the layout, geometry and reinforcing details of the existing pile caps, as well as variation in column capacities and construction constraints. Consequently, the proposed retrofit designs for different pile caps also vary. For some pile caps the reinforced-concrete overlay is not needed and the added post-tensioning alone strengthens the pile cap sufficiently to force plastic hinging into the columns. For other pile caps, the post-tensioning is not needed, and an overlay alone is proposed. Typically the pile-cap retrofitting is designed for strength only, to force plastic hinging into the columns. In some cases, however, inelastic behaviour of the pile caps, either retrofitted with an overlay only or unretrofitted, may be acceptable. Laboratory testing has been recommended to assess the ductility capacity of the inelastic pile-cap response.

Where column plastic hinging is possible, new steel jackets are provided over the potential plastic-hinge zone of the column as shown in Figure 4.4. The jackets improve the confinement of the concrete and restrain the column bars against buckling. Fibreglass/epoxy jackets have also been considered for use on the Thorndon bridge (BCHF 1994b). Compared to the pile-cap retrofit measures, the design criteria for the column jacketing are well established, as described in Section 3.3 of this report.

4.4 Evaluation and Retrofit of Multi-Column Piers

The first construction stage for the Thorndon bridge used four to five columns per pier. The columns of each pier are connected by a continuous pile cap below, and a continuous cap-beam above which supports the superstructure girders. A number of deficiencies are evident in these multi-column piers which are addressed with retrofit measures such as infill concrete walls, pile-cap overlays, and column jackets.

4.4.1 Seismic Assessment

Figure 4.5 shows the expected earthquake failure mechanisms for the typical multi-column piers of the unretrofitted Thorndon bridge.

4.4.1.1 Transverse direction

Under transverse-direction earthquake effects, a strong column–weak beam mechanism develops, as shown in Figure 4.5(a). The yielding zones of this mechanism, in the cap beam and pile cap, have not been detailed for ductile performance. Thus the inelastic rotation capacity of these elements is assessed as being deficient. Flexure/shear failures could develop in the pile cap and cap beam possibly leading to collapse. In addition, the beam-column joints of the cap beam may be vulnerable to failure, particularly if the overstrength of the cap beam is developed, or if the cap beam alone were to be retrofitted.

4.4.1.2 Longitudinal direction

In the longitudinal direction, the multi-column piers are vulnerable to pile cap failures, as shown in Figure 4.5(b). This potential pile-cap failure is similar to that described in Section 4.3 for the single-column piers. The inelastic rotation capacity at the pile-cap yielding areas is diminished because the bottom of the pile cap is much more heavily reinforced that the top of the pile cap.

The heights of the multi-column piers are different along the length of the bridge. For the shorter columns, a column shear failure could preclude a pile-cap failure, as shown in Figure 4.5(c). Virtually no inelastic displacement capacity occurs in such a failure mode, and several catastrophic bridge collapses have occurred in earthquakes caused by the shear failure of short columns.

4.4.2 Proposed Retrofit Measures

The proposed retrofit measures for the multi-column piers of the Thorndon bridge are shown in Figure 4.6.

4.4.2.1 Transverse direction

For transverse-direction earthquake effects, reinforced concrete walls are infilled between the columns of the pier.

