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USING HISTORICAL DATA TO AID IN THE EVALUATION OF STRUCTURAL PERFORMANCE OF BUILDINGS

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ABSTRACT

As earthquake engineers develop new technology and complex analytical techniques to evaluate the resistance of buildings to earthquake ground motion, available historical information is often neglected, underused, or ignored. If assessment and strengthening of existing buildings are done without full use and consideration of available data from past earthquakes, we have to ask ourselves, "What are we really learning from past earthquakes?" Information from past earthquakes is critical to understanding how structures will perform in future earthquakes.

Use of historical data involves not only the use of past records and published reports, but also detailed probing. Examples include the deconstruction of building response recordings to reveal modal responses and changes in periods and amplitudes. Other examples include searching historical libraries for unpublished eye-witness accounts of post earthquake observations. This paper presents some useful methods employed by the authors and cites instances where historical data can enhance the credibility and reliability of estimates of the performance of buildings responding to damaging earthquakes.

Introduction

Today's building codes and standards for new construction are substantially more demanding and complex than those of previous generations --- which fact is not always to the benefit of all buildings. In addition to basic controversy regarding the need for some of the overly restrictive provisions of current codes, unintended consequences of these provisions can have a particularly detrimental effect on the inventory of existing buildings, especially historical buildings. As earthquake resistant design provisions are made more restrictive for new construction, there is an economic premium on costs that can possibly be justified because of the relatively low effect on the overall costs on the building. However, when these same provisions are applied on projects that involve the strengthening of existing structures the social and economic effect can be devastating --- in many cases resulting in the demolition of prized historic buildings. Over the decades there have been attempts to address the counterbalancing issues of reducing potential safety risks from earthquake hazards versus preservation of historic buildings. In most jurisdictions, buildings are presumed to be safe to occupy if they were in compliance with building code requirements in force at the time they were originally constructed and are continuously occupied.

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However, when these buildings undergo alterations, additions or repairs, triggers are sometimes applied that require seismic upgrades to buildings if the new work is substantial. Engineers and building officials have been struggling for decades to establish reasonable limitations on when upgrades are required and what level of upgrade is needed. Recently there has been a movement to establish building codes exclusively for existing buildings, but this is causing even more controversy. The authors have discussed the details of this controversy elsewhere (Paret and Searer 2005; Searer and Paret 2006). In the State of California, there is a state historical code that is intended to provide relief for the seismic upgrading of historical buildings from the prescriptive measures in codes governing new construction. Primarily, the state historical building code attempts to provide for equivalent safety, while allowing consideration of the attributes of archaic materials, which would otherwise not be acceptable to traditional building codes, but attacks on this code intended to undermine this relief are on-going.

When existing buildings are analyzed for their ability to resist seismic forces, engineers generally rely on current building codes or published guidelines. The lateral force provisions are generally taken literally and assumed to be rational and reasonable. However, these provisions are invariably developed by committees that are driven to err on the conservative side of safety. Sometimes these provisions, although made with good intensions, lead to unintended consequences that may be counterproductive to a good rational design. Because the bases of many provisions are not well documented, their justification and reason for being is not completely understood. Thus, practicing engineers are often hesitant to employ intuition and common sense by straying from a strict interpretation of the code provisions. This situation has lead some engineers to suggest a performance based approach to seismic design that takes into account how buildings actually perform in the real world. This reality check approach is especially important for historical buildings and is the topic of this paper.

Sample Case Histories

Puget Sound Naval Shipyard

In the early 1970s, the principal author was involved in a pilot project to evaluate the seismic vulnerability of the Puget Sound Naval Shipyard (PSNSY) facility in the state of Washington (Freeman *et al.* 1975). The project included 80 buildings -- of which a great majority was built between the beginning of the 20th century and the early 1940s -- and a wide variety of building types including brick, concrete, steel, wood, moment frames, braced frames, shear walls, and combinations thereof. Seismic design was most likely not considered for many of the buildings and some cases there did not appear a definable lateral force resisting system. Many buildings had dual or multiple lateral force resisting systems, some acting in parallel and others acting sequentially. Capacities of the structures to resist postulated earthquake ground motions were determined on the basis of on-site observations, review of available drawings, calculation of approximate force-displacement relationships, and engineering judgment. The postulated earthquake ground motions were based on site specific probabilistic response spectra. It had been noted that the site had been subjected to ground motion from two moderate earthquakes from sources near Olympia, Washington in 1949 and 1965. Peak ground accelerations (PGA) at PSNSY were estimated to have been in the neighborhood of 0.07g. The goal of the study was to estimate the vulnerability of these buildings to damage by future earthquakes. At the time of the study, the effective PGA for the maximum postulated earthquake for the site was 0.2g and the 10% probability of exceedance in 25 years was 0.075g. Follow-up studies included estimates of cost-benefits that might occur if buildings were seismically upgraded.

