Techniques of Seismic Retrofitting For Concrete Structures

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Abstract
Recent earthquakes, starting with the 1971 San Fernando Earthquake in California, left major destructions, damaged the infrastructure, and raised questions about the vulnerability and design practice of structures, especially concrete structures. Design codes have being updated to include seismic previsions but structures built before 1971 have to be retrofitted. The focus of this paper is concrete structures. Surveys done after earthquakes have shown that the major problem with concrete structures is columns. Pre-1971 detailing left column with lack of confinement as well as lap-slice in plastic hinge regions creating potential failures in flexure strength and/or ductility, and in shear. Other critical structural elements include, but are not limited to, gravity design frames, footings, shear walls, connections, and beams. There are two major categories of retrofit options for concrete structure; local and global methods. Local methods focus at the element level on a particular member that is deficient and in improving it to perform better. Those methods include adding concrete, steel, or composite to the outside of the member. All three methods are effective but each present some disadvantages: concrete is labor intensive, steel requires heavy construction equipments, and composites have high initial cost. Global methods concentrate at the structure level and retrofit to obtain a better overall behavior of the entire structure. The different global techniques are addition of shear walls or steel bracings, and base isolation. All three methods are effective. Shear walls are usually an expensive solution but they are flexible in their distribution allowing them to be hidden in the architecture. Steel bracings allow for openings but their connections to the existing structure can be problematic. Finally, base isolation is an option that is becoming increasingly popular and that provides good behavior in earthquake for low to mid high structures. The different systems presented all have some advantages and disadvantages and the option chosen for the retrofit depends on the existing structure requirement. The different system presented can be combined to provide more efficient and more flexible retrofit schemes.

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1. Introduction

Since the 1970 several major earthquakes have caused heavy damage in California and in the rest of the World. Those have brought forth the vulnerability of structures and raised awareness everywhere about the need for seismic retrofitting. Design codes now include seismic provisions, and buildings build today are designed to withstand earthquakes. But many structures that were build before the 1970’s need to be retrofitted.

Attention needs to be given to both steel and concrete structures but this paper will focus on concrete structures and their retrofit methods.

After earthquakes, surveys have analyzed damaged and collapsed structures to understand their failure mechanisms. There are two major types of retrofit methods that can be used. The first are local methods that focus on the member level. They include an analysis of the structure to find the deficient elements and the retrofit of these elements. Local retrofit methods include the addition of concrete, steel, and composite. The second set of methods is a global approach that retrofits the entire structure to improve its overall behavior. Those methods include addition of shear walls or steel bracings, or the use of base isolation.

The results of the damage survey will be presented first. Then attention will be given to the retrofit methods, their description and advantages, as well as example of their application. The local methods will be presented first followed by the global ones.
2. Structure Failure Mode in Earthquake: A Survey of Typical Damage

2.1. General

Several recent earthquakes have reminded us of how vulnerable structures really are. In the United States, the 1971 San Fernando earthquake (6.6 Richter scale) hit California causing 65 deaths and over $500 millions in property damage\(^1\). Again, on the 17\(^{th}\) of October 1989, early in the morning, the Loma Prieta (7.0 Richter Scale) hit, shaking the San Francisco Bay Area causing 63 deaths, and creating more then $5.9 billions in property damage. Again, on January 17\(^{th}\), 1994, in just 10 devastating seconds, the Northridge earthquake (6.7 Richter scale) killed 57 people and destroyed or severely damaged 40,000 structures causing billions of dollars in losses. Other countries such as Mexico (Mexico City, 1985) Japan (Hansin-Awaji Kobe, 1995) and Turkey (Kocaeli 1999, Bolvadin 2002, and Bingol 2003) have equally suffered damaging earthquakes\(^2\).

After major earthquakes, the damages are surveyed and analyzed to understand the cause of the structural failure in the hope of gaining more understanding and avoiding collapses in future earthquakes. In the United States, the most critical elements became apparent after the 1971 San Fernando earthquake and have been linked to design practices before 1971. Five years later, in 1976, the United Building Code (UBC) caught up and made mandatory some important earthquake design elements such as ductile detailing. Subsequent changes have been made to building codes and structures build today have better chances of surviving major earthquakes.

Older structures build before 1971, are or have been surveyed to assess their deficiencies and to consider potential retrofit. Several structures have been retrofitted and underwent earthquake where they suffered only limited damages showing that retrofits work. Examples of such structures are some of California’s bridges. In 1994, when the Northridge Earthquake hit, the vivid images of the aftermath and of the destruction of seven major freeway bridges in and around Los Angeles only told part of the story (See Figure 2.1-1). The state of California’s department of transportation (CALTRAN, a

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leading figure in earthquake works in the United States) had before the earthquake retrofitted 122 bridges many of which were in the Los Angeles area and did not suffer any major damages\(^3\). Most of the bridges that were damaged were on the list of bridges to be retrofitted but work had not started on them. Post-earthquake analysis revealed that they could have survived had they been retrofitted in time\(^4\).

![Aerial view of interstate 5 collapse at Gavin canyon (Northridge earthquake, 1994)](image)

**2.2. Structural elements**

**2.2.1. Columns and piers**

The 1971 San Fernando earthquake left many structures damaged with columns and piers failures that often resulted in the collapse of the structure. Some of the major deficiencies in both columns and piers are listed below (See Figure 2.2-1).

- **Inadequate flexural strength:** Before 1971 lateral force coefficients were generally less then 10% resulting in high potential ductility demand.

- **Inadequate flexural ductility:** This type of failure comes from a lack of confinement of the concrete core followed by a failure in the plastic hinge region. This defect is a major design flow and is directly linked to pre-1971 practices which required, for transverse reinforcements of columns, No. 4 bars spaced 12 in (0.3m) on center. This was applied to every columns regardless of geometry (circular or rectangular) or dimensions. Also, the general practice was to close the transverse reinforcements by lap-slice. This technique does not provide good anchorage for the rebar and under pressure the bars deform and open up. More effective techniques for closing rebar include welding or anchoring (bending back into the concrete core). Those deficiencies limit the ultimate curvature in the plastic hinge region of the column to
the strain at which the cover concrete starts spalling (0.5%). As the longitudinal strain increases the hoops steel unravel and the already small amount of confinement is even further reduced.

- **Undependable flexural capacity**: Longitudinal lap-slices were only design for compression and are often located near the ends of columns. It was found that during earthquakes the longitudinal bars could also be subjected to high tension and that the locations where their slices are located are the same areas where plastic hinges will develop. (Current practices have lap slice located in the central portion of the column and designed as tension splice). Also, the length of the lap slices were traditionally 20 bar diameters which is insufficient to develop yield strength in the bars (especially when larger diameter bars are used). All of those elements lead to rapid reduction of flexural strength during cyclic loading.

- **Inadequate shear strength**: Shear failure develops principally in columns with a small height-to-depth ratio, those are either columns that were designed to be short or longer columns that are partially restrained by non-structural elements over a portion of their height (captive columns). Pre-1971 designs were based on elastic methods and used less severe shear requirements. As a result, the shear strength in columns is often less than that needed to develop flexural strength in the member. Shear failure are often brittle, they occur in the form of major diagonal cracking along the entire length of the column, along with the yielding of the longitudinal reinforcement.
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Figure 2.2-1: Columns and piers collapsed. A- San Fernando (1971). B- Loma Prieta (1989) Cypress street viaduct (Interstate 880) ²

The potential occurrence of the different failure modes depends on the height of the column, the geometry of the cross-section, the longitudinal and transverse reinforcement distribution, and the presence of stiffening elements.

2.2.2. Reinforced concrete frame

Practices for the design of frame structures in low to moderate seismic regions (eastern and central United States, but similar problems can also be found in structures located in the western United States) have been to design the structures for gravity load only disregarding lateral loads. This creates several deficiencies which are analyzed below (See Figure 2.2-2).

- Columns are weaker than their joining beam (weak column/strong beam behavior). This creates a structure with potential failures in a soft-story or column sideway mechanisms.
- Columns deficiencies are similar to the one discussed above (see section 2.2.1)
- Beam-column joints are often deficient. They have little to no transverse shear reinforcement and the positive (bottom) beam reinforcement is discontinued in the joints.

