



RISK ASSESSMENT AND RETROFIT OF EXISTING BUILDINGS

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SUMMARY

The structural risk assessment and evaluation process is broken into the steps of *Develop knowledge of as-built conditions*, *Determine local response characteristics*, *Create mathematical model*, *Perform global analysis*, *Determine acceptability*, and *Select retrofit procedure/Classify per evaluation procedure* and each step discussed. These steps, in general, are major topics of their own and only a few of the primary engineering issues that affect the state of the art in this area are included in this paper. The development of conceptual retrofit schemes is also discussed. Many aspects of earthquake engineering are directly related to risk assessment and retrofit, including performance based engineering, characterization of seismic hazard, performance of nonstructural systems, costs of retrofit, and public policy. These subjects and their relationship to risk assessment and retrofit are briefly described.

INTRODUCTION

Background

The categories of losses from seismic building damage are well known. The most important category of loss, of course, is the extent of casualties, and concern for life safety has been the primary driver for development of minimum codes and standards around the world. Of increasing concern, however, are monetary losses, either from costs of direct repair of damage or due to loss of the function of the building. Although not usually measured in monetary terms, damage and subsequent loss of historic fabric or loss of entire heritage buildings is also an important subcategory of monetary losses (i.e., non life-safety).

The potential for any of these losses represents a risk that must be identified to allow individual owners of buildings, institutions, and governments to weigh their importance against other demands for the use of limited resources. Performance based earthquake engineering, whether used for risk assessment and retrofit of existing buildings or design of new ones, is beginning to provide a framework for determination and measurement of these losses. However, formal systems incorporating performance based earthquake engineering are in their infancy and individual buildings continue to require engineering using available codes and standards.

In the case of new buildings, the basic configurations and design criteria needed to prevent catastrophic failure are well known for most building types around the world. Although building codes are still constantly being refined, the issues with new buildings are more associated with the economic and political will to implement and enforce design requirements rather than deciding what requirements are appropriate. For a small percentage of new construction, owners may request non-prescriptive designs aimed at seismic performance better or more reliable than the minimum standards of the building code.

However, it is the older buildings that were constructed with little or no seismic considerations that represent the largest risk to most communities. These buildings must be evaluated by owners or communities, their level of risk determined, and unacceptable risks reduced or eliminated. This paper focuses on the issues associated with detailed procedures for risk assessment and retrofit of individual buildings (*risk assessment* in this limited sense is also often referred to as *evaluation*). Many of the prescriptive forms of evaluation and related procedures for assessment of seismic risk for groups of buildings have been discussed elsewhere [Holmes, 1996].

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Further, there is currently much research and development activity documented in the literature directed at methods of predicting nonlinear building response (analysis) and the probable behavior of various materials (acceptance criteria). Although these subjects are included, the focus of the discussion herein is on the engineering processes associated with seismic evaluation and design of retrofit measures.

Related Subjects

The issues related to the seismic performance of the existing building stock are broad and encompass almost every aspect of earthquake research and professional practice. Although risk assessment or retrofit of individual buildings are at the heart of most of these issues, it should be acknowledged that there are areas of related study that stand on their own and will not be covered in detail here. Some of these separate areas of study are described below.

Performance based earthquake engineering

Traditionally, seismic evaluations and most building retrofits have been intended to protect life safety or to at least reduce the risk of casualties in the design earthquake. In the past decade, interest has increased in reducing economic losses from damage, either due to repair costs or lost use of the building. The cost of retrofit, often over 25% of the value of the building, has also caused interest in studying costs and benefits of several levels of retrofit (and the resulting different levels of expected performance). This interest in prediction of the level of damage expected in a building before and/or after retrofit has caused development of formal frameworks for performance based seismic engineering in several countries [Fajfar, 1997], [FEMA, 1977], [PEER, 1999], [SEAOC, 1995], [Shimazu, 1999]. This development work is gradually moving toward procedures previously created for performance based building codes that have been partially implemented in other disciplines such as fire protection [Holmes, 2000]. Detailed analytical risk assessment and retrofit as discussed herein are most properly placed within a framework of performance based engineering because both earthquake demands and performance levels can be varied in that format. However, the many issues and complications of such a framework are not discussed in this paper.

Characterization of Seismic Hazard

It is assumed that the seismic hazard is characterized by an elastic response spectrum at a minimum. Some nonlinear analysis methods may also require inelastic design spectra or time histories representing the site seismic hazard. There are many issues associated with determination of seismic hazard, both concerning the technical methodologies used and the policy decisions regarding the level of risk. The state of the art in these areas is documented elsewhere.

An issue that needs improvement in earthquake engineering is the interaction between structural analysts and ground motion specialists to advance the characterization of ground motions to include potentially more useful parameters, such as duration and convenient measurement of near fault pulses. Some study of these needs have been completed [ATC, 1997], but more is needed.

Costs of Retrofit and Cost-Benefit Studies

Unlike new construction, where adding seismic design requirements is not an over-riding cost issue, seismic retrofit of buildings is commonly expensive and disruptive. The cost of retrofit is therefore the primary determinant in selecting acceptable risk for evaluation standards and for selecting the minimum performance objective for retrofit public policy or for retrofit of individual buildings. Studies have shown that "typical" retrofit costs have large ranges, with coefficients of variation of about 100%, even when buildings are subdivided into fairly specific categories [Hart, 1995]. This is not unreasonable considering that existing buildings can have a wide variety of deficiencies, with some buildings requiring far more retrofit than others. Usually the economic feasibility of retrofitting any individual building can only be determined by a building specific evaluation to determine the extent of deficiency; often descriptions of building specific at the conceptual level will provide adequate scope for cost estimating purposes.

Procedures for calculating life-cycle cost-benefit ratios for seismic retrofit are straightforward. However, determination of the parameters necessary to perform such studies, including the annual probability of occurrence of various ground motions, the damage to the structure before and after retrofit, casualty rates and the monetary value of casualties, and the extent of building downtime, is difficult and require considerable engineering judgement. With few exceptions, such calculations yield benefit to cost ratios of less than one,

which is often counter-intuitive. Therefore, these studies are often used only to compare alternate actions, or are accompanied by risk adverse analysis (where the consequences of large losses are highly weighted).

Nonstructural Elements and Systems

The performance of nonstructural elements and systems in buildings is important due to their contribution to economic losses and their role in maintaining building function. However, other than for heavy exterior cladding that affects structural response or creates a falling hazard, these elements have generally not posed a high risk to occupant life safety. Seismic protection is provided by anchorage to the structure and by assuring compatibility with structural drift. In cases of extreme functional importance, internal fragility of equipment is also considered. Except when heavy or stiff systems interact with structural response, seismic performance of nonstructural elements can be separated from consideration of structural evaluation and retrofit. The unique issues associated with nonstructural building systems are not discussed in this paper.

Reliability

Seismic codes and standards traditionally have not been developed on the basis of reliability methods such as Load and Resistance Factor Design (LRFD) due to the complexity of the design process and the poorly defined variability of many of the parameters. The need to express the probability of failure of "code-designed" buildings has been acknowledged since the mid-1970s when ATC 3-06 [ATC, 1978] included a discussion of the problem in its commentary. Recently, progress has been made in this regard by the SAC Steel Project [SAC, 1999]. In order to develop performance based guidelines for the design of steel moment frames, a method was developed to measure the probability of collapse of a building (or group of buildings) from various ground motions [Hamburger, 2000]. Both the large variation in maximum building displacement due to similarly scaled time histories and the various uncertainties of calculations are considered. Although the method applies only to a narrow definition of a single performance level—collapse—the framework can probably be extended to other performance parameters. Many engineers, fearful of the implied "promises" of performance based design, feel that it is imperative to develop such a framework to describe the various performance levels. Actions must be developed to make such decisions.

Policies and minimum standards for evaluation and retrofit

Selection of minimum standards for seismic performance in existing buildings, either when buildings are being evaluated or retrofit, is controversial. The results of evaluation of most buildings fall on a continuous scale of increased risk of losses, except when a collapse mechanism is apparent. Selection of acceptable risk by policy-makers is difficult and commonly falls to the engineering community by default in the form of codes and standards. Issues most often considered when setting public policy are the risk to life safety, the historic significance of buildings, and some measure of the benefit/cost relationship of retrofit. It is hoped that the availability of better engineering tools for evaluation and retrofit, particularly in the framework of performance based engineering, may enable planners and politicians to refine targets for mitigation of seismic risk to efficiently reflect the best interests of the community or region. Such clarified public policies could provide valuable input into development of local codes and standards.

Economic ratings of buildings

Many banks and insurance companies worldwide use Probable Maximum Loss (PML) as a measure of seismic risk to buildings. PML is a statistically based parameter, measuring direct damage as a ratio of repair costs to replacement cost. Although first defined by Steinbrugge [Steinbrugge, 1982], many PML calculations rely on ATC 13 [ATC, 1984], a compilation of expected damage to buildings in California based on expert opinion. Some computerized calculation methodologies for certain building types have been updated with actual earthquake damage statistics, but little useful data exists. Although the determination of a PML is often called an evaluation, and the PML is a measure of risk, there is little relationship between PML and the type of analytical risk assessment discussed here. Eventually, individual building analysis may yield sufficiently detailed predictions of damage to allow direct translation to economic losses. Currently this translation is done by engineering judgement, if at all.

Regional or portfolio loss estimation

Regional loss estimates are used to raise local awareness for post earthquake emergency planning and response, and to study regional mitigation measures. Building damage functions are often statistically based, not directly

tied to evaluation procedures and have the same shortcomings as Probable Maximum Loss calculations. A more analytical loss calculation methodology was developed for HAZUS, the FEMA funded national loss estimating method developed for the U.S. [Kircher, 1997]. This methodology is based on a pushover analysis technique that is also often used for evaluation and retrofit of existing buildings. This will allow a more direct link between analytical evaluation procedures and modeling of building inventory in a region. In the future, performance based earthquake engineering holds promise to provide more direct links between different evaluation procedures.

