Government of Canada Gouvernement du Canada

Office of Critical Infrastructure Protection and Emergency Preparedness

Bureau de la protection des infrastructures essentielles et de la protection civile

# Seismic Hazard, Building Codes and Mitigation Options for Canadian Buildings

## Acknowledgments

This publication has been prepared for:

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**Emergency Preparedness**

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June 2001

This material is based upon work supported by the Directorate of Research and Development (DRD) in the Office of Critical Infrastructure Protection and Emergency Preparedness (OCIPEP), formerly Emergency Preparedness Canada, under Contract Reference No. 2000D020. Any opinions, findings, and conclusions or recommendations expressed in this material are those of the author(s) and do not necessarily reflect the views of the Office of Critical Infrastructure Protection and Emergency Preparedness.


Catalogue No.:  D82-73/2002E-IN
ISBN: 0-662-32761-6

## Executive Summary

The extent of casualties and economic losses during recent earthquakes worldwide has been staggering, with loss of lives in the tens of thousands, and loss in economy in the billions of dollars. The February 28, 2001 earthquake near Seattle, which rattled buildings and occupants in Vancouver, could be viewed as a reminder to people living in Canada's most active seismic zone, the Pacific coast. However, other parts of Canada, such as regions along the St. Lawrence River and Ontario/Quebec border, are also potential sites for moderate to strong earthquakes. As a case in point, the 1988 Saguenay earthquake in Quebec was the strongest event in eastern North America within the last 50 years.

This study consists of three related documents dealing with the built environment and seismic aspects of building codes in Canada, primarily the 1995 National Building Code of Canada (NBCC). The consequences of recent earthquakes and their effects on the buildings have exposed a common concern: older buildings may be susceptible to significant damage. This in turn could result in traumatic loss of life and property and lead to a difficult and protracted recovery period both from a human and an economic perspective. Conversely, newer construction designed and built using more stringent code requirements may be less susceptible to significant damage.

In an effort to provide Canada's seismic protection community with a state-of-the-art knowledge base on the seismic hazard assessment and mitigation for buildings, three tasks were carried out within the scope of this project. These include a review of:

- variances in seismic requirements for existing buildings with respect to applicable codes and regulations in Canada;
- emerging technologies for the seismic retrofit of buildings; and
- current screening methodology.

Part A, which deals with "variances in seismic requirements for existing buildings," reveals that the codes and regulations in place for seismic protection of existing buildings are less stringent than the requirements for new construction. The report concludes that in order to prepare communities for seismic events, and maintain their sustainability through better readiness, proactive rather than reactive codes and regulations are needed. This can help improve seismic performance of existing buildings in Canada.

Part B, a "review of technologies," discusses seismic upgrade techniques as well as research issues concerning seismic retrofitting of buildings. Building seismic retrofitting is a relatively new activity for most structural engineers. The retrofitting of a building requires an appreciation for the technical, economic, and social aspects of the issue. Changes in construction technologies and innovations in retrofit technologies represent an additional challenge to engineers in selecting a technically, economically and socially acceptable solution.

Conventional upgrading techniques usually include the addition and/or strengthening of existing walls, braces, frames, and foundations. Adopting these techniques often leads to heavy demolition, lengthy construction time, reconstruction, and occupant relocation with all the associated direct and indirect costs. It is often the indirect costs, the environmentally hostile approach, and the inconvenience associated with conventional techniques that deter building owners and custodians from committing to seismic retrofit.

Part C reviews seismic “screening methodology,” and is a review based on the 1993 NBCC seismic screening document and how it relates to building codes in Canada.

This three part document summarizes recent development efforts and innovative technologies for mitigation of a given building’s seismic hazard susceptibility. Advanced materials, systems and techniques have been extensively investigated, and, to a lesser extent, applied in seismic retrofit projects. The current gap between research advances and application benefits is principally due to the lack of state-of-the-art knowledge bases available to both research and practicing engineers. As such, the benefits of utilizing innovative technologies as technically, economically and socially acceptable solutions for seismic hazard reduction have not yet been fully realized. This report is a first step in providing Canada's seismic protection community with a state-of-the-art knowledge base dedicated to seismic retrofit mitigation options for Canadian buildings.

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# 1.0 Part A – Variances in Seismic Requirements for Existing Buildings: Codes and Regulations

## 1.1 Background

Building code requirements for seismic design have become more stringent over the past three decades. The seismic base shear specified in the National Building Code of Canada for buildings in regions of high seismicity has increased by as much as 100 per cent since the early 1970's. Improved design methods and requirements are expected to reduce the damage in newer buildings to acceptable levels, in the event of a moderate to strong earthquake. However, older buildings designed to satisfy older codes, which are known to provide inadequate safety, are likely to be vulnerable to severe damage or total collapse under strong seismic excitations. Recent earthquakes in Northridge, California (1994); Kobe, Japan (1995); Turkey (1999); and JiJi, Taiwan (1999) have revealed that an earthquake does not necessarily have to be the "big one" to cause widespread destruction, especially among the older buildings. Past earthquakes have also demonstrated that these older building would have survived past earthquakes, in most cases, with reasonable upgrades.

The stock of pre-1980's buildings is believed to be many times larger than the number of newer building which were designed and built in accordance with more recent codes. While substantial advances have been made in the development of newer codes, which were intended for new buildings, it appears that relatively little effort has been made for the existing and potentially vulnerable old buildings.

It is said that, "disasters do not happen to well-prepared communities." One aspect of determining the readiness of a community is to assess the applicable codes and regulations on seismic requirements of existing buildings. This report provides a summary of the variances in seismic requirements for existing buildings with respect to applicable codes and regulations in Canada.

## 1.2 Codes and Regulations for New Construction

Under the terms of the Constitution Act, regulation of buildings in Canada is the responsibility of the provincial and territorial governments. Building and fire codes developed and issued by provinces and territories have largely been based on the National Building Code of Canada (NBCC 1995) [National Research Council of Canada 1995] and the National Fire Code of Canada [National Research Council of Canada 1995]. Through the Municipal Act, the authority to enforce building regulations is granted to municipalities, who name a Chief Building Official to administer the building code, and a Fire Chief to administer the fire code. The status of adoption of the current NBCC 1995 by provinces and territories is identified in Table 1. In general, the seismic requirements for new construction in Canada are rather uniform (as defined in section 4.1.9 of NBCC 1995).

**Table 1** Adoption of the 1995 National Building Code of Canada *(NBCC 1995)*.

| Jurisdiction | Adoption of NBCC 1995 | |
|---|---|---|
| Newfoundland | Yes | Adopted by major cities. No provincial building code. |
| Nova Scotia | Yes | Adopted April, 1997 with minor amendments. |
| New Brunswick | Yes | Adopted May, 1998. |
| Prince Edward Island | Yes | Adopted by the 2 major cities, but not the province. |
| Québec | Yes | Adopted September, 2000. |
| Ontario | Yes | Adopted April, 1998, with substantial changes.<br>OBC Part 11 addresses renovation work. |
| Manitoba | Yes | Fire Commissioner administers both the building and fire codes. |
| Saskatchewan | Yes | Adopted July, 1998. |
| Alberta | Yes | NRC publishes the ABC & AFC.<br>1995 NBCC adopted June, 1998. |
| British Columbia | Yes | 1998 BC Code based on the 1995 NBCC. |
| Yukon Territory | Yes | Codes adopted without amendment by Commissioner's Order pursuant to Fire Prevention Ordinance. Reference is to the 1985 NBCC and NFC as amended from time to time. |
| Northwest Territory | Yes | Adopted March, 1997.<br>Fire Marshal's Office administers both the building and fire code. |
| Nunavut Territory | Yes | Adopted April, 1999. |
| Federal Jurisdiction | Yes | The1995 NBCC & NFC referenced by Canada Occupational Safety and Health (COSH) regulations as pursuant to the Canada Labour Code. |

## 1.3 Codes and Regulations for Existing Buildings

The NBCC 1995 has been adopted by most provinces/territories with little or no variances, as illustrated in Table 1. There are provinces, noticeably Ontario and British Columbia, which publish their own provincial codes. Vancouver and Montreal are the only two municipalities that have their own Charter to enact building bylaws. By virtue of the Charter, the City of Vancouver has outlined specific requirements for seismic protection of existing buildings. Variances in seismic requirements for existing buildings are discussed below, with respect to the applicable codes and regulations in Canada.

### 1.3.1 NBCC 1995

While the NBCC 1995 is a model building code for new construction, its requirements are not retroactive. In other words, seismic requirements of the NBCC 1995 are usually not enforceable on an existing building unless the building is undergoing major changes, such as changes in occupancy or structural alterations, which have impact on seismic performance. Although Commentary K in the NBCC 1995, (which legally is not part of the code) addresses the concerns of structural evaluation and upgrading of an existing building to achieve a level of performance as per the intent of the code, it does not specify the circumstances which would require a structural evaluation of an existing building.

According to Commentary K in the NBCC 1995:

- Part 4 (including seismic requirements) is written primarily for the design of new buildings or new additions, and not for the evaluation and upgrade of existing buildings.

- Buildings designed and built in accordance with previous codes may be considered acceptable provided that (a) the building or its use is not altered in such a way as to affect its structural behaviour or to increase the loading on the structure, and (b) the previous code or standard essentially satisfies the life-safety requirement of the current code or standard. In the case of seismic requirements, building or components designed and built in accordance with the NBCC 1970 may be considered as satisfying the life-safety intent of the current requirement.

Commentary K in the NBCC 1995 also recommends that, should an evaluation be required to assess the seismic performance of an existing building, National Research Council's "Guidelines for Seismic Evaluation of Existing Buildings" [National Research Council 1992] be followed. Both Commentary K and NRC Guidelines suggest the use of a reduced load factor of 0.6 as a trigger criterion for seismic upgrading. If an existing building does not possess the required capacity in resisting 60% of the seismic load as specified in the NBCC 1995, the building should be upgraded to – preferably – 100% of the NBCC 1995 value.

### 1.3.2 Québec

The earthquake live loads specified in section 10.4.1.3 of Gazette Officielle du Québec [Québec, 2000], as well as subsection 4.1.9 of Part 4 of the NBCC 1995, do not apply to existing buildings under alteration when:

- this alteration does not result:
  - in an increase in building height;
  - in the modification of any structural wind-bracing element that ensures lateral stability.
- the building, after alteration, can resist a live load due to seismic forces that is at least equal to 60% of what is specified in subsection 4.1.9 of Part 4 of the NBCC 1995.

### 1.3.3 Ontario

While Part 4 of the Ontario Building Code (OBC) [Ontario 1997] is similar to Part 4 of the NBCC 1995 in terms of seismic requirements for new buildings, the OBC 1997 includes a new section (part 11) specifically for the renovation of existing buildings. The intent of part 11 is two-folded; (a) to ensure that the performance level of the existing building after renovation is no less than the performance level before renovation, thus extending the service life of the building, and (b) to provide a reasonable and acceptable approach to allow renovations. Part 11 applies to buildings that have been in existence for at least five years and are undergoing renovation.

According to Section 11.3.1.1 on "Material Alternation or Repair of a Building System" and Section 11.4.1.1 on "Performance Level," "the performance level of a building after construction (alteration) shall not be less than the performance level of the building prior to construction (alteration)." If after the construction (alteration):

- the major occupancy changes to a different major occupancy;
- the occupancy load increases by more than 15%, or
- live load increases due to change in use within the same major occupancy,

remedial measures shall be taken to maintain the performance level prior to construction. However, section 11.5 allows compliance alternatives. Accordingly, provided the municipality's chief building official is satisfied that it is impracticable to comply with section 4.1.9 on "Live loads due to earthquake", the seismic provisions do not apply.

### 1.3.4 British Columbia

British Columbia's Building Code [British Columbia 1998] adopts the NBCC 1995 with amendments to reflect circumstances unique to British Columbia. BCBC's amendments are mostly related to accessibility and building envelope, with no changes to NBCC 1995's seismic provisions (i.e. no alternations to part 4 of the NBCC 1995).

There are no specific seismic requirements regarding the existing buildings in the Province of British Columbia. Municipalities within the province generally follow the City of Vancouver's requirements on seismic evaluation and upgrade for existing buildings.

### 1.3.5 City of Vancouver

Vancouver uses its own Charter to enact bylaws, which specify requirements for seismic evaluation and upgrade of existing buildings undergoing rehabilitation or major occupancy changes [Vancouver 1999]. Buildings which were originally constructed pursuant to a building permit issued after January 1, 1980, are exempted from these requirements. Under section 10.2.4 "structural upgrading of buildings," existing buildings shall be considered for seismic upgrading when:

- the renovation cost of the rehabilitation exceeds 200% of the value of the building as determined by the British Columbia Assessment Authority;
- the work includes a major addition; or,
- the work includes a change of major occupancy.

A detailed structural analysis and the proposed remedial work to an existing building must be submitted to the city for approval. The proposed remedial work is to bring the structure up to the standards required by Part 4 of the by-law (i.e. as per Part 4 of NBCC 1995 on "Live loads due to earthquake").
When the proposed alterations to an existing building:

- cost between 100% and 200% of the actual value of the building as determined by the British Columbia Assessment Authority;
- does not include an addition; or,
- does not include a change of major occupancy,

a structural survey of the existing building, instead of a detailed structural analysis, is adequate. The structural survey includes an evaluation of the building in conformance with the NRC publication "Guidelines for Seismic Evaluation of Existing Buildings" [NRC 1992]. NRC Guidelines do not require an existing building to be upgraded if the building structure has a load carrying capacity equivalent to 60% of that required by the NBCC 1995. If the building has a load carrying capacity less than 60% of NBCC 1995 value, the building should be upgraded to, preferably, 100% NBCC 1995 value. In this case, the structural survey shall include proposed remedial measures to the existing building.

The City may relax the requirements for seismic upgrading of an existing building, but not to less than 75% of the seismic force level as stipulated in the seismic provisions of the City Bylaw for new buildings. One of the major criteria for the relaxation is that, "it can be demonstrated that the total upgrade required to comply with the seismic requirements would be usually difficult to achieve."

### 1.3.6 Federal Jurisdiction

Canada Labour Code is developed and issued under federal jurisdiction for federal properties. For existing buildings, Canada Occupational Safety and Health Regulations – Part II, on "Permanent Structures" are made pursuant to the Canada Labour Code – Part II. The purpose of the Labour Code is to prevent accidents and injuries linked with, or occurring during, the course of employment in a federal jurisdiction. Part II – Division I on "Buildings" states that "the renovation of any building or part of a building shall, to the extent reasonably practicable, meet the requirements of the National Building Code."

The "reasonably practicable" clause allows certain degree of flexibility regarding the requirements for the seismic upgrading of existing buildings. Should seismic upgrading of a building be deemed necessary, the extent or level of the upgrading is determined not only by the NBCC 1995 provisions, but also by considering whether the required work is economically, technically, and socially acceptable or feasible. Table 2 lists various codes and regulations that pertain to the seismic requirements for existing buildings. Also included in Table 2 is the NBCC 1995's Commentary K on structural evaluation and upgrading of existing buildings.

**Table 2** Summary of Seismic Requirements for Existing Buildings

| Jurisdiction | Seismic requirements for existing buildings |
|---|---|
| Québec | • similar to NBCC 1995 and Commentary K for existing buildings<br>• Seismic upgrading (compliance to NBCC 1995 seismic requirements) is not required for an existing building undergoing alteration if: (a) the alternation does not result in an increase in building height and in the modification of any structural wind-bracing element that ensures lateral stability, and (b) the building, after the alteration, can resist a live load due to seismic forces that is at least equal to 60% of what is specified in subsection 4.1.9 (Part 4 NBCC 1995). |
| Ontario | • part 11 for existing buildings undergoing renovation<br>• performance level after construction is no less than performance level prior to construction<br>• Compliance alternatives allows for exemption of seismic requirements, subject to approval by the municipality's chief building official. |
| British Columbia | • no specific seismic requirements regarding existing buildings<br>• Municipalities within the province generally follow the City of Vancouver's requirements on the seismic evaluation and upgrading for existing buildings. |
| Vancouver | • requires seismic upgrading of an existing building to NBCC 1995 value when: (a) the total cost of rehabilitation exceeds 200% of the value of the building, excluding the land value, (b) the work includes a major addition, OR (c) the work includes a change of major occupancy<br>• requires seismic upgrading of an existing building to NBCC 1995 value when the existing building has less than 60% of the seismic resistance as required by the seismic provisions of the City Bylaw for new buildings, and that: (a) the total cost of rehabilitation exceeds 100% but does not exceed 200% of the actual value, OR (b) work does not include an addition or change of major occupancy<br>• does not require seismic upgrading of an existing building when the existing building has at least 60% of the seismic resistance as required by the seismic provisions of the City Bylaw for new buildings, and that: (a) the total cost of rehabilitation exceeds 100% but does not exceed 200% of the actual value, OR (b) work does not include an addition or change of major occupancy |
| Federal Jurisdiction | • The renovation of an existing building shall, to the extent reasonably practicable, meet the requirements of the NBCC 1995 (as per Canada Occupational Safety and Health regulations Part II on Permanent Structures).<br>• The above requirements are made pursuant to the Canada Labour Code. |
| NBCC 1995 Commentary K | • Part 4 was written primarily for the design of new buildings or new additions.<br>• Buildings designed and built in accordance with 1970 or later versions of NBCC may be considered acceptable provided that the building or its use is not altered in such a way as to affect its structural behaviour or to increase the loading on the structure (Commentary K).<br>• 0.6 or 60% of NBCC 1995 value can be considered as the triggering criterion for seismic upgrading of an existing building: buildings with seismic load carrying capacity of less than 0.6 NBCC 1995 value should be upgraded, preferably to 100% of NBCC 1995 value. |

### 1.4 Summary

The preceding section has provided a brief review of the variances in applicable codes and regulations pertaining to the seismic protection of existing Canadian buildings. Building code requirements for seismic lateral forces for high seismic areas in Canada have increased by as much as 100% since the early 1970's. In the event of moderate to strong earthquakes, buildings designed and built in accordance with previous building codes are not expected to provide the same level of safety as newer buildings, which were designed and built in accordance with the more stringent requirements defined in more recent codes. Previous codes and regulations were less stringent than the current requirements for construction of a new building. An obvious and urgent concern to the seismic protection community remains: the challenge of dealing with a large stock of older and potentially vulnerable existing buildings. It is clear that earthquake preparedness of Canadian communities, as well as the sustainability of such communities through increased readiness, requires proactive rather than reactive codes and regulations. These in turn would not only improve the performance of new and existing buildings in Canada, but also trigger the requirement for seismic upgrade of a particular building based on performance deficiencies rather than the cost of rehabilitation or changes in building occupancy.

## 2.0 Part B – Seismic Upgrading Technologies for Buildings: A Review

### 2.1 Introduction

Many existing buildings, constructed according to older codes, lack adequate seismic resistance, and may pose severe life safety hazard during seismic events. These buildings were primarily designed for gravity loads and were often inadequately detailed to resist seismic forces. Typical deficiencies of older reinforced concrete buildings, especially those built before 1970, include columns with insufficient shear strength, lack of confinement in flexural hinge zones, inadequate lap splices for longitudinal reinforcement, strong beam-weak column structural systems, and beam-column joints with inadequate shear resistance.

Past earthquakes, including the 1971 San Fernando earthquake, the 1989 Loma Prieta earthquake, the 1994 Northridge earthquake in California, the 1995 Kobe earthquake in Japan, the 1999 Turkey earthquakes, and the 1999 JiJi earthquake in Taiwan, caused severe damage to, or collapse of, many buildings designed according to older codes. A large number of severely damaged buildings were located in areas where moderate seismic ground motions were measured, which indicated that the resistance of older buildings is not sufficient even for moderate seismic excitations. Given this experience, comprehensive retrofitting programs were undertaken in countries with high seismic hazards, especially in Japan and the U.S.A., particularly California.

The lessons from past earthquakes for the behaviour of existing buildings are very important for Canada due to the following two reasons. First, strong earthquakes can occur in Canada. The west coast of British Columbia and the St. Lawrence valley in eastern Canada, where a large population is concentrated, are known to be seismically active [Associate Committee of the National Building Code, 1995 (NBCC 1995)]. Second, it is well recognized that the seismic resistance of a large majority of existing buildings, especially those built before 1970, may be inadequate even for much smaller seismic motions than those prescribed by the current building code (NBCC 1995). Therefore, in order to mitigate the risk posed by older buildings and to provide safety to building occupants, seismic retrofit (i.e. rehabilitation or strengthening) of these buildings is needed.

A seismic retrofit program for a given region consists of three major phases:

- screening/prioritization to determine the necessity of carrying out a more detailed evaluation of the seismic performance of buildings,
- detailed evaluation in determining the extent and severity of seismic deficiency of individual buildings, and
- selection of (or designing) appropriate retrofitting techniques for different types of buildings.

The objective of a screening/prioritization scheme is to identify and rank all high-risk buildings in a specified region so that an optimum allocation of resources for evaluation/retrofit can be made. Major parameters that have effects on the risk are; the seismicity of the location, vulnerability, and importance of the building structure. A methodology for screening and development of prioritization scheme for building structures has been developed by the Institute for Research in Construction of the National Research Council of Canada (1993). As for the second stage for a seismic retrofit program, the National Building Code of Canada (NBCC 1995) and few existing technical guidelines are often used to conduct a detailed assessment of the seismic performance of a building. In terms of the third and final stage (i.e. development of retrofitting techniques), significant amount of research has been conducted, and various techniques have been developed. While the majority of these techniques have been developed specifically for bridges, they can be also used for building structures.

The objective of this report is to present state-of-the-art retrofit techniques for building structures. Techniques available for retrofitting reinforced concrete columns, beams, and beam-column joints are described. In addition, methods of strengthening building structures by using passive damping devices, and base isolation systems are presented.

## 2.2 Upgrading Structural Members Using Fibre Composites

The terms fibre composites, advanced composite materials, or fibre reinforced polymer (FRP) materials, are generally applied to synthetic fibre materials such as fibreglass, carbon fibres, and aramids embedded in a resin matrix (epoxy or ester). Fibre composites generally possess higher strength to weight ratio than conventional construction materials such as steel.

These materials have been primarily developed for use in aerospace and defence industries. As the material cost of fibre composites decreases and the demand for more effective and durable construction material increases, wider use of these advanced composite materials in civil structures is expected. Recent research and development efforts have led to many applications of composite materials for strengthening existing reinforced concrete structures. Externally bonded FRP plates were introduced in Germany and Switzerland in the mid-1980's as an alternative to strengthening reinforced concrete beams with steel plates (Nanni, 1995). Ease of construction and broader applications have made fibre composite sheets a more popular choice over plates. While plates are more appropriate for flat surfaces and beams, sheets can be used on round (such as columns) and larger (such as walls) surfaces more efficiently and effectively. The focus of this review is on the use of fibre composite sheets on the strengthening of reinforced concrete elements.

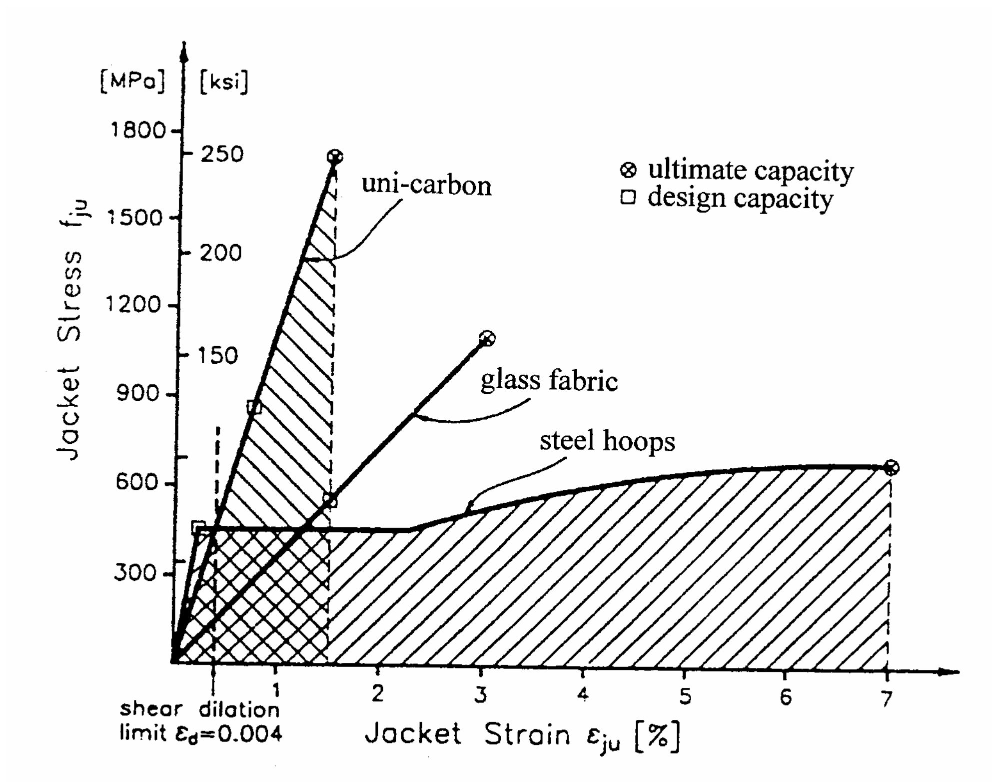
The primary load-carrying element within a composite is the fibre. Consequently, the fibre has a strong influence on the mechanical characteristics of the composite such as strength and elastic modulus. The resin provides a mechanism for the transfer of load among the fibres. It also protects the fibres from abrasion and other environmental and chemical attacks. The fibres can be oriented in a single direction (unidirectional) or several directions to optimize the performance of the composite.

There is large variation in the mechanical properties of composite materials. These properties are related to those of the fibre used in the composite, and to the fibre volume ratio (i.e. the ratio of the volume of the fibres to the total volume of the composite). In general, carbon fibre composites are characterized by larger strength and stiffness than glass fibre composites. Table 3 summarizes key properties for common composite materials. Figure 1 shows typical stress-strain relationships for carbon fibre and glass fibre composite materials. It can be seen from this figure that both of these composites are characterized with linear stress-strain relationships.