A gravity design frame will exhibit low-lateral strength resistance. This allows for large deformations and large inter-story drifts during moderate earthquake. In larger
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earthquakes, and because of the inadequate ductility of the columns, the frame will experience a brittle soft story or column sideways mechanism\(^5\).

![Image of building damage](image)

**Figure 2.2-2: Soft story collapse San Francisco, Northridge (1994)**\(^2\)

### 2.2.3. Other structural elements

There exists several other structural elements that have been observed to fail during earthquakes. Footing, shear wall, and coupling beams are just a few that might also experience deficiencies. Often, their failures though damaging will not result in the immediate collapse of the entire structure.

- **Footing failure:** Many older footings were only designed for gravity loads. As a result, they have several deficiencies. First, they are often undersized and vulnerable to overturning moments. Secondly, they do not have top reinforcements making them subjective to brittle failures. Thirdly, they are vulnerable to shear in both the footing and in the footing-column joint area. Finally, pile footing in older designs often did not have structural connections between the piles and the pile cap.\(^6\)

- **Coupling beam and shear walls:** Shear walls are most often damaged in shear and exhibit X-pattern cracks. Coupling beams can have inadequate capacity; particularly shear capacity, which are insufficient to develop flexural yielding of the beam\(^7\).
3. Local method

3.1. Concrete and Section Enlargement

3.1.1. General

Section enlargement consists in placing additional concrete around an existing structural element to increase its seismic resistance. This is the oldest method of seismic retrofitting. Typical applications include bridge deck, column wrapping, and joint strengthening. This method is easy and economically effective, but labor intensive.

3.1.2. Traditional concrete

Adding traditional concrete has been used as a mean of retrofit for many years. It is used to reinforce columns either by themselves or in the context of retrofitting gravity designed frames. It can also be used for other structural features such as foundation. It is used mainly when strengthening is needed.

As was discussed in section 2.2.1, columns are one of the structural elements that are often in need of retrofit in both buildings and bridges. This method has been used for a number of years and was, for example, widely applied after the 1985 earthquake in Mexico City. Numerous studies have been done in the past but this method has been proven and research has moved on to other materials. One example of such past studies was completed in 1994 by M. Rodriguez and R. Park on 4 RC columns at a 7/8 scale. (See Figure 3.1-1)

![Figure 3.1-1: Seismic load test - Strengthened by concrete jacketing (A- columns detail B- results)](image)

Figure 3.1-1: Seismic load test - Strengthened by concrete jacketing (A- columns detail B- results)
Different detailing and different situation (pre-damage vs. non damage) were tested and the results showed that the retrofitted columns exhibited higher strength and stiffness as well as higher durability and very good energy dissipation. They also showed that neither the detailing, nor the original state of the column, had much influence but that what was important was good surface preparation. Another variation of column retrofitting is to wrap the columns with a concrete jacket with added longitudinal and transverse reinforcements and in post-tension of the new longitudinal reinforcements.

Section enlargements have also been proven to be affective for the seismic retrofit of RC buildings that were originally only designed for gravity loads. As discussed in section 2.2.2, those buildings are subjected to soft-story collapse mechanisms, which come from a strong beam/weak column combination. The goal of the retrofit is to transform this system into a weak beam/strong column behavior which is preferable and can be done by adding strengthening to the columns. Research on 1/3-scale model, has shown that retrofitting the interior columns with pre-stressed concrete jackets obtains the desire behavior. (The exterior columns were less critical since they are subjected to less flexural demands and that they will be subjected to controlled story drift only, since the drift is controlled by the interior retrofitted columns). The experiment was successful and showed a change in behavior (nominal beam ratio strength from 0.6 to 1.59-5.85 i.e. weak beam/strong column behavior), a controlled inter-story drift and increased in the story stiffness and natural frequency.\(^5\)

Another structural element which can benefit from added concrete is foundations. A study of the problem was performed by McLean and Marsh\(^6\). The deficiencies were presented in section 2.2.3 and the recommended retrofit is to add a layer of concrete to the top of the footing. This will increase its shear resistance, by both allowing for the addition of top mat reinforcements which will provide negative moment strength, and by increasing the effective depth which will increase the positive moment capacity. For footings that are vulnerable to overturning, the seismic retrofit options are to enlarge them, or to provide additional piles, or to tie them down.
3.1.3. Shotcrete

A later development in section enlargement is shotcrete. It is a mortar or concrete pneumatically projected at high velocities onto surfaces. It was introduced in 1911 and has been used in retrofit applications for over 50 years. The invention of the shotcrete gun has been attributed to Carl E. Akeley and shotcrete comes in both wet and dry mixed forms. (See Figure 3.1-2)

![Figure 3.1-2: Shotcrete application](image)

Shotcrete’s main advantage is it eases of application especially in hard to access areas which result in a reduction of construction time and cost. It has a dense composition and has low shrinkage and low permeability which gives it a good durability. The main disadvantage of using shotcrete is that special attention and procedure is required in order to achieve a good quality product. These include placing thick sections in layers, using of a blow-man to help reduce rebound (when the shotcrete hit a hard surface some of the larger aggregate tend to ricochet and gather in the same spot), and requiring quality control and inspections. Finally, it should be noted that with shotcrete, as with all modes of repairs, attention must be given to the bond area and to the surface preparation of the existing concrete.

An example of the use of shotcrete was the retrofit of the Littlerock Dam (See Figure 3.1-3) in southern California.
The dam is a 28-arches historical structure designed by John E. Eastwood and build in 1924. The dam is actually located 1.5 miles (2.4 km) south of the San Andreas Fault and studies have showed that it lacks lateral stability. The retrofit was done in two parts. The first was the addition of gravity portion on the downstream side and the second, and the one of interest here, was the stiffening of the arched with a layer of bonded steel fibers reinforced shotcrete on the upstream side (See Figure 3.1-4). The shotcrete resulted in increasing the stiffness and the tensile strength of the arch faces. One element of concern was to obtain a full bond between the shotcrete and the existing concrete. The following steps were taken here to insure this: roughness of the existing concrete obtain by both sandblasting and chipping, moisture of the face of the dam for a period of 24 hours prior to applying the shotcrete, cleanness of the concrete water by washing it with fire hoses, and hardness. To insure an even stronger bond, an anchorage system was provided. It should be noted that, as previously recommended a blow-man, was provided to help prevent rebound.11

Figure 3.1-3: Littlerock dam

Figure 3.1-4: Littlerock dam restoration

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3.1.4. Polymer concrete composite

Another development in the concrete field is the use of polymer concrete (PC). PC are made from a polymer binder (usually a thermosetting polymer) mixed with a mineral filler; either sand (for mortar), or aggregates, gravel or crushed stone (for concrete). The material has several advantages such as high strength, low permeability, excellent resistance to chemicals and abrasion, and good adhesive properties. However, its disadvantages are cracking due to restrained volume changes, poor resistance to ultraviolet light, creep at high temperature, and additional cost. It is uses in the resurfacing deteriorated structures and as a compound in the repair of concrete structures. Even though it presents several advantages PC is more expensive and does not solve the extensive labor issue of using regular concrete.

3.1.5. High performance fiber reinforced cement composite

A more recent development is fiber reinforced concrete and even more recently of high performance fiber reinforced cement composites. High performance materials are defined as material exhibiting a post-peak strain hardening response associated with multiple cracks and high energy dissipation (quasi-strain hardening behavior). Advantages of using such materials are improvements in energy absorption, toughness, ductility, and cracking shear resistance. They are also good in accommodating differential elastic modules and thermal expansion as well as having a good ductility and a good crack distribution with small width. However, they come at a high initial price due to their labor intensity.