THE ENGINEERING PROCESS OF RISK ASSESSMENT AND RETROFIT

The Relationship between Risk Assessment and Retrofit

Analysis of response of the vast majority of buildings for the effect of seismic ground motion requires consideration of nonlinear structural behavior. It is simply too costly to keep building structural systems in the elastic range for significant earthquakes, and, in addition, an elastic superstructure response would put tremendous demands on foundations. Consideration of nonlinear response is particularly required for existing buildings, many of which have very little deformation capacity. As the realization grew that existing buildings pose a significant seismic risk, and evaluation and retrofit became extremely important, the need for explicit consideration of structural response in the nonlinear range became obvious.

Due to nonlinear behavior, as well as the generally complex and indeterminant nature of lateral force resisting systems of buildings, the design of the retrofit of a building is seldom a straightforward process, where a demand force is known and a resisting element is simply designed to suit. Typically, a wide variation in deformation capability among the new and existing elements requires a verification analysis of the proposed retrofit to assure that all significant structural components fall within acceptable force or deformation limits. The analysis process required for this verification is identical to that used for detailed *evaluation* of buildings; there are simply no new retrofit elements included in the case of a pure evaluation. In the vocabulary of performance based earthquake engineering, the evaluation process is directed at determining the performance level of the building for the design ground motion (or verifying that it meets a given performance level stipulated for the evaluation). Similarly, after a retrofit is properly engineered, it must be verified that the overall structure meets the intended performance level for the specified ground motion.

A convenient way to discuss the engineering issues of evaluation and retrofit is to break down the process into steps, as shown in Figure 1. For any given building, the steps may be done in different order and with differing emphasis, but generally engineers must deal with the issues associated with each of the steps shown. The steps are discussed in order below.

Develop Knowledge of As-Built Conditions

For detailed analysis procedures that explicitly consider nonlinear behavior, a good knowledge of the existing structure is essential. Assumptions that have sometimes been considered "conservative," such as assignment of low material strengths or assumption of non-participation in lateral resistance for certain elements could lead to incorrect prediction of element yielding strength, incorrect determination of the sequencing of yielding, incorrect calculation of limit states, and ultimately, to incorrect assignment of element demands and determination of the global performance level. Field investigation and testing, on the other hand, are often disruptive and expensive. Historic finishes and elements are sometimes involved. The correct balance is difficult to achieve. Guidelines for minimum field exploration and testing was an important issue when planning for the development of the *NEHRP Guidelines for Seismic Rehabilitation of Buildings* [FEMA, 1997] as documented in *Development of Guidelines for Seismic Rehabilitation of Buildings: Issues Identification and Resolution* [ATC, 1992]. However, as discussed below, it is difficult to write guidelines or standards that appropriately consider the many situations

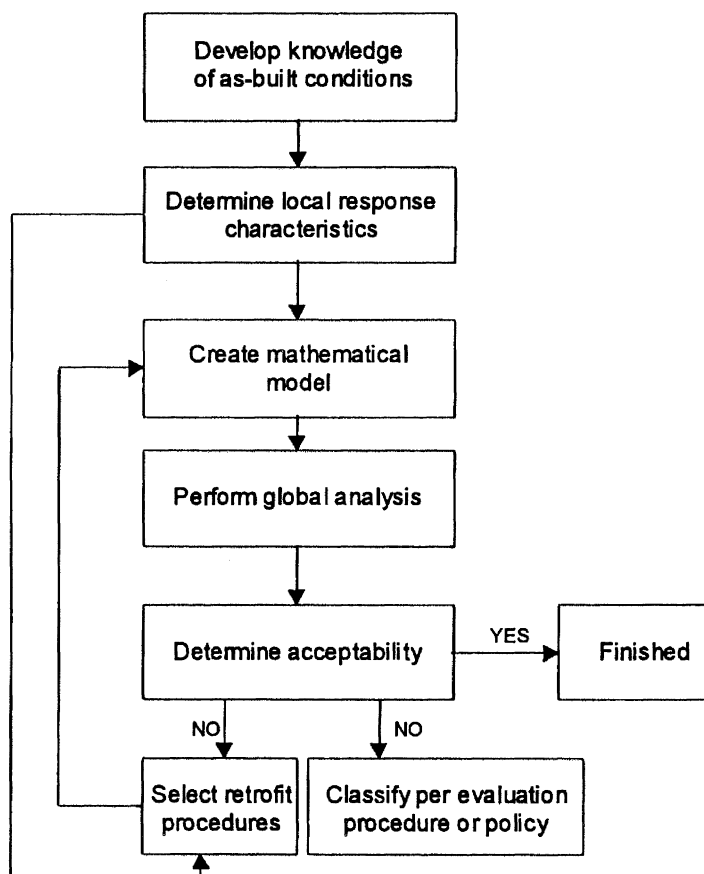


Figure 1: The engineering process of risk assessment or retrofit

that confront the design engineer, and the recommendation of the *NEHRP Guidelines* contains very conservative recommendations. There is no substitute for good engineering judgement and experience to properly deal with this issue. However, suggestions for approaches to this problem are offered in the following sections.

Regardless of the level of investigation done in the evaluation or design phases, unexpected conditions will be discovered during construction of retrofits. It is important that the design engineer is available during the construction phase to provide consistent and acceptable adjustment of details to account for these conditions. In certain cases, discovery during construction can be used as a strategy to confirm unknown or vague conditions while minimizing disruption from pre-construction investigation.

Buildings without construction drawings available

Obviously, the most difficult situation is when no construction drawings are available. In this case, neither the overall configuration of the building nor the material properties may be known, although a layout of most buildings is available due to the ongoing need to document uses, rentals, or remodels of the spaces. The overall dimensional accuracy of such layouts should be verified. The exact configuration of elements that have significant strength or stiffness must be determined, whether traditionally classified structural or nonstructural. Field investigation, including limited removal of finishes, will be necessary. Constant engineering judgement as to the progression of destructive investigation must be exercised, rather than a pre-decided program that will invariably be overly aggressive. Massive masonry elements may require small diameter coring to determine the presence and nature of core materials.

Once the basic configuration of significant structural elements is known, further investigation or testing should not be planned until a preliminary analysis is completed that will determine the approximate lateral capacity of the structure and the importance of various elements, both for lateral and vertical load carrying purposes. If the structure is relatively flexible, and the installation of a new, stiffer lateral force resisting system is feasible, a minimum of additional investigation may be necessary, as needed to determine the deformation capacity of significant vertical load carrying existing elements. If existing elements are stiff, the possibility of their use as

primary lateral force resisting elements must be established by preliminary analysis, and the sensitivity in the overall analysis to their strength determined. A rational program of field-testing can follow. Limited numbers of stiff, but weak elements may be economically removed or decoupled from the structural system, eliminating the need for detailed investigation and testing.

Methods for in-situ testing of existing materials, particularly unreinforced masonry, are well developed and documented in the literature. More efficient and less destructive methods than are currently available for common materials such as reinforced concrete and steel weldments are needed.

An example of an early reinforced concrete building for which construction drawings were not available is described in Figure 2. In this example, certain stiff but brittle infill elements were present. After a limited field investigation, it was decided to introduce a stiff new lateral system, and the existing vertical load system was then field-investigated in a limited program only to determine deformation capacity.

Buildings with construction drawings available

Given a reasonably complete picture of the configuration and detail of the structure from construction drawings, the configuration and material strengths must still be verified in the field. In this case, confirmation that the basic structural configuration was constructed as drawn is relatively easy. Material properties, however, may vary from those specified on the drawings. Reasonable guidelines for this purpose are not available and are difficult to develop. When such guidelines are developed by consensus code writing or standards procedures, they generally error on the conservative side [BSSC, 1999].

Generally, confirmation is desired of general conformance with specified strength, and a determination of the extent of variability. However, this information is only of interest if it has significance in the strength, stiffness, or global behavior of the structure. It is recommended here that testing programs be developed only after initial analysis is completed. This analysis should reveal the general sensitivity of the structural response to material strengths and the locations that may be most critical. Knowledge of material specifications and construction techniques of the era that the building was constructed often will suggest the likely variability of properties, also affecting the extent of the testing program.

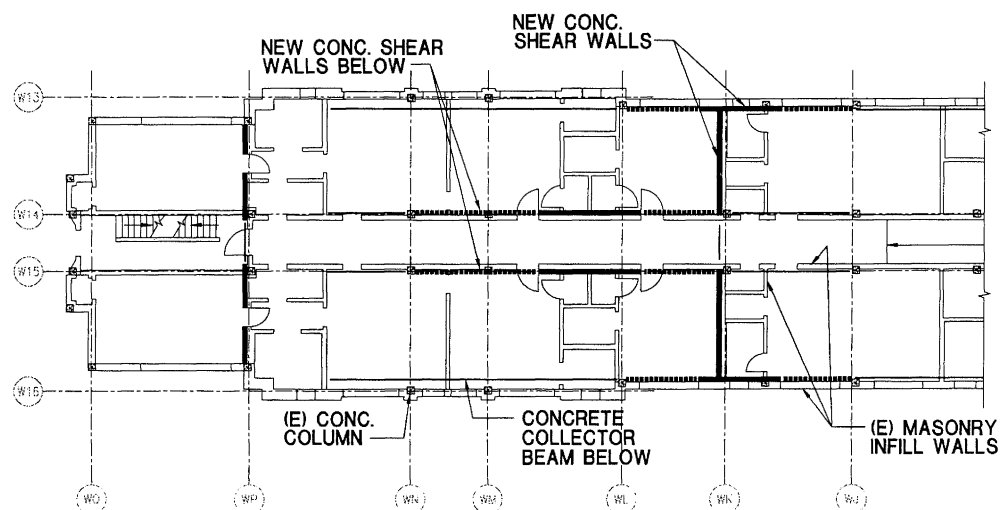
An example of a very large concrete structure, for which construction drawings were available, is described in Figure 3. The size of the structure, date of construction, and commercial use would suggest that deviations from the drawings and detrimental variations in strengths may be present. Certain seismic deficiencies were obvious and preliminary calculations for the proposed retrofit scheme indicated that nominal strengths indicated on the construction documents would provide adequate performance of existing elements. An initial testing program was therefore established that was far less extensive than recommended in FEMA 273 [BSSC, 1977]. As indicated in Figure 3, testing beyond this initial program was not required, saving the owner considerable money and reducing the disruption to the tenants of the building.