**Table 3** Properties of advanced composite materials *(Priestley et al., 1996)*

| Material | Modulus of Elasticity (GPa) | Ultimate Tensile Strength, $f_u$ (MPa) | Ultimate Strain, $\varepsilon_u$ (%) |
|---|---|---|---|
| Fibres | | | |
| Carbon | 160 - 270 | 1400 - 6800 | 1.0 - 2.5 |
| Aramid (Kevlar 29) | 62 - 83 | 2800 | 3.6 - 4.0 |
| Glass | 81 | 3400 | 4.9 |
| Polyethylene (Spectra 900) | 117 | 2600 | 3.5 |
| Resin | | | |
| Epoxy | 2.0 - 4.5 | 27 - 62 | 4 - 14 |
| Vinylester | 3.6 | 80 | 4 |

**Figure 1** Typical mechanical characteristics of column jackets in hoop direction *(Seible et al. 1997)*




### 2.2.1 Upgrading Technique

Fibre strengthening technique by wrapping an element with fibre composite sheets is a relatively simple process. While installation of the composite strengthening system may vary among various manufacturers and installers, the process generally involves the following steps:

- Inspect surface condition of the member that needs to be strengthened.
- Repair cracks and spalled regions with epoxy injection and mortar.
- Prepare surface of member (with hand grinders and wet blasting if necessary) to remove projections and to ensure a proper profile.
- Apply primer and putty to ensure good adherence of the fibre sheets.
- Apply first coat of saturant.
- Apply fibre sheets on to the surface in a manner that is similar to hanging wallpaper.
- Apply second coat of saturant after the sheets have been properly cured, usually within an hour.
- Repeat steps 6 and 7 until the required number of layers of fibre sheets has been installed.

### 2.2.2 Columns

A large amount of experimental research has been performed to determine the effectiveness of column retrofit using jackets of fibre composites. Figure 2 shows retrofit of rectangular and circular columns with fibre composite jackets. In general, tests on circular columns retrofitted with composite-material jackets have resulted in improved confinement, ductility, and shear strength. Experimental results of the effects of fibre composites on flexure, shear, and lap-splice debonding behaviour of columns are reviewed.

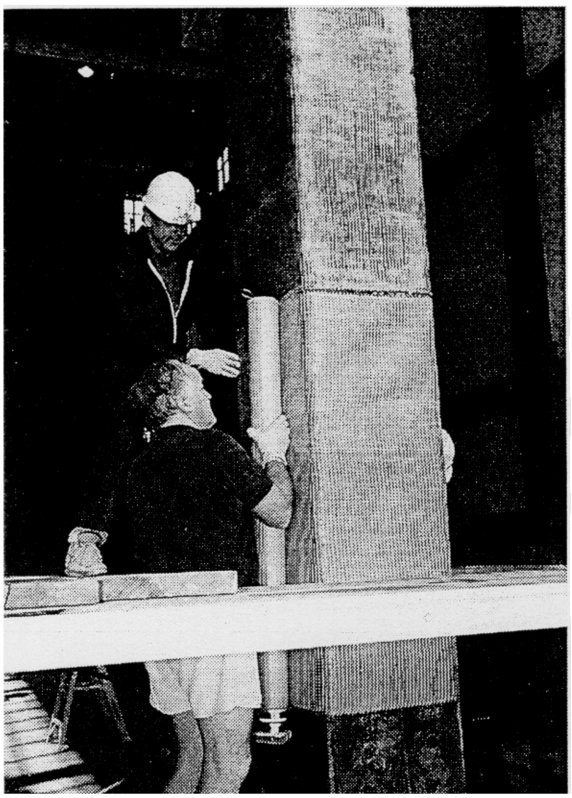
Among a number of experimental investigations (Priestley et al. 1996; Seible et al. 1997; Saadatmanesh et al. 1996, 1997a, 1997b), some results from the study conducted by Seible et al. (1997) on carbon fibre jackets are presented here. Three sets of 45% scale-model bridge columns were tested by the researchers to evaluate the effectiveness of carbon fibre jackets for the three possible column failure modes (i.e. due to flexure, shear, and lap splice de-bonding). Each set consisted of two specimens, one representing "as-built" column, and another one representing retrofitted column. Results of some of these tests are illustrated in Figures 2 to 4, which show the effects of column retrofit on flexure, shear, and lap splice de-bonding behaviour.

*Flexure:* The lateral load-displacement responses (i.e. hysteresis loops) of "as-built" and retrofitted columns, obtained from horizontal cyclic loading applied at the top of columns, are shown in Figures 3(a) and 3(b), respectively. Figure 3(b) shows that the carbon jacket significantly increased the ductile capacity of column. While the maximum displacement ductility ratio of the "as-built" column is 3 (point "d" in Figure 3(a)), the retrofitted column shows maximum ductility of about 7 (point “g” in Figure 3(b)) without any cyclic capacity degradation. By considering the areas enclosed by the hysteresis loops of the "as-built" and retrofitted column, it can be seen that the carbon jackets provided a large enhancement in energy absorption capacity of the column.

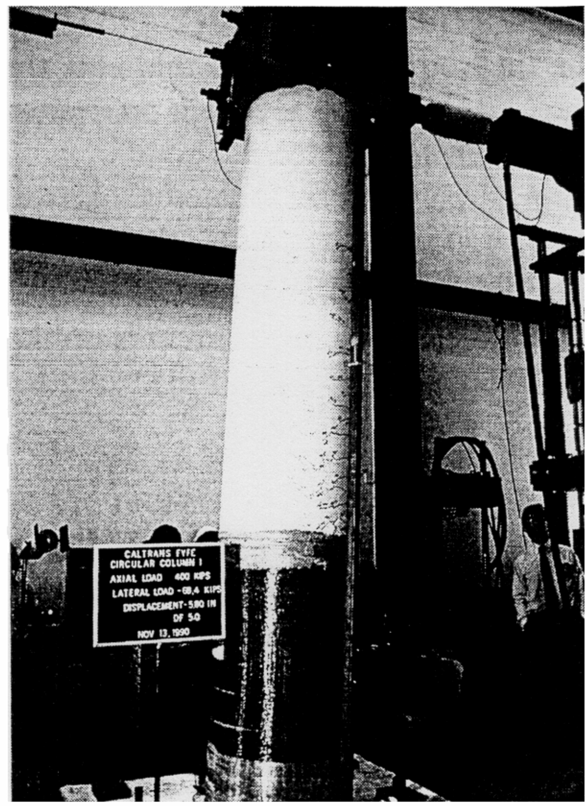
*Shear:* The hysteresis loops from the tests of "as-built" and retrofitted columns (in double bending) are shown in Figures 4(a) and 4(b), respectively. The effectiveness of the carbon fibre jacket is very obvious from these figures. As can be seen from Figure 4(b), the jacket provided ductile behaviour of the column even at relatively large inelastic deformations. The retrofitted column experienced maximum displacement ductility of about 10 (point "h" in Figure 4(b)) without any strength degradation. This ductility is more than three times larger than that of the "as-built" column (ductility of 3 as per point “c” in Figure 4(a)). The shapes of the hysteresis loops of the retrofitted column also clearly indicate that the carbon jacket provided a substantial energy dissipation capacity of the column. It is obvious from these results that properly designed carbon fibre jackets can prevent shear failure and provide improved ductile behaviour of retrofitted columns.

*Lap-Splice Clamping:* Figure 5 shows the hysteresis loops obtained from the tests of retrofitted column. The loops for the "as-built" column are not included in this figure since they are not given in Seible et al. (1997). However, from the load displacement envelopes shown by Seible et al. (1997) [see Figure 6(d) in the paper] it can be observed that the "as-built" column failed at a very low deformation (i.e. displacement ductility ratio of approximately 1). On the other hand, Figure 4 shows that the carbon fibre jacket provided ductile behaviour with stable hysteresis loops up to a ductility ratio of 10, indicating that this kind of retrofit is well effective for columns with inadequate lap splices.

**Figure 2** Retrofitting with composite-materials jackets (Priestley et al. 1996); (a) Retrofitting rectangular column: hand-layup; (b) Retrofitted circular column-base lap splice region.



(a)




(b)

**Figure 3** Lateral load-displacement response of "as-built" and retrofitted column for flexure (Seible et al. 1997); (a) "As-built" column; (b) Carbon fibre jacketed column.

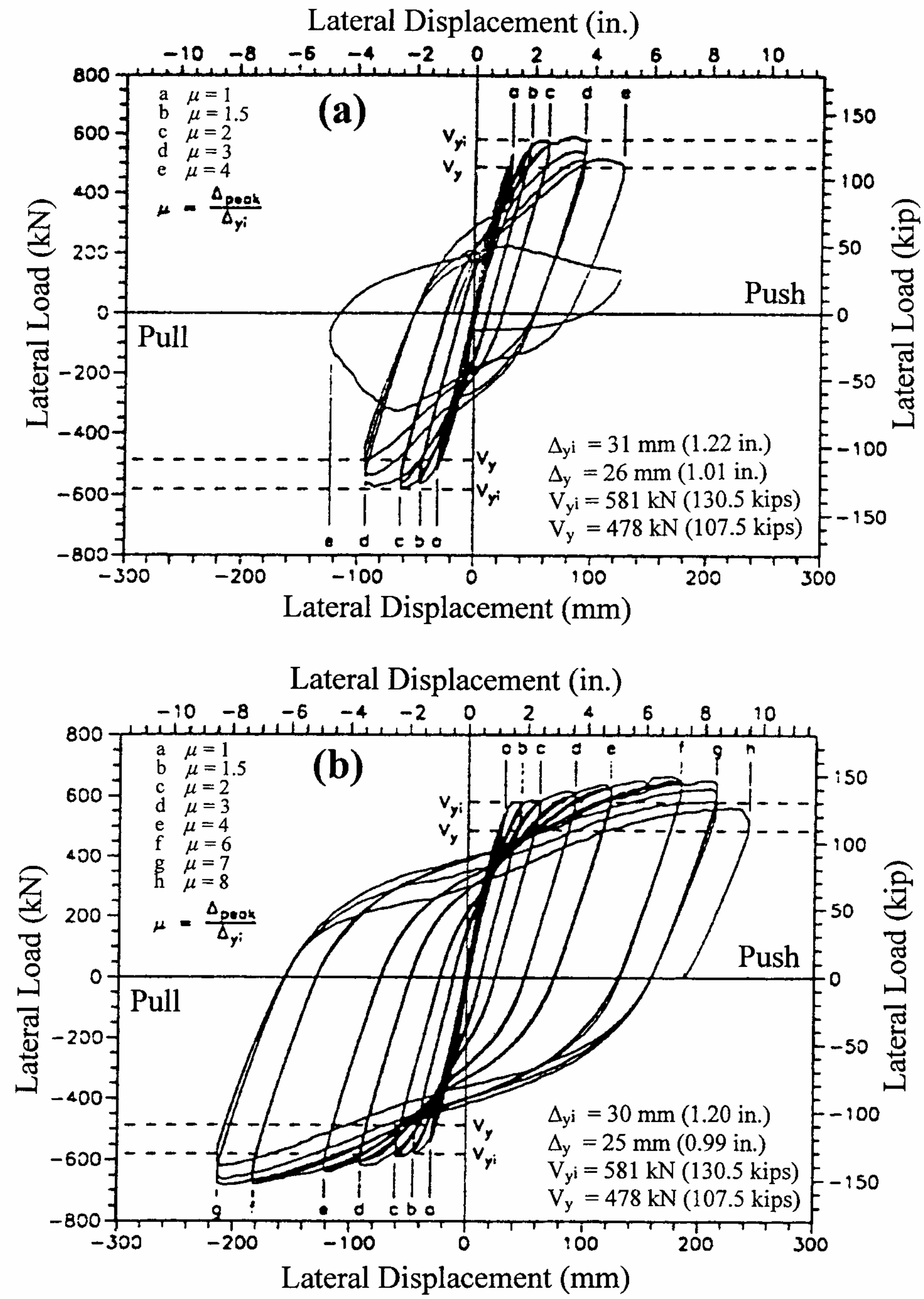


**Figure 4** Lateral load-displacement response of "as-built" and retrofitted column for shear *(Seible et al. 1997)*; (a)"As-built" column; (b) Carbon fibre jacketed column.

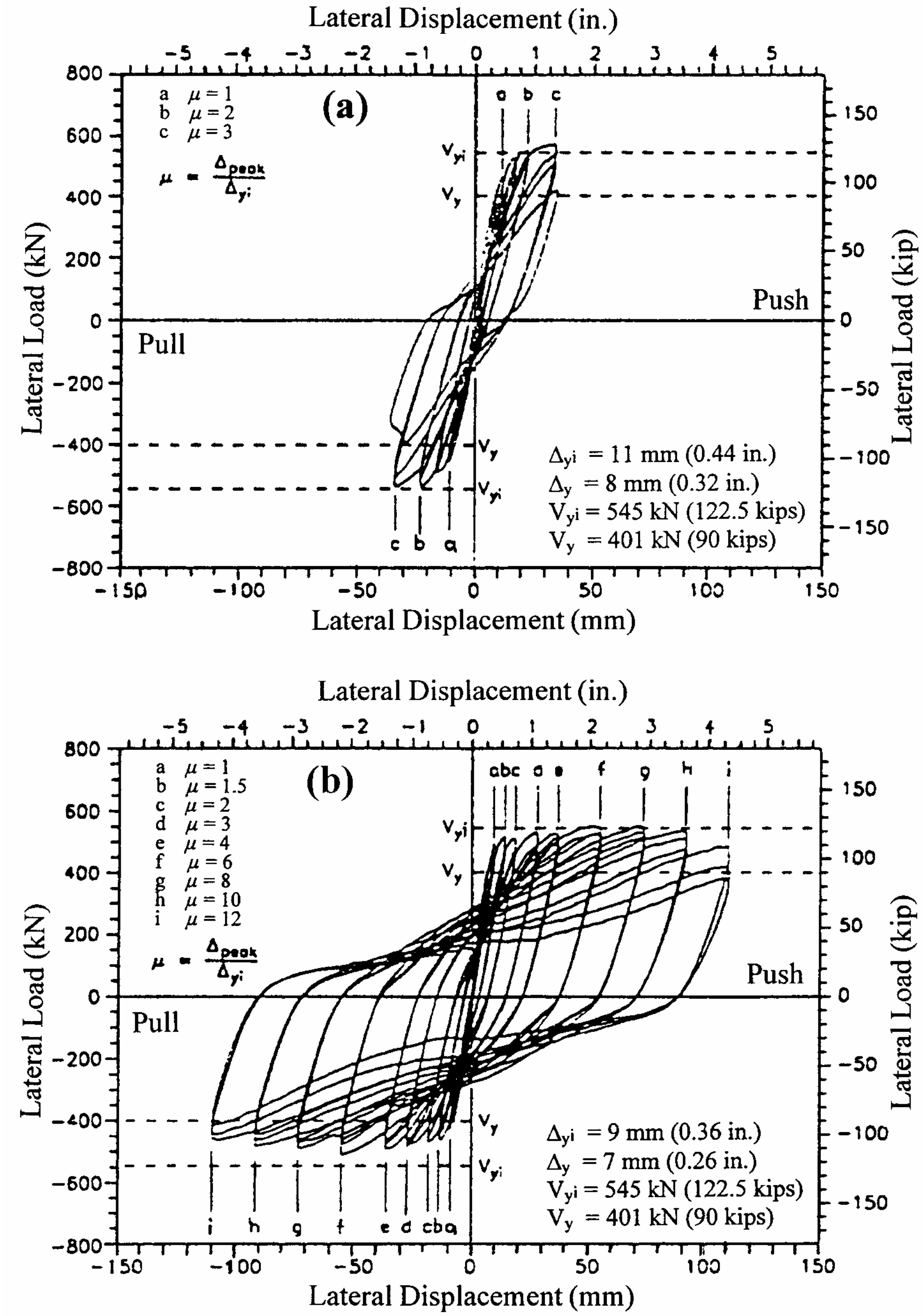
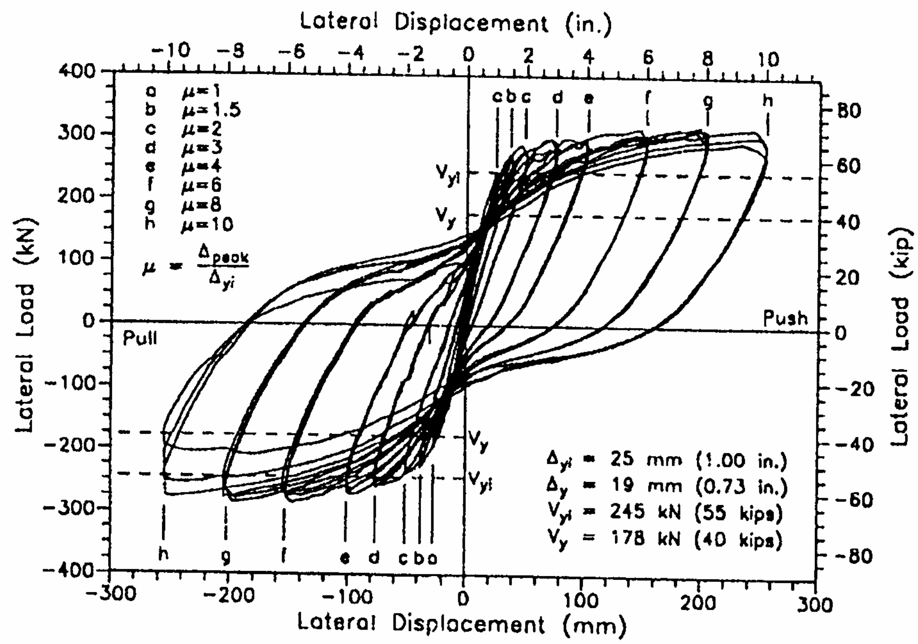


**Figure 5** Lateral load-displacement response of retrofitted column for lap splice clamping *(Seible et al. 1997).*




### 2.2.3 Beams

The initial application of fibre composites as seismic retrofit material was in beams. During the mid-1980's, beams of building structures were rehabilitated in Germany and Switzerland using fibre composites (Nanni, 1995). Externally bonded fibre reinforced polymer (FRP) plates were used as a replacement for steel plates to strengthen reinforced concrete beams. Since then, various types of fibre composites were manufactured and new techniques were developed for retrofit of concrete beams. In addition to FRP plates, wrapping with FRP flexible sheets was also employed.

A review of the literature indicated that a large amount of experimental research was conducted during the 1990's to gain an understanding of the behaviour of strengthened beams with FRP plates and sheets. Strengthening for both flexure and shear were investigated extensively (Arduini and Nanni 1997; Mukhopadhyaya et al. 1998; Buyukozturk et al. 1998; Khalifa et al. 1998; Spadea et al. 1998). However, in all experimental studies, only static loads were applied.

*Flexure:* Retrofit for flexure consists of bonding FRP plates or sheets on the tension face of beams. The length of the plate (or sheet) is normally the same or slightly shorter than the length of the beam. Similarly, the width of the plate is normally the same or somewhat narrower than the width of the beam. Experimental studies show that while the moment capacity can be increased by strengthening for flexure, this can trigger a shear failure as a result of the increased flexural strength.

The failure due to shear crack at the end of the plate and the failures due to debonding of the plate are most common failure modes. To avoid these failures, the FRP plates (or sheets) should be properly anchored. Figure 6 shows three types of anchorage systems that were used in the experimental studies conducted by Spadea et al. (1998), Muchopadhyaya et al. (1998), and Arduini and Nanni (1997). The anchorage system used by Arduini and Nanni (1997), shown in Figure 6, is employed by wrapping the three sides of the beam by two FRP sheets (i.e. U-wraps). In addition to the anchorage of the FRP sheet along the bottom face of the beam, this system also increases the shear strength of the beam.

*Shear:* While the flexural capacity can be increased by the use of a proper anchorage system, the resistance of the beam can be limited by its shear capacity. Therefore, when a given flexural member needs to be strengthened, the combined effects of strengthening for both flexure and shear should be considered. Experimental studies have shown that FRP sheets are also effective in enhancing the shear strength of reinforced concrete beams. A variety of FRP shear reinforcement configurations are available. Figure 7 shows several of these configurations. There are options to decide which surfaces are to be used for bonding [Figure 7(a)], whether to use continuous reinforcement or series of strips [Figure 7(b)], and whether mechanical anchorage is necessary [Figure 7(e)]. In addition, since the FRP is an anisotropic material (that is, characterized by its high strength in the direction of fibres), the fibres may be oriented in directions that are best suited for reinforcing against shear cracks [Figure 7(c)]. Alternatively, it may be beneficial to create pseudoisotropy by orienting the fibres in two perpendicular directions [Figure 7(d)].

Figure 8 shows some results from experimental investigations of the effectiveness of carbon FRP sheets for strengthening reinforced concrete beams. The dimensions and reinforcement of the specimens are shown in Figure 8(a). Load versus mid-span deflection curves for the "as-built" (without strengthening) and strengthened specimens are illustrated in Figure 8(b). The bottom curve (M1) corresponds to the "as-built" specimen. The three curves (MM2, MM3, and MM4) are for specimens strengthened for *flexure*, by FRP sheets placed at the bottom faces of beams without any anchorage. The top curve (MM5) corresponds to the specimen that was strengthened for flexure with FRP sheets placed at the bottom face, and U-shaped FRP sheets placed to cover the bottom as well as vertical faces of the specimen [Figure 7(e)]. The U-shaped FRP sheets increase flexural strength while providing anchorage against debonding of the bottom sheets and also strengthening for *shear*. It can be seen from Figure 8 that a properly strengthened beam with FRP sheets, such as that for specimen MM5, provides a large increase in flexural capacity.

**Figure 6** Three types of anchorage used in experimental studies:

(a) By steel U-channels and strips *(Spadea et al. 1998);*
(b) By steel U-channels and bolts *(Mukhopadhyaya et al. 1998);*
(c) By FRP sheets wrapped around the three sides of the beam *(Arduini and Nanni, 1997).*

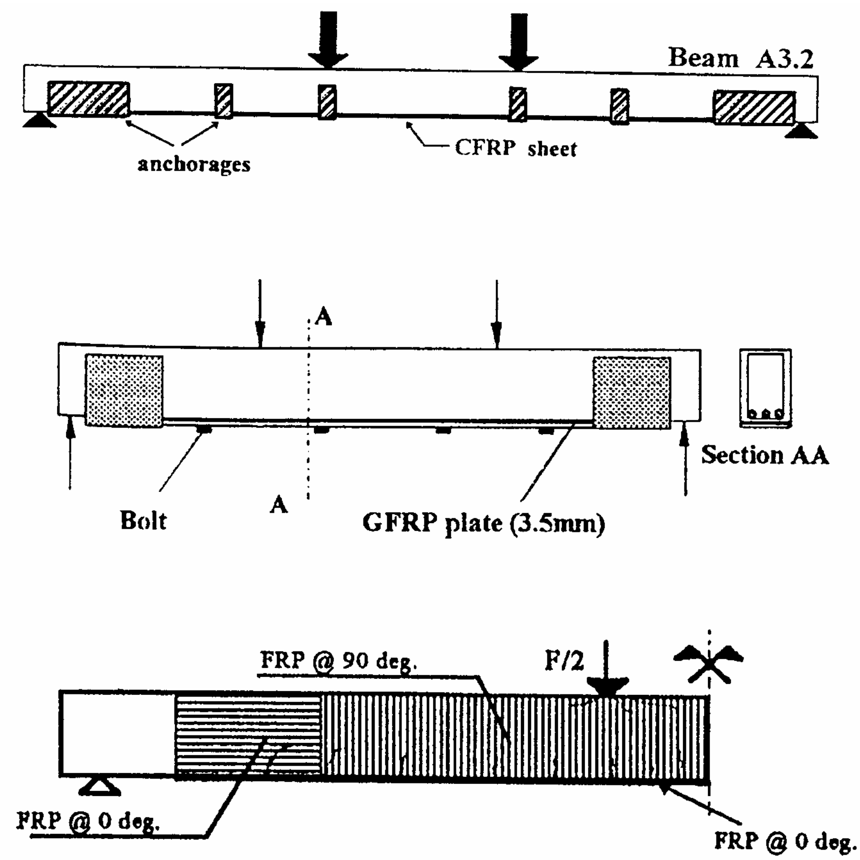


**Figure 7** FRP shear reinforcement configurations *(Khalifa et al. 1998)*

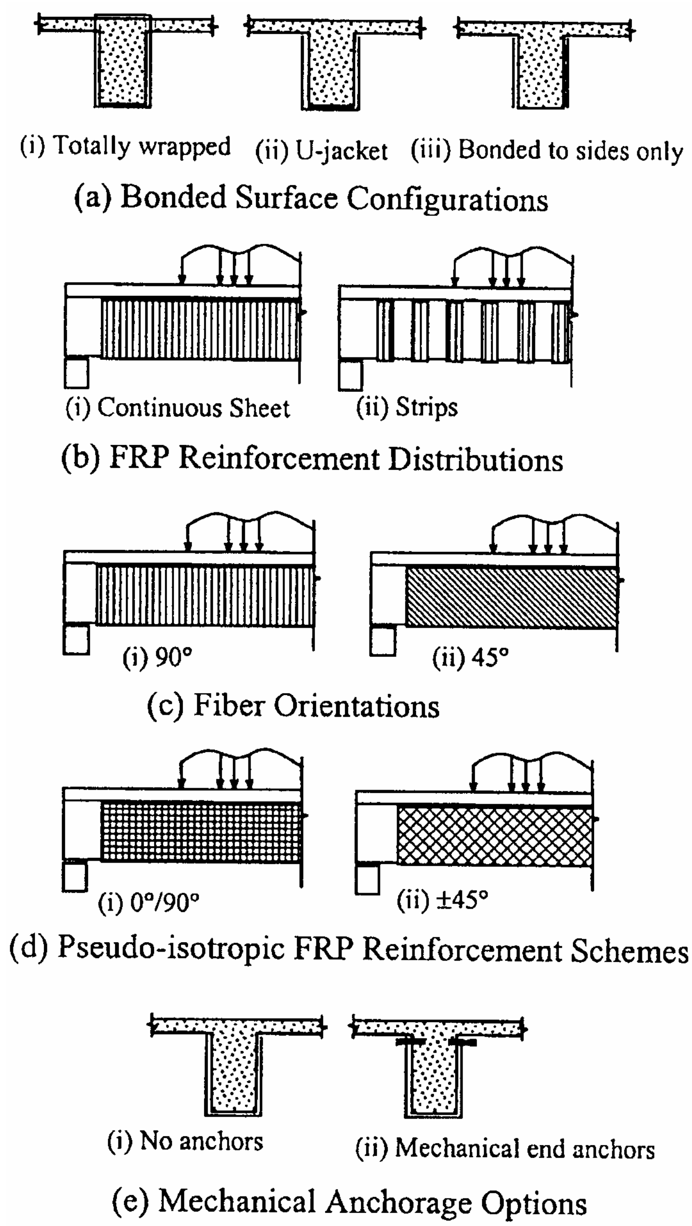
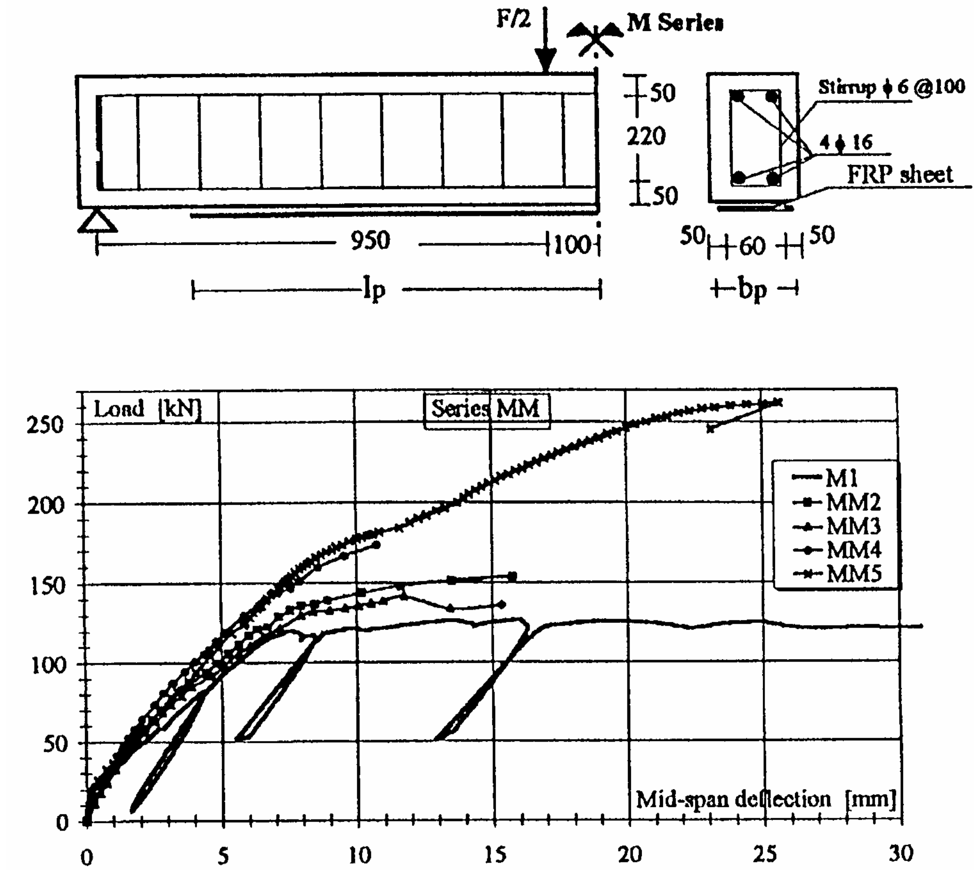


**Figure 8** Specimen dimensions and load versus midspan deflection curves
*(Arduini and Nanni, 1997)*




### 2.2.4 Beam-Column Joints

Investigations on retrofit of beam-column joints using FRP sheets are very limited. Very recently, Pantelidis at al. (2000) reported results from an experimental study conducted at the University of Utah, Salt Lake City. Reversed cycling loading was applied to two half-scale specimens, which were typical of 1960's construction with inadequate confining reinforcement. One as-built specimen and one retrofitted with FRP composite were tested to investigate the effectiveness of the FRP strengthening for shear.