If traditional code approach evaluation techniques had been applied to these buildings, the conclusion would have been that few, if any, could survive even a moderate earthquake without severe damage. However, there were no signs of damage caused by the Olympia earthquakes of 1949 and 1965, and it was determined that a different approach was needed. An intuitive approach that seemed rational at the time was to graphically compare the earthquake demand to the first mode pushover capacity. By converting a capacity pushover curve from force-displacement to spectral acceleration-spectral displacement, the capacity curve could be plotted on the same graph as the response spectrum. The process of plotting the capacity curve with varying damped response spectra later became known as the

Capacity Spectrum Method (Freeman 1998 and Mahaney *et al.* 1993)

Contrary to a number of recent publications that describe the utility of pushover analysis negatively, representation of the capacity of a structure to resist lateral forces by a pushover curve can predict with reasonable accuracy and significantly aid understanding how a structure will perform under limit state conditions. First, a realistic linear estimate of first yielding is made. Then elements are allowed to yield until a reasonably acceptable maximum nonlinear displacement is estimated. The response spectrum at five percent damping generally represents linear-elastic response of the structure. Increased damping can be used to approximate an inelastic response spectrum that accounts for reduction in demand due to energy absorption from inelastic hysteretic behavior (Freeman 2006). An example shown in Figure 1 indicates that the response of the structure to an MCE design earthquake will be at a ductility of 2.5, which equates to a demand at 20 percent effective damping.

In the late 1970s a very similar study using the same procedure was done for the Mare Island Naval Shipyard in the San Francisco Bay Area of California. Some of the buildings were later damaged by the 1989 Loma Prieta earthquake. In a post-earthquake investigation, it was found that the damage compared closely to that predicted by the earlier pushover-based vulnerability study (Scott 2006).

Van Nuys Holiday Inn Reactions to 1971 and 1994 Earthquakes

San Fernando, California Earthquake of February 9, 1971

The Van Nuys Holiday Inn was one of the many instrumented buildings for which seismic response motion was recorded during the San Fernando earthquake of February 9, 1971. The seven-story reinforced concrete frame structure was designed circa 1965, prior to the defining of ductile reinforced concrete by the building code. (Freeman 1972 and 1978).

A fairly extensive evaluation of the building was made after the 1971 event that included analysis of the accelerograph recordings, analysis of the structure, and analysis of structural response. Conclusions from investigation included the following:

- The elastic limits of structural system were exceeded during the earthquake; the system suffered a permanent loss of stiffness.
- The yielding of beams prevented possibly serious column failures. The structure responded in a generally ductile manner, although not specifically designed as such.
- On the basis of some analysis results, one might have expected more damage than was actually observed, for example, peak response accelerations and forces were 4 to 5 times design level and peak displacements were from 8 to 15 times design. Although there was a significant amount of nonstructural damage, little structural damage was observed. An interesting observation was made several months after the earthquake when exposure of the perimeter frames to rain provided clear indications at the ends of some of the beams of some fine cracks that had not been observed previously under dry conditions. This seemed to validate the conclusions of the analysis by demonstrating that flexural yielding likely had occurred.
- Responses of the structure to future earthquakes will vary considerably from the response to the February 9, 1971, earthquake. The initial periods will be longer and the amplitudes of response could be either greater or smaller for a similar size earthquake, depending on the characteristics of the ground motion.
- This study also showed that nonstructural elements played a significant role in the performance of the building to earthquake ground motion. The partitions absorbed a significant amount of force and energy before cracking. During the initial portion of the ground motion, they reduced the response period by about 30%.

Northridge, California Earthquake of January 17, 1994

In 1994 the Van Nuys Holiday Inn was again damaged by an earthquake. This time the damage was much more severe than the damage caused in 1971. This building has since been the subject of many studies and has been evaluated by many researchers (Freeman *et al.* 1999 and Gilmartin *et al.* 1998).