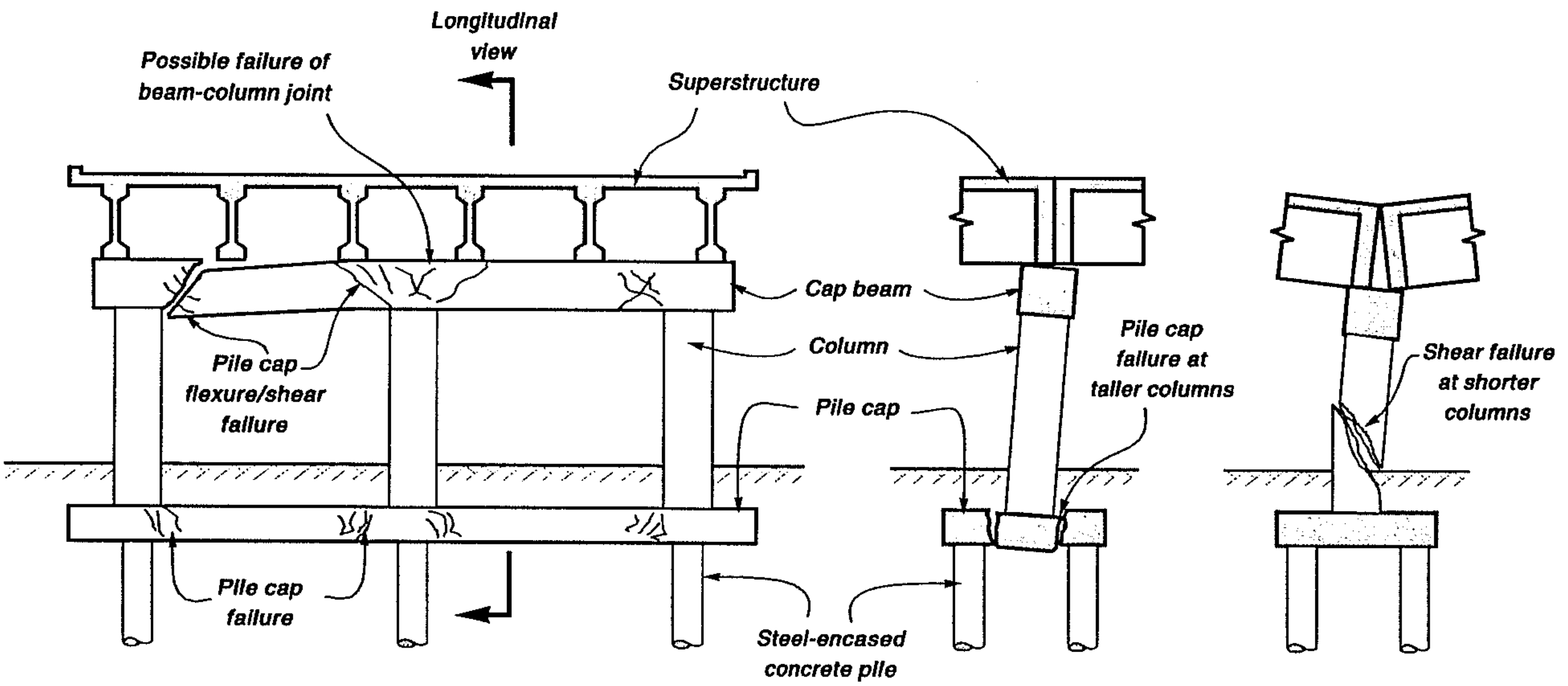
The infill walls are dowelled into the cap beam above and the pile cap below. The addition of the walls causes an entirely new inelastic mechanism to govern the seismic response of the pier. It is a ductile flexural hinging of the existing piles, as shown in Figure 4.6(a). The existing piles were constructed with steel jackets (or sleeves) which confine the reinforced-concrete pile core, providing excellent inelastic rotation capacity. The infill walls are designed for the overstrength of the pile-hinging mechanism and effectively prevent failures in the cap beam, pile cap, or beam-column joints.

4.4.2.2 Longitudinal direction

For longitudinal-direction earthquake effects, full-height steel jackets are added to the columns, and a reinforced concrete overlay is added to the existing pile cap. The steel jackets prevent shear failures from occurring in the columns. The overlay is used to strengthen the pile cap and force an inelastic mechanism elsewhere in the pier.