There is no transverse reinforcement within the joint core, and the longitudinal bars of the beam are not adequately anchored in the connection. During the load test, the column was subjected to an axial load to simulate the gravity load, and reversed cycling load was applied to the beam at the free end. The second specimen was retrofitted using FRP sheets to improve the shear capacity and ductility of the joint (Figure 9). The load-drift hysteretic relationships from cyclic tests of as-built and retrofitted specimens are shown in Figures 10(a) and 10(b), respectively. It is apparent that the retrofitted specimen performed significantly better than the as-built specimen. The maximum drift for the retrofitted specimen is approximately 7%, which is two times larger than that of the as-built specimen. However, although the retrofit increases the drift capacity, the hysteresis loops of the retrofitted specimen show significant pinching (Figure 10(b)), which is not desirable.

**Figure 9** Composite retrofit layout *(Pantelidis et al. 2000)*

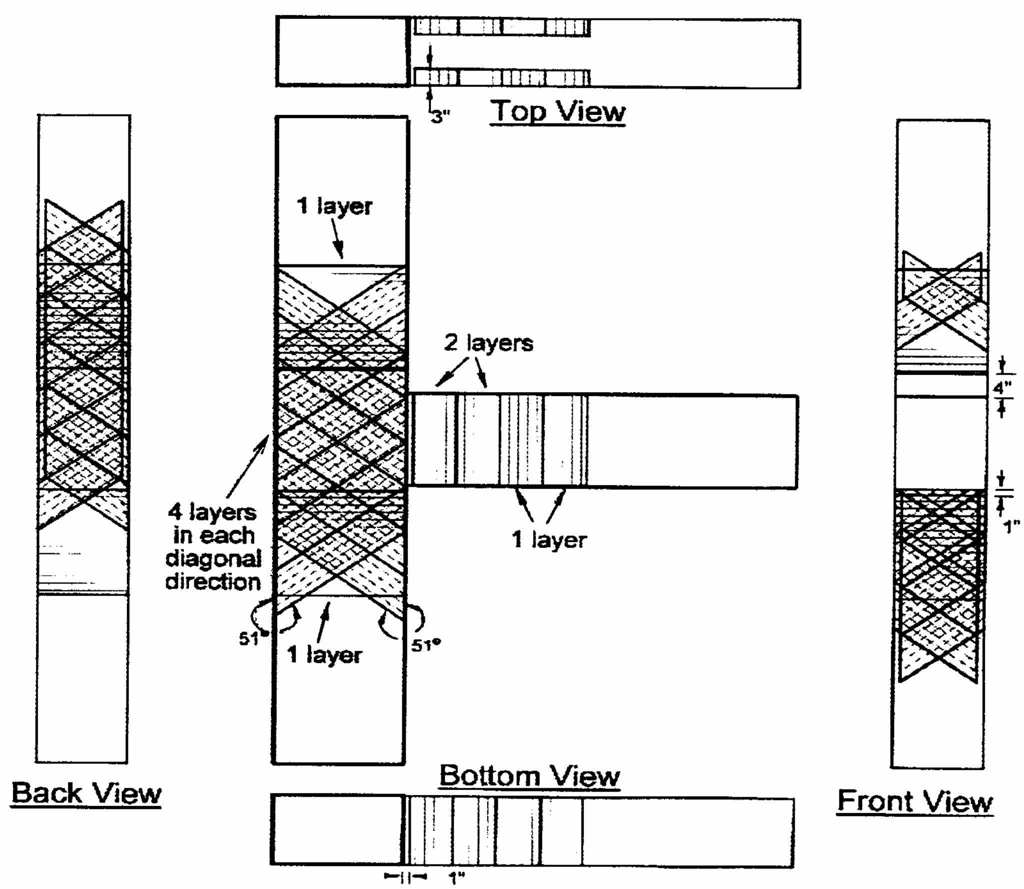
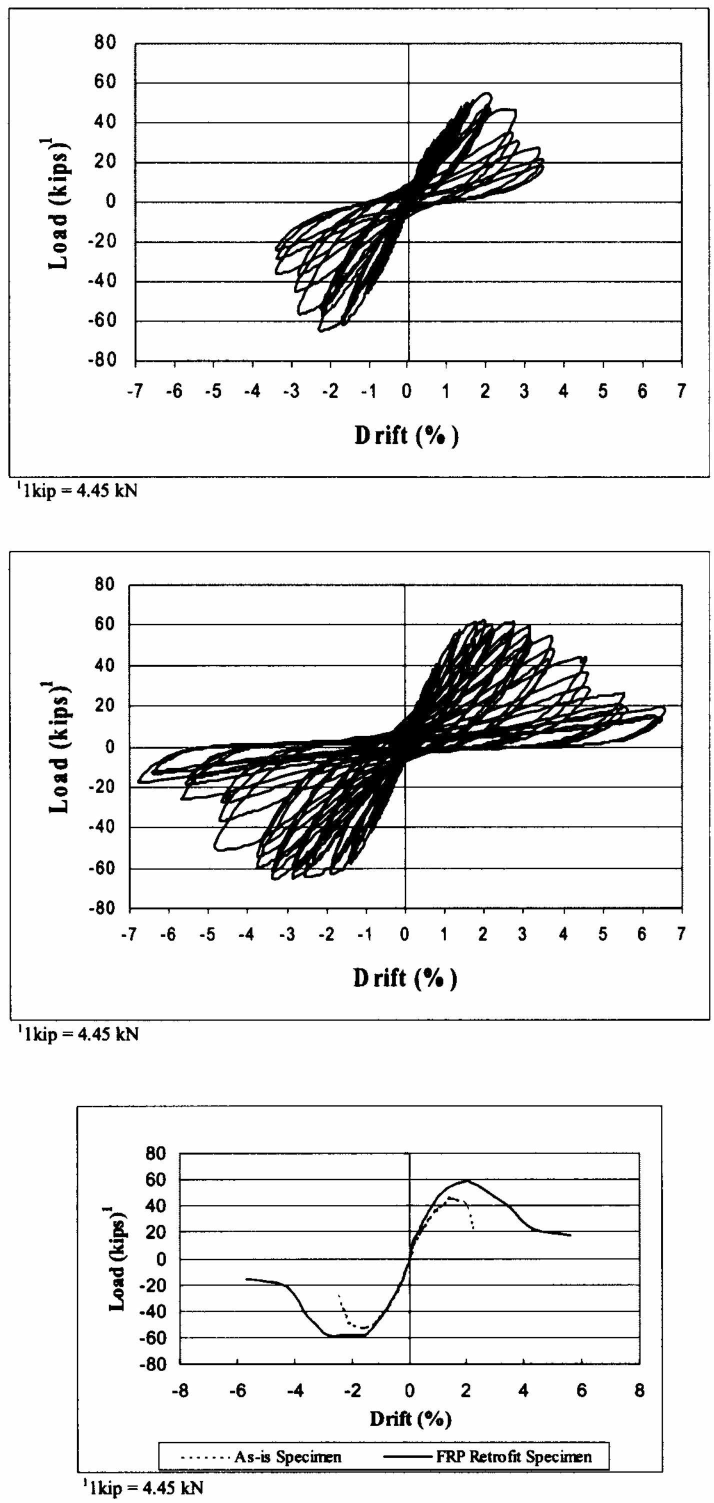


**Figure 10** Experimental results for as-built and retrofitted specimens:
(a) Load-drift hysteretic response for as-built specimen,
(b) Load-hysteretic response for retrofitted specimen, and
(c) Backbone curves *(Pantelidis et al. 2000).*



### 2.2.5 Shear Walls

Preliminary investigation on the use of fibre composites for seismic strengthening of reinforced concrete shear walls was carried out at Carleton University and the federal government's Department of Public Works and Government Services Canada (Lombard et al, 2000). Four series of tests were carried out on half-scale reinforced concrete shear wall specimens, including a control wall, a repaired wall (repairing the control wall which has been previously tested and damaged), and two strengthened walls. The first strengthened wall consisted of application of one vertical layer of carbon fibre sheet on both sides of the wall. The second strengthened wall consisted of the application of one horizontal and two vertical layers of carbon fibre sheets on both sides of the wall. All wall specimens had identical dimensions of 2.0x1.5x0.1 m, design details and material properties. Figure 11 shows the test set-up and Figure 12 illustrates the anchoring system for carbon fibre sheets at the base of wall specimens.

Test results showed the effectiveness of repairing and strengthening reinforced concrete shear walls using fibre composite sheets. Figure 13 presents load-displacement characteristics of four wall tests. The yield loads (and yield displacements) for the control wall, repaired wall, strengthened wall #1, and strengthened wall #2 are 122 kN, 158 kN, 153 kN, and 210 kN (3.7 mm, 5.4 mm, 1.6 mm, and 2.4 mm), respectively. The corresponding ultimate loads are 178 kN, 320 kN, 258 kN, and 413 kN. Figure 11 shows that the displacements at ultimate loads are 18 mm, 40 mm, 24 mm, and 25 mm for the four walls tested, representing ductility ratios of 4.9, 7.4, 15.0, and 10.4.

Strengthening reinforced concrete shear walls with fibre composite sheets appears to be a viable alternative. Both the load-carrying capacity and the ductility of the strengthened and repaired walls improve with the addition of fibre composite sheets on both sides of the walls. However, these preliminary tests also show that the anchorage system between the sheets and the wall footing plays a major role on the effectiveness of this upgrading system. Inadequate anchorage system may not allow proper load transfer from the sheets to the adjacent elements, causing premature failure (peeling or pulling off of the fibre sheets) of the strengthened wall.

**Figure 11** Test set-up for shear wall tests *(Lombard et al, 2000)*

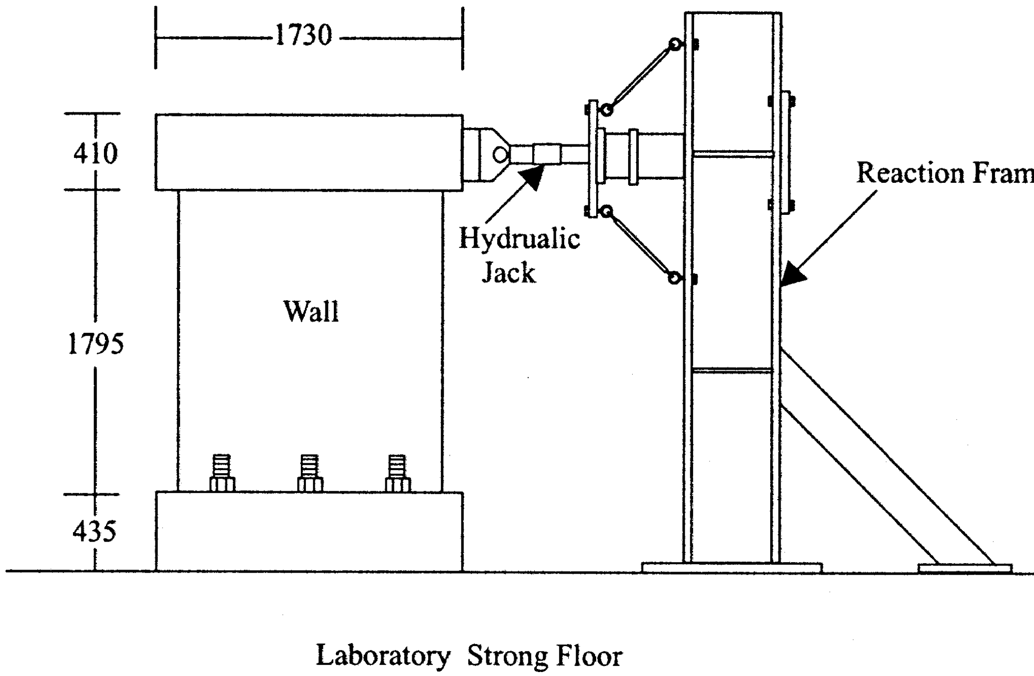



**Figure 12** Anchoring system for the carbon fibre sheets at wall base
*(Lombard et al, 2000)*

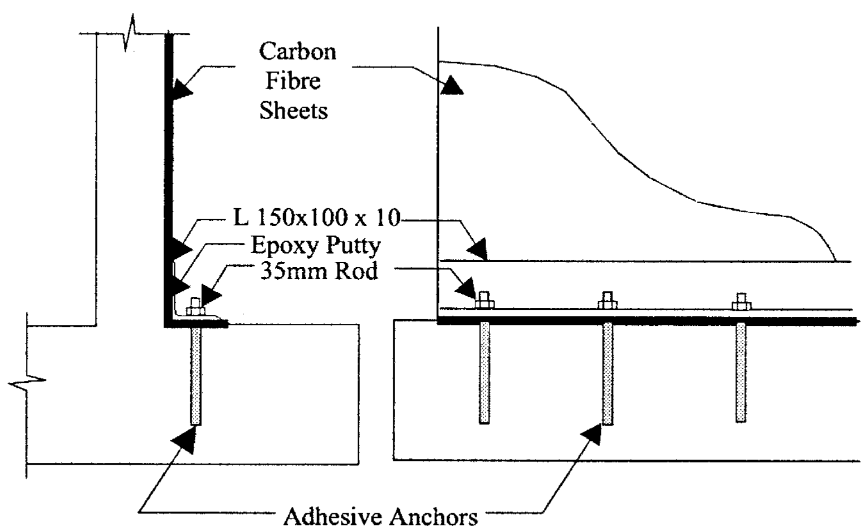
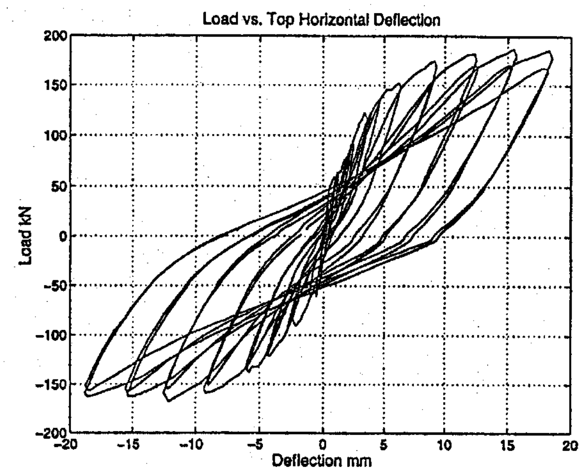


**Figure 13** Load-displacement curves from four wall tests *(Lombard et al, 2000)*

(a) Load-deflection curves for the control wall specimen




(b) Load-deflection curves for the repaired wall specimen

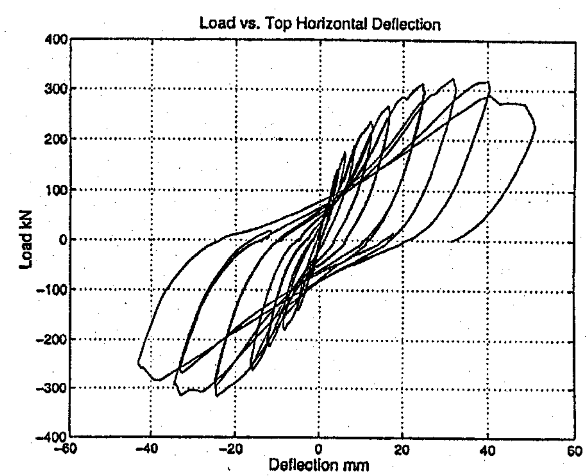
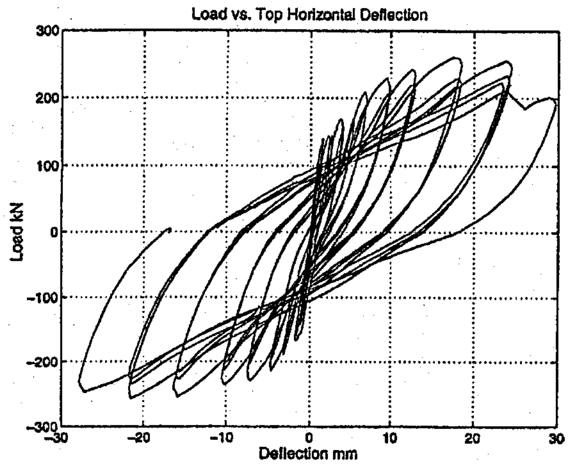
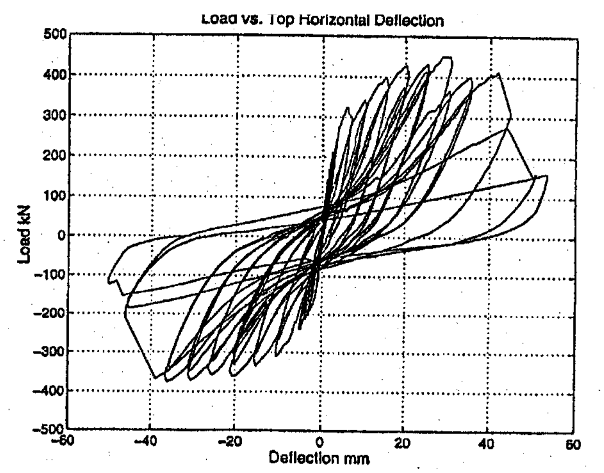


**Figure 13 (Continued)** Load-displacement curves from four wall tests *(Lombard et al, 2000)*

(c) Load-deflection curves for the strengthened wall #1 specimen




(d) Load-deflection curves for the strengthened wall #2 specimen




### 2.2.6 Summary

Fibre strengthening technology is among the most efficient and effective new technologies for seismic retrofits of columns. Its application is rather simple, non-intrusive to building occupants, and not labour intensive – making it one of the more desirable alternatives for seismic retrofitting existing buildings. Carbon fibre's non-corrosive characteristics and resistance to most chemicals give carbon fibre strengthening systems considerably longer life as compared to conventional materials such as steel (i.e. longer economical value over the long run).

While retrofitting bridge structures using fibre composites is rather common and well established, its acceptance in building application is not yet certain. Concerns for its application in buildings include:

- Long term durability in an enclosed interior environment;
- Fire resistance (potential smoke hazard);
- Dynamic behaviour; and
- Beam strengthening, including effectiveness of anchorage due to the presence of floor slabs, and on the effectiveness of strengthening when the beam is subjected to reversed cyclic loading.

The effectiveness of the FRP composites for retrofit of reinforced concrete beam-column joints has not been extensively investigated. Limited experimental results have indicated some improvements in the performance of retrofitted specimens. More comprehensive experimental and analytical research is needed to derive at conclusive recommendation. In addition, practical problems would arise in implementing the retrofit technique due to the presence of floor slabs at beam-column joints.

## 2.3 Upgrading Structural Members by Steel Jacketing

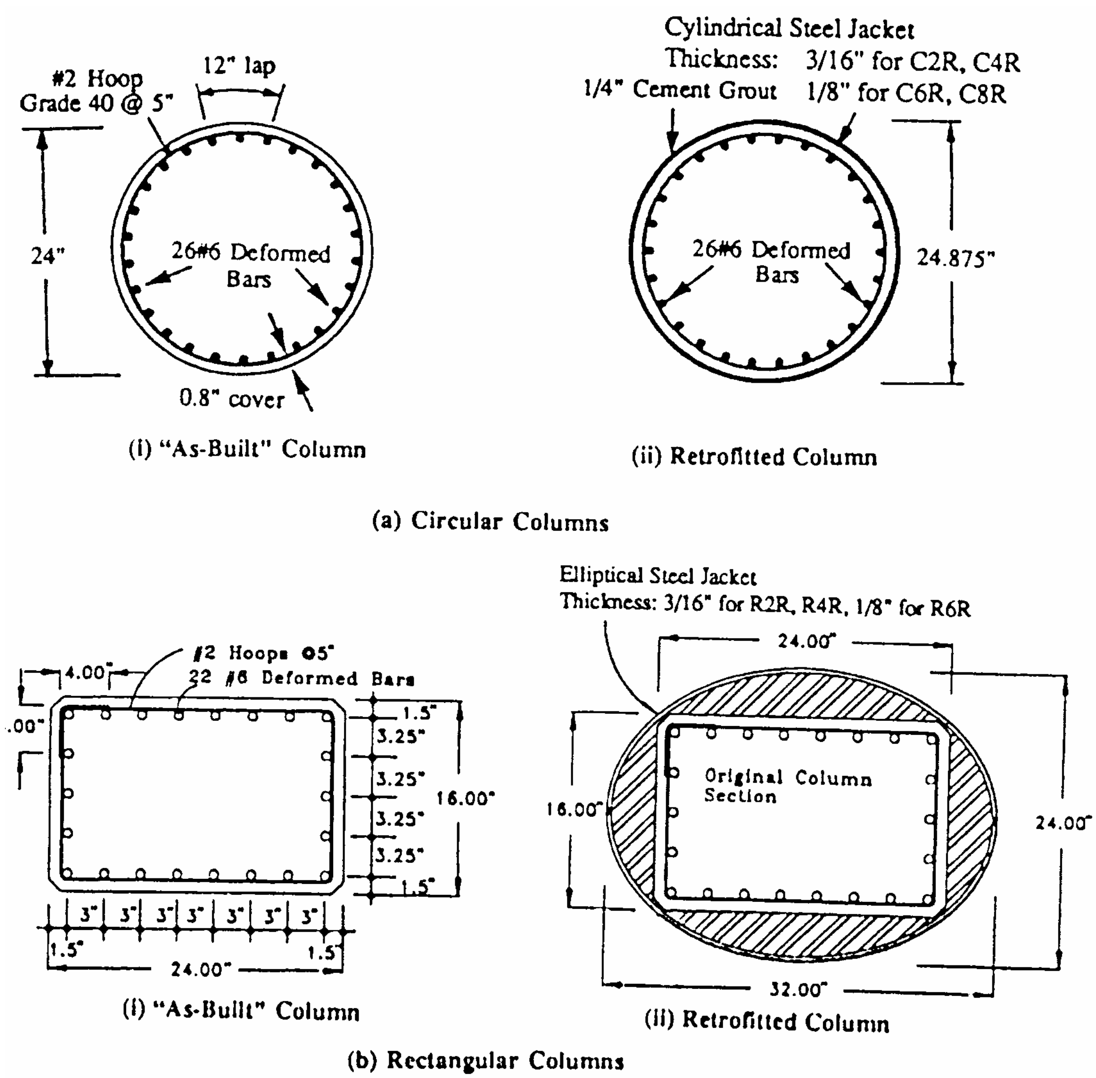
### 2.3.1 Upgrading technique

Steel jacketing has been widely used for seismic retrofit or repair of highway bridge columns. Details of individual steel jackets (such as jacket geometry and grout properties) may vary, although the general procedure and rationale for most steel jacketing systems remain more or less the same.

A deficient circular column is covered with pre-fabricated steel shells (or jacket) which are jointed together either by welding or mechanical connections. The small gap between the jacket and the column, which is usually less than 10 mm, is filled with grout to provide continuity between the jacket and the column. The new column section, which consists of the existing column section plus the new external steel shell, is now stronger (possessing higher load carrying capacity) and stiffer (attracting higher load, which is not desirable) than the original column.