In the 1971 strong motion records, the fundamental period in the longitudinal direction lengthened from about 0.7 seconds to 1.5 seconds due to cracking and inelastic action. In 1994, the period shifted from about 1.2 seconds at the beginning of the earthquake to 1.5 seconds --- until there was a significant loss of stiffness and the period suddenly elongated toward the 2.0 second range. This probably signified the severe cracking of columns on the south side at the fourth story that was observed after the earthquake.

In the authors' study, the strong motion records were analyzed by a process of deconstruction that measured cycles of the fundamental mode of vibration at the roof such that a pushover curve could be approximated directly from the response records (Freeman *et al.* 1997a). The results are summarized in Figure 2. The February 1971 response data is consistent with a spectral displacement (S_d) demand of about 4 inches. The balance of points on the pushover represent the first ten seconds of the 1994 earthquake, and reaches a peak S_d of about 6 inches. The upper bound envelope of the plot represents an equivalent pushover curve of the capacity of the building. Shown for comparison, analytically developed pushover data from other researchers were converted into S_a and S_d coordinates. Although there is a large degree of variance, the results do fit into a broad band that can be compared to the measured pushover. Additional analysis indicated that north side infill brick panels at the first story caused torsion in the building that contributed to the damage to the south side columns. Figure 3 shows 10 and 20 percent damped response spectra that represent the 1971 and 1994 earthquakes, superimposed on the pushover to illustrate the CSM procedure.

Hollister Warehouse Study on performance during the 1989 Loma Prieta Earthquake

Typically, seismic designs founded on code-type methods are based on the assumption that buildings have relatively rigid horizontal diaphragms with dynamic response characteristics that are governed by the stiffness of the vertical elements of the lateral force resisting system (e.g., UBC, NEHRP, etc.). Tilt-up construction does not satisfy this basic design assumption because the horizontal diaphragms tend to be flexible relative to the in-plane stiffness of the walls, thus tilt-up buildings respond to earthquakes dynamically in a very different manner from a typical rigid-diaphragm building. By examining the strong motion records for a well instrumented tilt-up, the behavior of the structure can be identified and compared to what the building code requirements would predict for design forces and displacements. A Hollister, California tilt-up, instrumented by the California Strong Motion Instrumentation Program (CSMIP), was studied using the Morgan Hill, the Hollister, and the Loma Prieta events. The studies focused primarily on the response of the transverse direction of the building (Freeman *et al.* 1997b).

The dynamic response of tilt-up buildings to seismic ground motion can be conceptually broken down into the following three components:
(1) The in-plane walls are relatively rigid and generally do not appear to amplify the ground motion; (2) The roof diaphragm has a predominant response based on its own natural in-plane period of vibration and is driven by the response at the top of the in-plane walls; and (3) The out-of-plane component of the wall panels are driven from their base by the ground motion at their out-of-plane period of vibration and at the top by the accelerations of the roof diaphragm. Thus, there are three governing modes of vibration for each axis of the building: that of the horizontal roof diaphragm, the out-of-plane walls, and the in-plane walls.

By the use of various simple techniques, including deconstruction of instrument records, the periods and amplitudes of the primary modes of vibration and their interrelationships for the Hollister tilt-up were identified. Load displacement characteristics of the components, such as a pushover curve for the diaphragm, were approximated for use in estimating linear and nonlinear properties for verification and use in analytical procedures. Results that were presented in this study represented only the transverse direction of one particular building; results will vary for other tilt-ups with different dimensions and aspect ratios. However, such studies help bridge some gaps between research and professional practice.

Engineers and researchers should spend more time understanding how buildings work by combining high-tech methodologies with old-fashioned hands-on procedures