For the more lightly reinforced columns, flexural hinging of the jacketed columns is expected to govern the seismic response. For the more heavily reinforced columns a pile uplift and foundation rocking mechanism may govern seismic response. These possible mechanisms are shown in Figure 4.6(b). The pile cap strengthening is designed for the lesser of the overstrength of these two possible mechanisms. Because of the uncertainty in pile uplift values, a high overstrength factor is used for the foundation rocking mechanism.

Figure 4.5 Expected earthquake failure mechanisms for typical multi-column piers of the Thorndon bridge, before retrofitting.

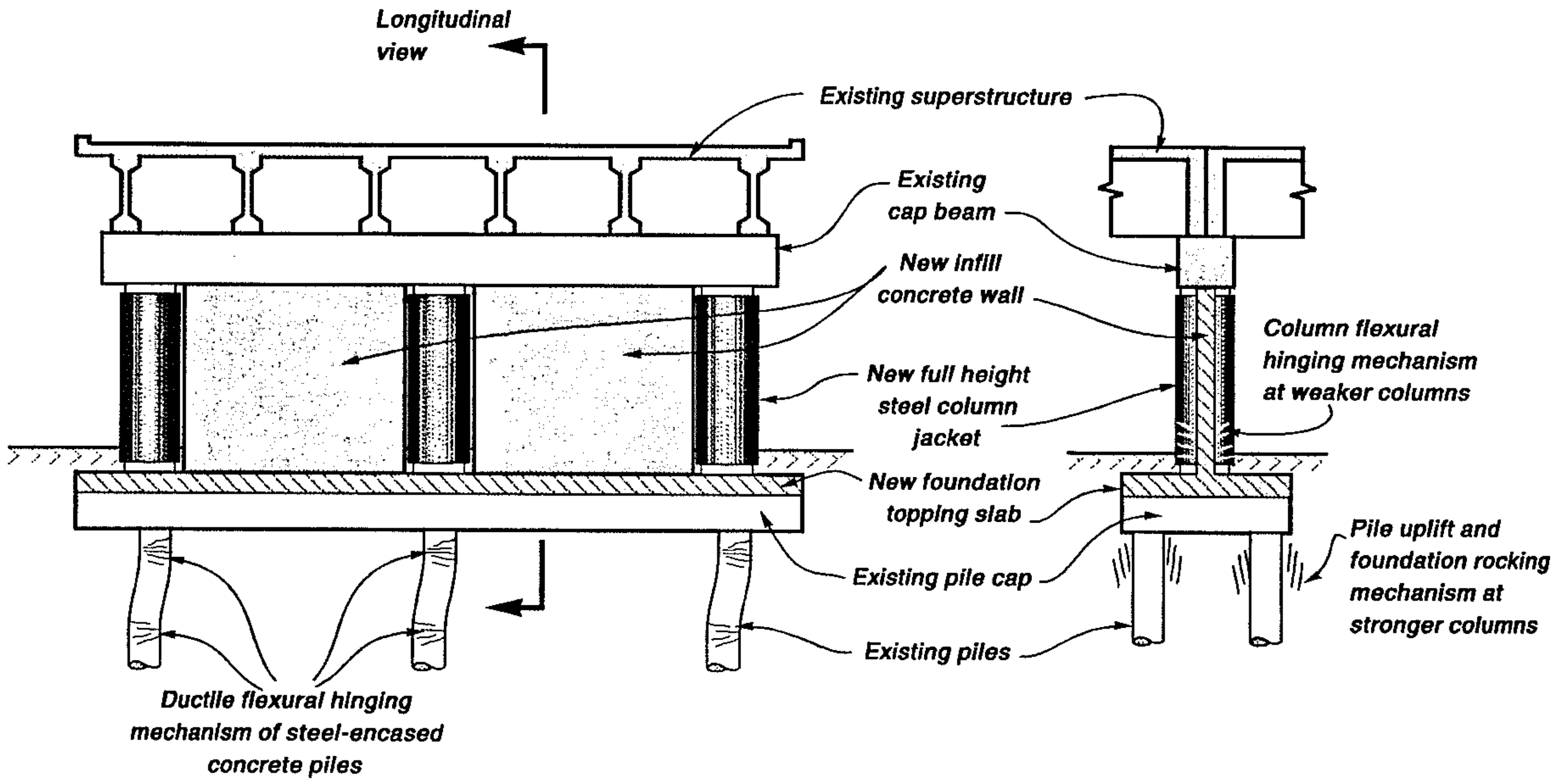


(a) Transverse direction

(b) Longitudinal direction taller column

(c) Longitudinal direction shorter column

Figure 4.6 Retrofit measures for multi-column piers of the Thorndon bridge, and expected earthquake response mechanisms, after retrofitting.



(a) Transverse direction

(b) Longitudinal direction

4.5 Other Proposed Retrofit Measures

Sections 4.2, 4.3, and 4.4 of this report describe the evaluation and typical retrofit of superstructure linkages, single-column piers, and multi-column piers for the Thorndon bridge. Additional retrofit measures are proposed for the bridge (BCHF 1994b), some of which are briefly described in this Section 4.5.

4.5.1 Structural Retrofit Measures

4.5.1.1 Seat extensions at ramps and abutments

Two of the most vulnerable areas of the Thorndon bridge are the seating conditions of the two ramps where they meet the main structure. No linkage bolts have been provided at these locations. The recommended retrofit measure to address this deficiency is to add reinforced-concrete seat extensions dowelled into the existing ramp support seats. The details of the existing structure make such a retrofit relatively straightforward.

The seat extension was considered preferable to the option of adding new linkage bolts at the ramp connections. The ramp connections represent major geometric and stiffness discontinuities in the bridge structure where large relative movements are prone to occur. Attempting to restrain these movements with linkage bolts is of doubtful use because failure of the linkage bolts could not be