Determination of Local Response Characteristics

The procedures discussed in the paper assume that some form of nonlinear analysis will be performed. Although this may not be necessary for small, simple buildings, the "art" of evaluation and retrofit of existing buildings has progressed to the point where nonlinear behavior should be considered in many cases. Even if a nonlinear analysis is not contemplated, the relationship of element capacities and likely element failure modes should be studied to identify potential brittle global failures or unexpected nonlinear response.

Before global structural analysis is begun (usually with mathematical modeling on a computer), as much as possible should be learned with limited studies of local conditions and capacity relationships. These studies can expose potentially serious flaws in the lateral system and will determine the level of detail required in the mathematical model. As a result of these studies, retrofit measures are often inserted into the first model and the as-is building is never analyzed, as is shown as an alternate path in Figure 1.

Interconnectivity, such as connections of lateral force resisting elements to diaphragms and foundations, is typically not modeled and this assumption must be preliminarily verified. A load path should be identified and connections reviewed for brittle or ductile behavior. Brittle connections could invalidate the model and a



PARTIAL PLAN OF GUEST WINGS

Figure 2: Example of Building With No Construction Drawings

Original construction drawings were not available for a U-shaped guest wing of a resort hotel built in 1927. A plan of one leg of the U is shown above. The structure is framed with concrete columns, beams, and slabs, and has masonry infill on the perimeter and along the corridors. A reasonable program of field investigation led to a retrofit scheme with a high degree of reliability despite lack of detailed drawings of the concrete construction.

The program began with visual observation of the typical configuration of structural members, often through holes that had previously been cut in plaster finishes for routine maintenance and repair. Finishes were removed in some locations to obtain dimensions of typical concrete beams and columns, and to inspect the general quality of the infill masonry. Small diameter holes were drilled into walls in various locations throughout the building to determine whether they were constructed of masonry, concrete, or other materials. Once the overall configuration was established, preliminary analysis indicated that the extent of masonry infill could have a significant effect on response in the long direction of each wing.

Additional field investigation of the masonry construction found that the masonry was low quality field cast concrete units poorly laid-up without reinforcing. A masonry prism was removed from a wall and tested to determine its compressive strength and modulus of elasticity. In a few cases small areas of concrete cover were chipped off beams and columns to determine approximate reinforcing bar size and layout. A limited number of concrete cores and steel reinforcing bar samples was also removed and tested.

Further analysis indicated that, given the construction quality and properties, the infill system would not form adequate or reliable lateral force resisting elements, and strain limits to prevent life threatening local failure were estimated. New concrete shear walls were designed to provide sufficient strength for structural stability and to limit drifts to prevent unacceptable levels of damage to the infill system.

The apartment complex described in Figure 10 was built under commercial pressures and at a time when material quality may not have been carefully monitored. Confirmation of material strengths assumed in preliminary calculations was extremely important. FEMA 273 Guidelines [FEMA, 1997] would require an extensive testing program due to the large size of the building, which was separated into three structures by expansion joints. Such a testing program was undesirable, both from a cost standpoint and due to the associated disruption to tenants. A reduced program of testing was undertaken as shown below,

Material	FEMA 273 recommendation	Preliminary Program
Concrete slab cores	72 cores	22 cores
Masonry grout cores	60 cores	15 cores

The results of the preliminary program are shown below:

CMU Grout		Concrete Slabs	
Location	Strength in psi*	Location	Strength in psi*
Bldg I: 7th floor	4 440	Bldg III: 2nd floor	5 090
Bldg I 6th-7th floor	4 320	Bldg III: 3rd floor	4 270
Bldg I 6th floor	4 330	Bldg III: 4th floor	3 630
Bldg I 5th-6th floor	4 750	Bldg III: 5th floor	4 440
Bldg I 5th floor	4 560	Bldg III: 6th floor	4 780
Bldg II 1st floor	4 110	Bldg III: 7th floor	3 570
Bldg II: 2nd floor	5 450	Bldg III: roof	5 170
Bldg II: 3rd floor	5 400	Bldg II: 2nd floor	7 250
Bldg II: 4th floor	3 340	Bldg II: 3rd floor	6 440
Bldg II: 5th floor	3 880	Bldg II: 4th floor	6 220
Bldg III: 1st floor	4 880	Bldg II: 5th floor	5 330
Bldg III: 2nd floor	4 180	Bldg II: 6th floor	3 940
Bldg III: 3rd floor	5 680	Bldg III: 7th floor	4 430
Bldg III: 4th floor	5 930	Bldg II: roof	4 690
Bldg III 5th floor	7 170	Bldg I: 2nd floor	5 670
		Bldg I: 3rd floor	3 800
		Bldg I: 4th floor	5 290
		Bldg I: 5th floor	5 760
		Bldg I: 6th floor	5 370
		Bldg I: 7th floor	4 985
		Bldg I: 8th floor	5 050
		Bldg I: roof	5 420
Overall average	4 828 psi	Overall average	5 027 psi
Coeff. Of Var.	20%	Coeff. Of Var	18.5%
Nominal strength	2 500 psi	Nominal strength	4 000 psi

*1 psi = 0.00689 MPa

The CMU grout strength, although exhibiting a relatively high coefficient of variation, was considerable over nominal strength. These wall were bending critical so the high strength only increased the factor of safety against shear failure. No more tests were taken. The concrete slab strength was important for diaphragm shears. Nominal strengths were used in preliminary calculations and the slabs proved acceptable with the proposed retrofit scheme. The testing indicated a $1 - \sigma$ value equal to nominal strength, which was judged adequate. No further tests were taken.

Figure 3: Example of in-situ testing

decision must be made as to whether to model such components as-is or as-retrofitted. The possibility of yielding ductile connections also must be studied to determine if the model must include such elements. Calculation of capacities of various systems to deliver forces can often be used to determine if connections are critical. Examples of components to be considered include out-of-plane and in plane wall to diaphragm connections, wall diaphragm collectors, and base-foundation conditions of all lateral force resisting elements. The possibility of significant movement at the foundation-soil interface must also be initially considered, although seldom can final foundation modeling decisions be made at this step.

It is also necessary to estimate the yielding mechanism of lateral force resisting elements before a model is constructed. In concrete and masonry, it must be determined if construction joints or lap splices form a weak link. It must be estimated if bending capacity of concrete and masonry walls and columns can be developed prior to shear failure. In steel moment frames and braced frames, connections may not be able to develop the strength of the members, which will require special modeling of the connections, or discounting of member strength. In moment frames, it is useful to calculate the relative strengths of columns and girders to determine the potential for column hinging, potentially leading to early story-side-sway mechanism. In moment frames with high gravity loading, it is possible that hinging will occur away from the column-beam joint, which will require non-standard modeling of the beams.

The likelihood of various failure types as well as their probable location depends on the material strengths used when calculating capacities. Capacities of nonductile actions, such as shear in concrete, are normally calculated conservatively low. Capacities of ductile yielding mechanisms are usually *expected* values. For example, FEMA 237 [BSSC, 1997] recommends using nominal strengths (mean minus one standard deviation or less) and the use of strength reduction factors for *force controlled* (brittle) components, and mean values or greater (depending on the significance of strain hardening) for *deformation controlled* (ductile or semi-ductile) components.

Modeling

The overall methodology used for analysis and to verify acceptability must be reviewed to understand the potential impacts on modeling. Some methodologies inter-relate the *modeling*, *global analysis*, and *acceptability* steps of Figure 1 and include special rules for each step. This discussion assumes evaluation and retrofit procedures are developed from basic principles and that each step is attempting to obtain, from an engineering standpoint, a "best fit" with reality and are therefore somewhat independent. However, the more complex the model must be, for the purpose of simulating reality, the larger the potential for variability. In many cases, it is better to provide retrofit elements to fit reliable modeling than to attempt to model extremely complex existing conditions.

All elements that significantly affect the overall stiffness or mode shapes of the structure should be included in the model. Elements sometimes considered "nonstructural" such as concrete or masonry infill, light masonry partitions, or precast concrete cladding must be considered unless they are detailed to be decoupled from the structural system, or their strength and stiffness is insignificant.

The configuration and representation of individual components must consider the potential yielding modes determined in the previous step. For example, a node must be placed at the expected point of hinging in a beam, if not at the face of column. If nonlinear response is expected in a mode other than flexure, the model should be able to represent that behavior. If components of connectivity between lateral force resisting elements are expected to yield, this also must be represented.

The modeling of individual components can be complex, even for linear analysis, and the stiffness used for materials that crack must be carefully considered. Whatever nonlinear method of analysis is chosen, a method of modeling post yield behavior of each component must be chosen. Although there is much guidance in the literature concerning nonlinear behavior of certain materials (e.g. [Paulay, 1992], [ATC, 1999], FEMA 273 [BSSC, 1997] may be the best compilation of modeling guidelines available that covers all materials.

Analysis

The next step of Figure 1 is to apply an analysis procedure. Given a mathematical model, such an analysis procedure is needed to both predict the global response to a given ground motion (in this discussion, maximum displacement), and the distribution of that response over the structure (a design deformation or drift at each story that can be used to judge acceptability of the story or of individual components).

Figure 4 represents a standard pushover relationship, in which the lateral displacement of a given point on the structure (often the taken as the roof) is plotted against an ever-increasing base shear distributed vertically in a triangular or rectangular shape. Nonlinearity of individual structural components are modeled explicitly (often simplified to elastic-perfectly plastic) or nonlinearity is accounted for by incrementally changing the properties of elements of an elastic model. Although the details of pushover curves vary, as nonlinearity in the structure increases, the point on the curve that represents response becomes relatively insensitive to force and must be measured by displacement. As interest grows to predict narrow ranges of performance, particularly in existing buildings that do not have the configuration or detailing controls of new buildings, it becomes necessary to focus our analysis on deformation rather than force. Relatively simple methods of nonlinear seismic analysis based on deformation response have been available for decades (e.g. *substitute structure* [Shibata, 1976]), but the influence of force-based gravity design on seismic design methods prevented its introduction into building codes and standard practice.

