The Retrofitting of Existing Buildings For Seismic Criteria

by

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B.S., Civil and Environmental Engineering
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Submitted to the Department of Civil and Environmental Engineering
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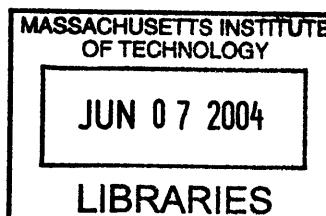
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ABSTRACT

This thesis describes the process for retrofitting a building for seismic criteria. It explains the need for a new, performance-based design code to provide a range of acceptable building behavior. It then outlines the procedure for retrofitting a building. This procedure begins with acquiring information about the existing building and its surroundings. The building owner or client then needs to work with the design professional to establish an acceptable performance level, or rehabilitation objective. A rehabilitation method must then be selected that determines how the building should be analyzed. The analysis of the building, including suggested rehabilitation strategies, must then be performed. Once the analysis indicates that the building will perform to its prescribed performance level, the rehabilitation strategies must then be implemented.

The thesis ends with a description of two buildings that have recently been retrofitted, or are in the processes of being retrofitted. It gives an overview of the selected rehabilitation strategies and the reasoning behind their selection.

Thesis Supervisor: Jerome J Connor

Title: Professor of Civil and Environmental Engineering

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I would like to dedicate this thesis to the wonderful and unique people I have met during my brief time at MIT, in the Civil Engineering Department and beyond. Special people who come to mind are the Dashpots, the Fly Girls, and my friends from Tang. This has been an experience that I will never forget.

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1. Introduction

Damage incurred during earthquakes is a concern for society as a whole in areas of high seismicity, not only in terms of loss of life, but also financially. The Golcuz-Izmit earthquake in Turkey in 1999 resulted in the loss of over 50,000 lives due to building collapses. These statistics, along with the results of other earthquakes such as the one that occurred in Taiwan in 1999, have revealed that buildings 'designed and constructed using codes that are now known to provide inadequate safety' are potential hazards[3]. In many urban areas, the number of buildings constructed prior to 1980 greatly outnumbers those that are built according to newer, more stringent, codes. The impact of this statistic on society is great; something must be done to make the older buildings safe or else human lives are at risk.

The financial impact of bringing older buildings up to current codes can be devastating as well. If this action, referred to as retrofitting, is not taken, however, the financial damage resulting from an earthquake can be even more devastating. For instance, the cost of the damage to buildings caused by the Northridge earthquake was $15 billion and $7 billion for Loma Prieta[9]. Obviously, there is no easy solution for this problem because there are so many factors that need to be considered. Additionally, there are many grey areas associated with retrofitting, such as who is responsible for the costs and what level of performance is acceptable for buildings. This second question, dealing with building performance during seismic events, is currently being addressed by several agencies involved with building code determination and public safety.

The Federal Emergency Management Agency (FEMA) has developed guidelines for retrofitting buildings to bring them up to the appropriate level of performance for seismic events: NEHRP Guidelines for the Seismic Rehabilitation of Buildings. This paper explores the process outlined in the Guidelines for the assessment of building deficiencies, the selection of the appropriate rehabilitation strategies, and the implementation of these strategies through two case studies of recent retrofitting projects.

Figure 1.1 Damage Due to Northridge Earthquake [14]

2. Argument for a Performance-Based Design Code

In the past, building codes have been based largely on "empirical and experienced-based conventions"[17]. Design analysis was performed for a single design event level using an equivalent base-shear method. The basis for analysis was the linear behavior of materials, which is inherently incorrect at predicting the behavior of the structural elements at their limit states, such as those experienced by the members during intense seismic events[8]. The only performance level introduced in the code was termed "life-safety", implying that the only requirement was that the building didn't collapse; the code did not address the issue of allowable deformations. The result is that although the loss of life during recent earthquakes, specifically within the US, was minimal, the financial costs incurred were unacceptable, as can be seen by the cost of the damage ($15 billion) due to the Northridge earthquake. The result in many cases is building owners who are disillusioned with their engineers because of unexpectedly low levels of building performance. When questioned, engineers find it difficult to justify the design procedure specified in the code. These financial losses account for a significant portion of the motivation within the design community to adopt a performance-based building code[8].

Performance-based engineering is a process in which the owner establishes a desired level of performance during specified service and seismic loads. This method is based on the idea that the structural behavior of a building can be realistically predicted given the spectrum of loading conditions it is likely to experience. Performance-based design differs from conventional design in

several ways. At the onset of the project, the owner evaluates the total life-cycle costs of the facility to justify greater design and construction costs up front. This approach is well-aligned with the current trend in the construction industry towards sustainability. Additionally, the process is more scientifically based than the empirical conventions of the past. It emphasizes the accurate characterization and prediction of behavior, which requires the use of a higher level of technology than what was used in the past[12].

These fundamental changes in the design process are beginning to be incorporated in the new versions of the traditional building codes. Several committees have been formed to provide standard procedures for performance-based design. *Vision 2000* (SEAOC 1995) is a project of the Structural Engineers Association of California with that purpose in mind. The *Guidelines and Commentary for Seismic Rehabilitation of Buildings* (ATC 1995) were developed by the Federal Emergency Management Agency (FEMA) to provide performance-based recommendations for retrofitting existing buildings. Both of these projects developed similar standardized performance level definitions, which are shown in Figure 2.1. It was important that these definitions be comprehensible to the layperson so that they could understand the level of performance they were requesting. When making a decision as to the level of performance of a building, some important parameters to consider include: the potential loss of life, the cost of repairing the building damage, and the amount of time the building cannot be occupied due to damage.

TABLE 1
DEFINITIONS OF STRUCTURAL PERFORMANCE

Performance Level		Description
NEHRP Guidelines	Vision 2000	
Operational	Fully Functional	No significant damage has occurred to structural and non-structural components. Building is suitable for normal intended occupancy and use.
Immediate Occupancy	Operational	No significant damage has occurred to structure, which retains nearly all of its pre-earthquake strength and stiffness. Nonstructural components are secure and most would function, if utilities available. Building may be used for intended purpose, albeit in an impaired mode.
Life Safety	Life Safe	Significant damage to structural elements, with substantial reduction in stiffness, however, margin remains against collapse. Nonstructural elements are secured but may not function. Occupancy may be prevented until repairs can be instituted.
Collapse Prevention	Near Collapse	Substantial structural and nonstructural damage. Structural strength and stiffness substantially degraded. Little margin against collapse. Some falling debris hazards may have occurred.

Figure 2.1 Structural Performance Levels Identified in New Codes[8]

Even with these standard definitions, the question still remains of how to translate these qualitative guides into quantitative information that is practical for design engineers. For example, a building owner may specify that the facility should be available for continuous occupancy after an earthquake. The resulting question is how to convert this request into a limit state for building analysis and design. As a transition between these qualitative definitions and the actual performance requirements for the facility design, a series of matrices were also incorporated in both guidelines that more adequately defined the performance level of specific building elements based on the previously mentioned parameters.

TABLE 2

ACCEPTANCE CRITERIA FOR LIFE SAFETY AND COLLAPSE PREVENTION PERFORMANCE LEVELS[1]

Performance Level	Primary Component	Secondary Component
Life Safety	75% of the deformation at which significant loss of lateral force resisting strength occurs	100% of the deformation at which significant loss of lateral force resisting strength occurs
Collapse Prevention	75% of the deformation at which loss of vertical load carrying capacity occurs, but not more than the deformation at which significant loss of lateral force resisting strength occurs	100% of the deformation at which loss of vertical load carrying capacity occurs

1.   The acceptance criteria indicated apply to buildings for which nonlinear analytical methods are used to predict component demands. An additional reduction factor, of 0.75, is applied against these acceptance criteria when linear methods of analysis are used to predict component demands.