Presidio of San Francisco Barracks Buildings Evaluation and the 1906 Earthquake

In the Presidio of San Francisco, California there are a series of unreinforced brick bearing wall buildings that date back to before the turn of the 20^{th} century. These buildings -- which experienced the 1906 San Francisco earthquake as well as the 1989 Loma Prieta earthquake -- had been used as barracks by the Army, but since the Presidio became a national park, are undergoing adaptive reuse. In the 1990s, the seismic vulnerability of these existing buildings was evaluated and methods for reducing risks to life safety were identified. A requirement for consideration in the development of any seismic upgrade method was to preserve the historical integrity of the buildings. During the studies, efforts were made to identify references that described the performance of the buildings during the 1906 earthquake. But it appears that after a message was sent to Washington, D. C. reporting that there was some damage to buildings at the Presidio from the 1906 earthquake, the reply from Washington was that "there was no damage". Rumor has it that there was concern at that time that the report of damage might indicate a state of weakness and the Spanish might attack the US to regain the Presidio. Thus, there were no historical eyewitness records preserved of how these buildings performed in 1906. Visual observations during site visits suggested that there may have been some brick replacement and repair in the distant past, but the timing of the work is uncertain and overall it appeared that the buildings survived the 1906 fairly well. The 1989 earthquake was relatively moderate and no damage had been observed. Recorded ground motion during Loma Prieta was available for one site within the Presidio, which along with acknowledgment of the 1906 earthquake, was used as a benchmark for evaluating the results of the analytical portion of the investigation. Because the available methods of analyzing these types of historic buildings is approximate at best, it is important to use any supplemental data that is available to aid in making a reality check of the results of the analysis.

Oakland Experience from the 1989 Loma Prieta Earthquake

General

During the 1989 Loma Pieta earthquake, downtown Oakland experienced a significant amount of damage. Many of the buildings were mid-rise (approximately 5 to 10 stories), constructed of unreinforced masonry, with and without concrete or steel framing, and generally built prior to the 1930's. There was ground motion recording station that appeared fairly represent the downtown area and that indicated peak ground acceleration in the neighborhood of 20- to 25 percent of gravity with a significant bulge in the ADRS response spectra in the range of 0.8 to 1.8 seconds. After the earthquake, engineers used a wide range of approaches to evaluate damage and approximating seismic vulnerability to future events. The City of Oakland established an earthquake repair ordinance and offered guidelines for evaluation procedures, but the overall ability of the engineering profession to properly evaluate damage and compute loss of capacity was questionable (Searer *et al.* 2006).

Government Building Evaluated Using FEMA 356/ASCE-41

Recently, some 15 years after the Loma Prieta earthquake, a large, 12-story government structure was evaluated to determine whether it poses a significant life safety risk in the event of a large earthquake; FEMA 356 (now ASCE-41) was used to assess the structure, and the assessment was the subject of a peer review.

A complicated non-linear static analysis of the structure in general conformance with FEMA 356/ASCE-41 was performed. After spending a significant effort analyzing the structure, it was concluded that in a future large earthquake, the building might have the following vulnerabilities: a soft/weak story on the first floor; weak third and 12th floors; and an unstable cupola. In addition, the engineer concluded that the building might experience compression buckling of columns due to multiple discontinuous walls, and diaphragm failure on at least three floors. Although the governing failure mode was not identified, it was concluded that the building would not be safe in a design-level or considered earthquake: a best-estimate of expected lateral strength was provided in the engineer's evaluation report, but was apparently not used to perform a reality check to ensure that the calculated capacity was reasonable. The elastic fundamental period of vibration of the structure was found to be about 0.6 seconds.

In reality, the structure rode out the 1989 Loma Prieta earthquake with very little damage, and although the downtown ground motion data described above was available to provide a calibration/reality check of the engineer's conclusions, no comparison was made. By simply converting the engineer's results from the nonlinear pushover to a capacity spectrum and plotting this best-estimate capacity against the demand from the Loma Prieta earthquake, a significant inconsistency was revealed. Use of the CSM procedure indicates that the structure should have experienced major damage as a result of the 1989 Loma Prieta earthquake; but it actually did not experience any significant damage. This demonstrates that the Phase III FEMA 356/ASCE-41 evaluation significantly underestimated the capacity of the structure (Figure 4). While some in the profession might argue on the accuracy of the CSM procedure, although time after time it has been shown to be consistent with reality, at the very least it indicates that a significant rethinking of the evaluation of this structure would be warranted prior to recommending an expensive upgrade.