prevented. There would be practically no upper limit to the force demands on linkage bolts in such a location. Instead of trying to restrain relative movements at the ramp connections, the seat-extension retrofit allows considerable movements to take place without span collapse. Reinforced-concrete seat extensions are also used at the bridge abutments, where it would be expensive and disruptive to replace the existing linkage bolts (BCHF 1994b).

4.5.1.2 Steel beam frames for Wellington fault offset

A seismic retrofit solution has been proposed to prevent collapse of the main bridge structure where it crosses the Wellington fault. The rupture of this strike-slip fault is predicted to cause a relative offset displacement of up to 5 m (16 ft) horizontally. The main bridge axis crosses the fault trace at an angle of 25 to 30°. The strike-slip offset of the fault would principally cause pulling-apart displacements of adjacent bridge piers with some relative transverse movement between the piers.

The proposed retrofit consists of frames built up of steel beams, dowelled and bolted to the undersides of the superstructure umbrellas on either side of the fault. Several of the linkage bolts at these umbrellas are replaced with slack linkage bolts. The steel frames act as 2.5 m-seat extensions to allow the relative pulling-apart and transverse movement of the piers on either side of the fault. The frames are designed with a shear-key stopper which, along with the slack linkage bolts, ensures that pull-apart fault offsets are distributed to each end of the span, and that transverse offsets are accommodated by the plan rotation of the suspended span.

The goal of the retrofit is only to prevent collapse and reduce the likelihood of casualties. The bridge spans above the fault would need to be replaced after an earthquake on the Wellington fault to correct roadway geometrics (BCHF 1994b).

4.5.2 Ground Improvement to Prevent Liquefaction

A major part of the Thorndon bridge site is susceptible to soil liquefaction. Several methods of improving ground conditions to reduce the likelihood of liquefaction were investigated by BCHF consulting engineers (BCHF 1994b). Their findings and recommendations are summarised in Sections 4.5.2.1-4.5.2.3.