Figure 2.3 Acceptance Criteria [8]

To further complicate the design process, owners can request different performance levels for different grades of seismic events. In the past codes, the buildings were designed only for the life-safety level during a single, "worst-case" earthquake. *Vision 2000* included an additional matrix to aid designers in determining how to design for the various levels of design earthquakes. An important criterion in this matrix is the classification of the facility based on its function. Three categories are defined: standard occupancy, emergency response, and safety critical facilities. See Figure 2.4 for the building performance requirements for the each category during the corresponding seismic events.
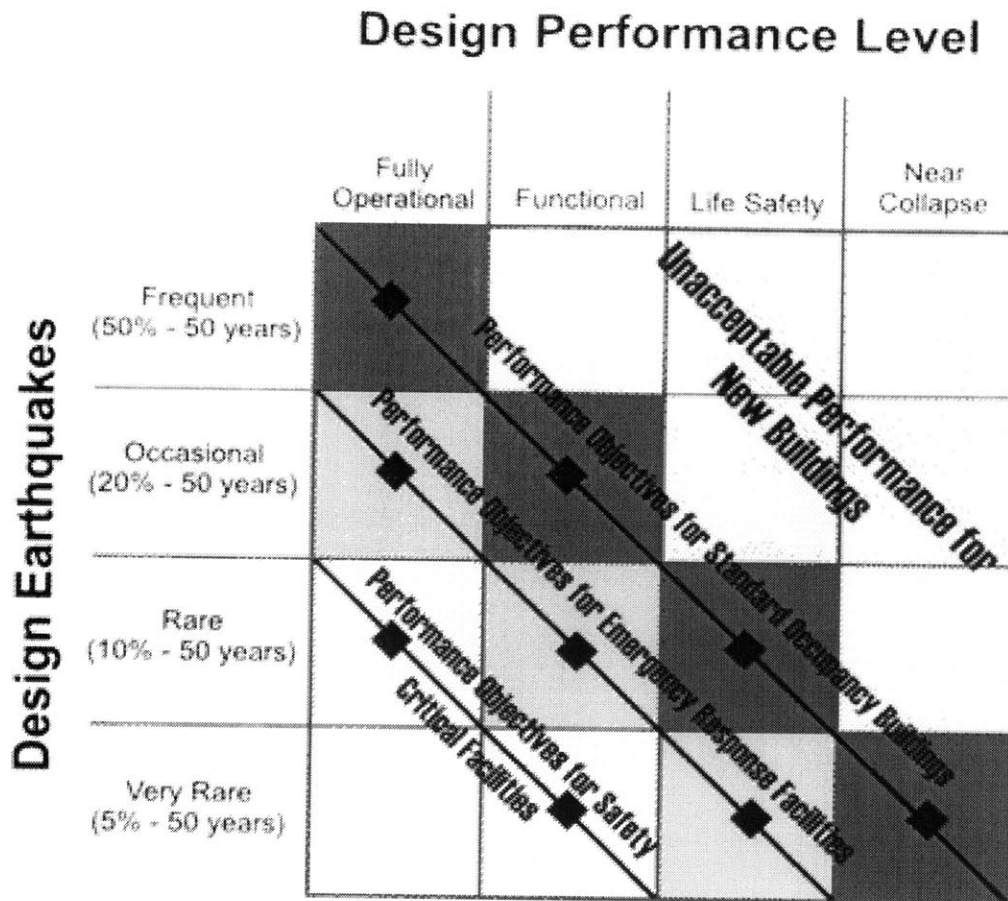
Design Performance Level

Figure 2.4 Design Performance Levels [10]

The new version of the International Building Code, IBC 2000, has begun to integrate some of the key elements in performance-based design. For instance, the code now requires that facilities be categorized using levels similar to those established in the *Vision 2000*. These categories have a corresponding additional factor of safety used in the seismic design procedure. Although the result is not a specific level of performance, it does imply that critical facilities must perform at a higher level during earthquakes than standard occupancy facilities.

This paper will focus on performance-based design as applied to the retrofitting of existing structures as outlined in the *Guidelines and Commentary for Seismic Rehabilitation of Buildings*.

3. Overview of Procedure

The *Guidelines and Commentary for Seismic Rehabilitation of Buildings* suggest a format for the basic approach to use when retrofitting a building for seismic concerns. This general procedure begins with obtaining information about the existing facility. The second step is determining a Rehabilitation Objective for the building and then selecting a Rehabilitation Method, either the Simplified Method or the Systematic Method. Using the selected method, an analysis of the building is then performed, including the suggested retrofit modifications, to make sure the revised building will perform adequately to meet current code requirements. If the building performance is satisfactory, the design should then be implemented (2-1)[6].

4. Acquiring As-Built Information

When performing a seismic retrofit on a building, it is crucial to have an extensive understanding of the structural system of the building in order to effectively predict its behavior during an earthquake. There are many ways in which information regarding the existing structure can be obtained. The first, most obvious way, is to acquire the original construction documents for the facility as well as any documentation on previous modifications. However, this can be difficult for older buildings. These documents should include explicit details of as-built conditions, number and placement of hidden structural items such as reinforcing bars and bolts, as well as a set of the specifications. Performing at least one site visit is required as well. During this visit, the information on the construction documents (CD's) should be verified. Pictures or sketches of existing conditions, especially those that differ from the design indicated on the CD's, should be made. It is also important to understand the design codes and reference standards upon which the design documents were based in order to understand the theory behind the intended structural behavior. Examination and testing, both destructive and non-destructive, should be performed to verify the material properties of the building components to ensure accurate modeling. Additionally, interviews with the building owner or tenants, the architects and engineers of record, and the original contractor, could provide important supplemental information[6].

Once the necessary information is obtained, the building configuration should be reviewed. This should incorporate the structural components, meaning

the gravity and lateral-load resisting systems, as well as the nonstructural elements. It is necessary to understand the nonstructural elements because they can contribute to the overall stability of the facility, even though they might not have been incorporated in the initial strength design. Including the nonstructural elements in the analytical model of the building is one of the ways in which the *Guidelines and Commentary for Seismic Rehabilitation of Buildings* differs from current building codes for new construction. The intended load paths should be identified and particular notice taken of instances of irregularity in the structural system because these irregularities are often times the cause of substantial building damage. These are all necessary considerations for the building behavior on a global scale, but it is also necessary to consider the individual element behavior on a local scale[6].

In terms of local structural behavior, it is important to verify the material properties of the individual structural elements. "Component deformation capacity must be calculated to allow validation of overall element and building deformations and their acceptability for the selected Rehabilitation Objectives" (2-25)[6]. This becomes especially crucial when non-linear analysis techniques are implemented. The material properties can be verified using destructive or nondestructive evaluation techniques.

Despite the acquisition of such substantial information regarding the condition of the existing facility, it is acknowledged that it is impossible to completely understand the behavior of the building and its individual components. In response to this fact, the *Guidelines* established a coefficient, κ,

called the knowledge factor. There are two possible values for κ. The first value of .75 is intended for facilities where only a minimum level of knowledge about existing conditions is available. When there is a substantial amount of information available, the value of κ is 1.0 [6].

In addition to the condition of the building itself, it is also important to understand its surrounding environment, including the soil characteristics and its possible interaction with other existing buildings. The characteristics of the soil on which the building is located can be a very large factor in seismic rehabilitation. This is because the soil has the capability of magnifying ground motion to very extreme levels. Hence, if adequate geotechnical data is not available, subsurface investigations on the site should be performed. If the facility is adjacent to other buildings, then the interactions between the structures should be understood to increase the accuracy of the analytical model[6].

5. Specifying a Rehabilitation Objective

A Rehabilitation Objective is the combination of a desired building performance level and a specified seismic demand. It is up to the owner to determine the Rehabilitation Objective with the cooperation and the advice of the design professional. This selection process is the defining characteristic of performance-based design because it allows the owner to determine the desired level of performance rather than relying on the life-safety performance category pre-established in most building codes. The *Guidelines* defines the Building Performance Level as the extent of damage to both the structural and nonstructural components of the building. As such, the overall Building Performance Level is a combination of a Structural Performance Level and a Nonstructural Performance Level. There are three discrete Structural Performance Levels: Immediate Occupancy (S-1), Life Safety (S-3), and Collapse Prevention (S-5). In addition, there are two Structural Performance Ranges, Damage Control (S-2) and Limited Safety (S-4), whose requirements can be determined through interpolation between the Structural Performance Levels. These are included to provide the owner with a wide range of possible performance levels. The Nonstructural Performance Levels closely resemble the Structural Performance Levels and are defined as Operational Performance Level (N-A), Immediate Occupancy Performance Level (N-B), Life Safety Performance Level (N-C), Hazards Reduced Performance Range (N-D), and a fifth level (N-E) which exists when the nonstructural elements are not addressed in the retrofit process[6].