Current Project

San Francisco Synagogue Survivor of 1906 Earthquake

The authors are currently working to save a monumental 102-year-old multi-story brick masonry synagogue in San Francisco that was threatened with closure due to non-compliance with an Unreinforced Masonry Building (UMB) Ordinance. The Ordinance required demonstrating that the building would protect life safety in the event of a major earthquake or upgrading the building to do so, despite the historical record showing that the building survived the Great 1906 Earthquake with relatively little damage. Much of the minor damage that did occur is still visible today as the historic finishes have remained intact since that earthquake. The structure consists primarily of thick, brick masonry perimeter walls, wood-framed diaphragms with diagonal and straight sheathing, and riveted structural steel trusses and columns that support the dome over the main sanctuary and a significant portion of the main roof and floor system.

The historical record is quite clear about the relative insignificance of the physical damage visited upon the building in 1906. As a result of its post-earthquake condition, the synagogue structure was used by the City of San Francisco as its Hall of Justice for about two years after the earthquake. Available written documentation indicates that the building required repairs at or around the roof costing about $1000, roughly 0.4% of the cost of the original building. While this fact on its own might ordinarily suggest that the intensity of ground shaking in the neighborhood of structure was not particularly damaging, other data belie this notion. Eyewitness accounts, for example, reveal that neighboring unreinforced masonry buildings of the Lane Hospital and Cooper College were rent by large cracks and complete loss of chimneys and cornices. Intensity contour mapping performed after the earthquake demarks the neighborhood shaking as generally moderate with a finger of moderate/heavy extending to the location of the Lane Hospital. On the basis of detailed studies, we concluded that certain unique dynamic characteristics of the structure -- not the modesty of the ground motions -- were largely responsible for the extraordinary performance of the synagogue structure (Paret *et al.* 2006).

Because the structure has seen such little remodeling and renovation since 1906, much of the interior damage inflicted by the earthquake remains visible today -- thus allowing study of the effects of the 1906 shaking. In the finished spaces, this damage consists almost exclusively of widespread diagonal and x-cracking in plaster on wood lath, though much of this is too fine to be meaningfully photographed. The most discernable plaster damage was found on a partition wall in an organ motor room. This, as well as similar but typically less visible cracking on the ornate ceilings throughout the building, indicates that the plaster finishes contributed to the resistance of earthquake-induced displacements.

The clearest evidence of structural damage from 1906 was observed in the main attic spaces above the balcony ceilings and just below the main roof. In these spaces, cracks were observed in the vicinity of several masonry wall pilasters, which occurrence suggests that during out-of-plane response, the walls were forced to partially “unfold”. This “unfolding” behavior was later confirmed by analysis. At one of these locations, steel bands had been added to suture the crack. More significantly, we also observed evidence of in-progress gable end wall failure: in two gables the “government anchors” that interconnect the timber roof framing to the masonry wall had dislodged and in one of those gables, permanent out-of-plane displacement of the masonry end wall had caused permanent relative motion between a steel “outrigger” beam and the end wall, resulting in a few square feet of dislodged brick and a concentration of horizontal and diagonal cracking where the beam was withdrawn from its bearing. In yet another gable, we observed the ends of numerous irregularly spaced steel veneer anchors on the interior face of the end wall, even though the ends of no other steel veneer anchors were observed at any other gables. This would appear to confirm an eyewitness report that the south gable end wall was damaged and was subsequently re-constructed. Thus, with respect to this building, the ground shaking during 1906 was apparently strong enough to just precipitate a failure sequence commonly noted in buildings with masonry gable end walls, but was insufficient to cause a full-fledged structural failure -- although a full-fledged failure might have only been narrowly avoided. We suspect that the reported $1000 damage repair to the roof was a manifestation of the damage to the south gable end wall. Nonetheless, comparison between this behavior and that of the adjacent Lane Hospital is striking in that it suggests that the subject building is a more competent building than many of its UMB brethren and perhaps that it should not quite so automatically be condemned simply for being a member of its class. Likewise, the comparison also suggests that at the root of its competence lies some interesting structural behavior waiting to be identified.

To study the response of the building and to develop an understanding of its true capacity, ground motion spectra for the 1906 and 1989 earthquakes were developed -- necessarily using different techniques. The 1989 motions were taken from records made by an instrument 1-1/2 blocks away from the subject building. The 1906 motions were ‘synthesized’ from two records from the Loma Prieta earthquake but from instruments located at approximately the same distance from the San Andreas fault as the subject building. Spectra from these two records were averaged and converted to the appropriate local intensity using the Engineering Intensity Scale (Freeman *et al.* 2004). The results demonstrate that the shaking during Loma Prieta was an insignificant fraction of the synthesized 1906 motions; that the 1906 motions likely fell primarily within Engineering Intensity VIII; and that the 1906 motions, though slightly smaller in the constant acceleration range, are nearly identical to the requirements of the UMB Ordinance. These findings generally conform to the damage intensity mapping performed after the 1906 earthquake.