For circular columns, the jackets are constructed in two half-shells slightly oversized for easy installation, which are welded in situ up to the vertical seams. For rectangular columns, the jacket is usually rolled to an elliptical shape, with the larger gaps between casing and column filled with concrete rather than grout (Figure 14). The elliptical shape is needed to provide a continuous confining pressure by passive restraint in potential plastic hinge regions. To avoid the jacket from bearing against the footing when in compression, a vertical gap of about 25 mm is typically provided between the jacket and the footing. Figure 15 shows a rectangular column being retrofitted with an elliptical steel jacket at the base to correct lap splice deficiency (Priestley et al. 1996).

**Figure 14** Retrofit of circular and rectangular columns with steel jackets; the columns shown in this figure were used in the experimental investigation conducted by Priestley et al. *(1994, 1994a)*




## 2.3.2 Columns

The effectiveness of steel jacketing for the upgrading of reinforced concrete columns has been investigated by many researchers. In 1987, a major research program was undertaken at the University of California at San Diego to study various retrofitting techniques for bridge columns in order to improve the seismic performance of existing bridges. A number of steel jacketed columns have been tested at the University of California at San Diego in order to investigate the effectiveness of steel jackets for enhancing the flexural capacity and shear capacity, and for preventing lap splice de-bonding in columns of older bridges. Selected results from these investigations are described hereafter.

*Flexure:* Chai et al. (1991) conducted experimental investigations on the performance of circular columns retrofitted with steel jackets. One set of the tests was specifically associated with improvement of the flexural capacity of columns. Figure 16 shows the geometry and reinforcement of the "as-built" columns used in tests for flexure. The length of the jacket used for retrofit of the column was 1.2 m (48 in.), such that the moment demand immediately above the jacket did not exceed 75% of the uncased flexural capacity.

Figure 17 shows the load-displacement curves of the "as-built" and retrofitted columns obtained by applying lateral cyclic loading at the top of each column. These figures show that the steel jacket provided a significant improvement in the flexural performance of column. While the "as-built" column performed relatively well until a ductility ratio of 4, the retrofitted column exhibited excellent behaviour up to a ductility ratio of 8, corresponding to a drift ratio of 6%.

*Shear:* Priestley at al. (1994, 1994a) investigated the effectiveness of steel jackets for retrofitting columns with inadequate shear strength. Both circular and rectangular columns were tested (Figure 16) with steel jackets over the full length. Circular jackets were used for strengthening the circular columns, and elliptical jackets were used for strengthening the rectangular columns.

Figure 18 is representative of the behaviour of "as-built" and retrofitted columns with steel jackets over full column length. Comparing Figure 18(a) for the "as-built" column and Figure 18(b) for the retrofitted column, it becomes apparent that a significant increase in strength and ductility is achieved with steel jacketing. While the "as-built" column experienced brittle failure at a displacement ductility ratio of 1.5, the retrofitted column showed excellent performance up to a ductility ratio of 8.

*Lap Splice Clamping:* Chai et al. (1991) also investigated the effectiveness of circular steel jackets in improving the performance of circular columns with inadequate lap splices. Columns were constructed with lap splices of 20 times the longitudinal bar diameter in the potential plastic hinge region (i.e. just above the footing) which was normal practice in pre-1970's construction. The length of the jacket was 1.20 m, as illustrated in Figure 16. The results from the cyclic testing of "as-built" and retrofitted columns are shown in Figure 19. The "as built" column did not posses practically any ductility, resulting in a brittle failure at a ductility of 1.5. The retrofitted column performed extremely well up to a displacement ductility ratio of 7.

*Rectangular Steel Jackets:* Based on the published material, the research for retrofit of reinforced concrete columns conducted at the University of California at San Diego is mainly associated with the use of steel circular and elliptical jackets. Research on rectangular jackets is very limited. Priestley et al. (1994) reported that "… Previous tests conducted mainly in Japan and New Zealand have shown that plastic buckling of the rectangular jackets tended to occur in the hinge regions when the columns were subjected to large cyclic lateral displacement, even when very thick jackets were used. Consequently, the rectangular jackets did not provide adequate confinement of the concrete and compression reinforcement in the plastic hinge region…" This is shown in Figure 20.

**Figure 15** Rectangular column-base lap splice retrofitted with a steel jacket
*(Priestley et al. 1996)*

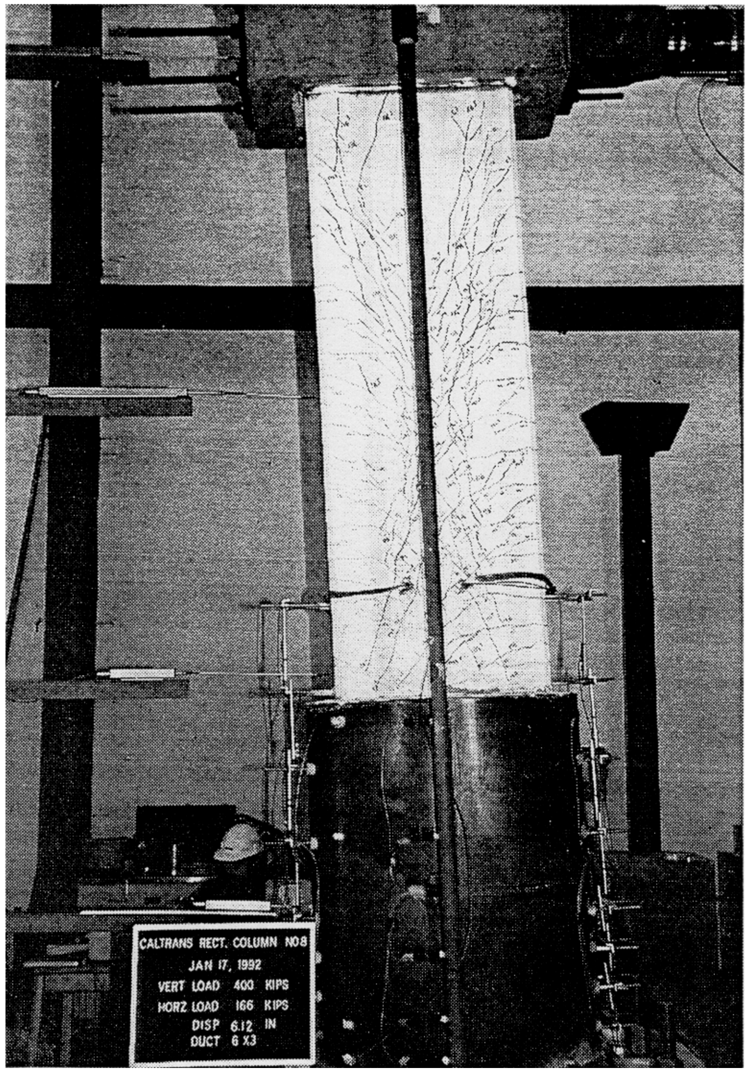


**Figure 16** Geometry and reinforcement of circular column used for flexural retrofit and retrofit for lap splice clamping by using steel circular jackets
*(Chai et al. 1991)*

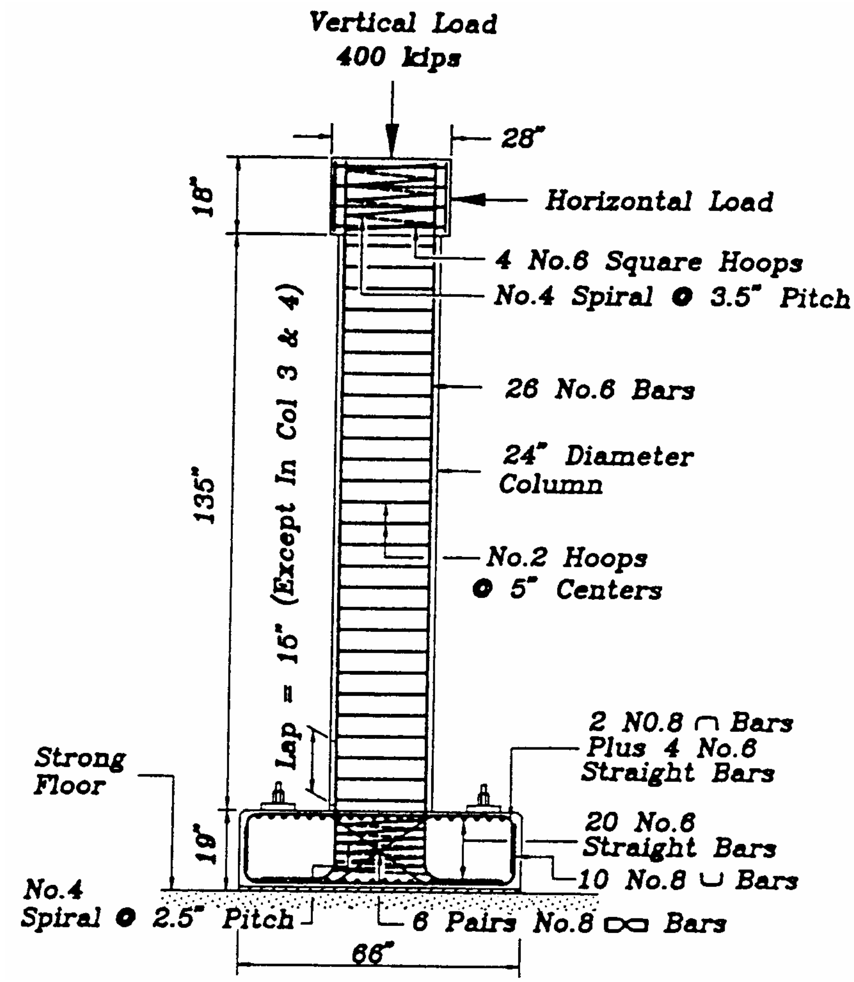


**Figure 17** Lateral load-displacement response of "as-built" and retrofitted circular column with steel jacket for enhanced flexural performance (Chai et al. (1991); (a) "As-built" column; (b) Retrofitted column; [Column No. 3 ("as-built") and Column No. 4 (retrofitted) in Chai et al. (1991)].

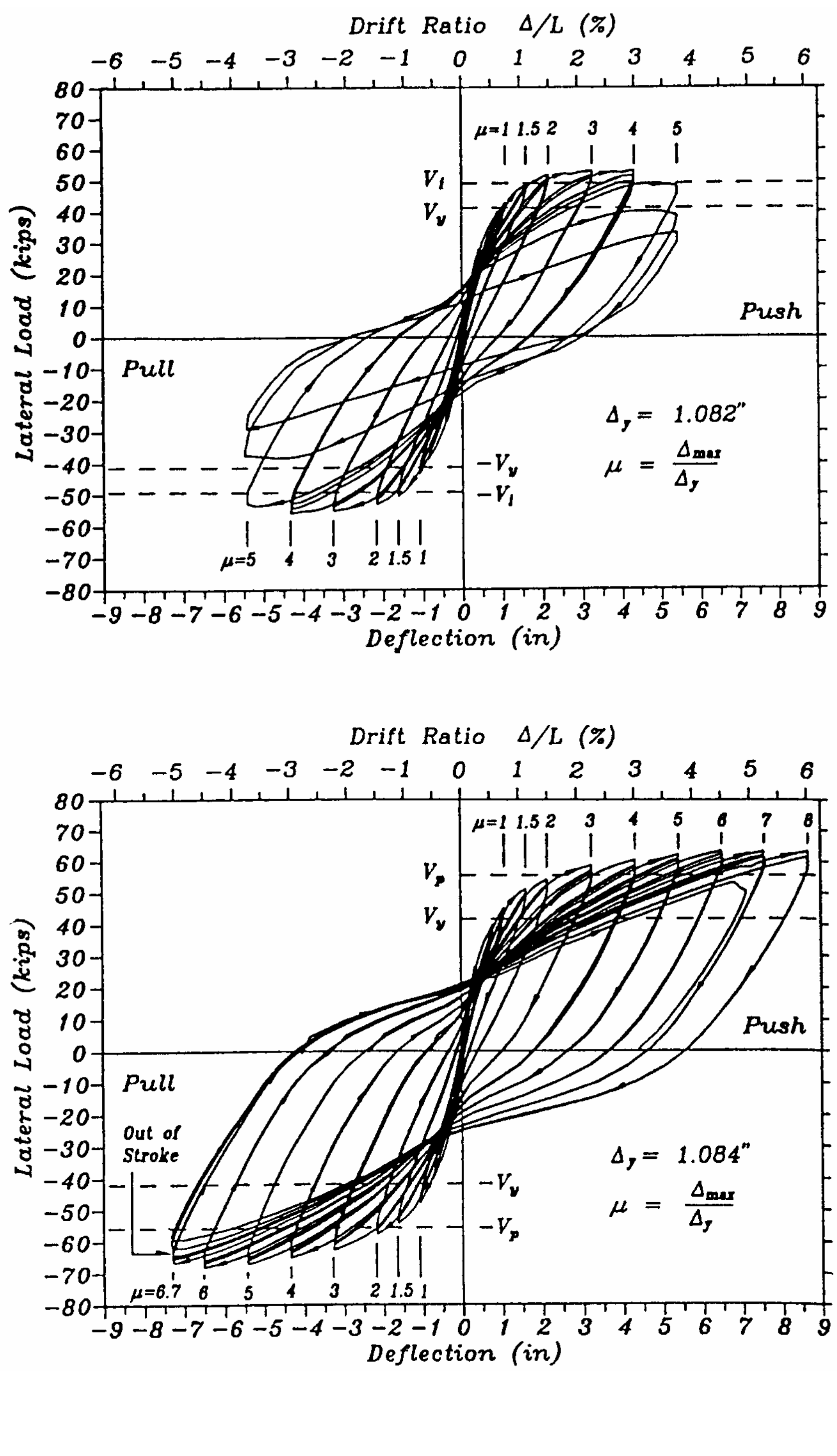


**Figure 18** Lateral load-displacement response of "as-built" and retrofitted rectangular column with elliptical steel jacket for enhanced shear strength (Priestley et al. 1994a); (a) "As-built" column; (b) Retrofitted column; [Columns R3A ("as-built") and R4R (retrofitted) in Priestley et al. 1994a].

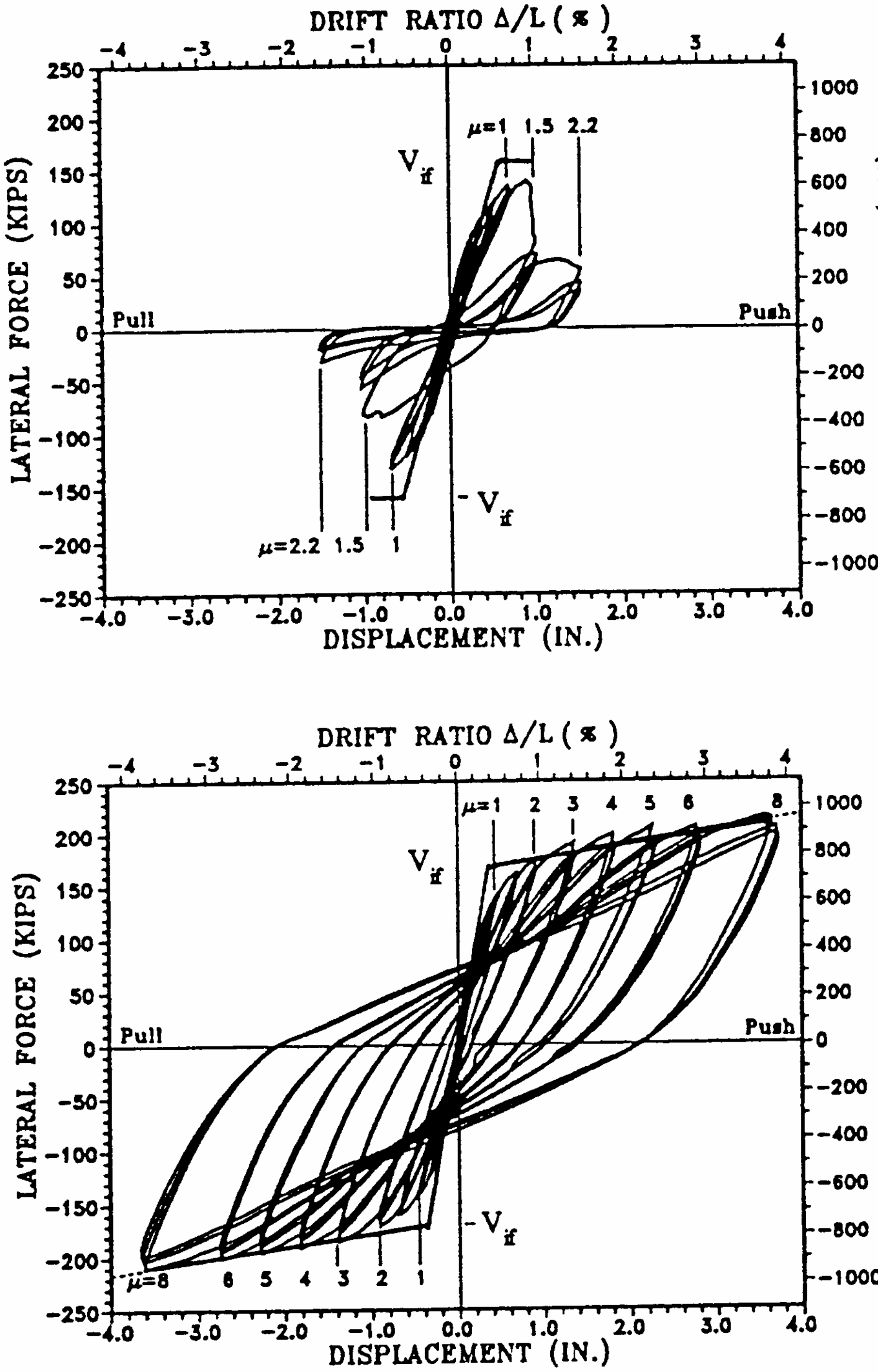


**Figure 19** Lateral load-displacement response of "as-built" and retrofitted circular column with steel jacket for lap splice clamping (Chai et al. 1991); (a) "As-built" column; (b) Retrofitted column; [Column No. 1 ("as-built") and Column No. 6 (retrofitted) in Chai et al. (1991)].

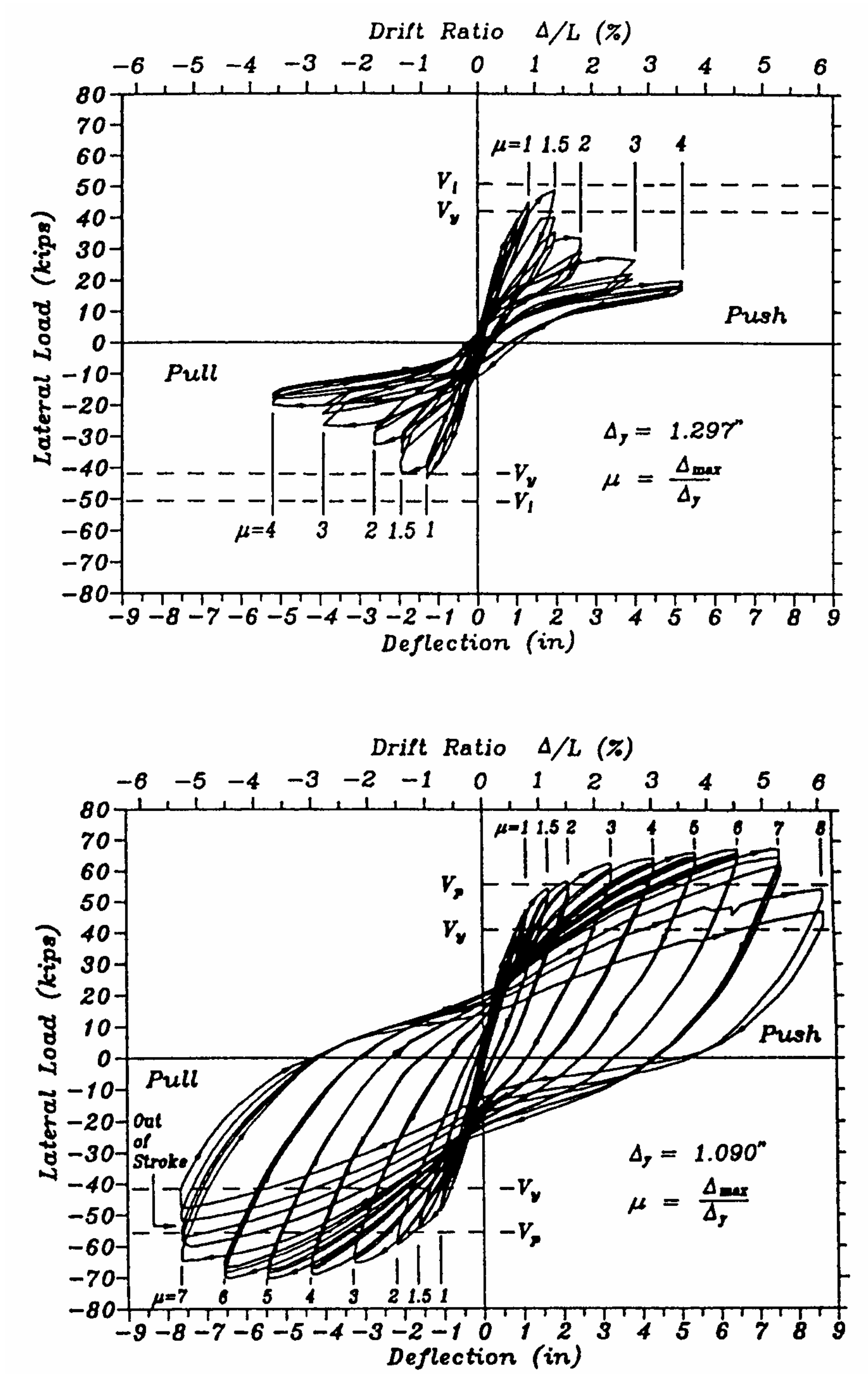
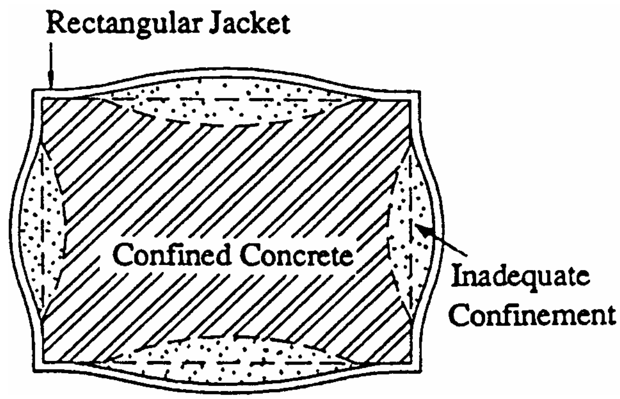


**Figure 20** Rectangular section confined by a rectangular jacket *(Priestley et al. 1994)*




### 2.3.3 Beam-Column Joints

During the past few decades, many experimental studies on the behaviour of typical interior and exterior joints under cyclic loading have been conducted. However, there has been very little research and experimental studies on joint strengthening techniques. A review of the literature indicated that one of the most comprehensive investigations for upgrading beam-column joints was conducted at McMaster University (Ghobarah et al. 1996, 1997, Biddah 1997). However, these investigations were conducted for a specific application (i.e. for joints of bare frames (without slabs) supporting large concrete ducts in nuclear power plants), and therefore the problems associated with joint retrofit for building structures which have floor slabs were not addressed. The method involves the use of corrugated steel jacket system as shown in Figure 21. The corrugated jacket is stiff and provides confining pressure by passive restraint in the joint region.

Ghobarah et al. (1996, 1997) presented experimental investigations conducted on three specimens, which were designated as J1, J3, and J4. The frame consisted of flexible columns, strong beams and weak joints, reflecting the non-ductile design of the1969 code. The ties inside the joint and column were about 16% of that recommended by the current CSA Standard for concrete design (CSA A23.3, 1994). All the specimens were of identical dimensions, representing one-third scale size of the actual beam-column joint (Figure 21).

Specimen J1 represented "as-built" conditions (i.e. no jacketing was applied to this specimen). Specimen J3 was encased in a corrugated steel jacket around the column and the beam to enhance its seismic behaviour. Non-shrink grout of 25 mm thickness was placed between the concrete and the steel jacket. Steel angles were attached to the beam at the column face to resist the outward confinement pressure from the concrete in the joint region. A gap of 20 mm between the column and the beam jacket was provided. Specimen J4 had the same jacketing arrangement as J3 for the column, but did not have any jacket on the beam. The assembling of the jacket for specimen J3 is shown in Figure 22. The details of the experimental research are reported elsewhere (Ghobarah et al. 1996, 1997).

The columns had hinge supports at both ends, while the beam tip was subjected to vertical cyclic loading (Figure 21). The beam tip load-displacement curves for the three specimens are shown in Figure 23. It can be seen from this figure that the behaviour of specimen J1 is poor, with pinched hysteresis loops and rapid strength degradation. The effect of confinement due to the jacket is evident in the hysteresis loops of specimen J3. Both the positive and negative flexural strength were reached and maintained for several cycles. The behaviour of specimen J4 represents a transition between the corresponding results obtained for specimens J1 and J3. The effectiveness of the beam jacket can be assessed by comparing the behaviour of specimens J3 and J4.