There are four combinations of Structural and Nonstructural Performance Levels that are commonly used as Building Performance Levels. These are Operational Performance Level (1-A), Immediate Occupancy Performance Level (1-B), Life Safety Performance Level (3-C), and Collapse Prevention Performance Level (5-E). Qualitative descriptions of various performance levels are provided in the *Guidelines* to guide the owner's decision regarding the desired performance. These include details about what type of damage to expect, such as the extent of cracking in the facades and the amount of permanent drift sustained by the structure during the earthquake. See Figure 5.1 for an example of a typical description of a performance level. To create a Rehabilitation Objective, a Building Performance Level should be combined with an Earthquake Hazard Level[6].

Table 2-3   Damage Control and Building Performance Levels

	Building Performance Levels			
	Collapse Prevention Level	Life Safety Level	Immediate Occupancy Level	Operational Level
Overall Damage	Severe	Moderate	Light	Very Light
General	Little residual stiffness and strength, but load-bearing columns and walls function. Large permanent drifts. Some exits blocked. Infills and unbraced parapets failed or at incipient failure. Building is near collapse.	Some residual strength and stiffness left in all stories. Gravity-load-bearing elements function. No out-of-plane failure of walls or tipping of parapets. Some permanent drift. Damage to partitions. Building may be beyond economical repair.	No permanent drift. Structure substantially retains original strength and stiffness. Minor cracking of facades, partitions, and ceilings as well as structural elements. Elevators can be restarted. Fire protection operable.	No permanent drift; structure substantially retains original strength and stiffness. Minor cracking of facades, partitions, and ceilings as well as structural elements. All systems important to normal operation are functional.
Nonstructural components	Extensive damage.	Falling hazards mitigated but many architectural, mechanical, and electrical systems are damaged.	Equipment and contents are generally secure, but may not operate due to mechanical failure or lack of utilities.	Negligible damage occurs. Power and other utilities are available, possibly from standby sources.
Comparison with performance intended for buildings designed, under the *NEHRP Provisions*, for the Design Earthquake	Significantly more damage and greater risk.	Somewhat more damage and slightly higher risk.	Much less damage and lower risk.	Much less damage and lower risk.

Figure 5.1 Building Performance Levels [6]

Earthquake Hazard Levels can be established in one of two ways, using a probabilistic method or a deterministic method. In most areas of the country, it is more practical to use a probabilistic method using data that is obtained from recent United States Geological Survey (USGS) national earthquake hazard maps. In 1996 the USGS developed probabilistic maps for ground motions due to seismic events corresponding to three chances of exceedance within a set number of years: a 10% chance in 50 years, a 5% chance in 50 years, 2% chance in 50 years. The *Guidelines* added an event corresponding to a 20% chance of exceedance in 50 years. These four scenarios constitute the established Earthquake Hazard Levels and correspond to mean return periods of approximately 75, 225, 500, and 2500 years. The earthquake level with the 2500 year return period is sometimes referred to as the Maximum Considered Earthquake (MCE); two-thirds of the MCE is prescribed in most building codes as the required value for seismic design of new buildings (1-5)[6].

For facilities located in close proximity to known faults, a deterministic method for obtaining an Earthquake Hazard Level is more appropriate. The deterministic method implies using specific response spectra acquired through recording ground motions of past seismic events. The result is a more realistic prediction of ground motion for the specified site. This method must be used if the Nonlinear Dynamic Procedure is implemented as the analysis method. See Section 6 for more on analysis methods[6].

There can be more than one Rehabilitation Objective for each project. For example, a facility could be required to perform at a collapse prevention level for the 2500 year return period and at an immediate occupancy level for an earthquake with a return period of 225 years. Since the Rehabilitation Objective establishes the design requirements, an analytical evaluation of the retrofitted building should be performed for each objective. See Figure 5.2 for a table of possible Rehabilitation Objectives[6].

Table 2-2   **Rehabilitation Objectives**

		Building Performance Levels			
		Operational Performance Level (1-A)	Immediate Occupancy Performance Level (1-B)	Life Safety Performance Level (3-C)	Collapse Prevention Performance Level (5-E)
Earthquake Hazard Level	50%/50 year	a	b	c	d
	20%/50 year	e	f	g	h
	BSE-1 (~10%/50 year)	i	j	k	l
	BSE-2 (~2%/50 year)	m	n	o	p

k + p = BSO
k + p + any of a, e, i, m; or b, f, j, or n = Enhanced Objectives
o = Enhanced Objective
k alone or p alone = Limited Objectives
c, g, d, h = Limited Objectives

Figure 5.2 Rehabilitation Objectives [6]

6. Determining a Rehabilitation Method

Once a decision is made regarding the level of performance desired for the building, a method must be chosen to by which to analyze the structure. The *Guidelines* specify two rehabilitation methods: the Simplified Method and the Systematic Method.

6.1 The Simplified Method

The Simplified Method is intended for buildings which only need to meet minimum performance requirements, such as the Limited Rehabilitation Objectives outlined in the *Guidelines*. Limited Rehabilitation consists of either a Partial Rehabilitation in which only a portion of the lateral-force-resisting system is addressed, rather than the entire structure; or a Reduced Rehabilitation effort where the entire structure is examined, but not to the extent that the facility reaches the requirements of the Basic Safety Objective (BSO)(2-6)[6].

The Simplified Method can also be used to obtain the Life Safety Performance Level for a BSE-1 earthquake provided that the facility meets certain requirements. The first requirement is that the building conforms to one of the Model Building Types as well as the specifications concerning number of stories, regularity, and seismic zone. Additionally, the building must be inspected according to FEMA 178 (BSSC, 1992) and the Simplified Method addresses all the deficiencies identified in the evaluation. If all these requirements are not met, the Systematic Method is necessary (2-28)[6].

6.2 The Systematic Method

The Systematic Method is an iterative process that involves creating a model of the structure, applying the design solutions to the model, and then analyzing the structure with the alterations to ensure that the building meets all the Rehabilitation Objectives. The specific steps are as follows: the first step is to analyze the existing structure to determine whether it meets the objectives as-is. If the structure is found to be deficient, one or more rehabilitation strategies are selected to overcome the deficiencies. A preliminary design is developed that implements the selected rehabilitation strategies. The structure, together with the retrofit design, is then reanalyzed. If the analysis indicates that the rehabilitation strategies are sufficient and the design allows the building to perform in a way that achieves the Rehabilitation Objectives, the cycle is complete, otherwise additional rehabilitation strategies are introduced and the rest of the iteration is completed. This process is repeated until the Rehabilitation Objectives are achieved (2-28)[6].

7. Performing the Analysis

An analysis of the building must be performed to determine how the building will react to the prescribed ground motion indicated in the Rehabilitation Objectives. All elements of the building that are designed to carry either lateral or gravity loads should be incorporated in the analysis. There are four analysis methods outlined in the *Guidelines*: Linear Static Procedure (LSP), Linear Dynamic Procedure (LDP), Nonlinear Static Procedure (NSP), and Nonlinear Dynamic Procedure (NDP), which is also known as nonlinear time history analysis[6].

7.1 Linear Procedures

Although linear procedures may be used for most rehabilitation strategy, the process is limited to very regular buildings. The *Guidelines* require that there not be any discontinuities in the lateral-force-resisting system and no interstory torsional strength irregularities. However, if it can be proven that the demand placed on the structural elements during the specified earthquake hazard does not exceed the ductile limit of the member, i.e. the building will behave elastically, then a linear procedure can still be used. This can be a tedious process and it is advised that if the building is irregular, a nonlinear procedure is used from the beginning[6].