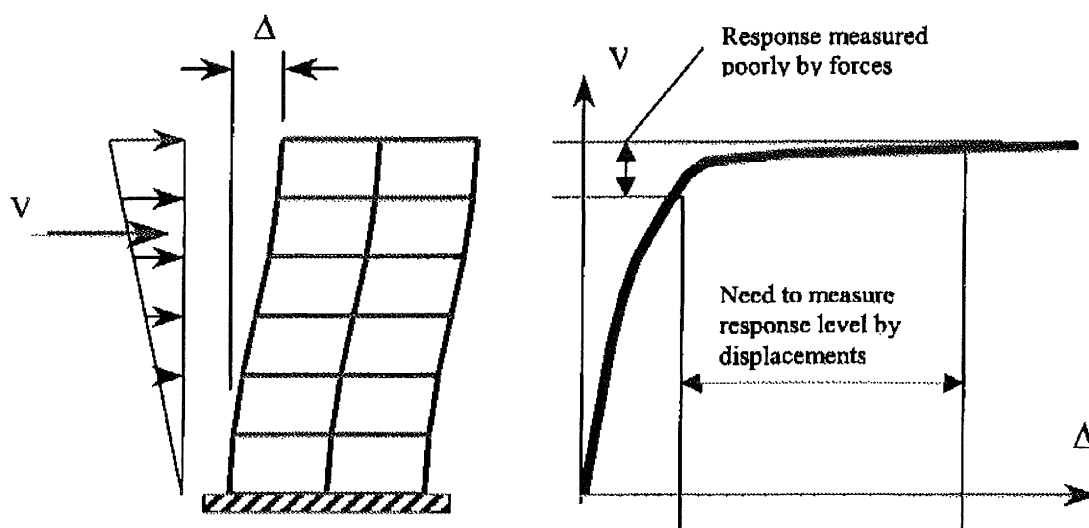


Figure 4: Typical pushover curve

There is no consensus on the best analysis method for engineering offices considering reliability, applicability to a wide range of structures, and practicality. An extensive effort was made in development of FEMA 237 to develop such a method resulting in the Nonlinear Static Procedure, based on converting elastic response to inelastic response using statistically derived coefficients. However, there is still disagreement over the values and application of the coefficients and concerns about appropriate accounting for higher mode effects. Many simplified nonlinear analysis methods are now described in both the engineering and research literature. Most use elastic response amplitudes of an equivalent single degree of freedom system to estimate maximum nonlinear response (e.g. [ATC, 1996], [Priestley, 1995], [Teshigawara, 1999]). Some reduce the elastic spectra by estimating the amount of equivalent viscous damping provided by inelastic deformation. Recently, it has been suggested that such reductions in demand are better represented by using constant-ductility inelastic spectrum ([Reinhorn, 1997], [Aschheim, 1999], [Chopra, 1999]). However, there is no benchmarking analysis method available that captures the complex nonlinear behavior exhibited by some elements, and many of the checks and cross checks of different methods are themselves performed using simplified methods. It is therefore difficult for engineers, needing analysis methods immediately, to determine the accuracy, reliability and limitations of each method.

The Capacity Spectrum Method of analysis ([Freeman, 1998], [ATC, 1996]) has been noted by many engineers for its intuitive simplicity and visual aid in determining retrofit design strategies. The method plots both structural capacity (as a pushover curve) and seismic demand (as an elastic spectrum with various levels of damping) in coordinates of spectral acceleration versus spectral acceleration, as shown in Figure 5. The intersection of the pushover curve and the appropriately damped spectrum represents the spectral displacement demand of the particular ground motion. As noted above, improvements to the method than retains its main appeal have been suggested by several researchers. Although the method is useful, a conceptual pushover analysis, in which the approximate order and significance of yielding is determined, coupled with thorough

studies of local response characteristics (as discussed above) will also provide adequate information for preliminary design, especially for an engineer experienced in evaluation and retrofit.

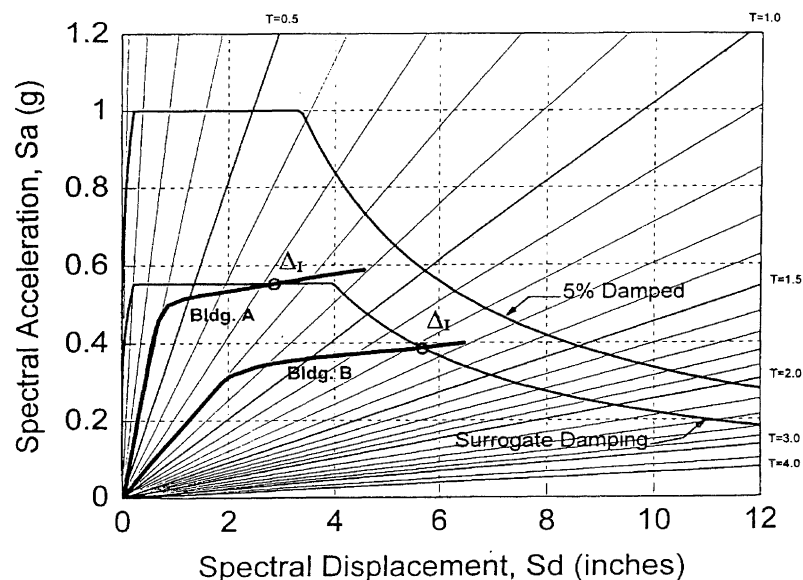


Figure 5: SASD format used for capacity spectrum

The key analysis tool is the pushover, from which deficiencies can be identified and alternative preventive measures determined. Although applicable to a large majority of buildings most in need of retrofit, it is important to note its limitations. First, the analysis is primarily based on first mode response. Although a rectangular load pattern can be used to test sensitivity of the system to failures from higher mode effects, it is difficult to deduce the sequence and extent of damage from such analysis. FEMA 273 suggests supplementing a pushover with a linear spectral analysis for structures over 100 feet (30 m) tall. A full nonlinear response history analysis is another, more time-consuming, alternate. Secondly, the pushover characteristics of structures with significant torsional response may be difficult to analyze. General purpose 3-D computer programs that model element nonlinearity for use in pushover analysis are not available at this writing. A three dimensional incremental linear pushover is possible, but, except for simple structures, such an analysis is time-consuming, heavy in book-keeping, and prone to error. However, severe torsion should be reduced or eliminated by retrofit anyway, and the issue often goes away. Caution should be exercised where existing elements with a potential for stiffness degradation are used to resist significant torsional effects.

Given a pushover curve of a structure, it is still necessary to estimate the amplitude of the deformation demand from the design ground motion. This point on the pushover curve will be used to establish the deflected shape of the structure for checking acceptability of local force and inelastic deformations. As discussed above, there are many methods for predicting inelastic displacement proposed. For example, there were six proposed methods for displacement based design presented in one recent workshop alone [Fajfar, 1997]. It is recommended that engineers making evaluation decisions or basing retrofit designs on maximum displacements undertake a thorough study of the basis and limitations of the method used, and if possible, use alternate methods to check results. Due to variations of methods as well as large variation in individual ground motion records, it is also recommended that retrofit schemes for collapse prevention be made as insensitive as possible to exact values of predicted displacements.

Determination of Acceptability

Acceptability is based on whether the results of analysis indicate that the structure has exceeded the desired limiting damage state. Although Vision 2000 [SEAOC, 1995] and FEMA 273 [BSSC, 1997] have set a series of performance levels, the levels between first yield and collapse are difficult to define. As performance based earthquake engineering progresses, it may be necessary to use fragility relationships similar to those used in the HAZUS loss methodology [Kircher, 1996], or other probabilistic characterizations of damage, to measure performance between these limiting states.

When using nonlinear analysis, acceptability is determined by comparing calculated local deformations to deformations predetermined to be appropriate for the target performance level. As previously mentioned, realistic mean-level yield capacities are normally used to estimate the extent of inelastic deformation. Brittle elements are allowed no inelastic deformation and are often further penalized by being assigned lower bound capacities. Mean capacities are often taken as 25% or more greater than nominal (specified by standard or specifications) values. Nominal strengths themselves represent lower bounds, but are often reduced further by code-mandated strength reduction factors.

Assignment of inelastic limits is often done by simplifying actual hysteretic behavior into several standard types, as shown in Figure 6. Limits on component deformation can be set from these simplified curves to prevent yield, to prevent degradation, to prevent complete failure, or at some other point chosen on the curve (e.g. life safety). Local acceptable inelastic deformations can also be determined using strain limits and basic principles, finite element modeling of sub-assemblies, or project specific testing. In any case, as discussed below, the relationship between these component acceptability limits and global performance levels is poorly defined.

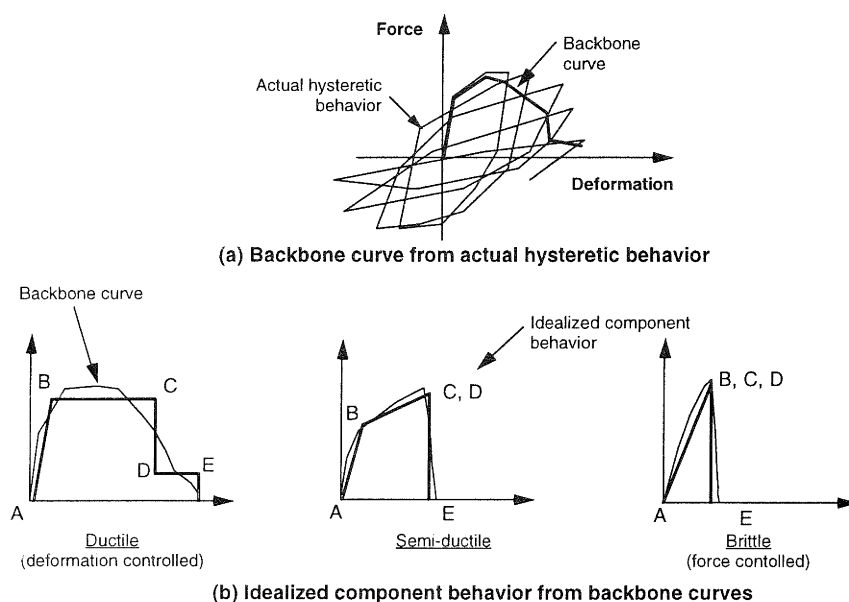


Figure 6: Simplified material behavior

Acceptability of traditional elastic structural design is measured on a local, rather than global basis—that is, each action (moment, shear, etc.) on each component (beam, wall, etc.) is *designed* to not cause “overstress.” This concept does not apply well to categorizing the damage state of an entire structure, as is necessary for evaluation and retrofit using performance based earthquake engineering. Performance levels useful to building owners measure global parameters such as the ability to occupy and use the building (*immediate occupancy*), protection of life safety (*life safety*), the cost of repairs (*damage control*), or preventing collapse (*collapse prevention*). Damage to a limited number of components, even beyond a preset failure state, does not necessarily mean failure of the entire building to reach the desired performance level. For example, in Figure 7, it is clear that a global $P-\Delta$ or a story $P-\Delta$ mechanism can define collapse. But the other “failure” modes shown may or may not represent or precipitate collapse. On the other hand, complete failure of individual elements as pictured is clearly undesirable and is to be avoided. Similar, but more complex issues arise when defining performance levels with limit states less than collapse.