Two examples of high performance reinforced concrete materials are slurry infiltrated mat concrete (SIMCOM) and slurry infiltrated fiber concrete (SIFCON). SIMCON is a pre-placed continuous stainless steel fiber mat infiltrated with a cement based slurry. The mat are pre-fabricated and brought to site in large rolls. The pre-fabrication has the advantage of providing a control environment when placing the fibers. This allows for construction of mats with specific fiber orientation, contributing to their strength. The procedure is simpler then normal reinforced concrete since the frame work is reduced. (See Figure 3.1-5)
SIFCON is pre-placed short steel fibers which are then infiltrated with cement based slurry. In this method, the fibers are placed on site such that their orientation cannot be controlled and a greater fiber volume is required. Typical applications are beam-column connection and column wrapping. They result in an increase in the strength and in the energy absorption capacity of the retrofitted members. The advantages of such methods are that they are less labor intensive than conventional concrete or steel jacketing and that they used standard method and construction equipments. However, they are new and still in the development stage, they will probably have an important impact on the field once research is completed.

Another example of the use of micro fiber high performance reinforced concrete is thin repairs. Thin repairs are difficult in the sense that they do not usually carry load; but that they have to insure strain compatibility, which will result in the development of high stresses. Also, they need the following three properties: the ability to stop degradation of the existing structure especially steel corrosion, the creation of a proper bond with the existing structure, and the durability and capacity to withstand sever climatic conditions. Research at the University of British Columbia (Professor Banthia and Research Associate Cheng Yan) has tried to implement those repairs using fiber reinforced cement composite containing high volume fractions of micro fiber such as carbon, steel or polyvinyl alcohol and polymer. A significant improvement of the bond strength was noticed. This cannot be due physically to the fiber (since none actually bridge the gap) but they improve the quality of the interface by, for example reducing shrinkage. It was
noted that the kind of fiber has little significant and that the surface need proper preparation.16

3.2. Addition of steel

3.2.1. General

Addition of steel is often applied in the form of plates or jackets. Steel tendons can also be used in external pre-stressing.

Advantages of using steel include that it does not add significant weight to the structure (or to its footings) in comparison with concrete, and that it saves on construction time (no curing time). The members are pre-fabricated offsite and are more rapidly installed which is less disruptive to the building occupants then other techniques. Disadvantages are linked to construction issues: steel can be labor intensive, time consuming, and require heavy equipments to handle thousands of tons, as well as having a more difficult maintenance. A typical column retrofit will take an average of 2.5 days excluding site excavation and painting. Also because of construction methods, the retrofit design is often governed by site installation demands rather then by necessary retrofit needs. One issue is that the steel needed could end up being heavier and stronger then the retrofit requirements to prevent it from buckling under its own weight during lifting and placing. Each element could also have to be limited in their size and require splices and a more complicated design so that they are not to heavy to be carried. Steel also requires continuing maintenance especially in corrosion protection. It results that steel is often an expensive option4.

3.2.2. Steel Jacket: Column

Steel jacket can be used to retrofit both column and joints. Column retrofit will be discussed first.

After the 1971 San Fernando Earthquake, reinforced columns were recognized as a structural element that needed more attention (see section 2.2.1). Retrofit of columns using steel jackets has been extensively studied in the 1990’s, mainly in the context of
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bridge columns. This research was primarily founded by CALTRAN (California Transportation Department). They have shown that this technique provides good overall behavior with increase ductility, shear strength, and energy dissipation. This method is now widely used in the United States and in Japan.

The principal behind this technique is that the steel jacket acts as a passive confinement reinforcement. The jacket will prevent the concrete from dilating, forcing it in lateral compression and increasing its compressive strength, its effective ultimate compressive strain, and its ductility. For circular columns, the method uses two semicircular half sections that are field welded along the entire height of the jacket. A gap of about 1 inch (2.5 cm) is left between the column and the jacket. It is filled with a cement-based grout that will ensure a good bonding and composite behavior. Use of expensive grout instead of the cement base one does not improve the performance. A gap of 2 inches (5 cm) is also left between the bottom of the columns and the top of the footing to avoid possible bearing of the jacket on the footing. For rectangular columns the retrofit options are to either use a rectangular jacket or a circular (or elliptical) jacket. In the case where a rectangular jacket is used the procedure is the same, and two L shaped panels are field welded together. For circular (or elliptical) jackets, the gaps created are larger and should be filled with concrete instead of grout (See Figure 3.2-1). It should also be noted that depending on the application conditions and failure mechanisms partial jacket or steel collar may be used.

Figure 3.2-1: Seismic retrofit of a rectangular section

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The design and requirements for the steel jacket depends on the mode of failure, flexural or shear. Each mode is discussed below.

### 3.2.2.1. Flexural strength failure and/or flexural capacity

The first failure type is inadequate flexural strength and/or flexural capacity. This type of failure comes from a lack of confinement of the concrete core followed by a failure in the plastic hinge region. Current practices have column designed with closely spaced lateral reinforcements. In comparison, they behave with an increase in compressive strength and in ultimate strain of the core (the strain goes from 0.003 to 0.005), as well as with the development of stable hysteresis loops. To provide the same type of confinement to the deficient columns, lateral reinforcements would need to be placed outside the column, weld lap, and cover with shotcrete. A comparable (if not better) reinforcement can be provided by steel jackets and are simpler, less expensive, and more esthetic then the previous described method.

The flexural capacity at the base of a column is related to the confinement level in the plastic hinge region. When cracks occur at the concrete-steel interface, a steel jacket will provide a radial pressure through passive confinement. (See Figure 3.2-2)

![Figure 3.2-2: Confining action of steel casing](image)

Ignoring the contribution of existing hoop reinforcement, the radial confining stress at yield due to the jacket is given by:

$$ f_i^c = \frac{2 \cdot f_{sj} \cdot t_j}{(D_j - t_j)} $$
Where $f_i$ is the maximum confining stress from the steel jacket,

$$f_w$$  

is the yield strength of the jacket,

$t_j$  

is the thickness of the jacket, and

$D_j$  

is the outside diameter of the jacket.

The compressive strength of the confined concrete can then be estimated by

$$f'_c = f_c \left(2.254 + \frac{7.94 f_i}{f'_c} - \frac{2 f_i}{f_c} - 1.254\right)$$

Where $f'_c$ is the confined compressive strength of the concrete,

$f'_c$  

is the unconfined strength of the concrete, and

$f_i$  

is the maximum confining stress from the steel jacket.

The increase in the concrete compressive strength will improve the compressive deformation capacity and the flexural capacity of the column.

For columns that have a lap-slice region at the interface with their footing, the flexural failure occurs through a sliding mechanism between the longitudinal reinforcement and the dowel bars of the footing. Often this sliding occurs before the flexural capacity of the section can be reached. Similarly confinement of the area and application of a radial pressure will prevent sliding. Research has shown that a coefficient of friction of $\mu=1.4$ should be provided at the crack interface. To achieve this coefficient, prevent excessive dilatation, and avoid yielding of the steel jacket, the hoop strain should not exceed a strain of $\varepsilon_{sj} = 0.0015$. (It is important that the steel jacket doesn’t yield because yielding would increase its deformation, which would reduce the amount of provided reinforcement). The corresponding required thickness of the jacket needed is given by:

$$t_j = \frac{2.42 A_b f_j D}{4 pl_s \left(0.0015 E_{sj}\right)}$$

where $t_j$ is the thickness of the steel jacket,

$A_b$ is the cross-sectional area of the longitudinal bars,
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\( f_y \) is the yield strength of the longitudinal reinforcement,

\( D \) is the diameter of the steel jacket,

\( P \) is an equivalent diameter of the cracked concrete around the longitudinal bars,

\( l_s \) is the lap-splice length, and

\( E_{sf} \) is the modulus of elasticity of the steel jacket\(^{17}\).

It should be noted that other problems have also been noted with deficient footing during some of the experiments. Analysis and retrofit of those footing should also be considered (see section 3.1.1).

When flexural failures are considered, failure will occur at the plastic hinge and so the location of the failure can be predicted. Research has shown that just confining the critical area with a partial jacket could be sufficient for retrofitting the column (confining an area 1.5 times the splice length). Research also focused on rectangular column. A rectangular jacket will not provide enough confinement pressure to be as effective as a circular jacket and so it is recommended that for flexural failures a circular or elliptical jacket be used (the difference between elliptical or circular jackets are minor). In cases where this is not practical (especially in buildings were space is limited), it was found that rectangular jackets will enhance the response, especially if they are stiffened with at least four adhesive anchor blots: two at the top and two at the bottom\(^{20}\).