7.2 Nonlinear Procedures

Nonlinear procedures are applicable for rehabilitation strategies and for almost all building types. NSP can be used for all buildings unless they experience "significant higher-mode response" (2-31)[6]. In this case, a LDP analysis should be included in the procedure. The NDP is a very rigorous analytical process and therefore must be reviewed by a third-party professional with extensive knowledge of seismic design and nonlinear procedures. This process should not be used on wood frame structures or unless a comprehensive knowledge of the existing structure was acquired (2-31)[6].

8. Determining Rehabilitation Strategies

Buildings can require rehabilitation on a local or global scale, or both. After the analysis determines in which ways the facility is deficient, an appropriate rehabilitation strategy should be selected.

8.1 Local Strategies

Some buildings have sufficient overall lateral-load capacity, but certain individual members do not have adequate strength, toughness, or deformation capacity. In such instances, local rehabilitation is all that is required. Local rehabilitation measures include improvement of connections, member strength, and/or component deformation capacity. This is often times the most economical rehabilitation strategy when only a few of the building's components are deficient.

Local strengthening is intended to improve the performance of understrength elements or connections to enable them to resist the strength demands determined in the analysis, without changing the structure's response as a whole. Solutions to these local deficiencies include adding cover plates to steel beams or columns, providing additional clip angles to strengthen connections, and adding plywood sheathing to an existing timber diaphragm. Some corrective measures are intended to increase the allowable deformation of a component without greatly affecting its strength or stiffness. This can be useful when it is not desired to change the existing load paths or behavior of the building system. There have been some relatively recent and innovative procedures developed for this purpose[6].

Concrete structures designed to past codes have shown some deficiencies during recent seismic events. Occasionally they experience local failure due to a lack of confinement, insufficient lateral reinforcement, and inadequate reinforcing splices. To prevent these types of failures, steel or concrete jackets have been placed around the deficient columns. However, this method of rehabilitation increases the stiffness of the member, thereby causing it to attract more seismic loads during an earthquake. A different method of retrofitting that does not add stiffness to the element is wrapping the column with individual cable strands which are then prestressed to exert a uniform pressure on the column section to increase the confinement. This has been shown to increase the columns strength and ductility mainly due to the additional concrete confinement and the supplementary shear reinforcing (6)[3].
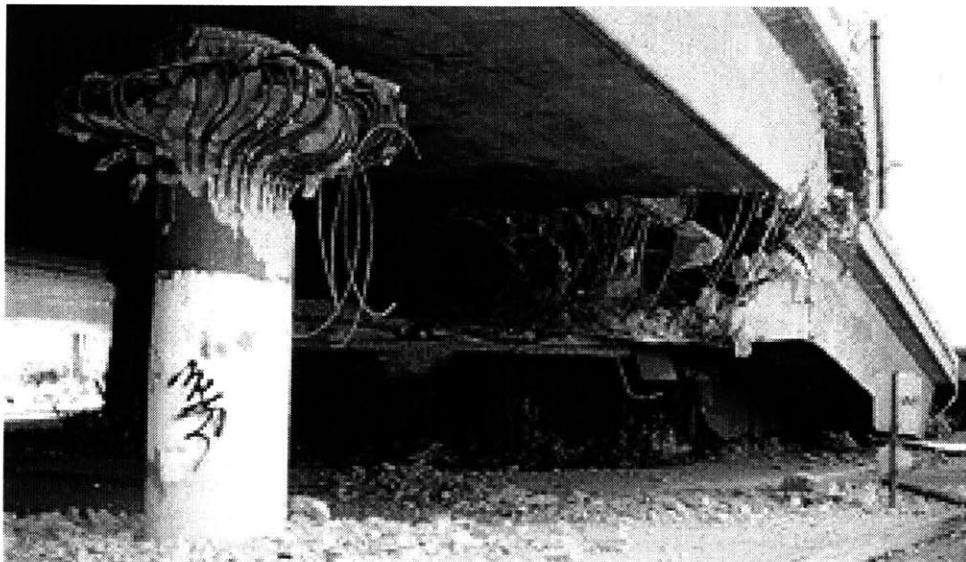

Figure 8.1.1 Structural Collapse Due to Lack of Confinement [4]

Figure 8.1.1 illustrates the failure of a column due to lack of confinement, which resulted in the collapse of this interstate during the Northridge earthquake in 1994.

Carbon fiber reinforced plastic (CFRP) is an advanced composite material with greater strength than steel, but much lighter. It can be applied to the exterior of concrete members such as columns or beams to increase their strength and ductility without adding stiffness to the element. The application process is very simple; it is applied in strips, similar to wallpaper, with the use of an epoxy. An additional advantage of this product is that it is very corrosion resistant and is therefore ideal in corrosive environments (5)[3].
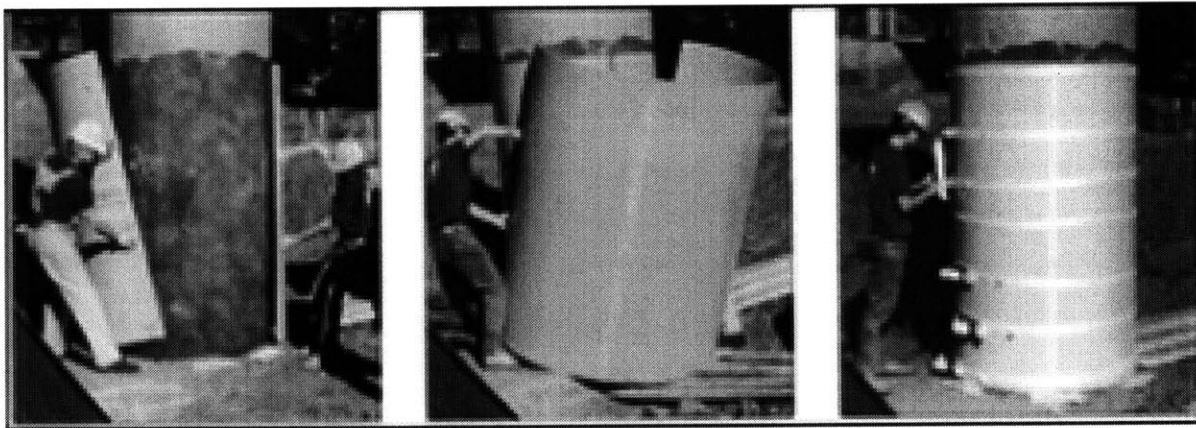

Figure 8.1.2 Installation of Prefabricated Composite Jacket [4]

Figure 8.1.2 is an example of the installation process of a prefabricated composite jacket. This is a very typical retrofitting technique for highway bridges in areas of high seismicity. In 1998, more than 3,800 columns on the Yolo Causeway in Sacramento, CA were retrofitted in this fashion. The result was an

increase in the bending strength of the column and it raised the ductility by 15%. The process can also be utilized for columns in buildings[4].
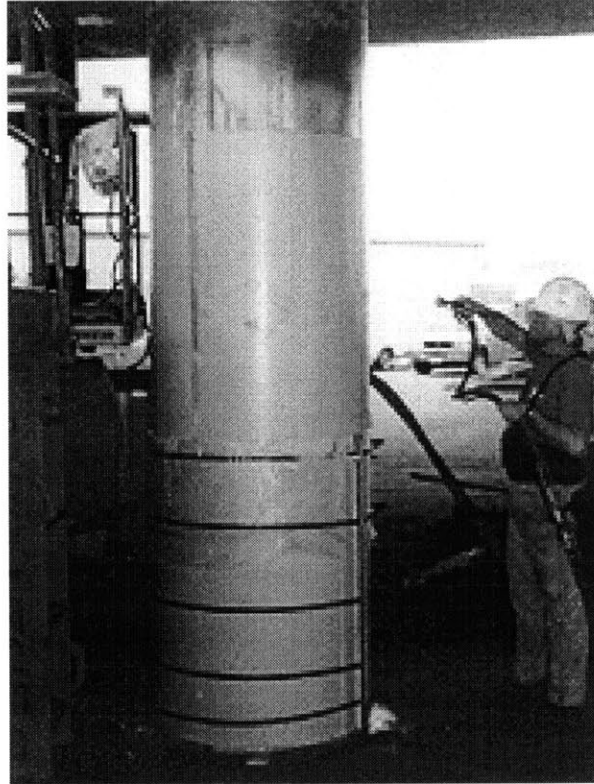

