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SEISMIC EVALUATION OF 32 INDUSTRIAL BUILDINGS BY SCREENING PROCESS AND ANALYSIS

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SUMMARY

Thirty-two buildings from the Puget Sound Naval Shipyard (PSNS), Washington, USA, were seismically evaluated by the two-tiered evaluation recommended by the U.S. Federal Emergency Management Agency (FEMA). The buildings, many of them large industrial shops and offices, were built between 1897 and 1980 and represent a variety of building types with the largest category being steel moment frame structure.

This paper presents the typical seismic deficiencies found during the evaluation and discusses the upgrade strategies and the affiliated costs in relationship to building age, size, type, and historic classification. The paper discusses the benefits and drawbacks of the seismic evaluation of large facilities, such as for PSNS.

The paper concludes that the seismic evaluation succeeds in identifying and quantifying the structural upgrade needs of the buildings to full code performance levels, but does not address alternative solutions that consider partial upgrades or less severe performance goals to optimize upgrade works for the available funding. It also concludes that, for owners of large facilities, the seismic evaluation alone does not offer a satisfactory basis for establishing upgrade priorities and implementation strategies without developing upgrade concept studies with affiliated cost estimates. The study found that the building age and structural type are the most influential parameters on upgrade costs. Whether a building within the same age group is listed as historic or not, seems not to influence the upgrade costs. Similarly, a larger building seems not less expensive to upgrade on a square foot price basis than a smaller building.

INTRODUCTION

PSNS was established in 1891 as a naval station at the Puget Sound in Bremerton, Washington. Today it is the second largest industrial facility in the state of Washington, USA, both in terms of plant investment and in the number of civilians employed. The facility comprises 125 buildings with a total of 4 million square feet. In 1999, the U.S. Navy concluded a seismic vulnerability screening study on core buildings of PSNS. As a result of the seismic screening phase, 32 buildings were selected for further detailed

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seismic evaluations. Of the buildings evaluated, the largest, most important to the mission of the shipyard are of steel frame or steel truss moment frame construction. These buildings are most typically described as industrial-type buildings composed of one to several long bays served by bridge cranes. The lateral load resistance is typically furnished by longitudinal bracing and transverse moment frames.

The seismic evaluation was a voluntary effort by the Navy to assess the performance of the core shipyard buildings during a major earthquake and to quantify the costs for their seismic upgrade. The evaluation and concept repair designs allow for the prioritization of buildings to be upgraded and create the basis for a strategy on how to integrate seismic upgrades into future budgets for maintenance and functional upgrades of the shipyard's building stock. First priority was given to buildings that not only were of high importance to the mission of the shipyard and pose the greatest risk of fatalities during a large earthquake, but those that also experience damage in the recent Nisqually earthquake. The upgrade work on these buildings was commenced in 2003.

Overview of Seismic Vulnerability of Building Systems

Over the years, detailed postearthquake reconnaissance investigations of building failures have provided revealing information on the physical attributes that affect the seismic performance of buildings. The following sections provide a brief summary of merits and deficiencies associated with basic structural systems commonly used in building construction. The information is presented to highlight potential problems that buildings may face when subjected to strong ground motions.

Reinforced Concrete Moment Frames

Seismic performance of reinforced concrete (RC) structures during moderate and severe earthquakes has ranged from minor cracking to complete collapse. This may suggest there is nothing inherent in concrete structures, which makes them particularly vulnerable to earthquakes. RC moment frames with infill masonry walls have demonstrated relatively good performance during earthquakes except for the presence of "captive columns" or the interaction with massive nonstructural infill walls. Examples of these situations were observed during the 1985 Mexico, the 1967 Caracas, and 1988 Armenia earthquakes.

Except for cases of poor detailing and construction, postearthquake inspections have revealed few instances of beam-column joint failures in moment-resisting frames. The main deficiency in concrete moment frames has been inadequate ductile behavior or shear capacity in either columns or beams. The performance of RC structures is not only very sensitive to but is even controlled by the amount and detailing of the reinforcement.