### 3.2.2.2. Shear failure

The second major type of failure in columns is shear failure. The shear force carried by a column section is the sum of the ones carried by each element i.e.

\[ V_u = V_c + V_s + V_n \]

Where \( V_u \) is the ultimate shear strength,

\( V_c \) is the shear strength of the concrete,

\( V_s \) is the shear strength of the steel reinforcement, and

\( V_n \) is the shear strength contributed by the presence of a vertical load.
To be able to increase $V_u$, either the existing shear resisting elements can be improved, or an additional shear resisting element can be added to the system. One such element is the steel jacket. The required shear strength to be contributed by the jacket is:

$$\beta V_{sj} \geq V_o - \beta^* (V_c + V_s + V_n)$$

Where $\beta = 0.7$,

$V_{sj}$ is the required shear strength of the steel jacket,

$V_o$ is the shear force induced by the maximum probable flexural capacity of the plastic hinge, and

$V_c$, $V_s$, $V_n$, are as described above\(^\text{17}\).

An estimation of $V_{sj}$ can be taken as a continuous transverse reinforcement of cross section equal to its thickness and with spacing equal to unity such that

$$V_{sj} = A_{sj} \left( \frac{F_{y sj}}{2} \right) \frac{d_{sj}}{s_{sj}}$$

Where $V_{sj}$ is the shear strength of the steel jacket,

$A_{sj}$ is the area of assumed squared ties,

$F_{y sj}$ is the yield stress of the steel jacket (to be conservative only $\frac{1}{2}$ of the stress of the steel should be considered. This is why that value is divided by 2),

$d_{sj}$ is the depth of the steel jacket, and

$s_{sj}$ is the spacing (here equal to unity).\(^\text{18}\)

The required thickness of the steel jacket can be back calculated from the $A_{sj}$ term of the previous equation. For columns subjected to shear failure the jacket should be applied to the entire length of the column. Different researchers have studied rectangular columns subjected to shear failure, and some of the results disagree. It would seem that for rectangular columns subjected to shear failure both rectangular or circular (or elliptical) jackets are possible and effective. It should be noted that rectangular jackets are often associated with de-bonding failure between the jacket and the concrete and that anchor bolts should be provided.
3.2.2.3. **Vertical earthquake motion:**

When a structure is subjected to earthquake loads it will principally be subjected to a large horizontal force. But, in an earthquake, a portion of the load also comes from the ground as a vertical load. Most codes, including American Concrete Institute (ACI), assume that this force will be counter-acted by the building gravity load. Surveys after earthquakes, however, have reported a vertical motion at the beginning of the ground motion, and questions have been reported as to whether vertical motion could be responsible for the circumferential cracks reported in reinforced concrete piers and columns. Studies have been made of the retrofit of columns and piers with vertical impulse motions and found that the jackets were effective.\(^{21}\)

3.2.2.4. **Examples**

The state of California and CALTRAN plays, in the United States, a major role in the rehabilitation of bridges. After the 1971 San Fernando Earthquake, they recognized the problem and started a retrofit program and founded researches. But it wasn’t until the 1989 Loma Preita earthquake that the severity of the problem was brought forth and that the necessary founding was allocated to start retrofits. The program identified the bridges and their vulnerability. The first columns that were retrofitted under this program were the Orange and Pomona freeway connectors. This was in 1991 and the retrofitted columns performed well in the 1994 Northridge earthquake\(^ {22}\). The retrofit was principally steel casing after the recommendations from the study made at the University of California San Diego. The retrofit went on to install thousand of steel jackets. (See Figure 3.2-3)

![Figure 3.2-3: Column Retrofit Using Steel Jacket: A. Test Column B. Field Application](image-url)

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3.2.3. **Steel Jacket: Connections**

One proposed rehabilitation technique for connection is to confine the weak connection in a corrugated steel jacket (See Figure 3.2-4). Similarly to column retrofits, the jacket would be constructed in two halves and field welded. The gap would then be filled with grout to provide both continuity and a good composite behavior. The main advantage of using a corrugated steel jacket rather than a normal jacket comes from an earlier confinement effect. When an element is reinforced with a flat steel jacket, the jacket does not provide confinement until the concrete expands (i.e. the concrete core becomes plastic). A corrugated jacket in comparison has a smaller axial rigidity such that the apparent lateral expansion is nonexistent when the jacket is loaded in the axial direction. The lateral confinement effect is then generated at earlier stages of loading, when the concrete is still behaving plastically. For the jacket to be effective, the steel jacket should be extended in the column above and below the joint for a minimum distance equal to the joint length. This method can also be used for confining beams or columns. It should be noted that in the case of beams, a gap needs to be provided between the beam and the jacket to minimize flexural strength enhancement, which would cause additional forces to develop in the joint and in the column (this gap has to also be provided in the beam section of the joint reinforcement).²⁴

![Figure 3.2-4: Joint retrofit using corrugated Steel²⁴](image)
3.2.4. Steel Plates

3.2.4.1. General

During the 1960's in Switzerland and Germany, a method for strengthening concrete structure by applying bonded steel plates was develop. The method is simple: a steel plate is bonded to the concrete surface by a two component epoxy adhesive creating a three phase concrete-glue-steel composite system. There are three important elements which need to be ensured during the placing of the jacket. First the concrete surface needs to be cleaned, then the epoxy should have a strength superior to the concrete (when failures happen it should be located in the concrete), and finally the plate must be long and thin to avoid brittle plate failure (or additional anchorage, such as bolts, can be provided at the end of the beam). Disadvantages of this method are the weight of the steel during construction and placement, the required false-work to support it during bounding, which might be important, and the issues of corrosion and fire protection with steel.

3.2.4.2. Beams

Research and field application have shown that attaching a steel plate to the tension side of a beam will increase its flexural capacity and its flexural stiffness as well as decrease its deflection and cracking (See Figure 3.2-5)

![Figure 3.2-5: Beam retrofit using steel plate](image)

3.2.4.3. Coupling beams:

As was discussed in section 2.2.3, coupling beams can be deficient. Experimental research focused on retrofittine the coupling beams by attaching a thin steel plate to only
one side of the beam to increase its shear capacity. The method was chosen because it is convenient, it requires access to only one side of the beam, and it provides minimum disruption to the building’s occupants and to the architectural finish. The retrofit significantly improves the strength, stiffness, displacement capacity, and energy absorption as long as the plate was both epoxied and bolted to the beam. It should be noted that this method may not be sufficient for ductile coupling beam and that it should only be used for coupling beams in moderate seismic regions. (See Figure 3.2-6)

![Figure 3.2-6: Coupling beam retrofit using a steel plate (A- retrofit; B- beam after testing)](image)

3.2.5. Steel Cable:

Steel cables can be used in two kinds of retrofits. The first use of cables is to prevent structure from moving from their support, the second is as an external reinforcement. Both will be discussed below.

The first method involves placing steel cables across hinges of segmental bridge. The idea is to limit the separation of the adjacent segment across the hinge and to prevent the bridge superstructure from falling off its support. This method was one of the first used by CALTRAN to retrofit bridges after the 1971 San Fernando earthquake. It was used on 1,262 bridges and was completed in 1989 and cost over $54 million.

External pre-stressing was already a method used in the 1950’s. It was abandoned for some times but has been “rediscovered” and with new advances in the pre-stressing it is now widely used in the United States and Japan. External pre-stressing has many advantages: it uses simple construction methods, involves less grouting problems, and
can be inspected and, if necessary, replaced throughout the life of the structure. Its main disadvantage comes from its openness which makes it vulnerable to corrosion, fire, and vandalism. This method has been used to either correct deflections or to strengthen existing structures. It will also provide additional resistance to cracking and fatigue. It is often used on bridges and can be placed inside box girder or on the outside of I-beams. (See Figure 3.2-7).