**Figure 21** Dimensions of test specimens *(Ghobarah et al. 1996)*

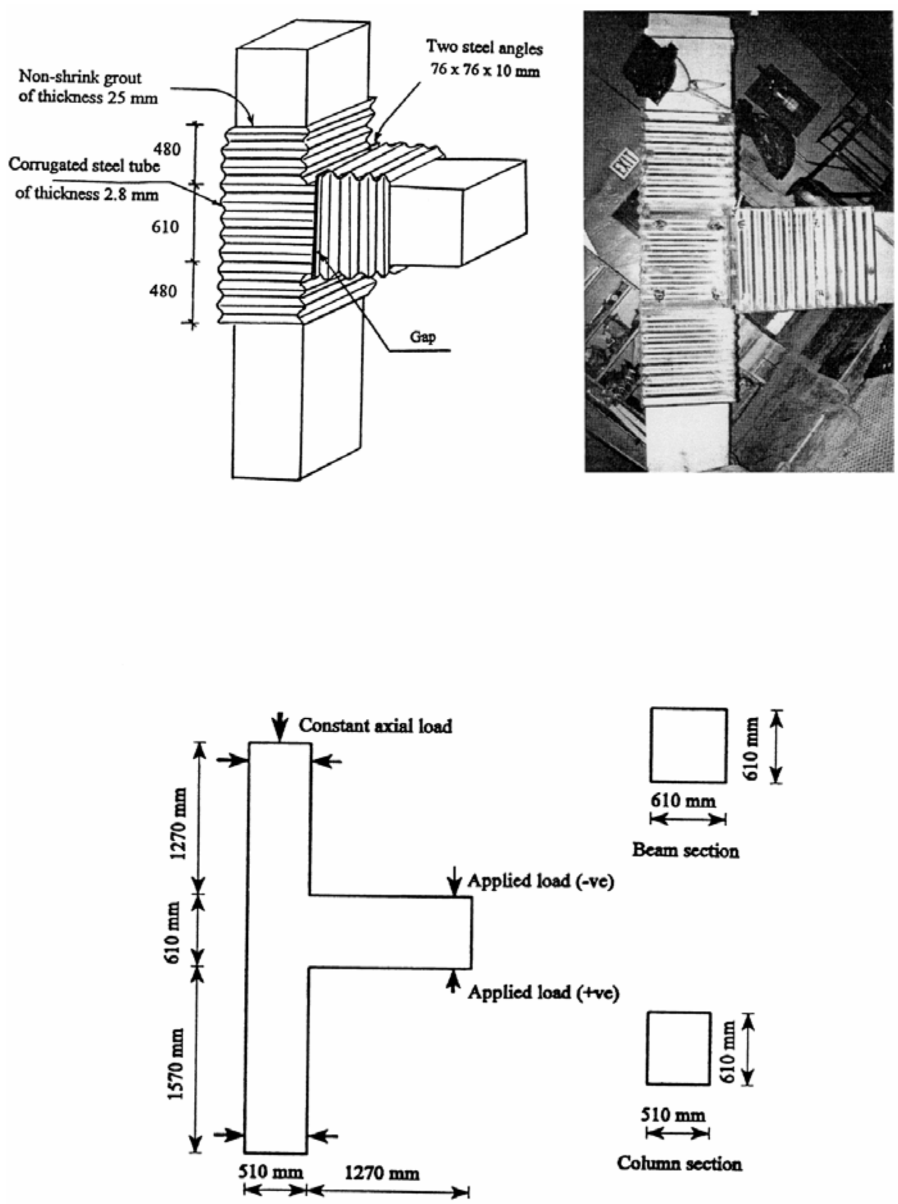


**Figure 22** Assembling of beam and column jackets *(Ghobarah et al. 1996).*

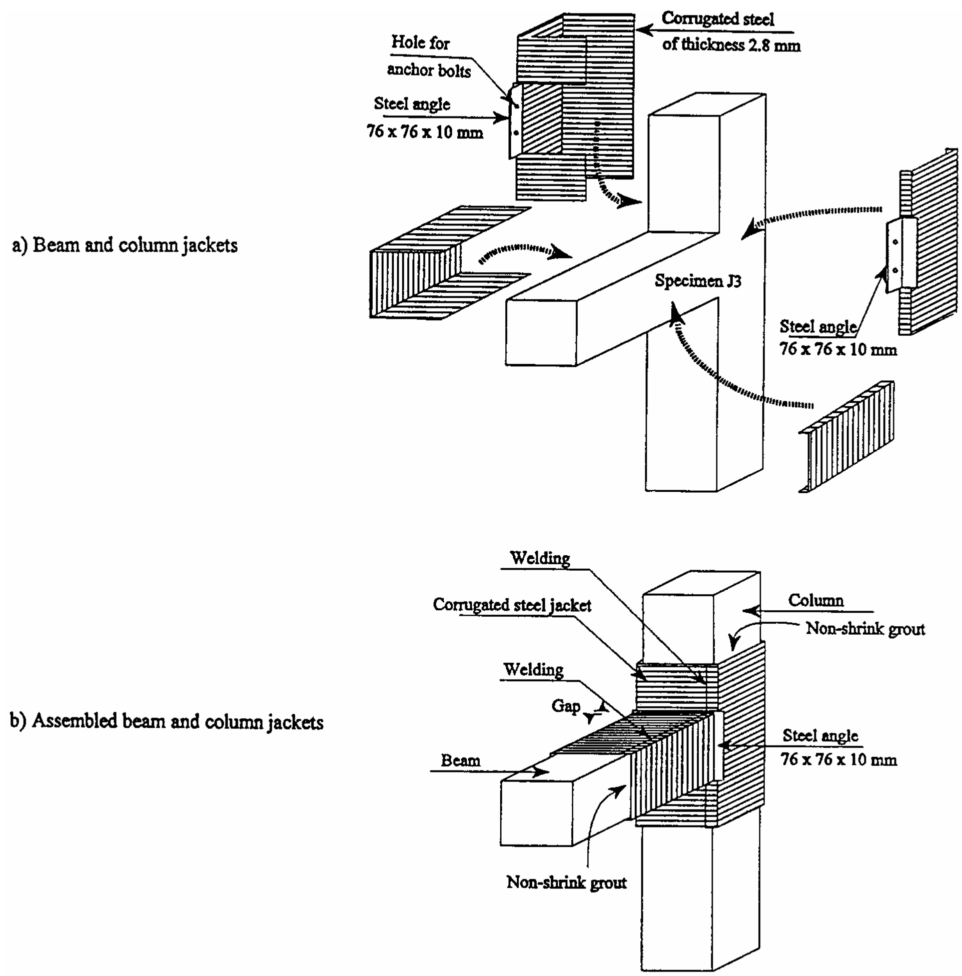
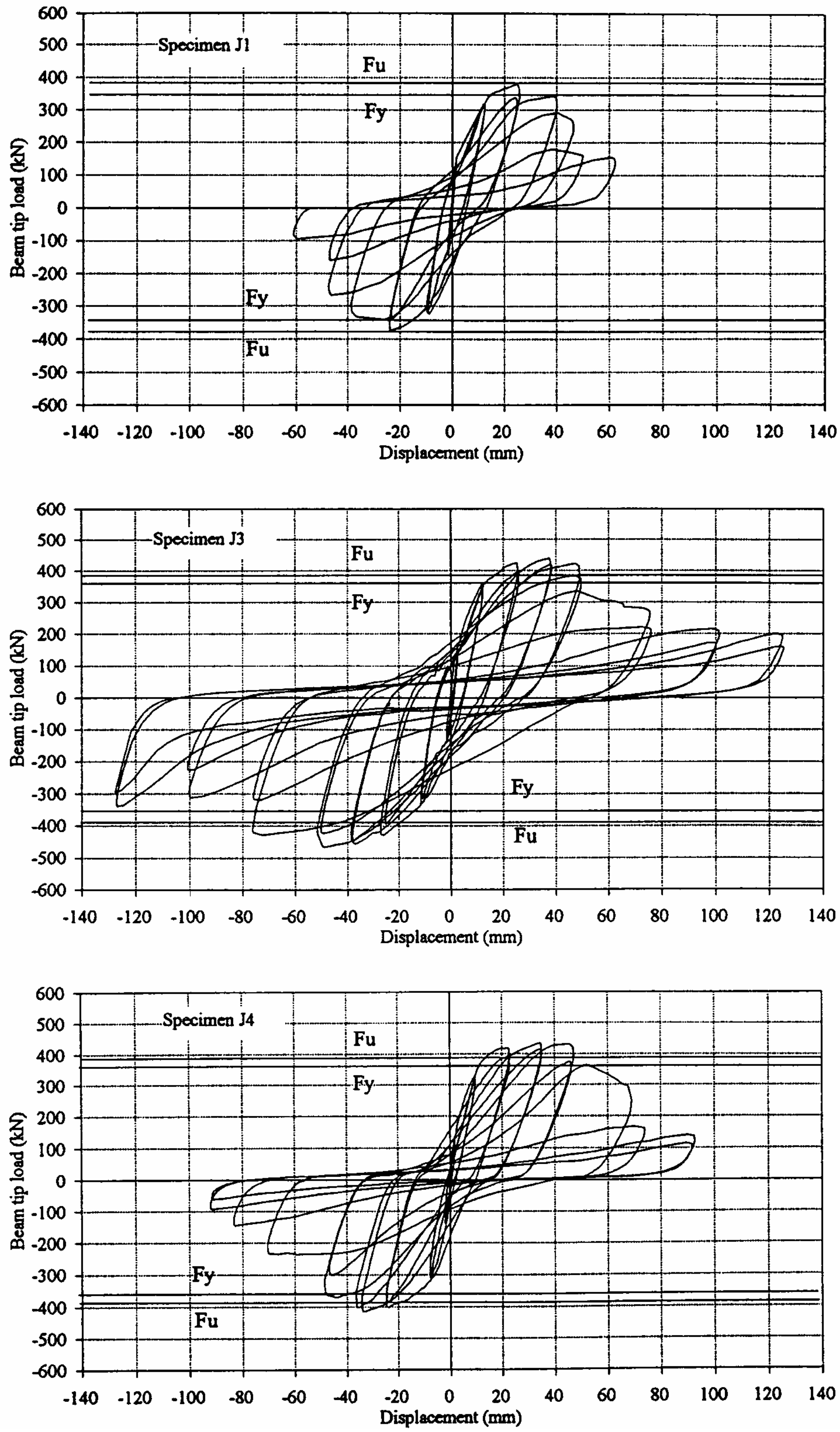


**Figure 23** Beam tip load-displacement relationships for specimens J1, J3 and J4
*(Ghobarah et al. 1996)*



### 2.3.4 Summary

Retrofitting circular columns with steel circular jackets, and rectangular columns with elliptical jackets, provides significant increases in seismic resistance of columns. The effectiveness of steel jackets is well illustrated both through experimental research and field observations during the 1994 Northridge earthquake. A number of retrofitted bridges with steel-jacketed columns were located in regions of intense ground shakings, with peak ground accelerations exceeding 0.25g. None of these columns were reported to have sustained any significant damage (Chai, 1996). However, this technique requires substantial labour and it is expensive. Given the high efficiency on one hand and the cost on the other hand, the use of steel jacketing might be justified for bridges or some industrial facilities where a small number of columns need to be retrofitted. However, for buildings where typically many columns require upgrading, this technique becomes costly.

Rectangular steel jackets for retrofitting rectangular columns also increase strength and ductility of columns. However, research results have shown that rectangular jackets are less effective than elliptical jackets.

Little research has been carried out concerning the retrofit of beam-column joints of older buildings. Retrofitting with corrugated steel jackets appears to be an efficient method for strengthening beam-column joints. However, this technique was developed for a specific application (i.e. for beam-column joints without floor slabs). Slab systems that are used in ordinary building structures pose a challenge for retrofitting beam-column joints.

## 2.4 Upgrading Reinforced Concrete Columns by Transverse Prestressing

### 2.4.1 Upgrading technique

Use of fibre composites or steel jackets, as discussed above, improves the performance of structural members through additional reinforcement and enhancement of passive confinement pressure. This passive lateral pressure is provided by the fibre or steel jacket, which encloses the member.

Performance of structural members can also be improved through external prestressing, which provides additional reinforcement, as well as active lateral pressure. A new technique developed at the University of Ottawa, called Retro-belt (Saatcioglu et al, 2000), involves prestressing concrete columns by means of placing high-strength steel hoops around the columns. The steel hoops, made out of seven-wire strands and specially designed anchors are placed around the column at specific spacing and are prestressed to a pre-determined stress level. The anchors, placed on the surface of the column, provide adequate anchorage for both ends of the strand. While the steel strand acts as additional shear reinforcement, the active lateral pressure improves concrete confinement, increasing flexural and shear capacity. The prestressing also provides adequate clamping force in the longitudinal splice regions, correcting the deficiency of inadequate lap splices often provided in potential hinging regions of existing columns. Figure 24 shows the elevation of a circular column retrofitted with external prestressing.

Figure 25 illustrates retrofitting of a square column using external prestressing by means of raiser discs. The additional hardware provided for rectilinear columns help produce near uniform pressure on faces of rectilinear columns. The hardware consists of hollow structural sections (HSS) used as external hoops, with raiser disks of different diameter welded on them to distribute the prestressing force uniformly on four faces of the column. The strands are placed directly on the raiser discs to develop perpendicular force components. The heights and locations of the raisers are calculated so that approximately equal force components are generated at the raisers.

**Figure 24** Retrofit of shear-dominated circular column *(Saatcioglu et al, 2000)*

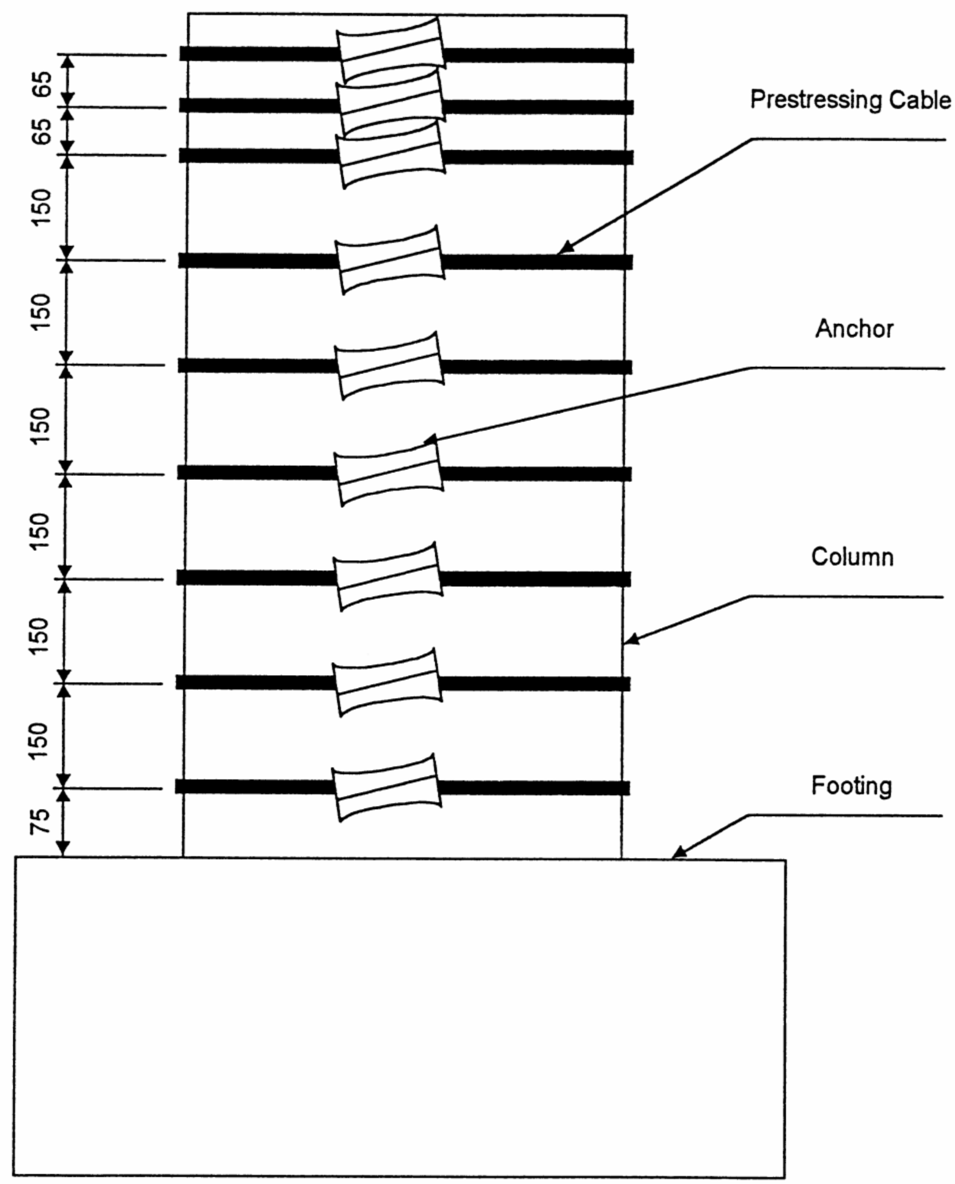
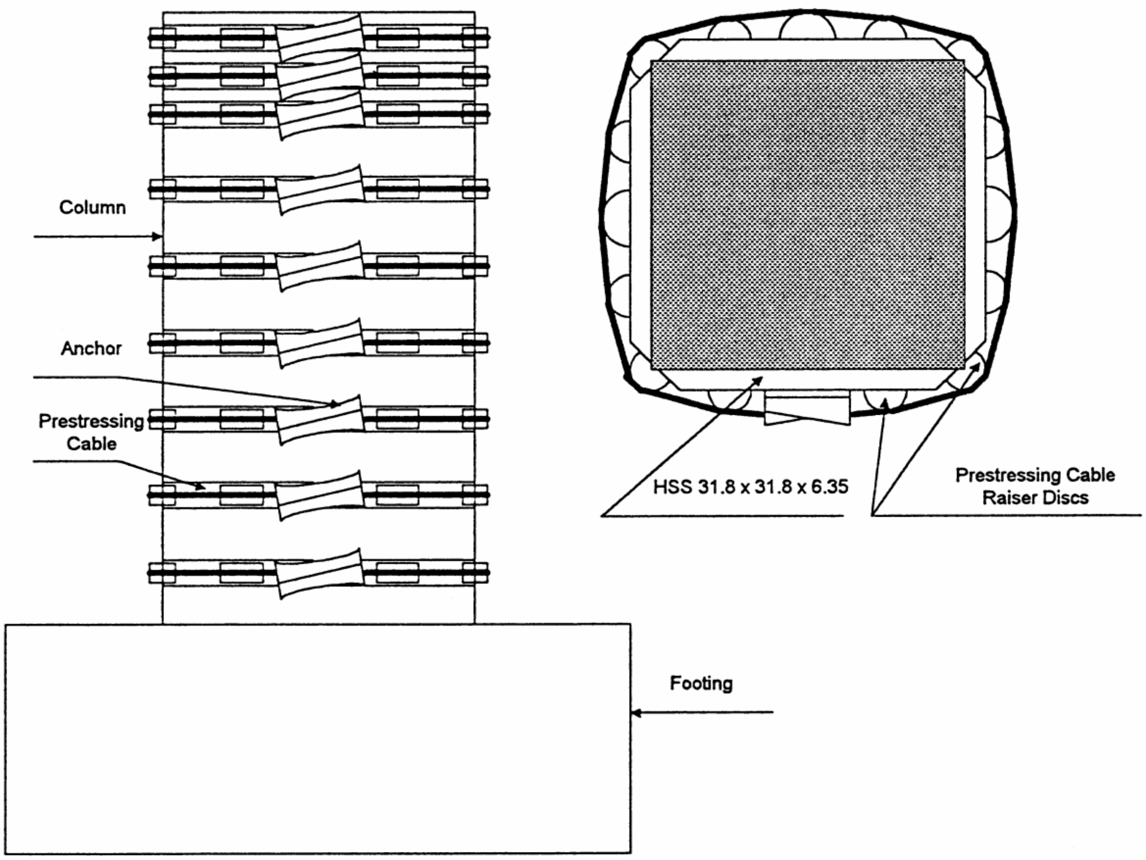


**Figure 25** Retrofit of shear-dominated square column *(Saatcioglu et al, 2000)*




### 2.4.2 Columns

Extensive experimental investigations on seismic retrofit of reinforced concrete columns have been conducted at the University of Ottawa (Yalcin 1997; Mes 1999; Beausejour, 2000; Saatcioglu, 2000). As a result of these investigations, a new retrofitting technique was developed. This technique consists of external prestressing, which provides active and passive lateral pressure to overcome lateral expansion in concrete under compression. It also provides a clamping force in reinforcement splice locations to improve bonding between the steel and concrete. In shear dominant columns, transverse prestressing counteracts diagonal tension caused by shear, thereby improving column shear resistance. The prestressing is achieved by means of high-strength seven-wire steel strands, individually prestressed by a small hydraulic jack, connected by specially designed anchors. The experimental program comprised of 19 full-scale bridge columns, with circular and square columns, tested under simulated seismic loading. The tests were conducted in three phases. The first phase consisted of shear dominant columns with continuous longitudinal reinforcement without any lap splices. The second phase included flexure dominant columns with continuous reinforcement, while the third phase consisted of flexure as well as shear dominant columns with lap splices.

*Shear:* Both circular and square specimens were tested under "as-built" and “retrofitted” conditions. The reinforcement used in the columns is representative of pre-1970’s design practice for bridge columns.

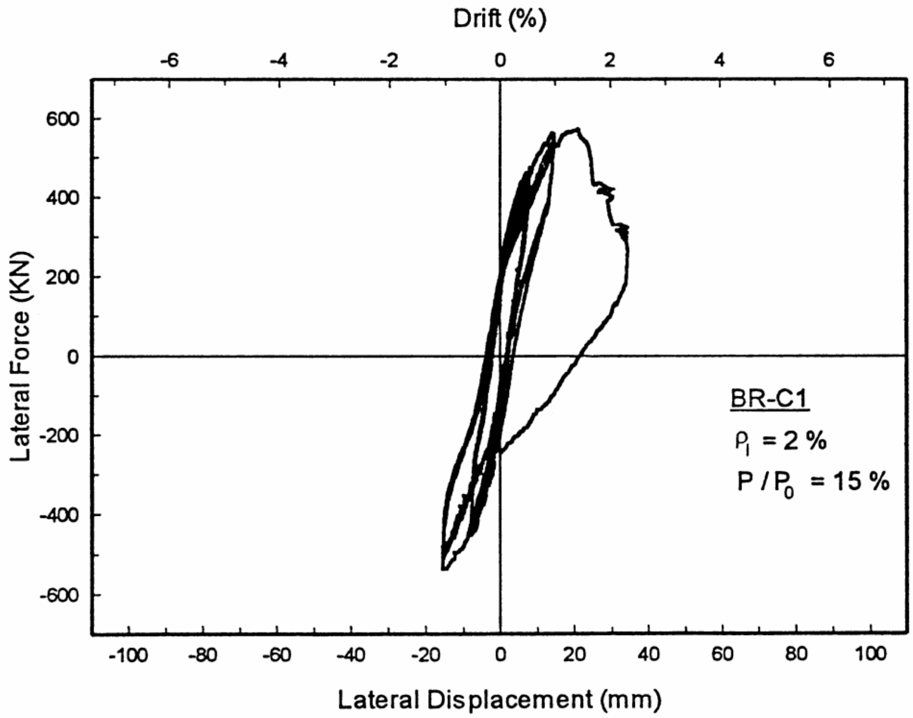
The retrofit of the columns was conducted using Grade 1720 MPa seven-wire strands. The strands had a nominal diameter of 9.53 mm. Different hoop spacing and level of initial prestressing were used as test parameters to determine the optimal retrofit solution.

The tests were conducted by applying a constant axial load, equal to 15% of the column concentric capacity, and incrementally increasing horizontal displacement cycles, applied at the top of the columns. Of the various tests, the best performance was obtained when 150 mm spacing was used with initial prestressing of 25% of the ultimate strand capacity. The hysteresis loops for such retrofitted columns, together with those for the companion "as-built" columns, are shown in Figure 26 (circular columns) and Figure 27 (square columns). The effectiveness of transverse prestressing becomes obvious in these figures. The retrofitted columns show much larger ductility and energy dissipation characteristics than those of the "as-built" columns.

*Flexure:* Tests of flexure dominant columns were conducted. Both circular and square column specimens, with "as-built" and "retrofitted" conditions, were tested. The "as-built" columns were constructed in accordance with the pre-1970 design practice for bridge columns. The test results are shown in Figure 28 for circular specimens and in Figure 29 for rectangular specimens. The comparison of Figures 28 and 29 reveals the superior performance of the retrofitted columns relative to the corresponding "as-built" columns.

*Splice Clamping*: Two square and four circular columns with lap splices at the bottom end of the columns were built and tested. The spliced reinforcement was located in the potential hinging region and would benefit from a seismic retrofit. The columns were externally prestressed in the transverse direction and tested under reversed cyclic loading. The specimens that reflected the “as-built” conditions could not sustain lateral drift greater than 1%. The longitudinal reinforcement became unstable at this stage of deformation and started to slip. This resulted in rapid strength decay. Columns that were externally prestressed showed improved behaviour and were able to sustain up to 5% lateral drift without any strength degradation. Figures 30 and 31 illustrate the effectiveness of external prestressing in providing sufficient clamping force within the splice region.

**Figure 26** Lateral load-displacement response of shear-dominated circular columns (Saatcioglu et al, 2000); (a) "As-built" column; (b) Retrofitted column (150 mm distance between strands and prestressing of 25% of the ultimate strand capacity).