Figure 8.1.3 Column Confinement [4]

Fiber reinforced cement (FRC) is made of a high strength fiberglass mesh and a thin layer of fiber reinforced concrete. It is similar to CFRP and can be applied in the same way. This retrofitting strategy is extremely effective in improving the seismic performance of unreinforced masonry walls (URMs). Unreinforced masonry walls are notorious for their poor performance in earthquakes. There is currently a lot of concern about these because a vast amount of the structures in the Midwest and also on the West Coast are

constructed out of URM. These structures are extremely susceptible to brittle failure and collapse because masonry has very little tensile strength of its own and is therefore unable to resist the deformations caused by earthquakes. Applying a layer of FRC to a URM can transform it into a reinforced masonry wall with substantially higher tensile strength and increased structural performance during an earthquake (5)[3].
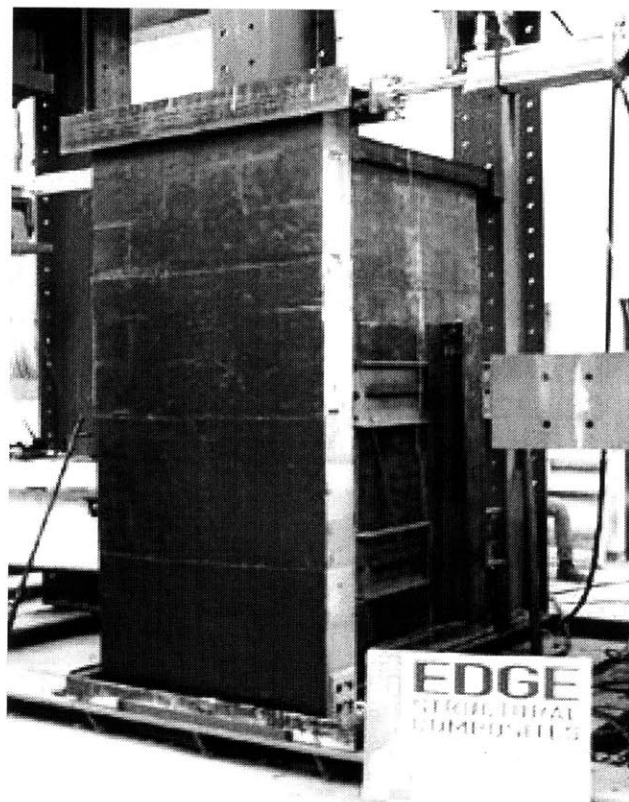

Figure 8.1.4 URM Rehabilitated with Reinforcing Fabric [7]

Figure 8.1.4 is an example of a research test being conducted on an URM. The failure load prior to adding the reinforcing fabric was on average 1600 lbs. After the application of the fabric, the failure load increased to 9 kips and the

mode of failure changed. The bolted connections at the base of the wall failed rather than the masonry itself. This demonstrates the great effectiveness of fiber reinforcement[7].

8.2 Global Strategies

There are several ways to improve the global performance of a building during a seismic event, some of them conventional and others more innovative. Three conventional methods are global structural stiffening, global structural strengthening, and mass reduction. Passive energy dissipation and base isolation are two innovative ways of improving the overall seismic performance of a building.

Global structural stiffening reduces the lateral deformation of a building during an earthquake. Flexible structures sometimes perform poorly in earthquakes because they lack the ductility or toughness required to resist the large lateral deformations that ground shaking can induce in the structural system. Hence, global structural stiffening is a good rehabilitation strategy for such buildings. There are several ways to accomplish this; adding shear walls or constructing new braced frames within the existing structural system are a couple of examples of how to implement this strategy.

Buildings which exhibit inelastic deformation at very low levels of ground motion are said to have inadequate strength to resist lateral forces. This can result in excessive damage to the structural system due to a mild earthquake. Global structural strengthening is required to mitigate this problem. The addition of shear walls and braced frames can compensate for the existing structures lack

of strength. However, these new structural elements can be significantly stiffer than the existing structure, causing them to attract nearly all of the lateral forces for the building. They must therefore be designed to resist such loads. Another alternative is to use moment-resisting frames which are more flexible and more compatible with the existing structural system. Due to their flexibility, they may not become effective in the building's response until the existing, more brittle members have already yielded[6].
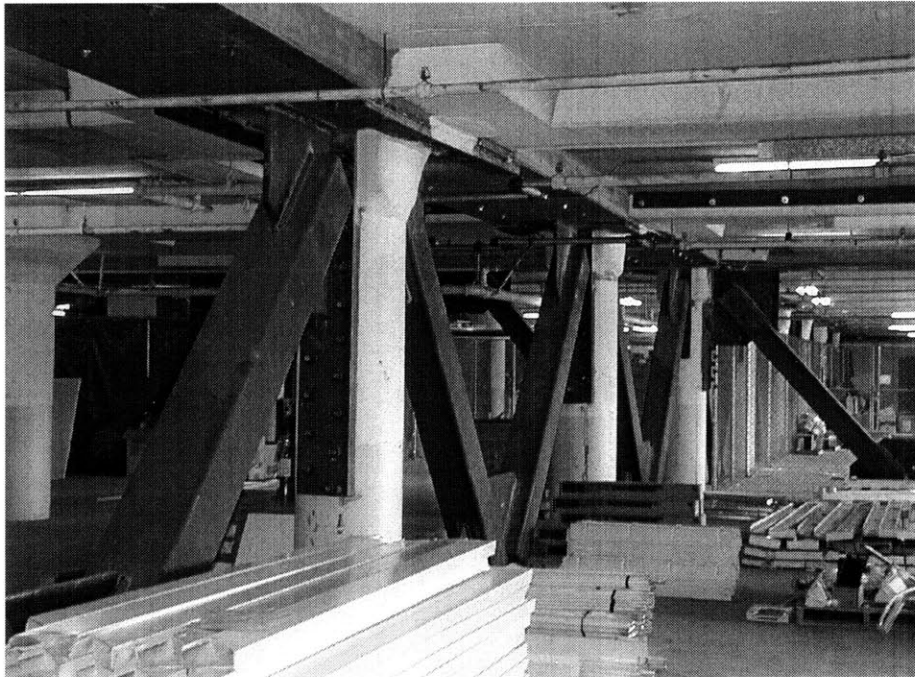

Figure 8.2.1 Chevron Bracing [15]

Figure 8.2.1 is an example of concentric chevron bracing that was installed in the Starbucks Headquarters in the SoDo district of Seattle. Seattle is located near the Nisqually fault. In 1995 the nine-story reinforced concrete slab-column structure with masonry infill walls was retrofitted to improve its

performance during earthquakes. On February 28, 2001 the area experienced an earthquake of magnitude 6.8. These retrofitting strategies probably prevented the building from experiencing extensive structural damage[15].

Reducing the mass of a structure can significantly reduce the inertial forces experienced by the structural system, thereby improving the overall building performance during an earthquake. This process can be used in lieu of structural strengthening and stiffening. Methods for reducing a building's mass include the removal of heavy storage and equipment loads, replacement of heavy exterior cladding and interior partitions with lighter substitutes, and the demolition of upper stories.

An important thing to consider when utilizing global strengthening or stiffening techniques, or mass reduction, is the effect of such measures on the building's natural frequency. Changing the natural frequency of a building can significantly alter its behavior during an earthquake. Before implementing these techniques, past earthquake histories located near the building site should be analyzed to determine the dominant frequency of the ground motion. If implementing these rehabilitation strategies results in a natural frequency of the building that is closer to the dominant frequency of the ground motion, more innovative design solutions should be reviewed.

Passive energy dissipation, or passive damping, can be a very effective means of rehabilitating a building on a global scale. Damping can be defined as "the process by which physical systems such as structures dissipate and absorb the energy input from external excitations,"(136)[5] which are in this case

earthquake loads. Adding additional elements that are specifically designed to absorb the earthquake energy reduces the amount of energy that must be absorbed by the existing structure, thereby increasing the overall performance of the structure during a seismic event. This can be accomplished through a variety of damping methods, including viscous damping, frictional damping, and hysteretic damping.