![External pre-stressing](image.png)

Figure 3.2-7: External pre-stressing

### 3.3. Composite Jackets:

#### 3.3.1. General:

Composites are new materials and research in the subject is on-going, especially for applications in the civil engineering field. Composites are non-isotropic and are made of a mixed between fibers and resin or epoxy. For every application, a specific design and composition has to be calculated. This is a complex process that requires the simultaneous consideration of component geometry, production volume, type of reinforcement, type of matrix, tooling, process, and market economy. The most common composite used in civil engineering applications are jackets or sheets.

When using composite the general expectations are light weight, high stiffness or high strength to weight ratio, as well as corrosion resistance, durability, low thermal expansion (at least in the fiber direction), and low maintenance. They can be used in marine environments and are usually applied without much disruption to the building or its occupants (often the structure does not have to be closed). The largest disadvantage is the
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high initial material cost. High strength high modulus fiber such as carbon can be very expensive (but that can be offset by considering the durability/no-maintenance capacity). Also composite are hard to inspect. Simple eye inspections might not reveal defects underneath the surface and complete inspections (with such methods as X-ray) can prove to be costly. Finally, since composites are just now developing in the civil engineering field, there is a lack of design criteria and code requirements such that structural engineers have to either rely on the design services of the material supplier, or on developing their own based on their research and experience. It should be noted that throughout the world (Europe, USA, Canada, and Japan) there are bodies that have been set up to draw up guidelines and design rules to deal specifically with the design of strengthening concrete structure using composites, however they have not been published yet\textsuperscript{26}.

3.3.2. Columns:

For columns, the same idea than for steel or concrete jackets applies. Composite jacket reinforcement acts in passive reinforcement to increase confinement, which leads to a significant increase in shear strength and ductility. They also inhibit rebar lap-slice failure and reinforce the axial strength of the members.

There exist a great number of composite column retrofit systems that have been researched, tested, and showed to be effective. They can be divided into two major categories:
- in-situ fabricated jackets,
- pre-fabricated jackets.

After installation, the system is often painted to both protect the composite from ultraviolet light and for aesthetic.

3.3.2.1. In-situ fabricated jacket:

The main advantage of in-situ fabricated jackets is that they can match the shape of the column precisely. However, they are usually longer to install in the field then prefabricated jackets and might require special attention for on site quality control and for
the curing of the composite jacket. Final appearance is dependent on the expertise of the field crew. Here are a few methods of in-situ applications:

- Unidirectional carbon fiber sheets wrapped longitudinally and transversely in the potential plastic hinge region. The carbon fiber sheets are attached to the concrete column using epoxy (See Figure 3.3-1).

![Figure 3.3-1: In-situ retrofit using carbon fiber](image)

- E-glass fiber straps applied around the column. E-glass is a more economical composite then carbon fiber. This system can be either pre-peg or applied dry with the epoxy added in the field. The E-glass/Epoxy system can then cure at an ambient temperature (See Figure 3.3-2).

![Figure 3.3-2: Epoxy-fiberglass carbon wrap. A- Laboratory testing. B- Field application](image)

One variation of this method is the use of glass-polyaramide-epoxy composite wrap around bladders, which are thin elastomeric bags. The bladders are wrapped around the columns and stressed by injecting them with cement grout. It should be note that with this method rectangular columns have to be made elliptical.
• Carbon fibers retrofit using automated machine to wrap carbon bundle, which results in a continuous jacket. The fibers used are per-peg tow, i.e., carbon fibers are pre-impregnated with a resin, and wrapped around the column using robo-wrapper. The robot rotates around the column in two different axes (radial and vertical) producing a hoop wrap jacket. This method ensures precise dimension and lay up, eliminating quality problems of hand layout. Latest versions of the robots now apply more strands simultaneously and can wrap a 20 ft (6.1 m) column in two hours. The columns are then cured by a radiant energy curing system, either electrically heated blankets or enclosure oven on site (for approximately 8 to 10 hours) (See Figure 3.3-3).

![Figure 3.3-3: Fiber wrap robot (A- up to 4ft column diameter, B- up to 7ft column diameter, C-up to 3 ft column diameter, and D- Curing system using electrical radiant heat)](image)

• This method was applied to the field for the first time in 1997 by XXsys Technologies, Inc. to retrofit six columns in the San Diego County, CA. The Nevada department of Transportation followed in the footsteps of California and on
November 14, 1996 approved the robotic column wrapping technique. This method has the potential of becoming the most cost effective column wrapping system because it provides a high strength, low weight and reliable wrap, while not being labor intensive or requiring heavy equipment.28

3.3.2.2. Prefabricated methods

Prefabricated jacket have been developed more recently and are also being researched. Prefabricated jacket are made and cured off site but are made for a specific column (i.e. diameter, high, special features). Their installation in the field is usually faster and quality control is easier then for in-situ methods. Special adhesives have to be used to attach the jacket to the column.

- Fiberglass and polyester resin jacket are made of stitch fiberglass sheets and strands and formed into rolls. The rolls are then encased in fabrics which can bend only in one direction. The mat is then wetted in resin and placed into forms. Once cured they are brought to the site and installed around jackets. They are bonded with epoxy. The system is light, can be installed in two hours, and will increase the load capacity by two. It will also protect the columns from corrosion and freeze thaw. This method was used for 3,500 concrete columns in the Yolo Causeway in Sacramento CA. The project cost $850,000 and the jackets were produced by Myers Technologies.31 (See Figure 3.3-4)

Figure 3.3-4: Prefabricated fiberglass shell field installation28
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- "Clock Spring" systems use an isothalic polyester resin. Several layers of shells are applied to provide the required ductility. A clamping system is used to hold the prefabricated shell tight until the adhesive cures (See Figure 3.3-5).

Figure 3.3-5: Clock spring system - full high shell

- Series of prefabricated E-glass fiber reinforced with composite cylindrical shells with slits. The shells are made of unidirectional glass fiber and 2-part epoxy. The fibers are arranged in such a way that 90% of the fibers are oriented in the circumferential direction and 10% in the longitudinal direction. When a column is retrofitted, the shells are opened and clamped around the column in sequences with their slits staggered (See Figure 3.3-6). High strength adhesive (such as urethane based) is applied to bond the shells to each other and to the column to form a continuous jacket. The slit for each layer are not butt-bonded. Research on this particular system was performed at the University of Southern California (L.A.) by Yan Xia and Rui Ma. They concluded that such retrofit system can delay typical lap-splice failures by significantly improving hysteretic responses and by increasing ductility. An analytical model was also created.
3.3.3. Other Structural System

3.3.3.1. Walls and Slabs

Studies have shown that walls could be retrofitted using composite. Walls made of masonry or reinforced concrete blocks are often deficient but research has shown that with the proper fiber configuration, strength and ductility could be achieved. The application can be limited to one side of the wall and still be effective. (See Figure 3.3-7)

Figure 3.3-7: Wall retrofit using composite

Slabs can also be a source of concern during earthquakes. Some research focused on retrofitting them using a combination of epoxy and carbon/epoxy laminates. Wide composite strips are staggered and placed in the two orthogonal directions. This distribution enhances the load distribution between the different laminates, maximizing the energy dissipation at failure, and avoiding sudden strength degradation from boundary failures. Spacings in between the strips (breathing zones) are left to avoid shear lag
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and degradation of the bond-line. This system can be applied for both repair and retrofit. When tested for gravity loads the retrofitted slab showed good results in both load strength and displacement\(^3\) (See Figure 3.3-8).

![Figure 3.3-8: Slab retrofit using composite](image)

### 3.3.3.2. Beams

Beams can be retrofitted to limit their deflection and crack development under service loads. They can also be retrofitted to increase their load capacity. One retrofit technique is to use a carbon fiber reinforced composite (CRFC), which is attached either to the tension side or fully wrap around the beam. Experimental results showed that the wrapping increased the flexural capacity by 30% without increasing the stiffness. When fully loaded to failure the failure mode observed is crushing of the concrete underneath the loading points, which is followed by tensile cracking of the laminate. Both fully or partly wrapped systems exhibited a sudden drop of load upon failure but the fully wrap beam continued to carry higher load than an unwrapped beam would\(^4\) (See Figure 3.3-9).