The reliability framework developed for the SAC steel project has been proposed to be applied independently to global collapse mechanisms and local collapse mechanisms that could occur in steel moment frames. These failure modes can be weighted in importance by adjusting the confidence level of *no-failure*. The methodology offers promise for combining local and global failure in a rational manner.

Currently, however, combining local and global behavior to define performance levels is overly complex for practical evaluation or retrofit of buildings and acceptance criteria is most often applied to individual elements. The concept of *secondary elements* was introduced into FEMA 273 to give engineers the flexibility to allow some components to “fail” the preset criteria. Elements can be called secondary and be considered acceptable if they are relatively insignificant in the determination of overall strength and stiffness of the structure and if they

can continue to carry gravity load to the deformation demand of the ground motion. The concept is similar to that used in new buildings for *elements not part of the lateral system*. The range of deformation for which the classification of secondary element applies is shown in Figure 8. The lack of available data concerning the vertical load capacity of various elements in this range prevents full use of the concept. More data on element behavior is needed to judge acceptability of both primary and secondary elements near collapse

Classify per Evaluation Procedure or Select/Refine Retrofit Procedures

As shown in Figure 1, there are several potential paths that could be taken following the check for acceptability depending on whether the process has been for evaluation of seismic performance under existing conditions or whether retrofit measures have been checked for acceptability.

In evaluation, if the acceptability check is positive, the structure is adequate and the process is completed. If the structure does not meet the acceptability requirements, it may simply be classified as "inadequate," or may be placed in a lower performance category, depending on the purpose of the evaluation procedure. Preliminary retrofit procedures necessary to meet the desired performance objective also may need to be defined, and after a conceptual design process, the analysis procedure would be repeated.

If the analysis procedure was done to confirm acceptability of a retrofit design, the design will be completed or refined, depending on the results of the acceptability check, as indicated in Figure 1.

CONCEPTUAL DESIGN OF RETROFIT MEASURES

Retrofit Techniques

There are many specific methods of intervention available to retrofit designers, both to improve the behavior of individual building components and to improve overall behavior [BSSC, 1992], [Sugano, 1996]. A complete listing of all techniques becomes a treatise on structural engineering because all materials and systems used in new construction can also be used in retrofit. The selection of the specific type of element or prefabricated hardware is dependent on local cost, availability, and suitability for the structure in question. It is thus also an extensive task to develop guidelines for such selection. Conceptual design techniques, on the other hand, can be systematically categorized and design strategies formulated.

Non-technical Issues

The solution chosen for retrofit is almost always dictated by building-user oriented issues rather than by merely satisfying technical demands. There are five basic issues that are of concern to building owners or users:

1. Seismic Performance
2. Construction Cost
3. Disruption to the building users during construction (often translating to a cost)
4. Long term affect on building space planning.
5. Aesthetics, including consideration of historic preservation.

All of these characteristics are always considered, but an importance will eventually be put on each of them, either consciously or subconsciously, and a combination of weighting factors will determine the scheme chosen.

Seismic performance

Prior to the emphasis on performance based design, perceived qualitative differences between the probable performance of difference schemes would be used to assist in choosing a scheme. Now, specific performance objectives are often set prior to beginning development of schemes. Objectives that require a limited amount of damage or "continued occupancy" will severely limit the retrofit methods that can be used and may control the other four issues.

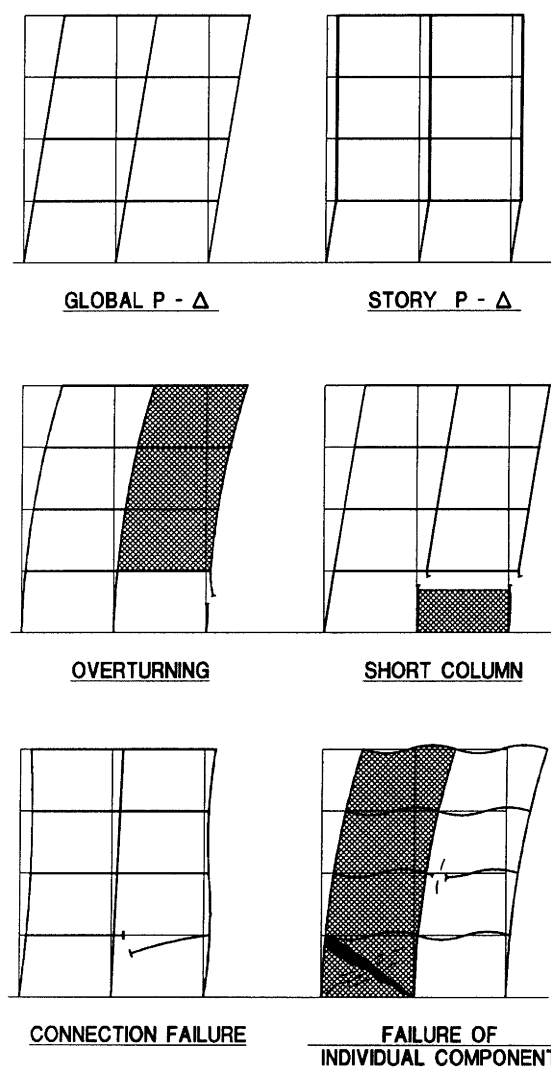


Figure 7: Various failure modes

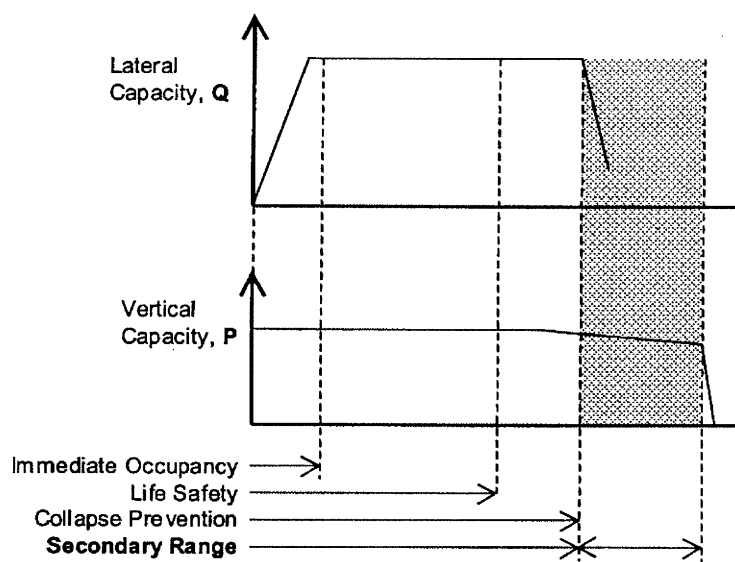


Figure 8: Range of behavior for secondary elements

Construction cost

Construction cost is always important and is balanced against one or more other considerations deemed significant. However, sometimes other economic considerations, such as the cost of disruption to building users, or the value of contents to be seismically protected, can be orders-of-magnitude larger than construction costs, thus lessening its importance.

Disruption to the building users during construction

Often retrofits are done at the time of major building remodels and this issue is minimized. However, in cases where the building is partially or completely occupied, this parameter commonly becomes dominant and controls the design.

Long-term effect on building space planning

This characteristic is often judged less important than the other four and is therefore sacrificed to satisfy other goals. Often the planning flexibility is only subtly changed. However, it can be significant in building occupancies that need open spaces such as retail spaces and parking garages.

Aesthetics

In historic buildings, considerations of preservation of historic fabric usually control the design. In many cases, even performance objectives are controlled by limitations imposed by preservation. In non-historic buildings, aesthetics is commonly stated as a criterion, but, in the end, is often sacrificed, particularly in favor of minimizing cost and disruption to tenants.

Figure 9 describes the evolution of a retrofit scheme based on several changes in the owner's weighting of these five characteristics.

Strategies

The primary focus of determining a viable retrofit scheme is on vertically oriented systems because of their significance in providing either lateral stability or gravity load resistance. Deficiencies in vertical elements are caused by excessive interstory deformations that either creates unacceptable force or deformation demands.

Given an initial understanding of the importance of the non-technical issues described in the previous section, alternate retrofit schemes can be developed. Retrofit actions can be classified into three types:

- A. Connectivity, consisting of assuring that individual elements do not become detached and fall, assuring a complete load path, and assuring that the modeled force distributions can occur.
- B. Modification of global behavior, usually decreasing deformations
- C. Modification of local behavior, usually increasing deformation capacity

These three types of actions balance one another in that employing more of one will mean less of another is needed. It is obvious that providing added global stiffness will require less local deformation capacity, but it is often less obvious that careful placement of new lateral elements may minimize a connectivity issue such as a diaphragm deficiency.