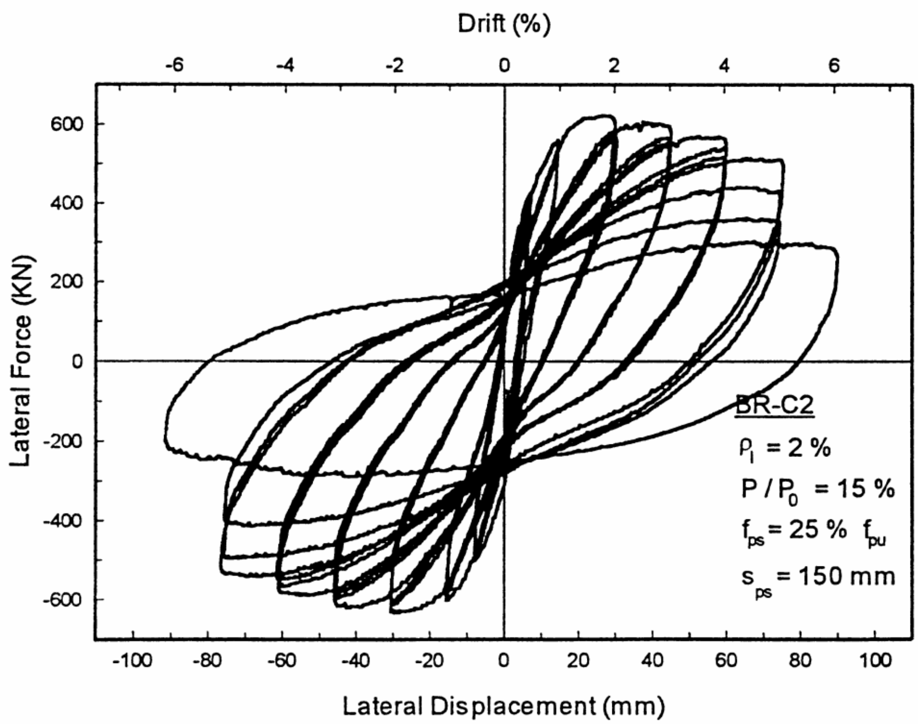
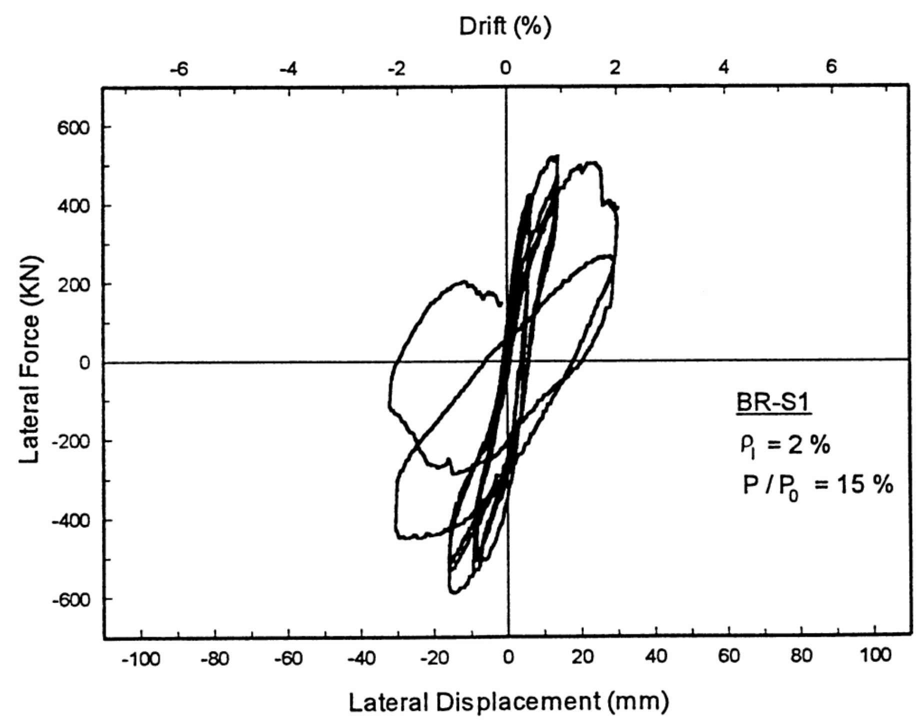



**Figure 27** Lateral load-displacement response of shear-dominated square columns (Saatcioglu et al, 2000); (a) "As-built" column; (b) Retrofitted column (150 mm distance between strands and prestressing of 25% of the ultimate strand capacity).



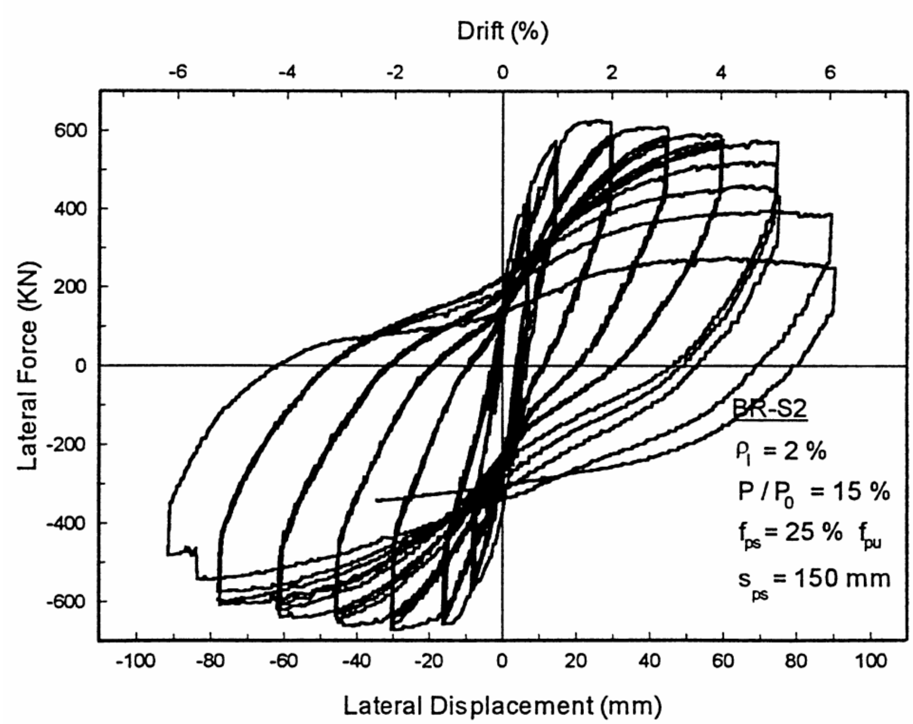



**Figure 28** Lateral load-displacement response of flexure-dominated circular columns (Saatcioglu et al, 2000); (a) "As-built" column; (b) Retrofitted column (150 mm distance between strands and prestressing of 25% of the ultimate strand capacity).

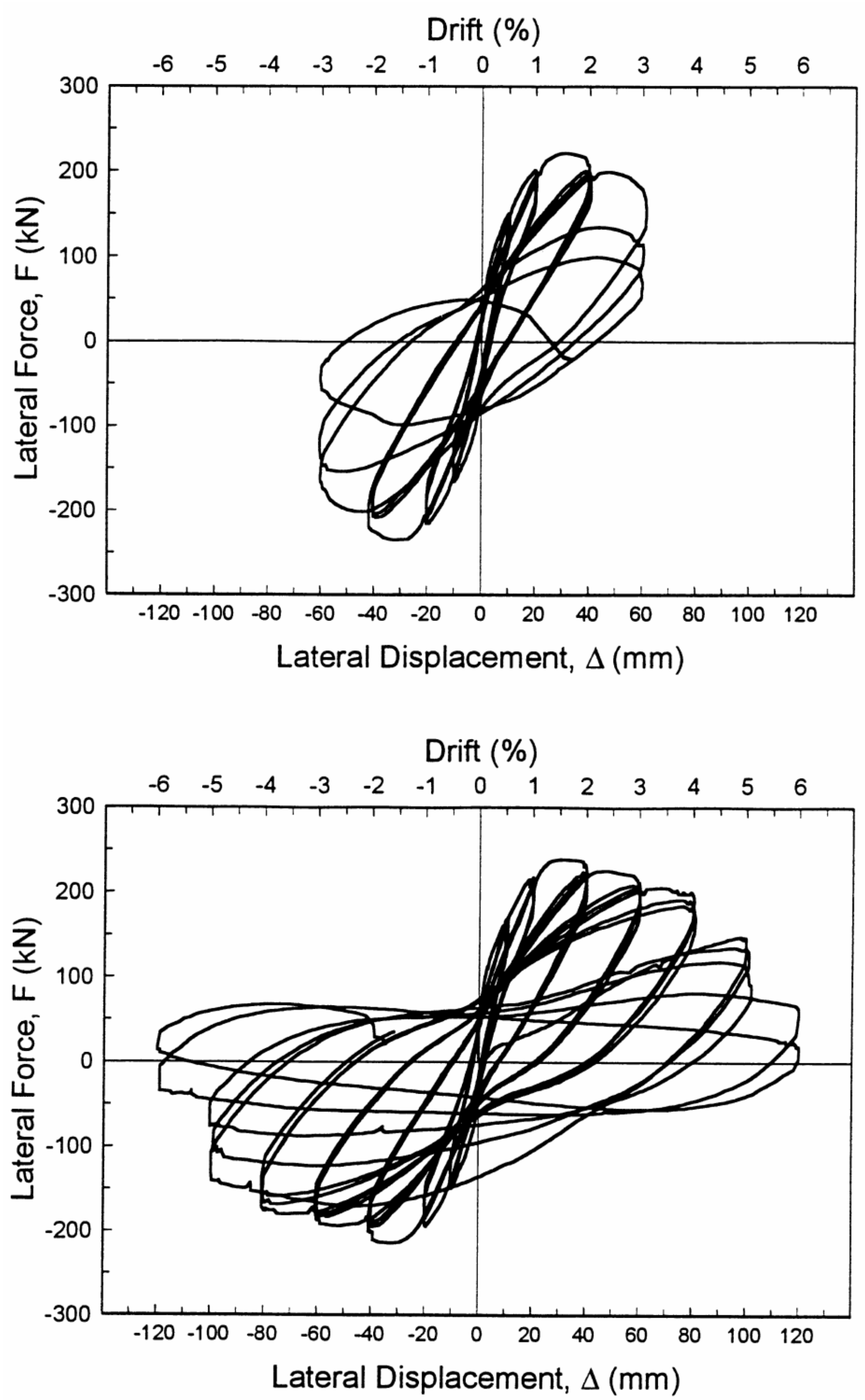


**Figure 29** Lateral load-displacement response of flexure-dominated square columns (Saatcioglu, 2000); (a) "As-built" column; (b) Retrofitted column (150 mm distance between strands and prestressing of 25% of the ultimate strand capacity).

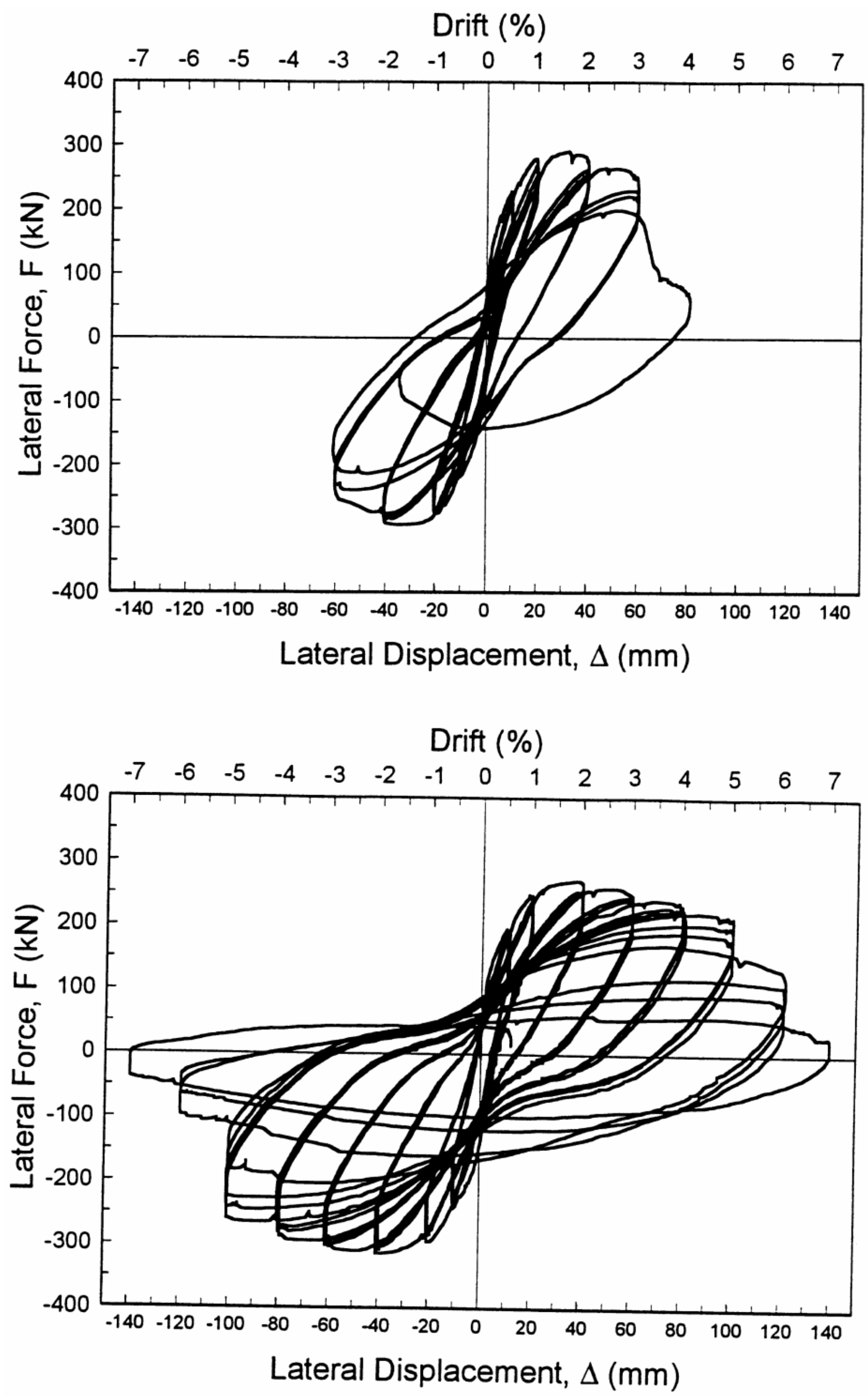


**Figure 30** A square column without retrofit *(Saatcioglu et al, 2000)*

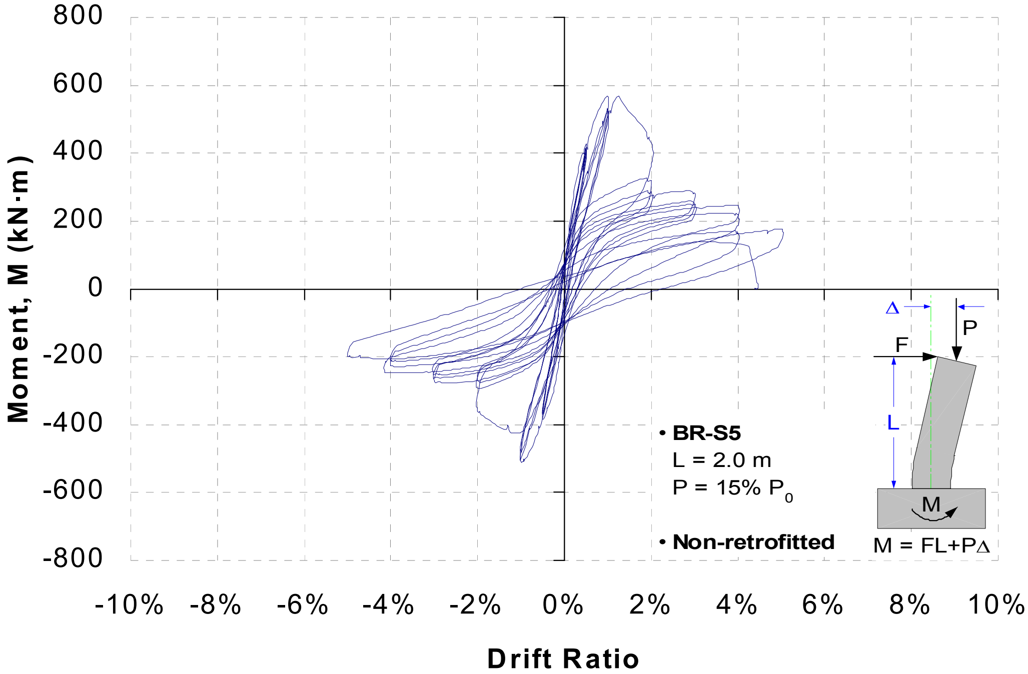
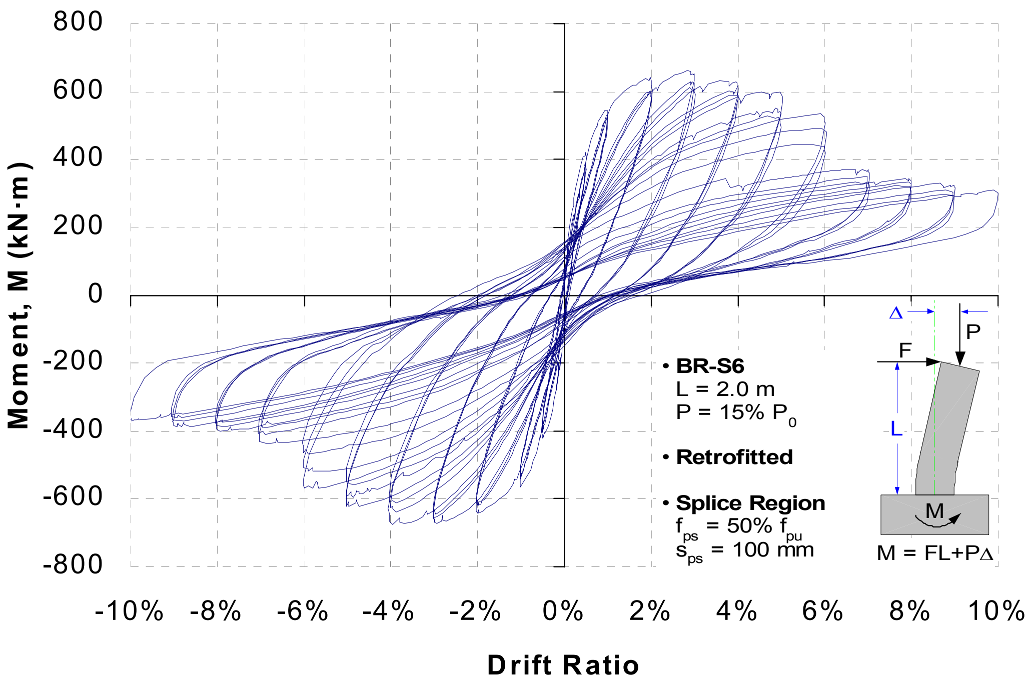



**Figure 31** A square column retrofitted with external prestressing *(Saatcioglu et al, 2000)*



### 2.4.3 Summary

This method is very promising for the retrofit of building columns. It is efficient and can be much more economical than steel jacketing. Installation of such a system can be less disturbing to the building occupants. This technique was developed during the last three years and its potential application on buildings has yet to be realized.

## 2.5 Upgrading Building Structures Using Damping Devices

### 2.5.1 Upgrading technique

In addition to the foregoing techniques for strengthening structural components (columns, beams, and beam-column joints), the use of damping devices represents a viable solution for rehabilitation of existing buildings. The function of a damping device in a building resembles to that of a shock absorber in an automobile. While the shock absorber reduces the effect of bumpy roads, the damping device reduces the impact of ground motions on the building's structure and occupants.

There are four basic types of damping devices: visco-elastic, friction, metallic and viscous. The common principle of these damping devices is to dissipate the earthquake-induced energy through heat energy, usually by means of friction between various materials. Damping device transfers the kinetic energy generated by the moving mass, or structure, to potential energy through friction/heat.

In visco-elastic and viscous dampers, a piston operates against a friction device (pads or fluid-filled chambers) to dissipate energy in the form of friction and heat. Friction dampers utilize the friction and heat generated when specially coated steel plates slide against each other, to release the earthquake-induced energy. Metallic dampers dissipate energy through inelastic deformation of the metal components. Many buildings have been recently retrofitted using friction and viscous dampers for improved seismic performance.

### 2.5.2 Friction dampers

Among the several types of damping devices, the friction dampers are rather widely used (Frederichs, 1997; Elliot et al. 1998). The principle of the work of these dampers is based on energy dissipation by friction. The friction dampers consist of a series of steel plates that are specially treated to develop the largest amount of friction. These plates are clamped together with high-strength steel bolts (Figure 32). During severe seismic excitations, friction dampers slip at a predetermined optimum load before yielding occurs in other structural members and dissipate a major portion of the seismic energy. Obviously, the predetermined loading, and the number of friction dampers for strengthening a given building, depend on the structural system and the seismic motion for which the building is required to be protected.

Several types of friction dampers are available, such as dampers for cross-bracing, diagonal bracing, and chevron bracing (Figure 33). For illustration, Figure 33 shows installed cross-braced friction dampers.

The effectiveness of friction dampers in reducing seismic effects on building structures has been tested in laboratories. The performance of a three storey steel frame, equipped with friction dampers, was investigated by Filiatrault and Cherry (1986) using a shaking table. Similar tests were conducted by Aiken et al. (1988) on a nine storey steel frame. In both cases, the friction dampers provided very satisfactory performance of the frames even for very strong shakings. Given this, friction dampers have been used for rehabilitation of a number of buildings during the last ten years.

The effectiveness of friction dampers for reducing the seismic effects is tested in labs. Strengthening with friction dampers can be done with a minor disturbance of the building occupants. A number of old buildings are already equipped with friction dampers. However, the performance of such buildings during actual earthquakes remains to be seen.

**Figure 32** A friction damper in action: when tension in one of the braces forces the damper to slip, the mechanism shortens the other brace, thus preventing buckling *(Friederichs, 1997).*

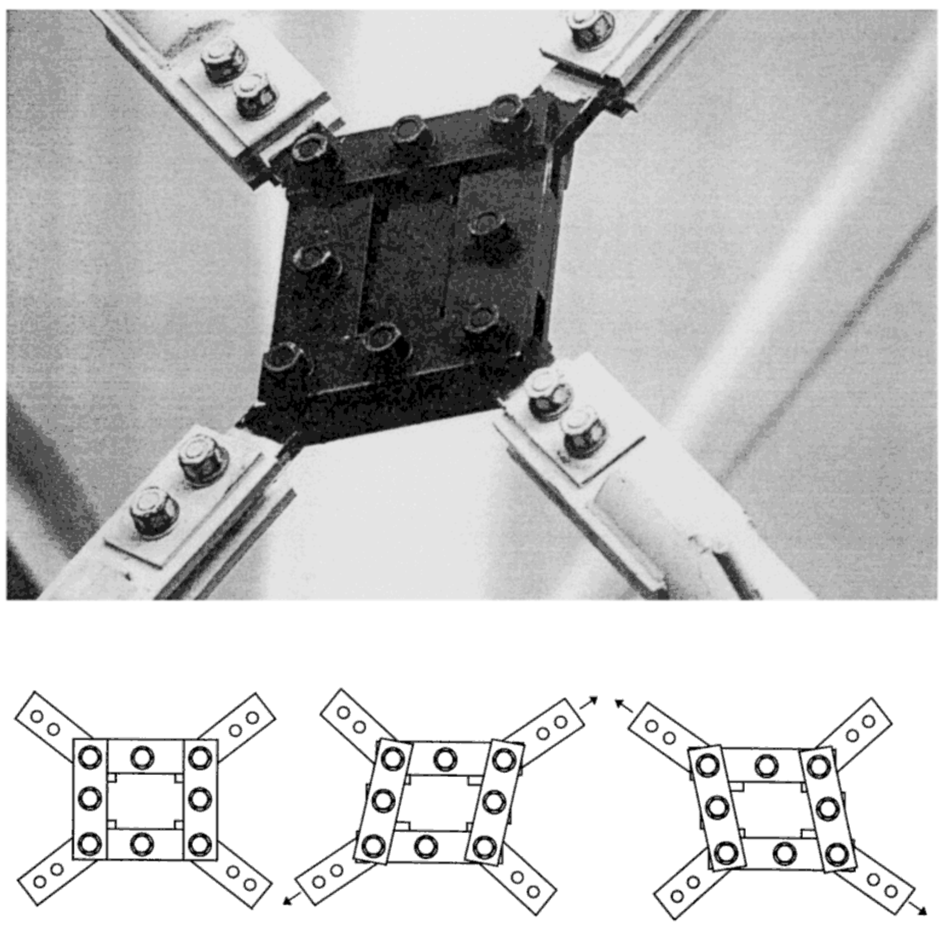
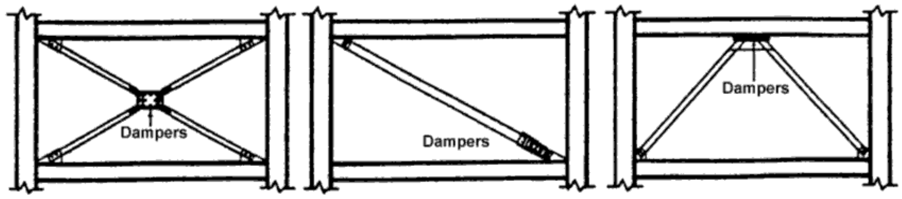


**Figure 33** Installed friction dampers (the braces along the wall in background of photo).




### 2.5.3 Viscous dampers

In simple terms, viscous dampers are rods moving back and forth inside a cylinder of viscous liquid, releasing earthquake-induced energy through friction between the rod, the cylinder, and the fluid. The common parts of a viscous damper consist of a solid stainless steel piston rod impregnated with Teflon®, an enclosed cylinder, and an operating fluid which is usually an inert silicone fluid permanently sealing inside the damper (Taylor and Constantinou, 2000). This is illustrated in Figure 34.

Dampers such as force-actuators can be attached to a structure through a threaded stud clevis-type mounting or a base plate mounting. Figure 35 illustrates a schematic layout of a damper brace system for a building (Keller). Figure 36 shows a damper brace system with two viscous dampers visible at the bottom of the brace (photo provided by Craig Keller of Taylor Devices, N. Tonawando, NY).

Force from viscous damping is dependent on the stroke velocity and can be out of phase with stresses generated by the movement of the structure. Damping force diminishes at maximum displacement (zero acceleration) of the structure. The maximum viscous force occurs at minimum or initial/original displacement (but maximum acceleration as the structure swings back) of the structure. The out of phase response is a very desirable feature of fluid viscous damping as it helps reduce building deflection and stress at the same time.