![Figure 3.3-9: Beam retrofit using carbon fibers](image)
Another retrofit technique uses both carbon fiber reinforced composite (for flexure) and glass reinforced composite (for shear). This method was used for the retrofit of Horsetail Creek Bridge, an 85 years old 3 spans reinforced concrete slab beam column bridge located in Oregon above the Columbia River gorge. The bridge had an inadequate structural design and its reinforcements were corroded. It was retrofitted with a FRP of both glass and carbon and was the first bridge in the United States that used these new materials to correct such severe deficiency\(^3\)\(^5\) (See Figure 3.3-10). The University of Oregon performed research and experiments on similar beams than the one retrofitted. It was found that retrofitting beams with either carbon or glass would help (30% increases in load) but that by combining both it would provide an even higher strength (150% of the original beam capacity).\(^3\)\(^6\)

![Horsetail Bridge](image)

**Figure 3.3-10: Horsetail Bridge\(^3\)\(^6\)**

### 3.3.3.3. Connections

Connection can prove to be one of the weakest links in the system. Studies done after past earthquakes have shown that failure of connections are responsible for the collapse of several major highways and freeways, as well as of several portal frame structures. Research has showed that the use of carbon fiber reinforced composite wrap around the connection (See Figure 3.3-11) increases the shear strength by 13% and allowed 86% more displacements then a similar unwrapped joint.\(^3\)\(^7\)
3.3.4. Other Methods Using Composites

3.3.4.1. Sprayed fiber polymer

The composite methods that have been discussed up to now are all polymers with continuous fibers. Their main disadvantage is that they are highly anisotropic, i.e. the material properties such as strength, modulus, or thermal stability are generally much higher in the fiber direction than in the normal direction to the fibers. This is exhibited by a brittle and low toughness behavior. To address those issues a sprayed fiber polymer is being researched by the University of British Columbia, (Vancouver Canada). The method studied uses a polymeric matrix simultaneously with short carbon fibers sprayed at high speed onto a concrete surface. It is in the nozzle unit that fibers are injected in the polymer stream. The sprayed composite is compacted pneumatically on application and then finished with a rigid roller (See Figure 3.3-12).

The research first tested the new material in tension and found that increasing the length of the fiber increased the elastic modulus and the tensile strength; such that the material can be tailored to desired properties. Of course, but that there is a trade off between strength and ductility. The retrofit scheme was also tested in compression. The
University of British Columbia tried the composite on a 46-year old concrete girder bridge, the Safe Bridge, near the town of Youbou on Vancouver Island. The bridge was retrofitted in 2001 and is now expected to last another 50 years and to be able to absorb three time as much energy during an earthquake. The bridge has been instrumented and is being monitored.\textsuperscript{39}

### 3.3.4.2. Epoxy Pressure Joint Rehabilitation

Epoxy pressure joint rehabilitation is a well known and common technique used to repair reinforced concrete beam-column joints that have been damaged in earthquake. It is effective in restoring strength, stiffness, and energy-dissipation capacity; it’s simple, efficient and inexpensive. The method consists in temporary bracing of the joints in place with steel braces, in repairing spalled concrete with standard epoxy mortar, in drilling a regular grid of 0.5 inches (12 mm) holes into the joint, in cleaning it with compressed air, and in pressure injecting epoxy into the drilled holes (at a pressure of 50 psi, 350 kPa). This method is efficient and has been extensively used in the US (since the 1964 Anchorage earthquake), in Japan, and in New Zealand.\textsuperscript{40}

### 3.3.4.3. Research in active confinement

Some researches are now focusing on active confinement. Self-stressing composites are made with memory materials that have the unique capacity of temporarily freezing elastic deformations. Deformation recovery is triggered by heat or electricity and large pre-stressing stresses are generated. The goal is to maximize member strength and ductility. Those methods are still at the level of primary research and are now being applied to new members.\textsuperscript{41}
4. Global Methods

4.1. Shear Walls

4.1.1. General

Shear walls have been used for over half a century in seismic retrofits and in new designs. They are sometimes made from steel plates or masonry infill but are most often made of concrete (either pre-cast panel or infill walls) with steel reinforcements in a regular rectangular pattern (See Figure 4.1-1). By definition, a shear wall is a vertical element that resists lateral loads in their plane, they receive those lateral forces from the diaphragms and transfer them to the foundation. They are subjected to both shear and bending forces and they resist an overturning moment. During an earthquake, they will help the building resist overall drift and story mechanism. If damaged, it will most likely be in shear and they will exhibit X-pattern cracks. Shear walls are usually an expensive solution but their locations in the structure is flexible and they can often be hidden in the architecture which make them good candidates for historical building retrofit.

![Shear wall construction](image)

Figure 4.1-1: Shear wall construction

Attention needs to be taken in their designs to their distributions throughout the structure, to their connections to surrounding members, and to the additional load they create in the structure. First, the distribution of shear walls is very important as different locations will result in different effect on the structure. For example, in a typical twelve stories concrete frame building, placing the shear walls at the outer corner will results in restraining the lateral drifts of the building but will not provide additional stiffness or
strength. For the same frame, placing the shear walls in the central bays will result in an increase in stiffening and strength\textsuperscript{43} (See Figure 4.1-2). The shear walls connections to the surrounding existing beams and columns are made using dowels. Attention should be given to those existing connecting members to ensure that they are strong enough to transfer the forces to and from the walls. The addition of shear wall will change the load paths through the building and different elements might need to support loads that they were not designed for, and as a result they need to be analyzed, and possibly retrofitted. Finally, shear walls are heavy and create additional dead load in the building. This can affect the structure and its foundation system.

4.1.2. Dual Frame Wall System

One particular system called the dual shear-wall uses both shear wall and coupling beams. This system is made up of two or more walls in parallel, interconnected by short (shear yielding) coupling beams and/or longer (flexural yielding) frame beams. Under an applied lateral load, such as an earthquake, the walls will displace laterally in opposite direction creating vertical and opposite displacements at each end of the coupling beams. This will force the beams into yielding long before the walls themselves start to dissipate energy. The system is designed such that the coupling beams throughout the building will yield at about the same time. After all of the beams have softened, the cantilever-wall will pick up the additional loads until yielding develops at their bases. The advantage of this system is that the plastic actions are spread throughout the building and that the beams and walls will take the damage preventing it from spreading to the other features of the building\textsuperscript{44}.
4.1.3. Examples

Examples of a seismic upgrade using shear walls are building No 6 and 7 of VA Medical Center in Bedford Massachusetts. Both buildings have similar structural system and were built between 1936 and 1939. The buildings are two stories high and are one-way concrete pan joists floor spanning between reinforced concrete beams and supported by reinforced concrete columns. The retrofit option selected was exterior shear wall. This option proved to be the simplest and least disruptive to the building. The walls could be placed at the exterior of the building just inside of the existing brick work. (See Figure 4.1-3)

Figure 4.1-3: Building 6 (A. elevation; B. interior view, exterior wall structure)

4.2. Steel braces

4.2.1. General

Steel braces are added to a non-ductile reinforced concrete structure to improve the structure’s strength, stiffness, and resistance to story drifts. Steel braces have been used for over 50 years in, for examples Mexico and Japan, and it has been showed that it is an effective system since the total strength of the concrete frame/steel brace system is more than the sum of the concrete system plus the steel system.

The advantages of using steel braces includes the possibility of accommodating openings, the small added weight (especially when compared to concrete shear walls), and the flexibility in its distribution, which allows for designing of the steel placement throughout the building to avoid such problems as shear concentrations. One additional
Advantage of the external systems is the possibility of performing most of the construction work from the outside which will limits disruptions to the building and to its occupants. The main disadvantages of steel brace systems are its initial cost and its continued need for maintenance especially for member exposed to the weather.

Several different bracing systems exist: concentric (X-shaped), eccentric (K-shaped), or even post-tensioned. They are made with double angles, L-sections, or tubular sections. Experimental studies have shown that the concentric systems add the highest strength.