Viscous damping is the energy dissipation due to the viscosity of the material and is a function of the time rate of change, or the velocity, of the corresponding displacement (139)[5]. Examples of viscous dampers, or dashpots, are prevalent in everyday life, such as shock absorbers on cars and the cylinders on screen doors that keep them from slamming shut. Viscous dampers used for seismic control are basically larger versions of these. A viscous damper is composed of a piston head and rod surrounded by a viscous fluid. The piston head is permeated and, as a force is applied to the rod, it pushes the piston head through the fluid. The fluid reacts by creating a resistive force that is dependent on the velocity of the motion. This is the mechanism that dissipates the earthquake energy[5].
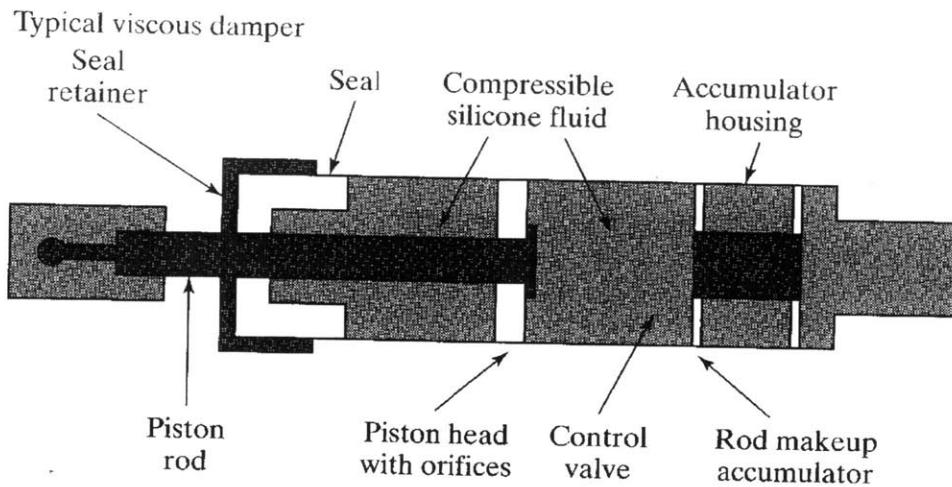
Figure 8.2.2 Schematic of a Viscous Damper [5]

Friction dampers dissipate energy in a different way than viscous damping, but similar to the way brake pads work in cars. This type of damping is a function of displacement rather than velocity, and is of a constant magnitude that depends on the coefficient of friction inherent in the material. To maximize the amount of energy dissipation that occurs, special friction pads are used with very high coefficients of friction. These pads are inserted in bolt-plate connections at the center of diagonal cross bracing within a structure, as shown in Figure 8.2.4. Differential drift between stories in a building well cause rotation in the connection, thus creating the displacement upon which the energy dissipation is contingent[5].
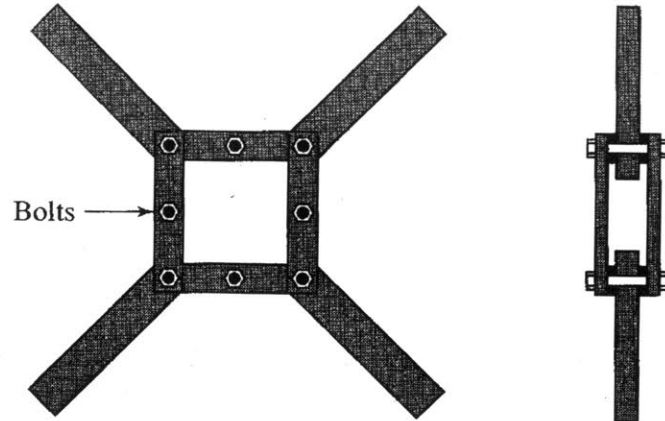
Typical friction damper

Figure 8.2.4 Friction Damper for Cross Bracing [5]

Hysteretic damping occurs when there is inelastic deformation of an element. During seismic events, inelastic deformation of the structural system is typically considered a very bad thing. However, adding additional structural members with yield strengths below the yield strength of the existing structural system can attract the seismic loads and dissipate the earthquake energy through inelastic deformation, thereby preventing permanent deflection in the existing structure. Hysteretic dampers are typically located as bracing elements between existing columns. One design consists of cross-shaped yielding metal core surrounded by a strong jacket but separated by a spacer material. This design allows the inner core to yield under separate deformation from the outer jacket, which must remain intact to prevent buckling[5].
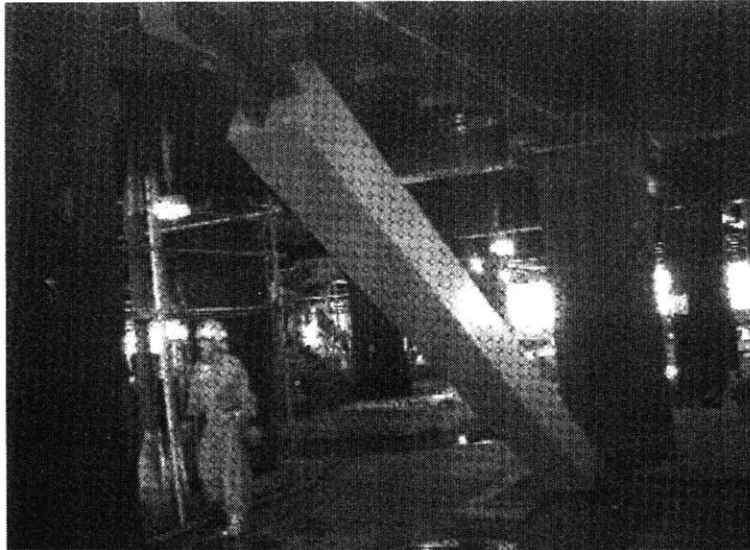
Figure 8.2.5 Hysteretic Damper [5]

There are several items to take into account when designing a passive energy dissipation system for building rehabilitation. The plan and vertical distribution of the selected damping devices must be included in the mathematical model of the building. The dependence of the devices on excitation frequency, ambient and operating temperature, velocity, and sustained loads must be accounted for in the analysis of the model. The effect of changes in the operating temperature of the device should be taken into special consideration. The properties of many dampers are generally dependent on ambient temperature as well as the rise in temperature due to cyclic response or earthquake excitation. For example, a rise in the temperature of a viscous damper changes the viscosity of the fluid within the damper, thereby changing its mechanical properties. Because of this, the analysis should be conducted

multiple times to observe the effects of the varying mechanical characteristics of the dampers[6].

Another aspect of the damper design that should be considered when designing the dampers are the effects of environmental conditions such as the effects of aging on the mechanical characteristics, creep, exposure to moisture and damaging substances, and fatigue. Fatigue is an especially important factor in the design of passive energy dissipation systems. This is true because a system could use a substantial portion of its energy dissipation capacity due to low-cycle fatigue caused by frequent subjection to wind forces. Subsequently, systems designed to dissipate energy in this way must be shown to behave in the linear elastic range for such wind forces[6].

Unless designed correctly, a passive energy dissipation system can actually be detrimental to a building system. If the damping system is not capable of deforming adequately during a large seismic event, it could induce greater localized stresses in the adjacent structural members, possibly causing building failure. The *Guidelines* introduces specifications to prevent this from occurring by requiring the dissipation devices to be able to sustain larger displacements than the maxima calculated for the Basic Safety Earthquake 2 (BSE-2), which has a 2% probability of occurring within a 50 year period. The percentage of increased capacity that is required is dependent on the redundancy of the supplemental damping system. If there are two or more damping devices per story in each direction, then the system must be designed for 130% of the calculated BSE-2 displacement or velocity, depending upon which criterion controls the damping

action. If there are less than two damping devices per story in each direction, then the capacity must be 200% of the BSE-2 (9-14)[6].