Design of a retrofit scheme is seldom a one step process. As shown in Figure 1, several trials are usually necessary to define a satisfactory design, both to appropriately balance the three types of retrofit actions defined above but also to determine the exact extent of each measure (number and length of walls, size of braces, etc.).

A knowledge of the existing building is essential, including a listing of seismic deficiencies based on building specific calculations or known performance of similar buildings, such as can be obtained from FEMA 178 [BSSC, 1992b] or FEMA 310 [ASCE, 1999]. Understanding of local element behavior as discussed above under *Determination of Local Response Characteristics* is also critical.

A pushover analysis has become a popular technique to aid in understanding the deficiencies of a building and for determining the most effective retrofit measures. An excellent discussion of these techniques as applied to concrete buildings is contained in Chapter 6, "Retrofit Strategies", of the Applied Technology Council's *Seismic Evaluation and Retrofit of Concrete Buildings* [ATC, 1996]. Some engineers prefer to create an initial pushover of the unretrofitted building in which they note local "failures" but ignore them in the continuing analysis in order to understand the potential benefit of local element strengthening (usually enhancement of deformation capacity). The extent and potential cost of local enhancement can then be judgmentally weighed against the benefits.

Capacity design techniques for the entire structure or for certain systems are also being employed in retrofit design. Capacity design concepts are built into many codes for new buildings; examples include design of connections for the capacity of steel braces, design of shear in ductile concrete to develop the bending capacity of the member, and requirements for "strong column-weak beam" in moment frames. Extended to an entire structure, capacity design would call for a clearly defined yielding element to act a fuse, with all other elements designed to stay elastic for the fuse loading. Such a design would suppress undesirable failure mechanisms regardless of the intensity of ground shaking [King, 1998]. It is usually difficult to provide a *global* capacity design for a retrofit due to the constraints of the existing conditions. An example of a *partial* capacity design, used to control the location of damage, is described in Figure 10.

Although experience and judgment will quickly lead to the development of alternate schemes, a systematic approach is also possible. Solutions to each deficiency can be developed using types A, B and C above, each of which can be evaluated for suitability by considering 1) overall feasibility and reasonableness, 2) satisfaction of demands of the five basic nontechnical issues discussed above, and 3) compatibility with retrofit actions needed for other deficiencies. This process should efficiently define a limited number of key choices that will lead to the most appropriate scheme. However, sometimes this systematic study will lead to a need for development and further study of several schemes. Often the lack of significance of the differences in options, or external time restraints, will lead to a choice at this point based on engineering judgement.

Retrofit Actions

Connectivity

Connectivity deficiencies are within the load path: wall out-of-plane connection to diaphragms; connection of diaphragm to vertical elements; connection of vertical elements to foundation; connection of foundation to soil. A complete load path of some minimum strength is always required, so connectivity deficiencies are usually a matter of degree. A building with a complete, but relatively weak or brittle, load path might be a candidate for seismic isolation design to keep the superstructure in the elastic range.

Yielding in connections within the basic load path can create profound complications on the overall building model. An early decision has to be made concerning modeling such behavior or preventing it by reducing demand or strengthening of the local connections. Demand can be reduced by adding vertical load resisting elements (reducing individual collector or foundation loads) or by seismic isolation.

The only location in the load path at which yielding is generally allowed is the foundation/structure interface. Allowing no movement at this location is expensive and often counterproductive, as fixed foundations transfer larger seismic demands to the superstructure. Most recently developed retrofit guidelines are attempting to provide simplified guidance to the designer on how to deal with this difficult issue [BSSC, 1997], [ATC, 1996].

Modification of Global Behavior

Modification to global behavior normally focuses on deformation. Overall seismic deformation demand can be reduced by adding stiffness in the form of shear walls or braced frames. A significant period shift is normally required to protect deformation sensitive elements in this way. New elements may be added, or created from a composite of new and old components. Examples of such composites include filling in openings in infill frames and using existing columns for chord members for "new" shear walls or braced frames. If existing lateral force resisting elements are to be used in conjunction with new ones to provide the required stiffness, the potential for degradation due to poor detailing in the existing structure must be considered. If loss of lateral stiffness of the existing elements will reduce the overall strength to levels that could cause P-delta instability, the existing elements should be discounted and additional new elements employed.

The deformation-demand of the ground motions can also be controlled by redistribution over the height of a structure. It is sometimes relatively easy to estimate the global stiffness that is obtainable with acceptable costs and disruption. If one or more interstory drifts are still unacceptable, it may be possible to redistribute stiffness vertically to obtain a more even distribution of drift. A soft or weak story is an extreme example of such a problem. Such stories are usually eliminated by adding strength and stiffness in such a way as to more closely balance the stiffness of each level and thus evenly spread the deformation demand over the height of the structure.

Seismic isolation is the supreme example of both the concepts of redistribution of deformation and capacity design. Essentially all deformation is shifted to bearings placed at the isolation level that are specifically designed for such response. The bearings limit the response of the superstructure, which can be designed to remain nearly elastic for this maximum load—a basic tenant of capacity design. The feasibility of providing isolation bearings that limit superstructure accelerations to low levels (0.10g-0.30g) not only facilitates design of superstructures to remain nearly elastic, but also provide a controlled environment for design of nonstructural systems and contents.

Global deformations can also be controlled by the addition of passive energy dissipation devices to the structure. Such devices are now available whose response characteristics are dependent on either displacement or velocity. Although effective at controlling deformations, large local forces may be generated that must be transferred from the device to structure and foundations. *FEMA 273*, [BSSC, 1997] includes a design method for utilizing these devices for retrofit. For a more theoretical treatment, see [Constantinou, 1999].

Modification of Local Behavior

Rather than providing retrofit actions that affect the entire structure, deficiencies also can be eliminated at the local, component level. This can be done by enhancing the existing shear or moment strength of an element, or simply by altering the element in a way that allows additional deformation without compromising vertical load carrying capacity.

As previously discussed, certain yielding sequences are almost always benign: beams yielding before columns, bracing members yielding before connections, bending yielding before shear failure in columns and walls. These relationships can be obtained by local retrofit in a variety of ways. Columns in frames and connections in braces can be strengthened, and the shear capacity of columns and walls can be enhanced to be stronger than the shear that can be delivered.

Concrete columns can be wrapped with steel, concrete, or other materials to provide confinement and shear strength. Concrete and masonry walls can be layered with reinforced concrete, plate steel, and other materials. Composites of glass or carbon fibers and epoxy is becoming popular to enhance shear strength and confinement in columns and to provide shear-only strengthening to walls. Such material must be used with caution however, because moment capacity can be inadvertently added, the material can be incorrectly designed to not be compatible with the existing element, or the system can be applied incorrectly in the field.

Similarly, bending (rocking) strength of unreinforced masonry walls can be increased by insertion of reinforcing or post-tensioning steel in field drilled cores. In this case, care should be taken to not change the behavior of the wall to one of initial shear or toe crushing failure, rather than rocking.

Another method of providing additional drift capacity is to provide a supplementary gravity support system for elements that might be unreliable at expected deformation levels. Supplementary support is a requirement in California standards for retrofit of unreinforced masonry buildings for concentrated wall-supported loads. An example where such a system was used in a concrete building is shown in Figure 11.

Lastly, deformation capacity can be enhanced locally by uncoupling brittle elements from the deforming structure, or by removing them completely. Examples of this procedure include placement of vertical saw cuts in unreinforced masonry walls to change their behavior from shear failure to rocking, and creating slots between spandrel beams and columns to prevent premature shear failure in the “short column”. An example of another type of reduction in stiffness to eliminate a seismic deficiency is described in Figure 12.

NEED FOR SIMPLIFIED AND/OR PRESCRIPTIVE METHODS

This paper describes complex nonlinear analytical and design method in use today by engineers specializing in seismic evaluation and retrofit of existing buildings. These methods are used on buildings where interaction between new and old elements is important or when performance is heavily dependent on existing, "nonconforming" elements. There is variability in the application of the methods, leading in some cases to lack of agreement among engineers and loss of confidence by owners.

Many buildings are still evaluated and retrofit using relatively simple force-based methods that have little or no explicit consideration of realistic displacement demands from ground motion, displacement compatibility among dissimilar elements, nonlinear response, or the subtleties of local versus global failure modes. There is little question that simple and efficient methods of evaluation and retrofit are needed so that the evaluation and retrofit process is not discouraged. There are many such procedures in use or proposed. A method is needed to benchmark these procedures and determine appropriate limits on their use so that results will be reliable, consistent, and credible.

CONCLUSIONS

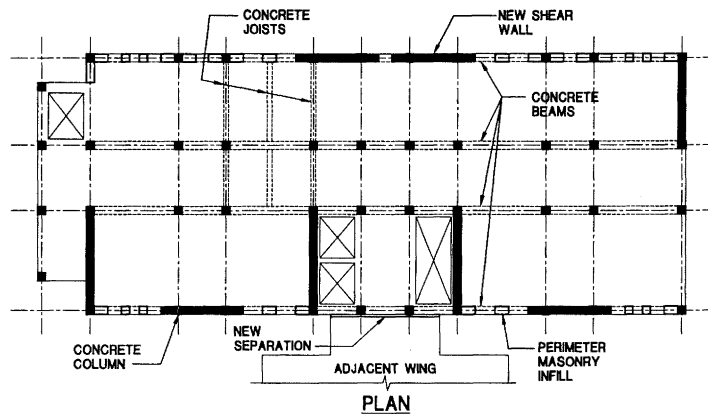
Risk assessment and retrofit include aspects of almost all of earthquake engineering, requiring close cooperation between disciplines, particularly in the area of acceptable risk. The growing understanding of the risk from existing buildings by owners and government is creating a demand for more efficient and reliable analytical methods, as well as innovative techniques for retrofit. A benchmark analytical tool is needed to test and set limitations on simplified nonlinear procedures. Reliable and consistent simplified linear or force based procedures are also needed for small, regular buildings.

When considering the entire spectrum of components and elements used in seismically deficient existing buildings, little cyclic test data is available from which analytical acceptability criteria can be set. A systematic program is needed to fill in the most significant gaps. For consideration of near-collapse conditions, much information is needed to determine gravity-load carrying ability of elements under severe deformations. The relationship between local and global failures and collapse needs further definition in order to not penalize redundant structures that may experience one or more local component failures.