Several elements need to be considered during design. First, the bucking and the yielding of the bracing members will need to be checked. Then, special attention should be taken for the connections, which will need to ensure safe transfer of the loads between the concrete frame and the steel bracing. Also, attentions must be given to how the loads of the existing structures are changing and how the existing members are going to react to those changes. For example, and in the case of controlled lateral drift, the axial force in the steel brace will create adverse lateral forces in the existing concrete members. Under such conditions, a column can now be subjected to a load which exceeds its axial capacity (in either tension or compression). The column then needs to be retrofitted.

A study by H. Abou-Elfath and A. Ghobarah focused on concentric frames and their behavior. The research studied such parameter as the amount of bracing and the steel distribution over the height of the frame. A typical bracing distribution is presented in figure 4.2-1 and consists of steel braces along distributed bays for the entire height of the structure.
Other configurations studied involved braces going up two bays two levels, all bays first level, and all level one bay. It was found that even though the typical configuration provides the structure with additional strength and stiffness it might not be the optimal option for bracing distribution. It is possible to improve the plastic mechanism of the building by providing a bracing configuration that forces the stories to contribute to the building overall deformation. It should be noted that distribution that have abrupt stiffness changes (such as all bracing in one level) should be avoided. The second half of the study focused on the effect of the amount of bracing on the response. It was found that increasing the number of steel bracing increases the performance of the building in spite of the fact it also increases the seismic demands due to a greater stiffness. An additional benefit of increasing the amount of bracing is the reduction of the drift, and therefore of the damages.  

### 4.2.2. Post-tensioned Systems

In a post-tension system the brace members are rods that have been initially pre-stressed. The main advantage of pre-stressing is that it reduces the likelihood that the brace will shorten to the point of becoming slack. Secondly, if the brace stays in tension, then the diminutions of the lateral stiffness are minimized. Finally, if the brace is pre-stressed to a high level, then the brace will yield and allow for major energy dissipation at a relatively small lateral drift. Research has shown that the level of pre-stress should be kept at 50% of the yield strength or higher to provide the most effective response.

One concern with pre-stressed bracing is the effect of the pre-tension on the surrounding members. During pre-stressing, forces are applied on the adjacent columns creating high shear demands. When the earthquake comes, it will increase the shear demands and since those columns are already under high demand they can fail prematurely.

### 4.2.3. Example of Application

One early example of the use of steel brace was one of the main buildings of the Tohoku Institute of Technology in Sendai, Japan. The building was damage in 1978 by the Miyagi-ken-oki earthquake. The building was repaired and strengthened with steel cross
braces which were placed eccentrically on both faces and from the outside. The braces were H section (they used a total of 50 tons) fastened by friction bolts. The retrofit took four months for both the steel fabrication and installation. An interesting aspect of this project was the research done by the university prior to the retrofit to verify that the system, new at the time, would be effective.48 (See Figure 4.2-2)

![Retrofit scheme A. Elevation B. Detailing](image)

**4.3. Base isolation**

**4.3.1. General**

Isolation is an old idea and is based on preventing interaction between two systems. It has been used for over 70 years to isolate vibration machineries from their surroundings. Its application to the civil engineering field is relatively new and substantial developments have been made since the mid-1980s.49 When applied to seismic, the idea becomes to prevent the ground motion from even entering the structure by using an isolating system which is achieved thru the use of bearings (See Figure 4.3-1). Those are generally located at the basement level between the foundation and the superstructure. They are placed underneath each column in frame structures or at a maximum allowable spacing in continuous structures. The system has the following three goals to accomplish:

- The fundamental frequency of the structure has to be changed and moved away from the dominant frequencies of seismic excitations typical of the area. This implies
increasing the flexibility at the base of the structure (in the horizontal direction) which will avoid resonance and reduce floors acceleration.

- Energy has to be dissipated to reduce the damaging effect on the structure.
- Minor lateral loads (i.e. wind loads) have to be accounted for and rigidity or energy dissipation under those loads have to be provided for.

![Figure 4.3-1: Deformation fixed vs. isolated in Earthquake](image)

It is generally accepted that base isolated structures will performed better than conventional structures in moderate to strong earthquakes. It is now being used and becoming increasingly popular in the United States and Japan.

Base isolation is limited by the following constraints. First, because the structure will experience large displacements, accommodations have to be provided to assure that this is possible without damage (i.e. by digging a moat around the structure) and these accommodations have to be ensured throughout the life of the structure. Then, vertical forces have to be considered. During an earthquake, a structure will experience both horizontal and vertical forces. The vertical forces, thought smaller, do have to be taken into account, and most isolators are not designed for them. This problem can be addressed for relatively small vertical forces through isolation distribution and proper connection detailing. For large vertical forces, as can happen for structures with high overturning moment, this problem might not be solve and isolation might not be possible. Finally, if a structure is already very flexible, increasing its flexibility might not help. Those constraints limit the type of building that base isolation is suitable for.
that are good candidates are low to mid rise (up to 10-12 floors) with a clearing of 4-8 inches (10-20 cm) around them and with limited service lateral loads. The type of soil supporting the structure also has to be considered, the stiffer the soil the better. Finally, from the last two constraints it seems obvious that high rise buildings are not good candidate for this type of isolation.\textsuperscript{50}

The main disadvantage of base isolation is that it is a relatively new technique. Even though it had been used extensively in Japan and has been shown to perform well in field conditions, it remains that in the United States only a few systems have experienced earthquakes in the field. Many laboratory tests have been performed, as for example by CALTRAN in 1996\textsuperscript{51} where a large number of different isolation systems were tried, but there are still some discussions as to their exact field behavior. Questions on the effects of ground motion acceleration on the upper story, of acceleration spikes (due to closeness of the epicenter) on the system, and of general field conditions still have to be answered.

Base isolation can and has been used for new constructions and retrofits. Examples of such retrofit will be mention in section 4.3.3.1 but the advantages of base isolation for a retrofit situation should be discussed. Base isolation has a low impact on the existing structure. A minimal addition of stiffness or strength might need to be added to the superstructure, but otherwise most of the work is located at the isolation level (usually the basement level), while regular activities can continue above (see section 4.3.3.1 Long Beach V.A. Hospital). This advantage is especially important when retrofitting historic structures since it will not disrupt the historical elements (ornamentations and features that might have been replaced with other retrofit schemes). Base Isolation is also good for highly-sensitive buildings such as hospital or microchip factories that cannot support any movements (see section 4.3.3.1 Conexant Semiconductor System Inc.). It can also help a structure with other problems such as torsion by designing the bearing location such that the center of stiffness corresponds to the center of mass. Finally, and for bridges, isolation can be easily implemented by converting the thermal expansion bearing into isolators.
The cost of isolation varies with the structure. For a new structure it is about 3% of the construction cost. For retrofitting of an existing structure, the cost is less. Its environmental cost is also low, creating less waste during construction.

4.3.2. Seismic isolation device

There exist a large number of isolation systems, 50\textsuperscript{52} or more patented isolators exist worldwide. The largest family of bearing is elastomeric. Sliding bearings are also becoming popular.

4.3.2.1. Elastomeric System

4.3.2.1.1. General

Elastomeric isolation systems are the most popular. They are made of layers of thin rubber sheets bonded to steel plates. The steel plates help carry the vertical forces by providing both load capacity and stiffness in the vertical direction. The rubber sheets provide the horizontal flexibility by shearing deformability. (See Figure 4.3-2 & 4.3-3)

![Elastomeric bearing schematic](image)

**Figure 4.3-2: Elastomeric bearing schematic\textsuperscript{50}**

![Elastomeric bearing (laboratory testing set up)](image)

**Figure 4.3-3: Elastomeric bearing (laboratory testing set up)\textsuperscript{53}**
The lateral stiffness of the bearing can be increased by increasing the number of rubber layers (not the thickness of each individual layers). However, to avoid a buckling failure, the height should be limited to half of the diameter. Natural rubber is a nonlinear viscoelastic material which can deform 300% without damages (See Figure 4.3-4).