**Figure 34** Cross section of a typical liquid viscous damper
*(Taylor and Constantinou, 2000).*

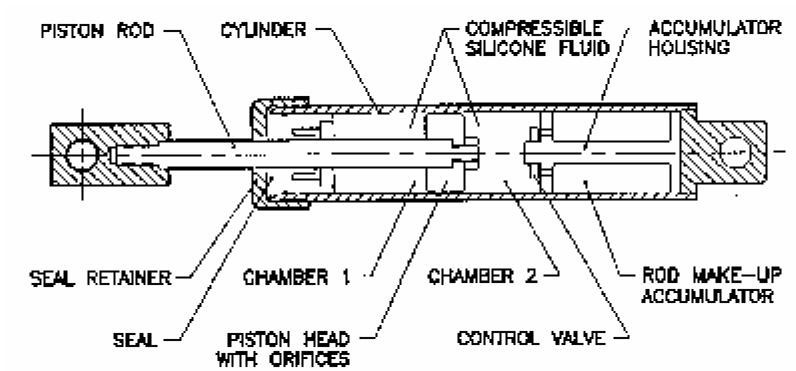



**Figure 35** Schematic illustration of viscous damping braces within a building
*(Keller 1998)*

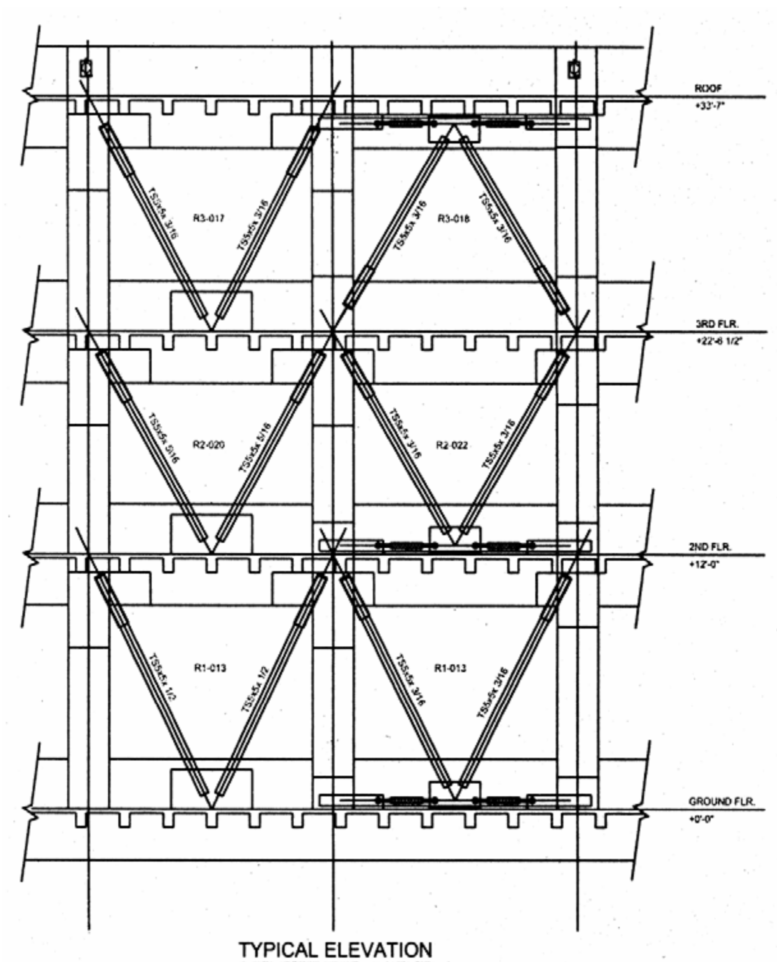
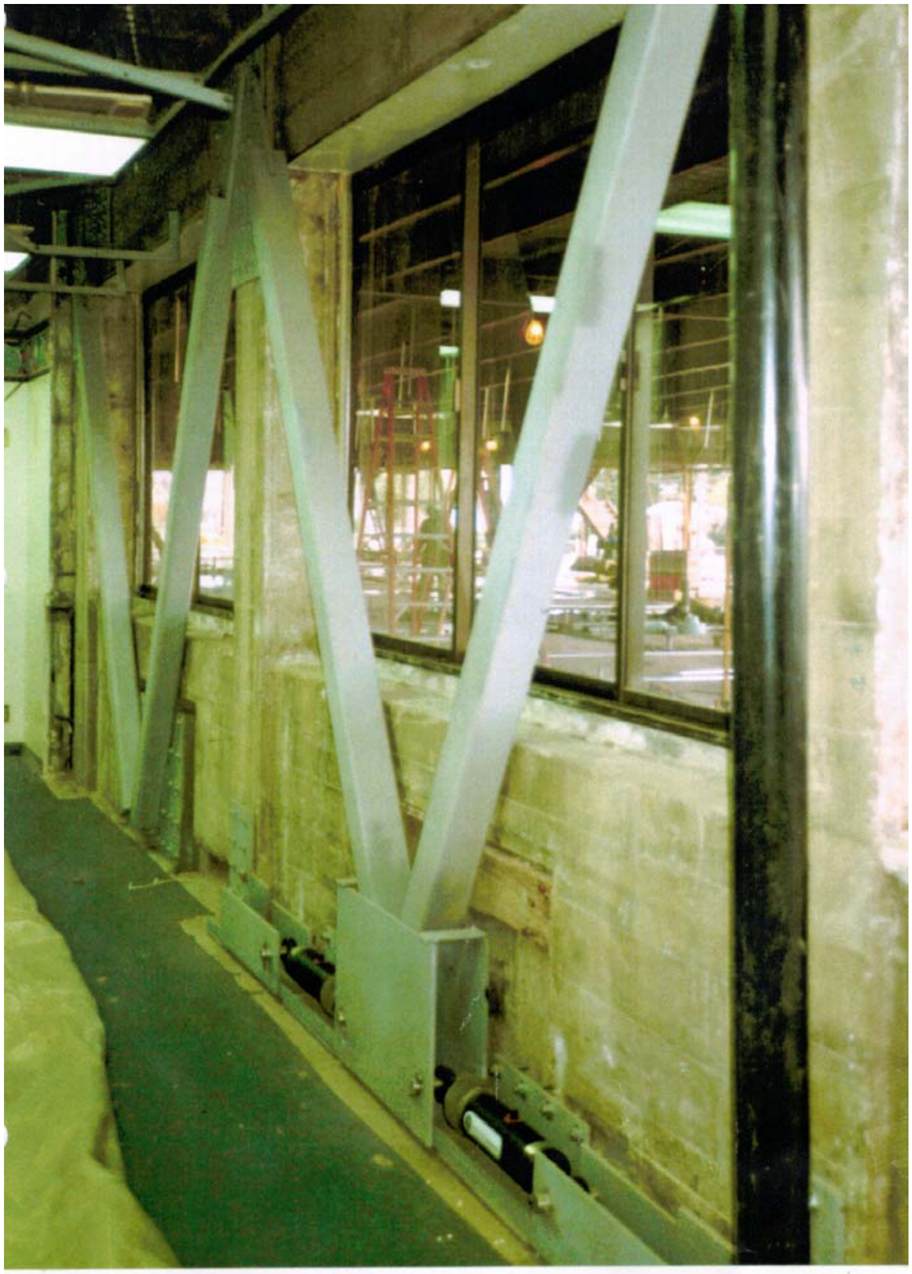


**Figure 36** A steel brace with viscous dampers
*(photo provided by Keller of Taylor Devices, N. Tonawando, NY)*.



### 2.5.4 Summary

Within the past 20 years, innovative technologies such as energy dissipation and base isolation devices have been developed and used to enhance seismic performance of buildings. Energy dissipation devices such as friction and viscous dampers can mitigate potential damage to buildings by absorbing a significant portion of the energy input to a building by earthquake shaking. Compared to base isolation devices, passive damping devices are applicable to a broader inventory of structures.

A new and evolving technology which integrates the damping devices, force actuation devices, sensors, controllers ,and real-time information processing, has been receiving considerable attention in recent years, mainly in Japan and the U.S.A. Almost all installations of such a system on new buildings are located in Japan. Most of these newer buildings have been subjected to earthquake events and their performance is promising. Although such a structural control technique is expected to provide enhanced structural behaviour for improved service and safety, applicability of this exciting and evolving technology on retrofitting of existing buildings remains unchallenged.

## 2.6 Upgrading Building Structures Using Base Isolation Devices

### 2.6.1 Upgrading technique

Damping devices, as discussed in the previous chapter, deal with energy absorbing systems which help dissipate energy that is induced from the ground to the building in the event of an earthquake. There are other devices, which can be used to release the earthquake-induced energy before the energy is being transferred to the building structure. These devices are generally referred to as "base isolators." These devices can be used to isolate the building base from the ground such that the impact of ground shaking upon the building structure is reduced to an acceptable level. Figure 37 illustrates the effect of base isolation (Zenitaka Corporation, 2000).

Base isolators usually possess the following characteristics:

- Low frequency motion with high damping;
- Maintenance-free mechanism to displace sideways, reducing energy (i.e. load) transferring from the ground to the building structure and returning to original position after an earthquake;
- Flexible enough to move sideways and stiff enough to sustain gravity loads and to remain stationary under wind loads; and
- Rigid connections between building structure and foundation.

Figure 38 shows the installation of a base isolator under a column (Taylor & Gaines, 2000). The three commonly used base isolation bearings for buildings are steel laminated rubber bearings, high damping rubber bearings, and sliding bearings. All three have the same basic effect: to allow the building to move independently of the ground motion.

**Figure 37** Effects of ground shaking on a building with/without base isolation
*(Zenitaka Corporation. 2000).*

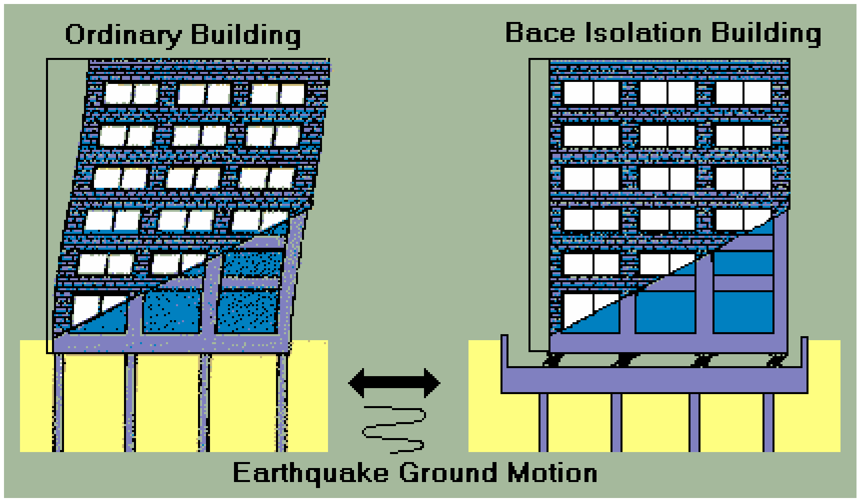


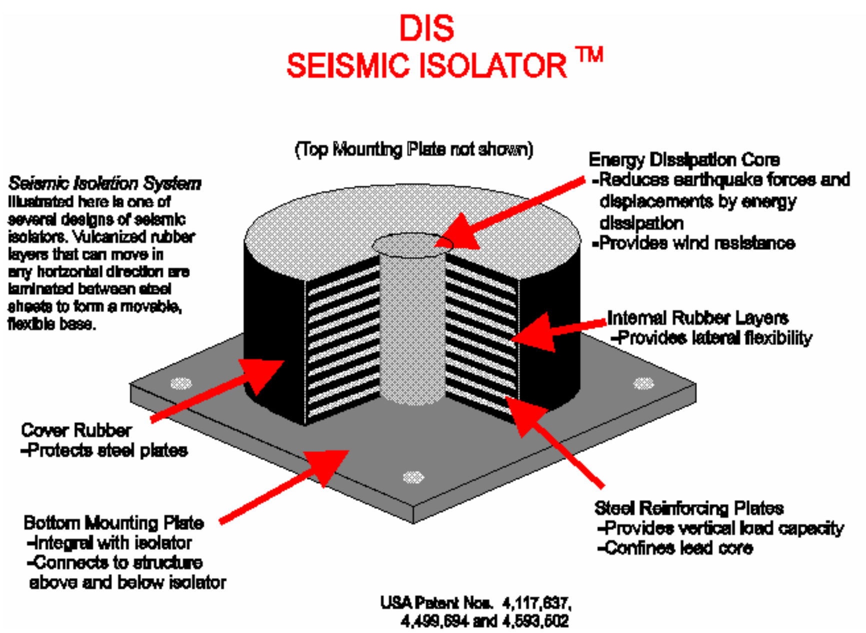

**Figure 38** A base isolation device being installed at the column base
*(Taylor and Gaines, 2000).*

### 2.6.2 Steel-laminated rubber bearings

The steel-laminated rubber bearings consist of alternating layers of rubber and steel bound together with a cylinder of pure lead, tightly inserted at the center of the steel/rubber layers, as shown in Figure 39 (DIS Inc., 2000). The steel and rubber layers are molded under heat and pressure (vulcanization process) into one unit, with the steel laminates permanently bonded to the rubber. The principle of this type of bearings is as follows:

- the rubber layers displace sideways, absorbing earthquake-induced energy, reduces load transfer from the ground to the building structure, and returns the building structure to its original position after the earthquake;
- the steel layers provide vertical load capacity and confine the lead core; and
- the lead core stops the structure from moving sideways under wind loads, absorbs a portion of the earthquake energy, and controls the lateral displacement of the structure.

**Figure 39** A typical steel-laminated rubber bearing *(DIS Inc., 2000).*




### 2.6.3 High damping rubber bearings

Similar to the steel-rubber bearings but without the alternating steel layers, the high damping rubber (elastomeric) bearings consist of only rubber layers of high damping characteristics, and a cylinder of pure lead tightly inserted at the centre of the rubber layers. Varying degrees of softness of the rubber permit different levels of movement.

The principle of this type of bearings is similar to that of the steel-rubber bearings, except that the modified rubber also provides vertical load capacity and helps confine the lead core. The all-rubber bearings are softer, allowing greater movement. The lead core absorbs some of the seismic energy, like the steel-laminated rubber bearings, and helps control the lateral displacement of the structure.

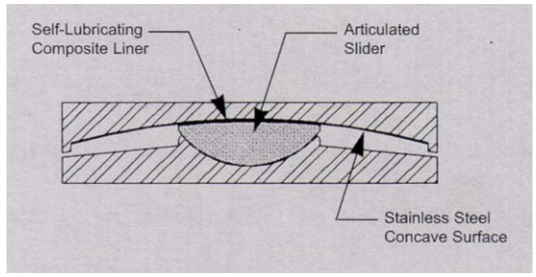
### 2.6.4 Sliding bearings

Sliding bearings resemble ball bearings, in that they consist of a slider which slides sideways to allow structures to swing gently from side to side. When the ground shakes horizontally during an earthquake, the seismic force is reduced (isolated) by the sliding bearings, so no more than friction force is transmitted to the building structure. The weight of the structure forces the slider to move back to its original position, thereby re-centering the building after an earthquake. One common type of sliding bearings is friction pendulum bearings.

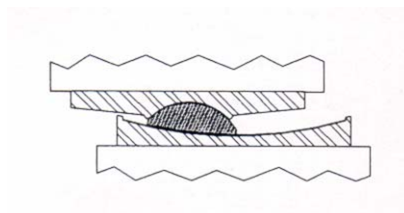
Friction pendulum bearing is a relatively new system that relies on pendulum motion and friction to reduce earthquake forces upon a building structure. In simple terms, friction pendulum bearing functions like a ball on a plate. It consists of a slider, which can be attached either to the footing below or to the building above, and a stainless steel concave surface (as illustrated in Figure 40(a) (Earthquake Protection Systems Inc., 2000)). There is a liner – which is made out of polytetrafluoroethylene composite or PTFE with a low coefficient of friction – between the slider and the concave surface (Figure 40(b)).

When the earthquake force exceeds the static friction, the slider moves along the concave spherical surface (Figure 40(c)). The slider's motion is similar to that of a simple pendulum, and causes the supported structure to rise as well. As the slider rises along the concave spherical surface, the bearing develops a lateral resisting force that is equivalent to the combined effect of a dynamic frictional force and a gravity restoring force. This provides the required damping to absorb the earthquake-induced energy.

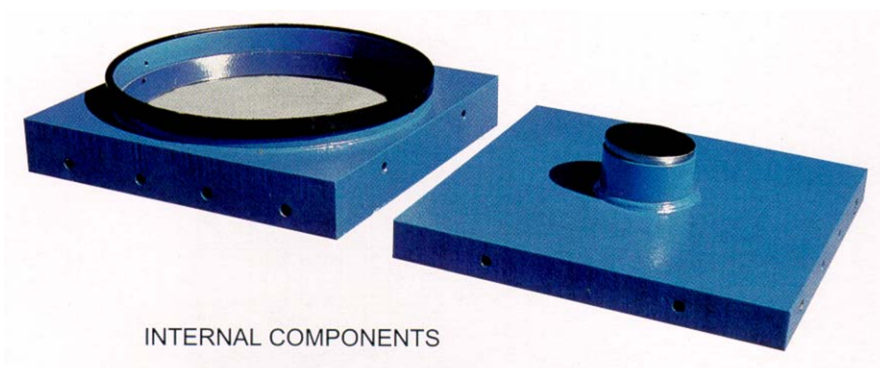
**Figure 40** A friction pendulum: (a) section of a bearing; (b) main components of a bearing; and (c) operation of a bearing *(Earthquake Protection Systems, Inc., 2000).*




**(a)**



**(b)**




**(c)**

### 2.6.5 Summary

During an earthquake, the base isolators deform, while the building above undergoes only gentle long period lateral motions and suffers no damage. Base isolators act as a flexible layer between the foundation and the building such that the ground motions have little or no impact upon the structure above it. Base isolation is the only engineering solution that can mitigate both interstorey drift and high floor accelerations.

Design and installation of a base isolator demands a sophisticated structural analysis/simulation and precise construction practice. The use of base isolators is most common for buildings of architectural significance, heritage consideration, valuable contents, and special operational requirements. Notable recent applications of base isolation systems include the seismic retrofit of the New Zealand Parliament buildings, U.S. Court of Appeals in San Francisco, and the city halls of Oakland and San Francisco.

The selection of appropriate types of base isolators for individual buildings requires thorough assessment of building characteristics, performance requirements, construction cost, and long-term maintenance and performance considerations. For example, base isolators are appropriate for rigid buildings, but not for flexible buildings. Addition of braces and shear walls may be required to make some buildings rigid enough to achieve proper base isolation.

Since rubber can harden or stretch, rubber bearings usually require periodic inspection to ensure its continued responsiveness. Friction pendulum bearings offer a lower profile than rubber bearings (about one-third the height) and are usually maintenance free. However, strong earthquakes may force the slider to stick at the edges of the plate or to slide off the plate.

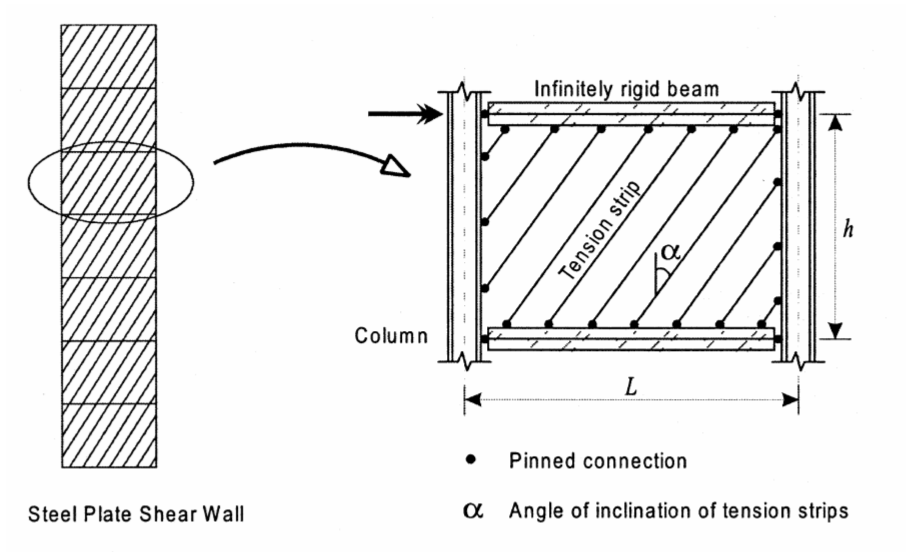
## 2.7 Upgrading Building Structures Using Steel Sheet Plates

### 2.7.1 Upgrading technique

Retrofitting building structures using various configurations of steel braces is rather common. The braces can be configured as diagonals, chevrons, knees, or X shapes. The braces can be concentric (connected to the beam-column joint) or eccentric (connected to the beam some distance from the beam-column joint). The steel braces, usually welded or bolted to stiff joints for load transfer, provide additional load carrying capacity to the original frame. However, the steel braces also add stiffness to the structure, attracting added load to the strengthened structure.

A new structural steel lateral load building system, called the steel plate shear wall (SPSW) – currently under development in Canada at the Universities of Alberta and British Columbia – is gaining interest for its potential application in the seismic upgrading of buildings. A SPSW element is essentially a thin steel in-fill panel bordered by the wide flange members of the column and beam frame. Figure 41 shows a typical steel plate shear wall panel and its representation by a parallel strip model (Rezai, Ventura and Prion, 2000). The system's lateral resistance is controlled by the post-buckling strength of the thin steel in-fill panels and the integral moment resisting frame. Extensive analytical and experimental investigations have shown that SPSW system exhibits stable hysteretic characteristics and that SPSW system can be a very effective energy absorbing lateral framing system.

**Figure 41** A steel shear plate panel and strip modelling
*(Rezai, Ventura and Prion, 2000)*




### 2.7.2 Summary

Expected low cost fabrication, speedy erection, and good energy absorbing potential make the SPSW system an attractive alternative for the seismic upgrading of existing buildings. While SPSW system can be easily integrated into existing steel frames, its suitability in concrete frames is still in the development stage.

## 2.8 Summary

The art and science of retrofitting a building for improved seismic performance is relatively new to the seismic protection community. Over the past 20 years, tremendous progress has been made in the research and development of innovative materials and technologies for improving the seismic performance of existing buildings through retrofitting processes. Many of the developed technologies have also been put to use in the seismic retrofit of numerous buildings.

The emerging technologies for seismic retrofit of buildings fall under two categories: global systems and local systems. Installation of global systems such as damping devices, base isolation, or steel shear plates, has an impact on the overall structural response to earthquakes. A local system, such as the use of fibre composites, steel jacketing, and column prestressing, improves the performance of individual structural elements such as columns, beams, and walls.

Since no two buildings are the same, the ultimate selection of suitable technologies for a specific building or its structural elements hinges on whether the methodology is technically (performance requirements), financially (cost-effectiveness in terms of construction cost and business/productivity losses), and socially (consideration for heritage, aesthetics, etc.) acceptable. Some technologies may be more effective for preventing seismic damage, while others may be more cost effective. At times, use of a combination of various technologies on buildings may be the most advantageous. Buildings may require both damping devices (to prevent the building from swaying too actively) and base isolators (allowing the building to sway in the event of a strong earthquake).

During the past few years, efforts have been made in developing innovative technologies for the seismic hazard reduction of buildings. Advanced materials, systems, and techniques have been extensively investigated, and, to a lesser extent, applied in seismic retrofit projects. The gap between research advances and application benefits is mainly due to the lack of a state-of-the-art knowledge base available to both research and practicing engineers. As such, the benefits of utilizing innovative technologies as technically, economically and socially acceptable solutions for seismic hazard reduction have not been realized. This report reviews the emerging technologies for the seismic retrofitting of buildings, and provides Canada's seismic protection community with a state-of-the-art knowledge base on seismic mitigation for buildings.

# 3.0 Part C – Seismic Screening Manual

## 3.1 Background

Generally speaking, a building would go through a three-step process before being retrofitted: screening, evaluation, and retrofitting. Screening helps prioritizing buildings such that buildings with the highest risk scores would warrant a more detailed analysis, while buildings with the lowest risk scores may be exempted from further investigation. The detailed analysis determines if and to what extent a building needs strengthening. Figure 42 illustrates the seismic mitigation procedure on screening, evaluation, and upgrading for buildings.

Screening entails assessing buildings to ascertain their level of seismic risk following a simplified procedure whose main objective is to determine if the building should or should not be subject to a more detailed investigation (i.e. step 2). The widely used methodology for screening in Canada is given in "Manual for Screening of Buildings for Seismic Investigation, 1993," developed by the National Research Council's (NRC) Institute for Research in Construction (NRC 1993). Its purpose is to establish numerically a Seismic Priority Index (SPI) – a ranking – which results from the addition of a Structural Index and a Non-Structural Index.

Figure 43 depicts the screening procedure in determining the SPI of a building. More details of the procedure are given in the next section of the report.

Major factors in determining the screening score are the building location, soil conditions, type and use of the structure, obvious building irregularities, the presence or absence of non-structural hazards, building age, and the building importance and occupancy characteristics. The benchmark for screening is the 1990 edition of NBCC.

While NRC's screening methodology has been found to give good indications of "Low," "Medium," or "High" risk for most buildings, there were cases where the seismic risk of buildings located in very high seismic zones were underestimated (as per communication with PWGSC seismic engineer on screening of federal buildings in British Columbia). Buildings in high seismic zones, with a medium seismic risk according to the screening methodology, were shown to be at high risk of failure under detailed evaluation. Possible explanations were that the seismicity factor, the structural type factor, and the categories of "Low, Medium, High" needed to be refined for buildings located in very high seismic zones.

NRC's screening methodology was based on the 1990 edition of the National Building Code of Canada or NBCC 1990 (NRC 1990). The current edition of NBCC was published in 1995 (NRC 1995). Besides, new seismic hazard maps have been developed and seismic requirements are being proposed by CANCEE (Canadian National Committee for Earthquake Engineering) for the new objective-based national code. The variance in code requirements over the years may have a potential effect on the validity of the screening procedure.