The *Guidelines* require a substantial amount of testing before a passive dissipation system can be implemented in a rehabilitation design. The testing must confirm the force-displacement relations and the damping values that were used in the design of the system. They are also intended to verify the robustness of the devices during extreme seismic events. Additionally, it is the responsibility of the engineer of record on the project to establish restrictive acceptance criteria for the damping devices, outside of which the devices will be rejected. This is necessary to ensure that the devices will behave as designed during an earthquake and therefore should be strictly enforced[6].

An increasingly popular global retrofitting strategy is base isolation. As the name suggests, base isolation is intended to *isolate* the structure from the earthquake-induced ground motion and acceleration, thereby reducing the total seismic forces experienced by the building system. Unlike a damping system, base isolation is intended to deflect the earthquake energy, rather than absorbing it mechanically. Base isolation can be particularly effective when retrofitting historical buildings because it is not as intrusive into the interior space of a building; large amounts of additional bracing inside the building or local strengthening and stiffening of individual members is not required. The application of base isolation in retrofitting can also be very beneficial if the contents of a facility are highly sensitive, such as highly sensitive equipment in hospitals and computer facilities. These "tend to sustain more damage when

conventional methods of seismic-resistant design are used and which, in many buildings, are much more costly than the structure itself"[11]. The two main categories of base isolation systems that are used in these and other situations are elastomeric isolators and sliding isolators.

The first type of base isolation system is a system of elastomeric bearings. There are several types of elastomeric isolators, including high-damping rubber bearings (HDR), low-damping rubber bearings (RB), low-damping rubber bearings with a lead core (LRB), and 'smart' isolators which include some kind of active or semi-active damping device. With the use one of these base isolation systems, "the building or structure is decoupled from the horizontal components of the earthquake ground motion by interposing a layer with low horizontal stiffness between the structure and the foundation. This layer gives the structure a fundamental frequency that is much lower than its fixed-base frequency and also much lower than the predominant frequencies of the ground motion"[11].

Sliding isolators work in a different way. Their purpose is to limit the transfer of shear between the foundation and superstructure, thereby limiting the amount of force that is transmitted by the earthquake into the structure. There are several different types of sliding isolation systems available today. One system, which has been implemented already in at least three buildings in China, uses a specially selected sand at the interface to limit the shear transfer. "The friction pendulum system is a sliding system using a special interfacial material sliding on stainless steel"[11]. Rolling systems are subset of base isolation that fits within the category of sliding systems. The *Guidelines* specify that these may be

flat assemblies or have a conical or curved surface. An example would be a ball and cone system[6].

There has been a substantial amount of research conducted on base isolation since the original conception of the idea in 1970's. Originally, the theory of base isolation was focused on two concepts: heavy damping and frequency separation[10]. However, these two concepts are not totally independent. In order to provide substantial damping, a strong connection between the superstructure and the substructure is required, making it difficult to decouple the action of the ground and the structure. Subsequently, research has focused on discovering how and when to apply the appropriate amount of damping. One experiment was performed using five base isolation systems each with a different type of damping: two lead-rubber isolators, one designed to withstand moderate ground motions and the other severe ground motions, a passive linear viscous damper with 27% of critical damping, an active isolation system, and a smart, or semi-active, isolation system. The lead-rubber isolators designed for severe ground motion was able to improve base drift, but resulted in amplified accelerations and interstory drift. Conversely, the other lead-rubber system did not amplify the accelerations or interstory drift, but was also not as successful at controlling base drift. The passive linear viscous damper also did not perform well at controlling base drifts, but did control absolute accelerations and structural drifts. The smart damping system was able to control both the base drift with the same effectiveness as the first lead-rubber system, but without adversely affecting the

accelerations and interstory drift. This implies that smart damper systems can provide protection against the whole range of seismic events[18].
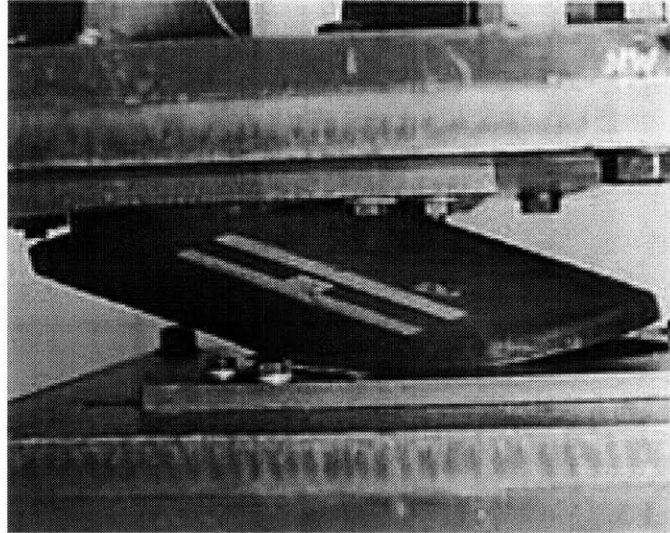

Figure 8.2.6 Elastomeric Base Isolator After Displacement

There are several factors to consider when designing a base isolation system. The design of the mechanical properties of the base isolators is dependent on parameters such as the axial loads due to gravity, the rate of loading, bilateral deformation, temperature, and aging. The *Guidelines* specify that these parameters should be used to determine the range of possible values for the stiffness and the damping of the isolation system. In addition to the mechanical characteristics of the devices, there are other concerns as well. One special concern is the ability of the system to deform in order to prevent the transfer of seismic loads and yet be able to resist the lateral loads induced by wind without displacing. Another consideration is the stability of the system under

vertical loads. The original isolators were composed of rubber, which made them 'bouncy' and created vibrational problems in the buildings. Consequently, new isolators are made with alternating layers of rubber and lead to help control the vertical deflection of the isolators under gravity loads. Base isolation is still a relatively new technology and therefore modeling and testing is still an essential part of the design process.

Although the theory of base isolation predicts reductions in seismic forces of a magnitude of 5 – 10 times that of the structure without isolation, the actual performance of structures with such systems has not been so promising in some cases. As reported earlier in the results of the experiment on damping, some base isolation systems can amplify the acceleration and interstory displacement within a building. This can be very detrimental to the contents of the building and cause great frustration with building owners who thought they were purchasing this new technology to prevent such damage. During the Northridge earthquake in 1997, some buildings with base isolation systems recorded maximum accelerations that were greater than the maximum acceleration of the ground. Some examples are the LA County Fire Command, with a high-damping rubber system, with recorded values of .35g compared to ground accelerations of .19g. The acceleration recorded at the Rockwell International Headquarters was .15g which is almost two times greater than the .08g recorded for the ground. These results indicate that there is still a need for additional research on this rehabilitation strategy[10].

9. Implementing Rehabilitation Designs

Two case studies were selected which provide examples of the implementation of several rehabilitation strategies. The first case study is the Centro Postal Mecanizado building located in Mexico City. The rehabilitation methods used for this case are relatively traditional and are performed on a local and global scale. The second case study is the Utah State Capitol. The methods used for this project are state of the art, and both local and global.

8.1 Case Study 1 – Centro Postal Mecanizado

The Centro Postal Mecanizado (CPM) building is a five story moment resisting space frame (MRSF) constructed out of rectangular (mostly square) reinforced concrete columns and 55 cm thick waffle slabs. The building was erected in 1970 and during the 1985 Mexico Earthquake it sustained some structural and non-structural damage. Vitelmo V. Bertero, and his associates at the University of California at Berkeley, performed an analysis of the most effective and economically viable solutions for upgrading the structure to resist future earthquake loads, using both traditional and innovative techniques, intending to restrict the building's behavior to acceptable elastic and inelastic levels, respectively.

The initial mathematical analysis of the building revealed that its fundamental period, T_1, was 2.14 s. This period was large for a five story building for several reasons. The inter-story height of the building was 7.2 m to create enough space for the postal equipment, and there were also large clear spans

between the columns, making the inherent stiffness of the building very low. Additionally, the elastic modulus of the concrete, E_c, was relatively low as well. The mass of the building was high due to the heavy postal equipment. In this case, the high fundamental period posed a problem because it was dangerously close to the predominant period of the earthquake's ground motion, T_g, resulting in a significant amount of energy being inputted into the structural system. There were three possible remedies for this problem: changing the fundamental period by (1) increasing the stiffness of the structure, (2) decreasing the mass of the structure, or (3) a combination of (1) and (2).