![Elastomeric bearing under deformation](image)

Rubber also has a rehabilitation force that returns the system to its original position. They are usually low maintenance but have high initial cost. They are not the most efficient type of isolators since they do not have an energy dissipation device or a mechanism to help with minor lateral service load. The bearing can be used in parallel with other systems (such as a damper) or be build with those devices incorporated such as lead plug bearing or high damping rubber bearings.

### 4.3.2.1.2. Lead Rubber Bearing (LRB)

LRB are made of thin layers of low-damping rubber bonded to steel plates with a lead cylinder firmly fitted in a hole at the center. The steel plate and the rubber function as described above, and the lead plug gives the structure both the rigidity it needs against minor lateral loads and an energy dissipation system. (See Figure 4.3-5)
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The initial stiffness comes from the lead plug and its high initial stiffness. The energy dissipation comes from the plug yielding which occurs at a low stress level 1,450 psi (10.5 MPa) (See Figure 4.3-6).

It should be noted that before the lead yield there will be no energy dissipation provided in the building. For sensitive structure (microchip factories or hospital) this can be an issue since the building will experience deformation under low lateral load such has minor wind load or micro-tremor. To solve the problem, a parallel damping system can be used. LRB are supplied in the United States by Dynamic Isolation Systems, Inc and are used on a great number of projects.

4.3.2.1.3. High-damping rubber bearing (HDRB)

In this system the low damping material used in typical elastomeric bearing is replaced with a high damping rubber. High damping rubber is a rubber compound (mechanical properties not affected) which has additional fillers such as carbon and resin embedded in
it. When shear is applied, the sliding molecules in the bearing interact with each other creating frictional heat and dissipating energy. This behavior is characterized by a smooth elliptical hysteresis loop and is available for both small and large strains. The main advantage of HDRB is that the energy dissipation starts as soon as the rubber starts deforming so for very low strains. However, the system does not provide any initial stiffness and small loads may result in larger displacements. Also, both the damping and mechanical properties can be dependent on temperature.

Finally, the two systems, LRB and HDRB, can be combined and the resulting system will have the advantages of both systems: stiffness and energy dissipation at low strain (it also prevent having to combine the system with any other parallel systems). There are two different ways of combining LRB and HDRB. The first is to combine them at the system level creating a lead high damping rubber bearing (LHDRB), which are layers of high damping rubber between thin plates of steel and with a lead plug fitted in the center. The second way is to combine the systems at the structure level by isolating it with both LRB and HDRB.

4.3.2.2. Sliding isolation system

Sliding bearings have been available and applied since 1993, and were included in AASHTO’s specifications in 1997\(^5\). The theory behind them is based on friction and they are made of stainless steel and Teflon. At low lateral forces, the bearing will provide stiffness to the structure. When the lateral forces increases and overtakes the friction forces (at a designed level) the system starts to move. The main advantage of the system is that the magnitude of the force transmitted to the building is based only on the coefficient of friction and is independent of the seismic event, making the system effective even in major event (See Figure 4.3-7).
Sliding isolation systems are relatively cheap, they are compact, and they can be used when high pull-out forces are needed. However, the systems have several disadvantages. First, they can be affected by fatigue, deterioration, and thermal changes. Secondly and most importantly, they do not have a mechanism that forces them back to their original position. If the bearing is in a first earthquake and finishes in a non-initial position then when the next seismic event hits, the bearing will not provide as much accommodation. Several options exist to remedy this problem. One is to combine the system with elastomeric bearings which as we saw before have restoring forces and will provide them to both systems. Another option is to use a friction pendulum system (FPS). FPS are made of two parts, a slider and a spherical stainless steel surface. The surface is upside down and its geometry generates the self-centering action, while its radius of curvature gives a specific stiffness and frequency to the system (See Figure 4.3-8). Energy dissipation is provided thought the friction on the sliding element.

Figure 4.3-7: Sliding isolation system

Figure 4.3-8: Friction pendulum system
4.3.3. Examples

4.3.3.1. Buildings

There are many examples of building that have been base isolated both in the United States and around the world. Here are a few key ones:

- **Foothills community law and justice center** is the first American structure to rest on base isolator (1984 - new construction - steel frame)
- **City and County Building in Salt Lake city** is the first retrofitted building in America (1987 - retrofit – un-reinforced brick masonry wall)
- **Rockwell International Corporate Headquarters**: The building is an 8 stories (+ basement) rectangular non ductile concrete frame (260,000 ft²) located in Seal Beach CA. The building was retrofitted with 42 LRB and 69 rubber isolators for a total cost of $48 millions. During the analysis a 3-D model was created which included the non-linear properties of the isolators. The project won a National Merit Award for Outstanding Civil Engineering Achievement from the American Society of Civil Engineering 53 (See Figure 4.3-9).

![Figure 4.3-9: Rockwell International Corporate Headquarters](image)

- **Long Beach V.A. Hospital**: The building, which is 12 stories and 350,000ft², is the core of the hospital. Its structural system is non-ductile concrete shear walls and after several studies the building was declared to be vulnerable to earthquakes. It was retrofitted in 1995 with 110 LRB, 18 rubber isolators, and 36 FPS for a total cost of $18 millions. The choice to isolate the building was made based on the need for such
a structure to experience no down time either during construction or in the even of an earthquake\(^5\) (See Figure 4.3-10).

![Figure 4.3-10: Long Beach V.A. Hospital\(^5\)](image)

- **Conexant Semiconductor System Inc.** Located in California’s Silicon Valley, the high-tech manufacturing center, a 1968 300ft by 400ft concrete frame building located on clay soil, was in compliance with the UBC (United Building Code) for safety but was still at high risk for damage to the equipment in this highly seismic area. In 1997, the retrofit began and consisted in strengthening the structure with external steel columns and to shield it form seismic acceleration by excavating underneath the building to install isolators below the foundation. The project’s most interesting feature is that all of the construction was performed with production continuing above 24 hours a day, 7 days a week.\(^5\) (See Figure 4.3-11)

![Figure 4.3-11: Conexant semiconductor system, Inc (A. factory, B. bearing in place)\(^5\)](image)

- **Takenaka Corporation headquarters in Umeda in Osaka (Japan)\(^5\).** The limits of isolation are here being tested. The “DT project” is a high rise building (27 stories –
425 ft at its highest point, 130 m). Its isolation used twelve linear sliding bearings, six LRB, and two dampers and was located above the basement. (See Figure 4.3-12)

**Figure 4.3-12: "DT project"**

### 4.3.3.2. Bridges

Recent earthquakes, such as the 1995 Kobe earthquake in Japan, have demonstrated the vulnerability of typical bridge bearings and the explanations to such numerous failures are still unknown. Bearings provide the bridges with seasonal thermal displacements and their failures have raised questions about their replacement and if they should be retrofitted with isolation bearings. Since the bearings have to be replaced anyway, to provide them with isolation bearings could reduce earthquake loads enough to avoid having to retrofit the columns and the foundations. They are two major concerns though with using isolator bearings. First, the isolators will lengthen the period of vibration of the structure, which will increase displacements, which have to be accounted for in the retrofit, with longer expansion joints and changes to the abutments. Secondly, if the bridge is supported on soft soils then isolator bearing might only make matters worse if the structure is already very flexible increasing its flexibility might only bring its natural frequency closer to the one of the earthquake rather than away from it.)
5. Conclusion

Concrete structures built before the 1970’s need to be retrofitted to withstand earthquakes. Six different methods of retrofit divided into two major categories, local and global, were presented. Local methods include addition of concrete, steel, and composite to a specific member to improve its response in a seismic event. All three methods are effective each also has some disadvantages: concrete is labor intensive, steel requires high maintenance during the life of the structure, and composites have high initial cost. Global methods retrofit the entire structure at once by adding shear walls or steel braces, or by using base isolation. Shear wall are labor intensive and expensive. Steel brace can be easier to implement but present some connection problems. Base isolation is effective and works well, but cannot be applied to all type of structures.

The choice of the method depends on the building, on its specific requirements, as well as its condition, location, and geometry. Several methods should usually be considered and compared to find the appropriate best one. To provide greater flexibility in the retrofit scheme, several methods can be combined and implemented together, combining the advantages of each.
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