On the basis of a displacement-ductility based Capacity Spectrum analyses, we concluded that with respect to in-plane behavior, even without considering global rocking and foundation flexibility, the four perimeter brick masonry walls would have had no trouble weathering the 1906 earthquake. With respect to out-of-plane behavior, however, the displacement-ductility based Capacity Spectrum analyses demonstrated that for three of the four walls, a displacement ductility of about 4 would have been required to weather the synthesized motion. However, because the analysis did not include the particularly unique circumstances of the fourth wall, namely the out-of-plane restraint afforded by a series of plaster sheathed timber-framed walls that enclose two stairwells, this wall appeared in the pushover curve to have been subjected to substantially greater displacement ductility demand. In general, these findings are consistent with the physical appearance of the stairwell walls, which are believed to have never been repaired since the 1906 earthquake. The greatest in-plane strain demands are located in the lower portions of each of the walls, yet to our knowledge, cracking in the lower reaches is essentially absent. The greatest out-of-plane strain demands occur in the upper reaches of the walls --- in the gable end walls and in the vicinity of the thickened pilasters --- precisely where the most significant structural damage was observed. As noted earlier, in appearance this damage suggests the initiation of a failure sequence; a sequence whose initiation is not unreasonably posited at a displacement ductility of around 4. With respect to the fourth wall, damage done to the supporting plaster-sheathed partitions indicates that this wall did in fact engage supplementary out-of-plane support in order to withstand the earthquake.

Given the general similarity of the demands imposed by the synthesized 1906 spectrum and the UMB Ordinance criteria, it might therefore be argued that the building does not require intervention to bring it into compliance. However, this argument neglects to account for some fundamental uncertainties as well as some potential seismic hazards at the property that cannot be realistically modeled. Amongst the uncertainties are the intensity of ground motion that actually occurred at the site during 1906 and the uncertainty regarding the particular characteristics of future ground shaking at the site. Since a displacement ductility of 4 for an unreinforced masonry wall is arguably on the cusp of unacceptability insofar as seismic risk, either of these uncertainties could alone be considered as sufficient reason for intervention. Other circumstances also weigh heavily on this issue. For example, the masonry, having significant cracks and locally dislodged bricks in the attic, is certainly in worse condition than it was prior to the 1906 earthquake. Moreover, there are ample indications that portions of the wall have been degraded by long term water ingress, which would not have been the case in 1906. Of significance as well is the general lack of reliability with respect to the existing floor-to-wall anchorage --- a critical link in the seismic resistance of the entire system. Some government anchors have already been damaged and most locations do not even seem to have anchors; rather, the floor-to-wall ties rely upon a friction fit between the floor/roof joists and the masonry pockets into which they are embedded. Today, we do not know which of these connections have already slipped, nor how much slip any of them might have experienced during 1906. For these and other reasons, we recommended and the client agreed that improving the seismic resistance of the building was necessary.

In developing the intervention philosophy, it was recognized that unreinforced brick masonry buildings with flexible diaphragms have distinctive dynamic characteristics, specifically modal separation between the in-plane and out-of-plane wall vibrations, the preservation of which during seismic strengthening can proffer tremendous advantages. In the case of the synagogue, a unique historic structure that experienced the Great 1906 Earthquake with relatively little damage, such preservation was made a performance-based priority during the course of developing a design to bring the building into compliance with the San Francisco UMB Ordinance. Without embellishment, it can be said that employment of this approach likely saved this important historic structure from the wrecking ball, since funding for a more traditional prescriptive approach was unavailable, especially given that such an approach would have effectively destroyed the heart and soul of the very building which needed to be saved.