The initial retrofitting strategy utilized option (3) while maintaining the structure's behavior within the elastic bounds. The solution entailed the removal of the heavy postal equipment and conversion of the facility into a typical office building while installing a system of diagonal cross-bracing. In addition, the columns in the braced bays were to be encased with steel jackets to increase their strength. An analysis of the 3D mathematical model incorporating the new retrofitting measures revealed that the new fundamental period of the structure was .60 s, compared to the existing period of 2.14 s. Additionally, the new bracing elements and the jacketing of the columns increased the lateral stiffness of the building by a factor of 9. The inter-story drift index of the retrofitted building, as determined through a time history analysis of the 3D model using the SCT record, was only 0.1%, which was significantly less than the 0.8% allowed by the 1985 Mexican Federal District Emergency Code.

In spite of these positive results, there were also some problems associated with the proposed retrofitting strategy. The implementation of the solution required approximately 750 tons of new steel, making it a very expensive endeavor. In addition, there was some concern about whether the existing foundations possessed adequate capacity to resist the additionally loads developed in the jacketed columns. If they didn't, the cost of this design would be even further prohibitive. It was evident that a more efficient solution was necessary.

To reduce the amount of additional steel required, an energy-dissipating strategy was explored. Dissipating the energy of the earthquake would reduce the total amount of internal forces developed in the columns and transferred to the foundations. This would ensure that no additional piers would be required, thereby simplifying the retrofitting process. Three types of energy dissipating configurations were explored: friction-damped diagonal cross-bracing, chevron bracing with friction-slip devices, and steel plate energy dissipaters. There were several problems associated with the friction-damped diagonal cross-bracing. Tests of the selected "Pall devices" revealed that they experienced significant out-of-plane vibration due to the concentration of mass at the connection of relatively slender cross-bracing members and the eccentricity of the devices themselves. Other issues involved the possible degradation of the friction material over time and the maintenance of the required level of pressure on the friction interface. Using chevron bracing with friction-slip devices solved the problem of out of plane vibration, but the maintenance issues still remained.

Although extensive research had been conducted on these devices, there were still uncertainties associated with their use. To avoid these uncertainties, another system with well-established and reliable mechanical characteristics was evaluated. The recently developed Added Damping and Stiffness (ADAS) elements were selected. Tests on these devices revealed that they possessed the ability to dissipate significant amounts of energy while sustaining extremely large numbers of yielding reversals without strength or stiffness degradation. Subsequently, they are ideal for implementation in buildings such as the CPM.

In conclusion, the use of innovative techniques, such as passive energy-dissipation and base isolation, can significantly reduce the cost of retrofitting. In many cases these methods will also reduce the intrusiveness of the procedure on the function of the facility that needs to be maintained. Selecting the most efficient and cost-effective retrofitting strategy is key to the successful implementation of the seismic rehabilitation of the structure[1].

9.2 Case Study 2 – Utah State Capitol

The Utah State Capitol was built in 1916 using reinforced concrete, which at that time was considered an innovative design and construction technique. Because the technique was new, the standards for design and construction were not fully developed and therefore, by today's standards, the design is substantially deficient. Specifically, the amount of reinforcing used in the building is roughly half of what would be required today. This is especially detrimental from a seismic perspective because the steel reinforcing is what provides the tensile strength required to prevent collapse during an earthquake. As it stands,

the building is currently very brittle and hence extremely susceptible to collapse due to seismic excitation. To make matters worse, the Capitol is located just 1 mile from the Wasatch Fault, which, according to the Utah Geological Survey, experiences an earthquake with a magnitude of 6.5 to 7.5 approximately every 350 years. Because of its location and the deficiencies of its design, the Utah State Capitol Preservation Board decided to conduct a $200-million renovation of the building, which includes a complete seismic retrofit of the building and also the addition of new four-story extensions to the east and west of the capitol.

Figure 9.1.1 Utah State Capitol [16]

As previously stated, the structural system of the building is constructed of reinforced concrete. The architectural design includes a central dome with a height of 165 ft. The exterior of the building is a granite cladding façade backed

by unreinforced masonry walls with large parapets made of stone and unreinforced masonry. This building is essentially a seismic nightmare, including many of the worst structural components in terms of earthquake behavior. Testing of the concrete has revealed that the quality of the concrete diminishes as the building increases in height. The strength of the concrete at the top of the dome was found to be as low as 250 psi, which is less than 1/10th the modern standard for concrete strength of 4,000 psi. This is especially bad because at that height, the earthquake accelerations would be amplified. The exterior of the building is potentially hazardous as well. The majority of deaths in America due to earthquakes have not been the result of building collapse, but rather that of bricks and other cladding falling off of building exteriors and onto people below. The heavy granite façade backed by unreinforced masonry has great potential to cause such tragedies. In addition to the challenges of the structural system, the fact that building was of historical value and therefore it was important to limit the affects of the rehabilitation on the appearance of the building was another important constraint. The retrofitting of this building posed a great challenge to Reaveley Engineers & Associates, the structural engineering firm chosen for the project.

The selected rehabilitation procedure includes both local and global strategies, as well as traditional and innovative techniques. The conclusion reached after an intensive study of various retrofit alternatives was to implement a base isolation system at the interface between the foundation and the superstructure of the building. This approach was selected in order to reduce the

amount of displacement and acceleration experienced by the building, and especially the dome. To supplement the base isolation system, a series of new reinforced concrete shear walls was designed to provide additional stiffness in the building. The shear walls were to be installed in a symmetrical pattern. They also aided the engineers in being able to 'tune' the structure in order to maximize the effectiveness of the base isolation system in limiting the amplifications of the ground accelerations in the dome.

The engineers worked closely with the architects to select locations for the shear walls that would be inconspicuous and not detracting from the aesthetics of the historical facility. In some locations, the existing unreinforced masonry walls that backed the granite exterior were removed and replaced with the concrete shear walls. This solved two problems by providing hidden locations for the shear walls and also creating a more substantial backing to keep the exterior intact during an earthquake. In places where this was not possible, steel bracing will be installed to secure both the granite façade as well as the heavy stone and masonry parapets. To add ductility and strength to the dome, it will be reinforced with shotcrete.

The existing foundation of the building, consisting of small, lightly reinforced concrete footings, was found to be inadequate to support the new base isolation system. It will therefore be replaced with a new heavily reinforced concrete mat foundation in a somewhat complicated construction process. The first step in the process is to create a two-way grid of concrete beams just above the existing footings. Parts of the foundation mat will then be cast between the

existing footings. Once these sections of the mat have cured sufficiently, jacks will be inserted between the existing footings and the foundation mat. Crews will then remove the remainder of the existing footings and complete the construction of the mat. The base isolation system, which consists of roughly 280 base isolators, will be installed on the mat. The type of isolation system has not been selected, but lead and rubber, high-damping rubber, and friction pendulum systems are being considered.

As with all retrofit ventures, the construction sequence and duration is imperative because the facilities which are being renovated typically need to remain open for business during the process. In the case of the Utah State Capitol, new and east and west additions will be constructed first. Once they are completed, the workers will be relocated to the new space while the retrofitting of the existing building is occurring. This entire construction process is scheduled to be completed in 2008[2].

10. Conclusion

The issue of retrofitting buildings is a very complicated one; it affects many aspects of society. There are controversies concerning when to make retrofitting a requirement, to what level must existing buildings be expected to perform, who should pay for the rehabilitation costs, and what are the best techniques for retrofitting. It will be a long time before these questions will be answered, and in many cases it will have to be decided on a case-by-case basis. FEMA has established a very useful set of guidelines for the assessment and design of seismic rehabilitation for buildings, but it is still up to society to determine when and how to implement them. The only certainty is that earthquakes will continue to occur. It is better to be prepared beforehand than to experience massive losses, both in terms of lives and economics, in the wake of a seismic event.

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