**Figure 42** Seismic Mitigation Procedure

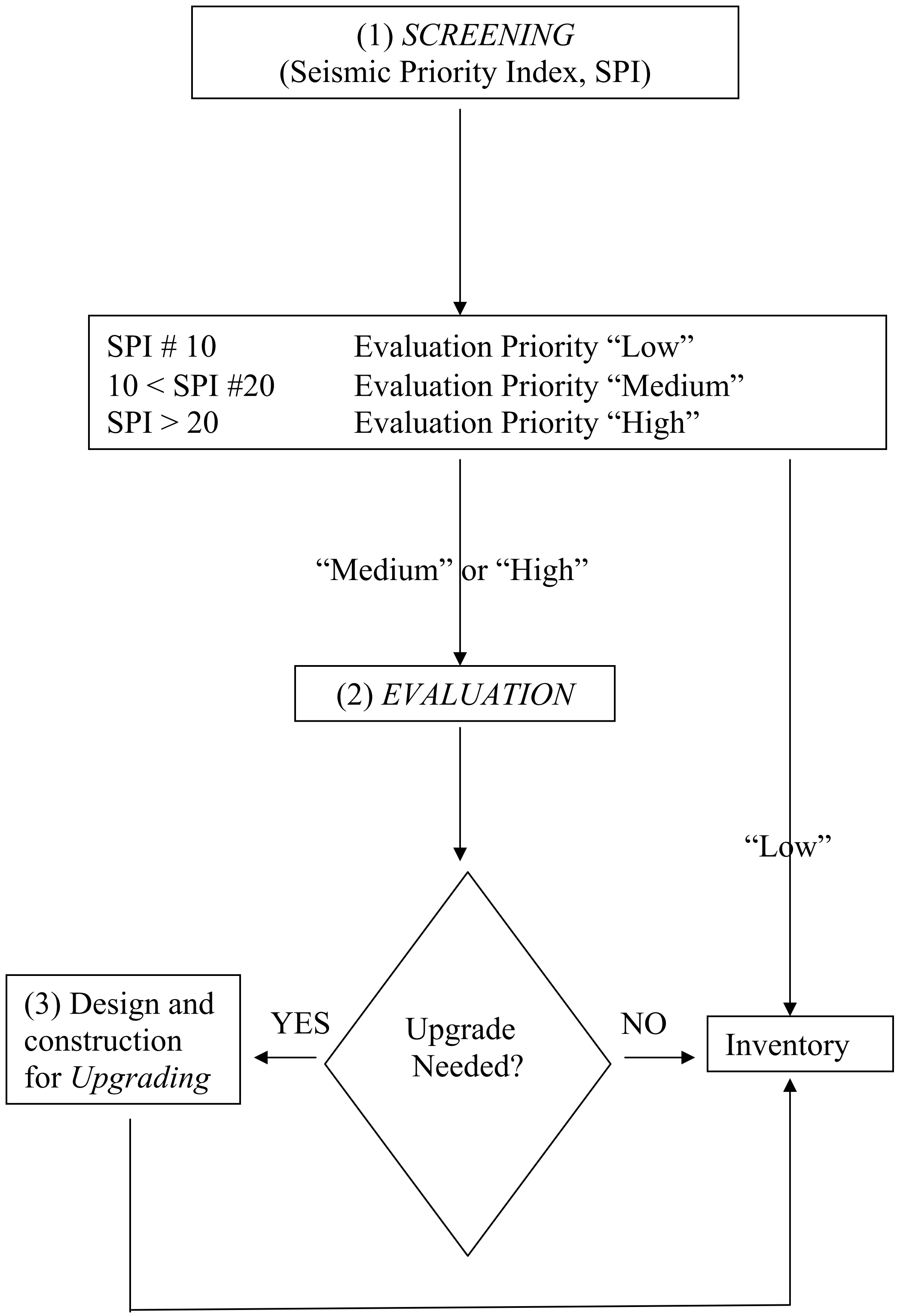
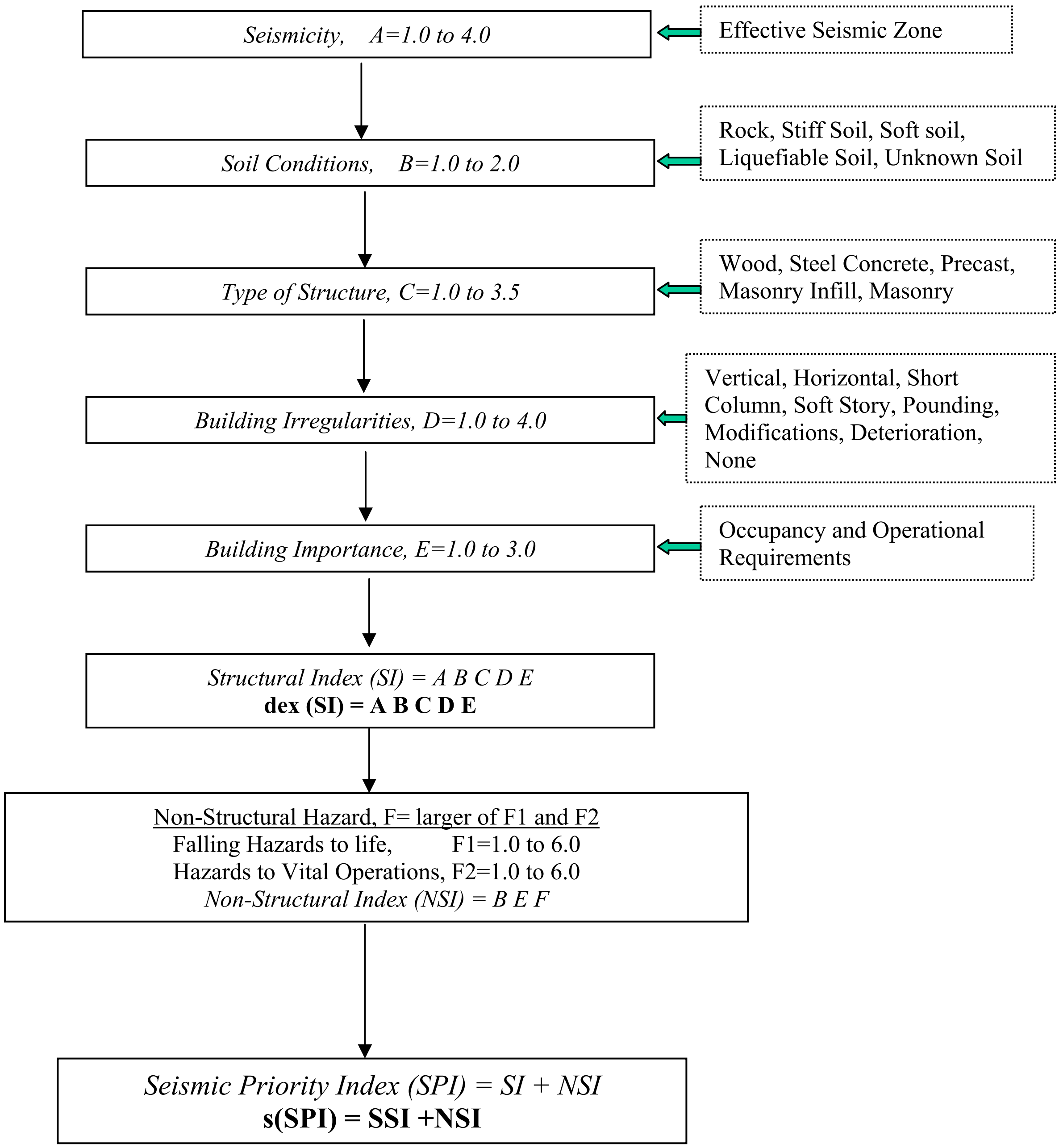


**Figure 43** NRC Screening Methodology to develop the Seismic Priority Index



This report summarizes the findings in reviewing the screening manual in accordance with the NBCC 1995, the new hazard maps and the proposed seismic requirements for the upcoming edition of NBCC, which is likely to become available in 2003.

## 3.2 Screening Parameters

The methodology of the screening manual is based on:

- identifying the main features of the building, its location, occupancy etc;
- the individual numerical factors associated with the parameters as identified in (1); and
- the combined risk index which is essentially the mathematical product of these individual numerical factors.

Information such as the year built and applicable NBCC are the key parameters in determining the seismic risk of a building. The information, as it relates to the design and construction practices of the existing building, is tied directly to the individual scores for other parameters. Other parameters considered in the screening process include:

- seismicity
- soil conditions
- type of structure
- building irregularities
- building importance (occupancy)
- non-structural hazards (life safety and operation requirement)

### 3.2.1 Seismicity

Seismicity effect is determined by the location of building and the applicable NBCC as given in Table 4. The seismicity of a location is determined by the effective seismic zone, which was defined in the NBCC 1990. The effective seismic zone is equal to $Z_v$ (if $Z_a$ is equal to or less than $Z_v$) or $Z_v+1$ (if $Z_a > Z_v$). $Z_a$ is the zonal acceleration and $Z_v$ is the zonal velocity for a specific location in Canada. The seismic parameter (A) can have a value between 1.0 and 4.0.

**Table 4** Effect of seismicity

| | | Design NBC | Effective Seismic Zone ($Z_v$ or $Z_v + 1$ if $Z_a > Z_v$) | | | | | |
|---|---|---|---|---|---|---|---|---|
| | | | 2 | 3 | 4 | 5 | 6 | |
| A | Seismicity | Pre 65 | 1.0 | 1.5 | 2.0 | 3.0 | 4.0 | A= |
| | | 65-85 | 1.0 | 1.0 | 1.3 | 1.5 | 2.0 | |
| | | Post 85 | 1.0 | 1.0 | 1.0 | 1.0 | 1.0 | |

### 3.2.2 Soil Conditions

Soil condition effect is determined by the type of dominant soil under the building and the applicable NBCC as given in Table 5. There are five different soil categories considered in the screening manual, namely rock or stiff soil less than 50 m deep, stiff soil greater than 50 m deep, soft soil greater than 15 m, very soft or liquefiable soils, and unknown soil condition. The soil condition parameter (B) can have a value between 1.0 and 2.0.

**Table 5** Effect of soil condition

| B | Soil Conditions | Design NBC | Soil Category | | | | | B= |
|---|---|---|---|---|---|---|---|---|
| | | | Rock or Stiff Soil | Stiff Soil > 50 m | Soft Soil >15 m | Very Soft or Liquefiable Soil | Unknown Soil | |
| | | Pre 65 | 1.0 | 1.3 | 1.5 | 2.0 | 1.5 | |
| | | Post 65 | 1.0 | 1.0 | 1.0 | 1.5 | 1.5 | |

### 3.2.3 Type of Structure

Effect of type of structure is determined by the type of structural system of the building and the applicable NBCC as given in Table 6. The screening manual considers both material and system of the building structure. Wood, steel, concrete, precast, masonry infill, and masonry structures are accounted for in the evaluation. The type-of-structure parameter (C) can have a value between 1.0 and 3.5.

**Table 6** Effect of type of structure

| C | Type Of Struc -ture | Design NBC | Construction Type and Symbol | | | | | | | | | | | | | C= |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | Wood | | Steel | | | | Concrete | | Precast | | MI * | Masonry | | |
| | | | WLF | WPB | SLF | SMF | SBF | SCW | CMF | CSW | PCF | PCW | SIW CIW | RML RMC | URM | |
| | | Pre – 70 | 1.2 | 2.0 | 1.0 | 1.2 | 1.5 | 2.0 | 2.5 | 2.0 | 2.5 | 2.0 | 3.0 | 2.5 | 3.5 | |
| | | 70 – 90 | 1.2 | 2.0 | 1.0 | 1.2 | 1.5 | 1.5 | 1.5 | 1.5 | 1.8 | 1.5 | 2.0 | 1.5 | 3.5 | |
| | | Post 90 | 1.0 | 1.0 | 1.0 | 1.0 | 1.0 | 1.0 | 1.0 | 1.0 | 1.0 | 1.0 | 1.0 | 1.0 | — | |

MI * = Masonry Infill
WLF = Wood Light Frame
WPB = Wood, Post and Beam
SLF = Steel Light Frame
SMF = Steel Moment Frame
SBF = Steel Braced Frame
SCW = Steel frame with Concrete shear Wall
CMF = Concrete Moment Frame
CSW = Concrete Shear Walls
PCF = Precast Concrete Frame
PCW = Precast Concrete Walls
SIW = Steel frame with Infill masonry shear Walls
CIW = Concrete frame with Infill masonry shear Walls
RML = Reinforced Masonry bearing walls with wood or metal deck floors or roofs
RMC = Reinforced Masonry bearing walls with Concrete diaphragms
URM = UnrReinforced Masonry bearing wall building

### 3.2.4 Building Irregularities

Effect of building irregularities is determined by the types of irregularities and the applicable NBCC as given in Table 7. Types of irregularities include:

1. Vertical irregularity (abrupt changes in plan dimensions over height)
2. Horizontal irregularity (irregular building shapes in the horizontal plane)
3. Short concrete columns (columns restrained by walls, resulting in columns with short length)
4. Soft storey (severe reduction of stiffness from storey to storey)
5. Pounding (separation between building less than 20 X Zv X number of storeys, in mm)
6. Major modifications (change in function and use or significant addition)
7. Deterioration (damaged or poor condition of structural elements)
8. None

The type-of-structure parameter (D) is determined by the combined effect of all building irregularities. The value of D is between 1.0 and 4.0. For a pre-70 building with both horizontal and vertical irregularities, D = 1.3 X 1.5 = 1.9. For a pre-79 building with horizontal irregularities, short concrete columns and a soft storey, D = 1.5 X 1.5 X 2.0 = 6.0 ==> 4.0 (D can only have a maximum value of 4.0).

**Table 7** Effect of building irregularities

| D | Building Irregularities | Design NBC | 1. Vertical | 2. Horizontal | 3. Short Concrete Columns | 4. Soft Storey | 5. Pounding | 6. Modification | 7. Deterioration | 8. None | D = |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Pre – 1970 | 1.3 | 1.5 | 1.5 | 2.0 | 1.3 | 1.3 | 1.3 | 1.0 | |
| | | 1970 – 1990 | 1.3 | 1.5 | 1.5 | 1.5 | 1.3 | 1.0 | 1.3 | 1.0 | |

*Note: D = product of all applicable values selected = 4.0 (maximum).*

### 3.2.5 Building importance

Effect of building importance is determined by the type and density of occupancy of the building and the applicable NBCC as given in Table 8. The building importance parameter considers post-disaster buildings and special operational requirements. Depending on the occupancy type and density of the building, the building importance parameter (E) has a value of between 0.7 and 3.0.

**Table 8** Effect of building importance

| | Building Importance | Design NBC | Low Occupancy N<10 | Normal Occupancy N=10–300 | School, or High Occupancy N=301–3000 | Post Disaster, Very High Occupancy N>3000 | Special Operational Requirement | |
|---|---|---|---|---|---|---|---|---|
| E | | Pre 70 | 0.7 | 1.0 | 1.5 | 2.0 | 3.0 | E= |
| | | Post 70 | 0.7 | 1.0 | 1.2 | 1.5 | 2.0 | |

N = Occupied Area X Occupancy Density X Duration Factor*

| Primary Use: | Occupancy Density | Average Weekly Hours |
|---|---|---|
| Assembly | 1 | 5 to 50 |
| Mercantile, Personal service | 0.2 | 50 to 80 |
| Offices, Institutional, Manufacturing | 0.1 | 50 to 60 |
| Residential | 0.05 | 100 |
| Storage | 0.01 to 0.02 | 100 |

*Duration Factor is equal to the average weekly hours of human occupancy divided by 100, not greater than 1.0.

### 3.2.6 Non-Structural Hazards

Effect of non-structural hazards is determined by the type of hazards (life safety or operational requirements) and the applicable NBCC as given in Table 9. Special considerations are also given to the type of structure (such as SMF – Steel Moment Frame and CMF – Concrete Moment Frame) and building irregularities (such as soft storey and horizontal irregularities). Value of the non-structural parameter (F) is the maximum of $F_1$ (due to falling hazards to life) and $F_2$ (hazards to vital operations). Both $F_1$ and $F_2$ can have value of between 1.0 and 6.0 (i.e. value of F is between 1.0 and 6.0).

**Table 9** Effect of non-structural hazards

| | Non-Structural Hazards | | Design NBC | None | Yes | YES* | |
|---|---|---|---|---|---|---|---|
| F | $F_1$ | Falling hazards to life | Pre-1970 | 1.0 | 3.0 | 6.0 | F = max ($F_1$,$F_2$) |
| | | | Post-1970 | 1.0 | 2.0 | 3.0 | |
| | $F_2$ | Hazards to vital operations | Any Year | 1.0 | 3.0 | 6.0 | |

*YES* - applies only if or more of the following descriptors are circled: SMF, CMF, soft storey, torsion (horizontal irregularities)*

### 3.2.7 Seismic Priority Index

The scoring system is made up of a structural index (SI) and a non-structural index (NSI). SI is related to possible risk to the building structure, and NSI is related to the risk of non-structural building components.

The structural index, SI, is calculated as follows:

$$SI = A \bullet B \bullet C \bullet D \bullet E,$$

where A, B, C, D and E account for effects of seismicity, soil conditions, type of structure, building irregularities, and building importance as defined and determined in sections 1.1 to 1.5.

The non-structural index, NSI, is calculated as follows:

$$NSI = B \bullet E \bullet F,$$

where F is the maximum value between $F_1$ and $F_2$ as defined and determined in section 1.6 above.

The seismic priority index, SPI, is equal to the sum of the structural index and non-structural index (SPI = SI + NSI). The seismic priority index is related to the seismic risk for a building as per the NBCC 1990 requirements. The seismic manual suggests that the potential seismic risk for a building is low with a SPI less than 10, medium with a SPI between 10 and 20, and high with a SPI higher than 20. It is desirable to conduct a detailed assessment of a building with a SPI of greater than 15. Buildings with SPI scores of greater than 30 can be considered high risk; Ian immediate assessment of the seismic performance of such buildings is required.

## 3.3 Effects of Changes Between NBCC 1990 and NBC 1995 on the Screening Parameters

The National Building Codes of Canada have been continuously modified and improved in the past. All these changes have had effects on the base seismic design base shear (i.e. the total lateral seismic force). However, significant changes were introduced into the 1985 and the 1990 editions of NBCC. These changes can be illustrated by considering the base shear specifications. The base shear, V, in the 1985 NBCC was specified as:

$V = vSKIFW$ where,

v = zonal velocity ratio
S = seismic response factor
K = structural system coefficient
I = importance factor (1 for buildings of normal importance)
F = foundation factor (1 for buildings on rock or stiff soil)
W = dead load

Note that in the pre-1985 editions, the seismic hazard for a given area (or zone) was represented by zonal acceleration, and the effects of the seismic motion on the total lateral force was represented by the seismic response factor, which was defined with a single curve for all structural periods. In the 1985 NBCC, the seismic hazard for a given location was represented by two parameters: the zonal acceleration ratio, a, and the zonal velocity ratio, v. The zonal acceleration ratio represented the ratio of the horizontal peak ground acceleration with a probability of exceedance of 10% in 50 years to the acceleration of 1 g. Similarly, the zonal velocity ratio represented the ratio of the horizontal peak ground velocity with a probability of exceedance of 10% in 50 years to the velocity of 1 m/s. The seismic response factor, S, in the 1985 NBCC was represented by a single curve for structural periods, T, longer than 0.5 s and was defined as $S = 1.5/T^{1/2}$. For periods below 0.5 s, the S factor was represented with three branches that were associated with three ranges of zonal acceleration to zonal velocity ratios, a/v (i.e. $a/v > 1$, $a/v = 1$, and $a/v < 1$) These three branches are defined as follows:

- For zones with $a/v > 1$, the S factor is represented by a plateau at a level of 4.2 for periods below 0.25 s, and a linear decrease from 4.2 to 2.1 between periods of 0.25 and 0.5 s respectively;
- For zones with $a/v = 1$, the plateau is at a level of 3.0 and the S factor decreases from 3.0 to 2.1 between periods of 0.25 and 0.5 s respectively; and
- For zones with $a/v < 1$, the S factor has a value of 2.1 for all periods below 0.5 s.

As an example, the seismic response factors for Montreal and Vancouver, as representative locations of seismic hazards in eastern and western Canada respectively, are defined as:

- For Montreal, $a/v > 1$ (i.e. $a = 0.2$, $v = 0.1$), and the short period plateau of the S factor is at a level of 4.2; and
- For Vancouver, $a/v = 1$ (i.e. $a = v = 0.2$), and the plateau of the S factor is at a level of 3.0.

For periods longer than 0.5 s, the S factor is the same for both locations, i.e. $S=1.5/T^{1/2}$.

In the 1990 edition of NBCC, the base shear was expressed as:

$V = (Ve/R)U$ where,

U = 0.6 is a calibration factor,
R = force modification factor (values range from 1 to 4), and
Ve = elastic lateral seismic force, which is given by
Ve = vSIFW

in which the parameters v, S, I, F, and W have the same meaning as those in the 1985 NBCC. It is important to note that the U factor was included in this equation in order to calibrate the 1990 NBCC base shear to that of the 1985 NBCC. The seismic zoning maps are the same as those in the 1985 NBCC. In other words, the base shear in 1985 NBCC and in 1990 NBCC is approximately the same.

No further changes have been done since 1990, and the current NBCC are practically the same as those of the 1990 edition. Therefore, the screening parameters associated with the 1990 NBCC are still valid.

### 3.4 Effects of the Proposed Seismic Requirements (for NBCC-2003) on the Screening Parameters

Recently, the Canadian National Committee on Earthquake Engineering (CANCEE) has considered a new hazard level for the upcoming code cycle. The new hazard level is based on 2% probability of exceedance in 50 years, and may be implemented in the upcoming edition of the code through site-specific uniform hazard spectra. The new hazard level is based on the new generation of seismic hazard maps generated by the Geological Survey of Canada (GSC). The next edition of the code is likely to become available in 2003. The proposed changes will result in substantial divergence from the current design practice, which is based on the provisions of 1995 NBCC. These changes may have a significant impact on the screening parameters.

An important aspect of the proposed changes in NBCC, aside from the new hazard level, is the use of Uniform Hazard Spectra (UHS) for the determination of seismic base shear. The earthquake hazard level considered in the current 1995 NBCC design requirements is based on an earthquake event that has 10% probability of exceedance in 50 years. This level of hazard is introduced into the design process by specifying an idealized response spectrum derived from the peak ground acceleration and peak ground velocity of a given seismic region. However, it has been shown by GSC that the hazard level, as indicated by return period of certain probability of exceedance, changes with peak spectral acceleration at different rates in different regions of Canada. For example, in Vancouver, representing western Canada, the peak spectral acceleration would not vary as rapidly with the return period as that in Montreal, representing eastern Canada. Therefore, basing the design spectrum on peak spectral values may imply different levels of hazard in different regions of the country. The alternative approach, recently proposed by GSC, is the use of site-specific Uniform Hazard Spectra (UHS).

The UHS have been generated for different locations in Canada, and for different hazard levels. These spectral curves provide a more accurate reflection of Canadian seismicity, and may lead to a more realistic determination of seismic demands.

The provisions for seismic design of structures in the upcoming 2003 edition of NBCC have not been finalized at the time of the preparation of this document. However, a draft document is under preparation with some recommendations (discussed in the following paragraphs) which will then be released for public review and comment. Upon feedback from this review process, there may be some changes introduced for further improvement and refinement of the document. Therefore, it is not possible to provide an exact assessment of the impact of the upcoming changes in NBCC 2003. However, it is possible to provide an approximate estimate of their impact on the seismic screening process. The draft recommendations for NBCC 2003 edition are likely to include the minimum lateral earthquake force V as calculated from the following formula:

V = S(T) Mv I W / (Rd Ro)

Where, V should not be less than:

S(2.0) Mv I W / (Rd Ro)

S(T) is the design spectral acceleration value, and is determined as follows:

S(T) = FaSa(0.2) for T ≤ 0.2 sec.
S(T) = FvSa(0.5) or FaSa(0.2), whichever is smaller for T = 0.5 sec.
S(T) = FvSa(1.0) for T = 1.0 sec.
S(T) = FvSa(2.0) for T = 2.0 sec.
S(T) = FvSa(2.0) / 2 for T ≥ 4.0 sec.

Linear interpolation is permitted for in-between values of fundamental period T.

Acceleration and velocity related coefficients Fa and Fv are specified as a function of soil site classification, as well as the design spectral acceleration values. The soil classification is specified as "Hard Rock," "Rock," "Very Dense Soil and Soft Rock," "Stiff Soil," "Soft Soil" and "Others," where the last category is intended for liquefiable soil, quick and highly sensitive clays, and other soils susceptible to failure or collapse under seismic loading. Sa(T) is the 5% damped spectral response acceleration for period T, and is based on a 2% probability of exceedance in 50 years. The Sa(T) values are obtained from the site specific UHS.

Coefficient Mv is specified to reflect higher mode effects, along with its associated based overturning moment reduction factor, J. The values of Mv and J are specified as a function of period T, and are also based on the ratio of spectral acceleration at 0.2 sec to that at 2.0 sec. [Sa(0.2)/Sa(2.0)]. They are presented for three different structural types, specified as "moment resisting frames or coupled walls," "braced frames," and "walls, wall-frame systems, and other systems." The higher mode effects are more prevalent in wall structures with longer periods.

W is the weight of the structure, as determined in the current NBCC 1995, as well as the NBCC 1990. The numerator of the above equation provides the elastic design shear force, and is divided by ductility and over-strength factors – Rd and Ro respectively – to arrive at the inelastic design force. The calibration factor (U = 0.6) used in the earlier editions of NBCC is eliminated from the base shear expression, since over-strength in structures believed to be included in this calibration factor is accounted for by factor Ro, and the change in the hazard level compensated for the calibration needed to conform to the previous history of successful applications. The importance factor, I, is specified as 1.5 for Post Disaster Buildings, 1.3 for schools, and 1.0 for all other buildings.

A comparison of static base shears computed based on the proposed provisions and those computed on the basis of NBCC 1995 indicates that some discrepancies do exist between the two sets of values. In certain cases this discrepancy can be quite significant, though the majority of the values appear to be comparable. The regions where the discrepancies exist are limited to only a few locations of sparsely populated regions of the west, and already low seismic risk regions in the east. The impact of these discrepancies should be assessed once the new provisions are finalized. Perhaps the most significant aspect of the proposed provisions is the use of an entirely different approach, and different classifications in terms of the spectral values, soil types, ductility, and over-strength characteristics of different types of structures, as well as the classification of building importance.

In addition to the revisions introduced to the calculation of equivalent static loads, dynamic analysis procedures are given prominence in the proposed code provisions. Unlike the previous editions of NBCC, the proposed edition specifies dynamic analysis as the primary method of analysis, limiting the equivalent static load analysis to regular structures, even though the minimum base shear established by the dynamic analysis is limited to 80% of that computed on the basis of equivalent static loads. The non-linear dynamic analysis is also included in the proposed code provisions for the first time in NBCC as a procedure that requires a special study. These improved analysis techniques are expected to improve the accuracy of seismic design in buildings.

It is clear from the foregoing presentation of the revisions proposed for adoption in NBCC seismic provisions that significant differences are likely to emerge in the upcoming edition of the code. This implies that the existing screening parameters will have to be revised in the future, and a new screening document will need to be developed.

## 3.5 Summary and Conclusions

The seismic screening procedure developed by NRC is currently the only procedure used in Canada for screening buildings as part of the seismic evaluation process. This procedure is outlined in a document published by NRC (1993), and includes the building location, soil conditions, type and use of the structure, obvious building irregularities, the presence or absence of non-structural hazards, building age, building importance, and occupancy characteristics as major factors in determining the screening score. The benchmark for screening is the 1990 edition of NBCC. The current edition of NBCC was published in 1995 (NRC 1995). The NBCC 1995 provisions for seismic design are evaluated in this report to assess the significance of changes in terms of their impact on the NRC seismic screening procedure. It is concluded that the changes introduced to the 1995 edition are not significant enough to invalidate the NRC screening document.

In addition, the draft seismic provisions currently being formulated by CANCEE (Canadian National Committee for Earthquake Engineering) for the new objective-based national code are reviewed to assess their impact on the existing screening process. It is suggested that revisions to the NRC 1993 screening methodologies may be necessary to reflect the impact of upcoming new provisions on seismic screening procedures employed in Canada.

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