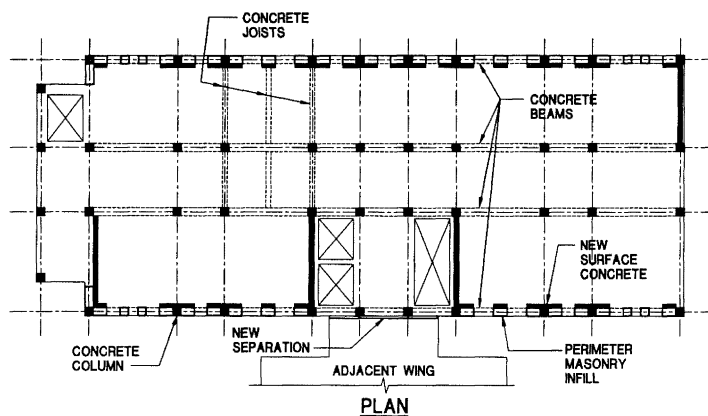
Despite the rapid advances in published guidelines and standards for evaluation and retrofit, engineers must continue to temper their decisions with experience and judgement, primarily based on field observations of real structures under loadings from real ground motions.

ACKNOWLEDGEMENTS

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SCHEME 1 - CONVENTIONAL SHEAR WALLS

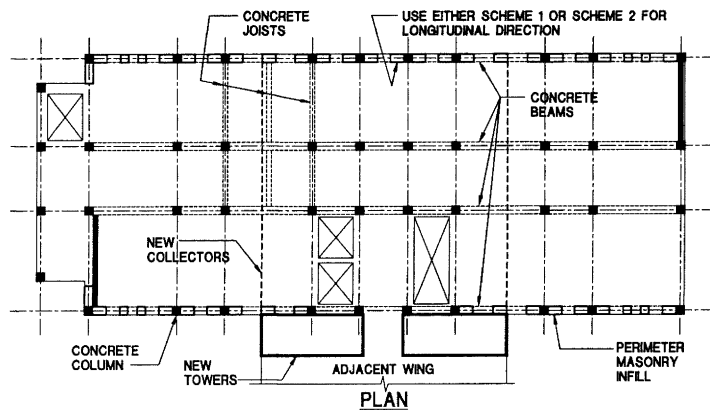


SCHEME 2 - PIER & SPANDREL CONCRETE WALL

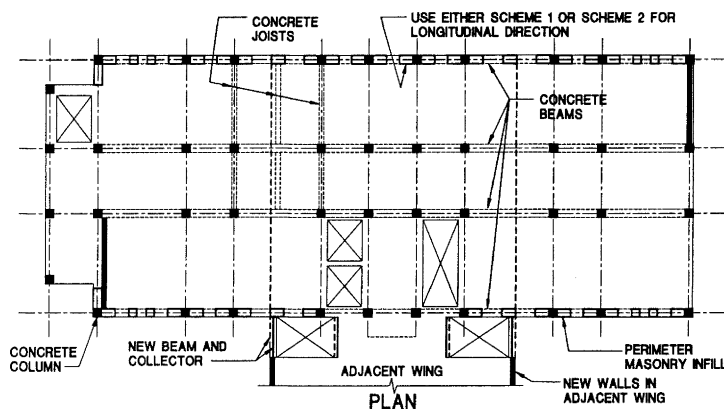
This is a seven story concrete building built in the early 1920s. It consists of two wings in a tee shape. The plans show one of the two wings, the second wing poor connected at the location shown. The building is a concrete frame with brick infill exterior walls and lateral forces were not apparently considered in the original design. Although not officially judged historic, the exterior was articulated and considered pleasing and a good representative of its construction era. As can be seen in the plan, the building has no lateral strength in the transverse direction other than the poor connection to the second wing, and was evaluated to present a high risk to occupants.

It was judged early that the vertical load carrying elements had little drift capacity and that stiffening with shear walls was the only feasible solution. The first two schemes shown are straightforward application of shear walls. The first concentrates the work to minimize disruption, but closes windows and creates large overturning moments at the base. The second distributes the longitudinal elements and preserves windows by using a pier-spandrel shear wall. Both schemes separated the wings into two buildings to allow future demolition of either to facilitate phasing for a new replacement building sometime in the future. The cost and disruption was judged high and the work would have to be phased upwards, evacuating three floors at a time to avoid the noise and disruption. (Continued Figure 9b)

Figure 9a: Example of effect of non-technical issues on retrofit schemes



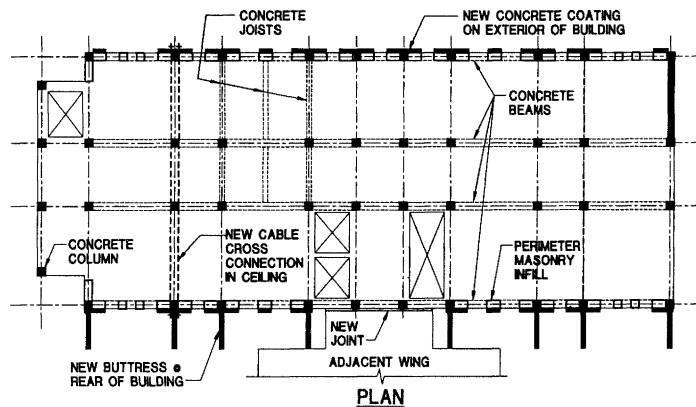
SCHEME 3 - STRENGTHEN CONNECTION TO ADJACENT WING ADD TOWERS



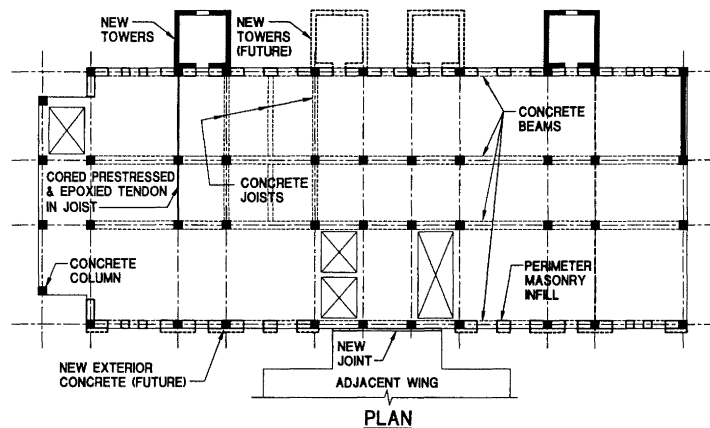
SCHEME 4 - STRENGTHEN CONNECTION TO ADJACENT WING WITH CONVENTIONAL WALLS

Schemes 1 and 2 required a complete lateral system in both buildings because of the separation installed at the wing intersection, which also caused difficult exiting issues. Schemes 3 and 4 were therefore developed, providing a strong inter-tie between wings and taking advantage of several new lateral elements to provide support to both wings. Scheme 3 featured new concrete towers as shown. Although the outside location was considered advantageous from a disruption standpoint, the towers closed windows and caused disruption to mechanical services. Scheme 4 was similar to 3, but eliminated the towers. Schemes 3 and 4 had less construction cost than 1 and 2, but disruption, in terms of phasing, caused essentially the same total downtime. (Continued Figure 9c)

Figure 9b: Example of effect of non-technical issues on retrofit schemes



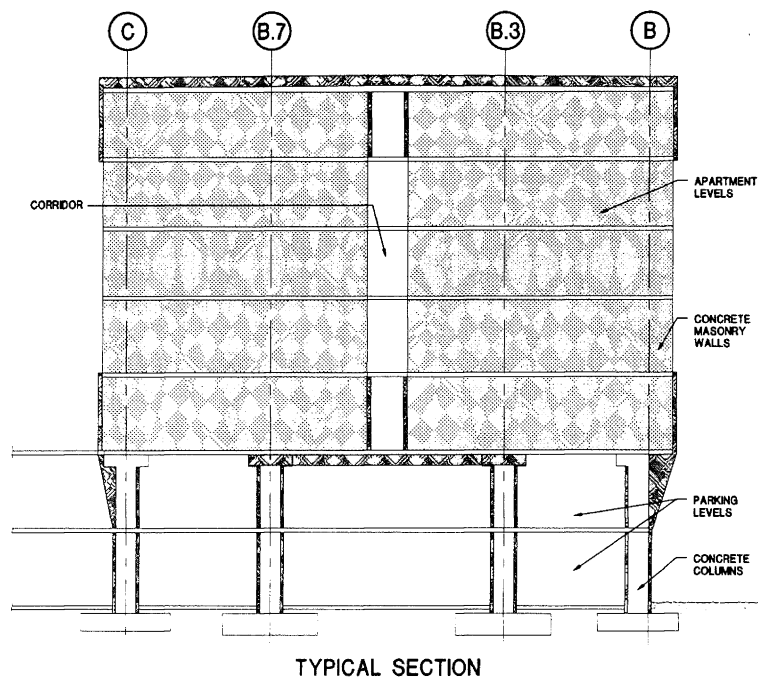
SCHEME 5 - EXTERIOR SCHEME AT 'REAR' OF BUILDING



SCHEME 6 - CHOSEN SCHEME

When the owner completed a study of the availability and cost of surge space in the area to facilitate the phasing required for schemes 1–4, it was discovered that the cost of moving and rental space was larger by far than the construction costs thus far budgeted. Occupant disruption thus became the primary control parameter for development of retrofit schemes and aesthetics, measured by preservation of the exterior appearance, was significantly reduced as a consideration. Scheme 5 was developed with buttresses on the off-street side of the building and longitudinal walls applied from the outside. Collectors to the buttresses were to be post-tensioned cables installed through conduit placed at night in the ceiling spaces. Access to the rear of the building was difficult, so aesthetic considerations were further relaxed to allow buttresses—that became towers—on the front side. Collectors to the towers were post-tensioned rods installed in cores drilled approximately 16 feet (5 m) into the building in the center of 12 inch by 16-inch (300x400 mm) joists. Both schemes 5 and 6 installed a seismic separation at the intersection of wings to facilitate partial replacement. At the request of the contractor, another scheme, 6a, was developed that replaced the concrete shear walls with steel braced frames, but it proved no more economical. Scheme 6 was selected for construction, although since replacement at that time appeared to be in the long range planning stages, the owner chose to construct only part of scheme 6, aimed at eliminating the obvious collapse mechanism in the transverse direction. As shown, only two towers were constructed and the only longitudinal strengthening provided was the weak-way strength of the towers.