Conclusions

When major earthquakes occur, there is a great emphasis on reporting spectacularly damaged buildings. In many cases, there is a rush to judgment and a drive to modify building codes to correct perceived loopholes. After the dust has settled and a fresh look as to the cause of the reported damage is made, it is often determined that some of the initial perceptions were incorrect. In many cases, the cause of observed failures were errors in the design process -- such as mathematical errors, dependence on computer programs without proper back checking, or lack of attention to details -- rather than deficient codes. Other causes include errors in construction or improper modifications. What is rarely emphasized is how well a great majority of the buildings performed. In other words, the older building codes and good engineering practices have served us well. It is also worth noting that many of the older buildings, some dating back before the era of modern seismic design, also perform well. It is often not realized by younger engineers that many engineers of earlier generations understood engineering mechanics and the basic concept of lateral force design, even though the codes of the day might appear to be deficient from today's perspective. Also, some building systems and materials are better than we think they are. In these modern times of high-speed computers and the rush to get things done, there seems to be a lack of interest to take the time and effort to look over what we are doing, and conduct a reality check of the results of the analyses. It is a shame that when so much data exists from past earthquakes, engineers often forget to use this data to bench-mark and reality-check their own analysis and predictions of future behavior.

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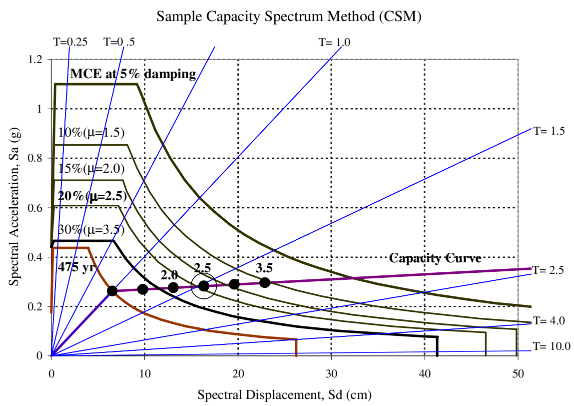

Figure 1. Sample Capacity Spectrum Method.

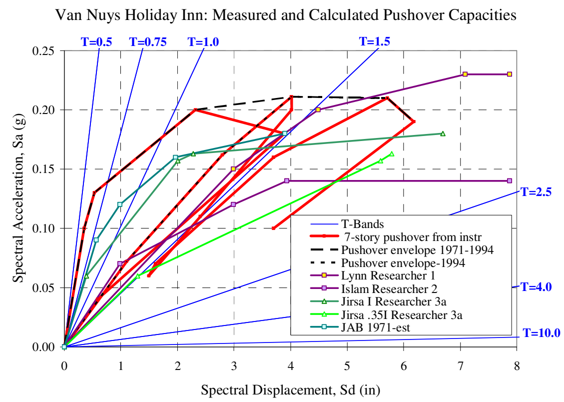

Figure 2. Van Nuys Holiday Inn Pushovers.

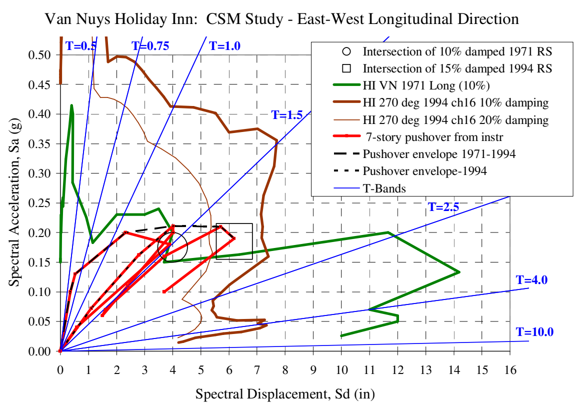

Figure 3. Van Nuys Holiday Inn CSM Study, EW.

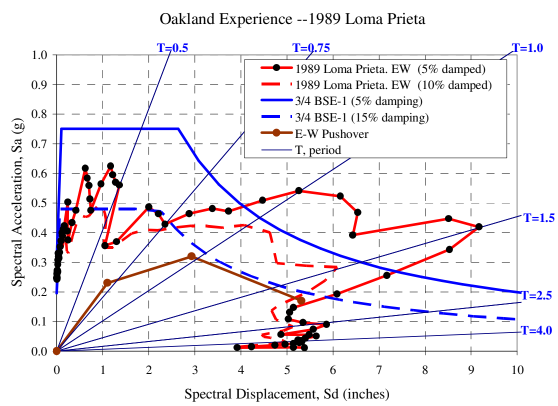

Figure 4. Oakland Experience, 1989 Loma Prieta.