4.5.2.1 Seismic assessment of ground

Underneath the northern half of the Thorndon bridge is a layer of beach sediments which is susceptible to liquefaction. Liquefaction of this soil layer would result in permanent slope displacements because the overlying reclamation fill material would move in a large-scale sliding block type of ground failure. The ground acceleration at which the beach sediments liquefy is assessed to be 0.16g in a magnitude M=7.4 earthquake. The slope displacements are predicted to occur in arcs forming scallops, eventually joining up to form a complete block movement. Where the original seabed profile is steep, this block movement may extend as far back as the original high water mark.

At a ground acceleration of approximately 0.2g, permanent slope displacements are assessed to be small, with magnitudes less than 25 mm. At higher levels of shaking the magnitude of the displacements increases, with displacements up to 1,500 mm predicted for ground accelerations of around 0.74g. The foundations of the overbridge are assessed to maintain gravity support at this higher shaking level, but the piles would suffer significant and irreparable damage.

Underneath the off-ramp of the Thorndon bridge is a layer of sandy hydraulic fill which is assessed to liquefy at a peak ground acceleration of around 0.19g in a magnitude M=7.4 earthquake, or at a ground acceleration of around 0.25g in a magnitude M=6.0 earthquake.

When the sandy fill deposit liquefies, the seawall retaining the fill is expected to fail by either overturning or sliding. This failure would permit a seaward movement of the body of liquefied material and the gravel rockfill above. The seawall may possibly move as much as 10 to 20 m. Ground movements were assessed to decrease with distance back from the seawall, but would remain large (in the order of metres) as far back as the off-ramp pier foundations. These ground movements would apply very large lateral loads to the off-ramp piles and pile caps, almost equal to the soil passive pressure, and the off-ramp would most likely collapse (BCHF 1994b).

4.5.2.2 Preferred ground-retrofit measures

To reduce the magnitude of the permanent slope displacements which could occur underneath the main bridge, the strengths of the liquefiable materials, which are located some 10 m below existing ground level, need to be improved. This improvement would not need to eliminate the permanent slope displacements, but would aim to reduce the magnitude of the displacements to a level at which the bridge foundations could maintain structural integrity.

The preferred method of ground improvement to the Thorndon bridge site to mitigate liquefaction effects involves a combination of stone columns (vibro-replacement) and jet grouting techniques. The stone columns would be implemented at discrete points along 10 m-wide strips on both sides of the overbridge where there are no headroom restrictions. The stone columns would extend beneath the interface beach layer, to depths of approximately 15 m.

- *Stone Column Technique:* A vibrating cell or "vibroflot" is lowered through the sandy deposit, partly driven and partly aided by the vibrating action and water jets liquefying the deposit locally. When at the desired depth, aggregate is fed down a central column and compacted in place from the bottom up. As the vibroflot is withdrawn a dense stone column is formed in place. Both the vibrating action of the vibroflot and the compaction of the stone column densify the surrounding sandy material to a level designed to prevent liquefaction. For the liquefiable area underlying the off-ramp, preliminary designs suggest installing stone columns at approximately 2.0-m centres on a triangular grid along 10 to 20 m-wide strips of ground both on the seaward and landward side of the off-ramp.

- *Jet Grouting Technique:* This could be used beneath the overbridge where headroom is restricted, to improve the strength of the weak fill and beach materials. For this technique, injection tubes are installed through the fill and beach materials to the desired depth, and then a system of high pressure water, air, and grout jets are employed to mix the material in place with the grout. The injection tubes are rotated and withdrawn to form columns of "soilcrete" of up to 1.5-m diameter. A portion of the in situ material is removed through the centre of the injection tube. The diameter of the injection tube is small in comparison to the diameter of the soilcrete column formed. The soilcrete columns would be constructed over approximately 50% of the ground area beneath the overbridge, over a depth of approximately 5 m centred at mid-depth of the liquefiable soil layer. The soilcrete columns are constructed so that adjacent columns are in contact with each other, forming the system of celled walls (BCHF 1994b).