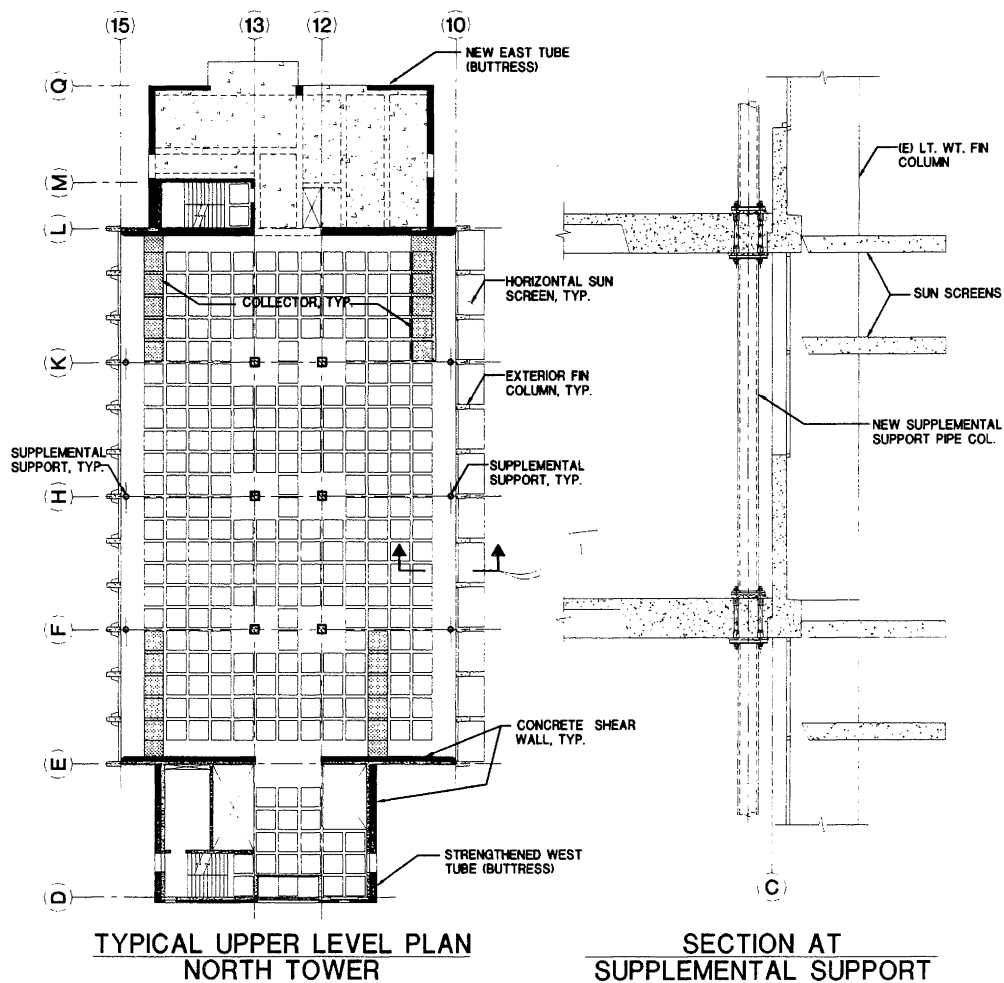
Figure 9c: Example of effect of non-technical issues on retrofit schemes



This example is a large and complex structure containing 721 apartments and parking for 760 cars and over 1,000,000 square feet (93,000 square meters). It is typically configured as four levels of apartments over three stories of parking, but it steps in two directions down a sloping site, creating a variety of cross sections. The apartment levels are constructed of concrete slabs supported by reinforced concrete masonry bearing walls. The bearing walls stop at the concrete parking levels, creating a typical condition of discontinuous shear walls, as shown in the section above. Retrofit measures are shown in the section shaded, so the original conditions are also evident. Although the lateral capacity of the shear wall levels was determined to be over $0.4 W$ (W = dead load), several undesirable failure modes due to the discontinuity reduces the lateral capacity of the section significantly below that. For lateral loads pushing towards the right of the section above, the footing for the column on line C uplifts first at approximately $0.2 W$, followed by separation (rocking) of the right wall from its concrete base due to a poor tension connection from the center corridor to line B3. At the point of rocking of the both walls, the compression capacity of the columns and footings on lines B and B7 are adequate. However, before the expected deformation demand of the design earthquake is met, undesirable damage to the coupling slabs and potential failure of the column at line B7 due primarily to bending occurs.

Capacity design concepts were used in the retrofit. The high strength of the wall structure was utilized and the undesirable failure modes at the level of discontinuity were prevented with the addition of new, ductile coupling beams at each pair of walls. A coupling beam was added below the apartment levels and on the roof to minimize disruption to the tenants. Columns below the discontinuity were also strengthened and new connections between the coupling beams and the wall system added. However, rocking of the whole system at the foundation level through uplift was allowed. The strength of the ductile coupling beams were set as a "fuse," slightly below the strength of the upper wall system, and elements with undesirable failure modes, such as the ties to the upper walls and the supporting columns in compression, were strengthened to avoid yielding.

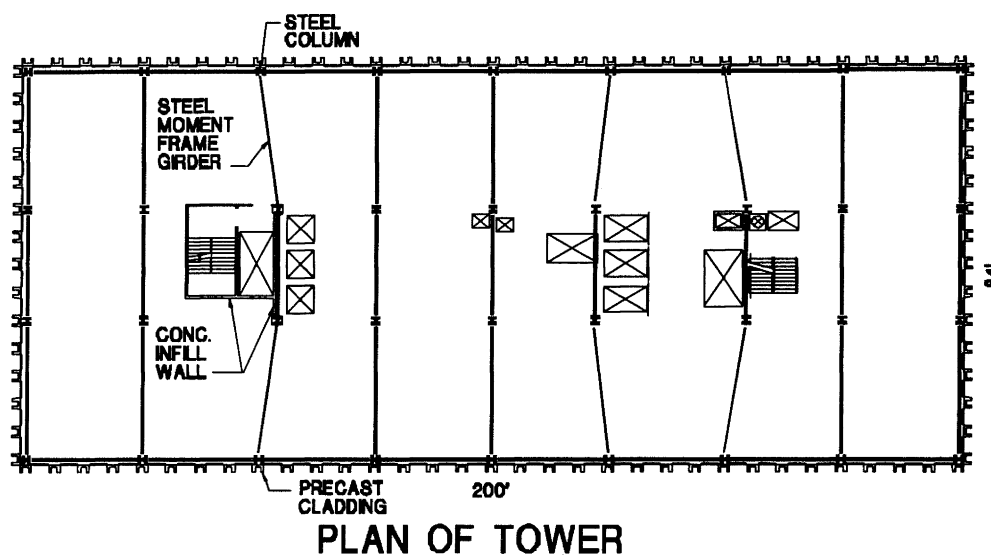
Figure 10: Example of capacity design applied to retrofit



The plan shown above is of a nine story wing of a larger complex. The building was built in the 1960's and is a combination of cast-in-place and precast concrete. A major architectural feature of the building is the use of closely spaced precast rectangular columns at the perimeter. These columns are constructed of lightweight concrete and were not reinforced to act as part of the lateral force resisting system. Estimates of properties of these columns using current guidelines would indicate a very low drift capacity before vertical load carrying ability becomes unreliable. The large tributary area affecting these columns and the lack of vertical load carrying redundancy caused the drift capacity of these columns to control the design of the retrofit.

The large number of columns and the complication of horizontal sun-shade-fins at mid-story (see the section above) dictated that local treatment of the columns to enhance deformability was uneconomical. However, adding sufficient stiffness to the building to control interstory drift to safe levels was also economically unrealistic. In this situation, drift limits that controlled the overall design were increased by adding a supplemental gravity support system at the exterior faces of the building (as shown in the section above). These new elements provided a reliable perimeter gravity support system at drifts that were economically achievable with new shear towers at each end of the building.

Figure 11: Example of enhancing drift capacity with new vertical support elements



This is a nine story building, constructed on a sloping site in 1964. Total building area is approximately 175,000 square feet (16,275 square meter). Its lateral load resisting system consists of five stories of steel moment resisting space frame above four levels of perimeter reinforced concrete shear walls. In the moment frame portion of the building, around the stair and elevator core at the west end of the building, there are a series of 6" thick concrete infill walls.

The building was evaluated using a FEMA 178 (ref) methodology (updated to consider post-Northridge steel frame behavior), and several deficiencies were found:

The floor to floor drifts in the north south direction at the east end of the building exceeded maximum allowable drifts at several upper levels of the structure, and certain of the steel moment-resisting frame connections in the transverse (north south) direction had overstressed panel zones.

- Several of the infill concrete walls between the steel columns and beams at the west end stair and elevator core were discontinuous below the 1st Level; columns supporting these discontinuous walls had insufficient strength to resist expected overturning forces from the walls.
- Several of these infill concrete walls were also overstressed in shear on various upper floor levels.

Strategic sawcuts were in placed the infill concrete walls to decouple them from the primary lateral system. This apparent "reduction" in the lateral force resisting system eliminated the seismic deficiencies in the building:

- Since these walls are no longer shear walls, the deficiency linked to their discontinuity is no longer applicable.
- Since they take no lateral load, they are no longer overstressed in shear.
- Since the infill walls occurred at one end of the building only, they caused a torsional irregularity in the building. When the walls are removed from the lateral system, the steel moment frame becomes a torsionally regular structure. Drifts are within allowable limits, and the steel frame panel zones are no longer overstressed.

Figure 12: Example of elimination of lateral elements to improve the seismic system

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