4.5.2.3 Alternative ground-improvement methods

In addition to the stone column (vibro-replacement) and jet grouting systems recommended above, additional ground improvement techniques were investigated. Compared to the preferred methods, these alternative techniques were generally found to be more costly for application on the Thorndon bridge.

Two of the alternative ground improvement techniques, displacement piling and compaction grouting, described below (BCHF 1994b) are similar to the stone column method in that their effectiveness lies in densifying ground material to prevent liquefaction and loss of strength. Like the stone column technique the methods involve treatment of the ground at discrete points on a (typically triangular) grid. The

- *Importance of experimental testing of retrofit designs*
  Experimental testing of new retrofit designs is important. Some retrofit measure designs, based on engineering judgement, were subsequently shown by testing to be ineffective. Additional research may be needed in the following areas:

  - Partial-confinement jacketing of columns with lap splices is now common in California, but further experimental and analytical research should be done to validate this retrofit concept for rectangular columns and shear-deficient columns, and to improve hysteretic response.

  - The evaluation of movement-joint behaviour in bridge superstructures is difficult. Experimental studies of the force-displacement capacities of typical movement-joint details, and studies of earthquake demands on movement joints would be useful.

  - Seismically deficient beam-column joints and column-foundation joints are difficult to effectively retrofit. Additional development and experimental testing of potential retrofit methods, such as added external prestressing, are needed.

  - Strengthening foundations and abutments to resist greater seismic forces has been common in California. These seismic strengthening measures are expensive and, paradoxically, little earthquake damage has been observed in bridge foundations. Further study of bridge abutment and foundation behaviour is warranted, including studies of foundation rocking response.

- *Innovative retrofitting of superstructure linkages*
  A novel scheme of retrofitting the superstructure linkages at the numerous movement joints of the Thorndon bridge has been developed. The retrofit uses high-strength, loose (slack) linkage bolts and a capacity-design philosophy to ensure that earthquake displacement demands can be distributed to several movement joints, rather than being concentrated at one location and possibly causing span unseating. The retrofit design eliminates the need for complex modelling of the structure or precise estimation of earthquake demands at movement joints, procedures which would be of questionable accuracy.

- *Capacity design retrofitting of columns and foundations*
  The proposed column and foundation retrofit measures for the Thorndon bridge illustrate the application of recent bridge seismic retrofit research and the benefits of a capacity-design approach to seismic evaluation and retrofitting. The design of the retrofit measures emphasises the development of desirable inelastic seismic response mechanisms, with less emphasis on computer modelling of the structure. This approach will make the structure less sensitive to the large uncertainties inherent in predicting earthquake force and displacement demands.

5.2 Recommendations

This report has shown that there are some gaps in the engineering community's knowledge of, and ability to provide, effective seismic-retrofit solutions. In some areas further research is needed, while in other cases sufficient research has been carried out but practising engineers may not be fully aware of the research results or their implications. Based on these findings, two recommendations can be made:

1. Further structural engineering research on the seismic evaluation and retrofitting of bridges should be funded and promoted. A high priority should be given to research on bridge features which are specific to New Zealand. Topics which merit investigation include:
   - Studies of typical superstructure movement-joint details for New Zealand bridges.
   - Studies of bridge foundation performance and retrofitting.
   - Further studies of concrete bridge structures with plain-round (undeformed) reinforcement.
   - Studies of partial-confinement retrofitting for bridge columns with lap splices.
2. Information on recently developed seismic evaluation and retrofit technology should be disseminated to practising engineers in New Zealand. In this way Transfund New Zealand could play an important role in transferring knowledge from research and implementation of international experience. The publication and distribution of the present report should help in this goal. But as seismic-retrofit research and implementation for bridges is currently developing at a rapid pace, this report should be periodically updated by Transfund New Zealand.

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