Precast Concrete Frames

Precast concrete frame systems can be classified in three categories.

1. Precast frames emulating cast-in-place moment frames
2. Precast frames other than emulating cast-in-place conditions
3. Gravity-type precast frames not expected to resist lateral loads directly

Emulating systems have been used since the mid-1950s. Deficiencies of emulating systems are consistent with those traditionally linked to cast-in-place frames. Nonemulating systems rely on dry joint, bolting, welding, or post-tensioning for assembly and, consequently, for adequate seismic performance. Frames of this kind may act alone or in conjunction with shear walls to resist lateral loads. Unfortunately, many double-tee and single-tee systems lack a complete seismic load path, in which brittle welded connections often become the weakest link. Precast "gravity" frames rely on either concrete shear walls, steel moment frames, or braced frames to resist seismic loads. Evidence from the 1993 Northridge earthquake showed that many parking structures built with this structural system collapsed due to the inability of the precast frames to resist any shear from building drift.

Reinforced Concrete Shear Walls
Reinforced concrete shear walls have behaved relatively well in earthquakes. This system can be classified in three categories.

1. Bearing wall
2. Shear wall
3. Shear wall-frame (dual) system

Structural performance of bearing wall systems during the 1985 Chile earthquake indicated this system may be very reliable if the percentage of wall area to total floor area is high. Shear walls and dual system buildings have also performed well except for buildings with wall plan irregularities that have exhibited excessive torsional demands on columns.

Masonry Bearing Wall Buildings
Unreinforced masonry (URM) bearing wall structures are probably the most seismic vulnerable form of construction worldwide. URM wall buildings are typically short period structures that lack the ability to withstand inelastic deformations, due to lack of reinforcement. As a result, structural damage is inevitable through a few cycles of seismic action. After strong ground motions, those structures that have not collapsed are often so damaged that demolition is required.

Both URM and inadequately reinforced masonry buildings have displayed poor performance in past earthquakes. For instance, structural damage in URM buildings during the 1989 Loma Prieta earthquake included out-of-plane brick failures due to high wall slenderness, poor anchorage to diaphragms, and low mortar strength; in-plane shear failures; and excessive diaphragm flexibility. These observations are of particular concern because the Loma Prieta earthquake was not a severe earthquake. Diaphragm separation from vertical support leading to partial wall collapse is one of the leading causes of human life loss during earthquakes.

Precast Tilt-Up Panel Buildings
The seismic performance of precast tilt-up panel buildings is closely dependent on their connections. Structural damage in tilt-up panel buildings during the 1964 Alaska and the 1971 San Fernando earthquakes included partial collapse of roof sections due to failure of panel-to-roof connections and collapse of wall panels due to failure of the panel-to-roof, panel-to-panel, and panel-to-foundation connections. The most common failure mode is roof separation from wall supports. Despite efforts to improve seismic design recommendations for tilt-up construction, damage in these types of buildings following the 1989 Loma Prieta earthquakes was still unacceptable.

Steel Frame Buildings
Lateral forces in steel frame buildings are carried by moment frame action, bracing, infill masonry walls, or a combination of these. The principal seismic deficiencies in steel moment frames are associated with inadequate beams, columns, and beam-column connection capacities. In addition, moment frames may exhibit excessive drifts, with accompanying racking of walls.

For years, it was believed that welded steel moment frame buildings were virtually invulnerable to earthquake loads. The 1994 Northridge earthquake changed this perception when postearthquake evaluations identified brittle fractures at beam-column connections in welded steel moment frame buildings ranging from 1 to 26 stories high and as old as 30 years. These observations were widespread over a large geographical area, including sites where only moderate ground shaking occurred. Buildings of this type did not collapse but both repair costs and indirect losses resulting from building evacuations led to considerable loss. For the case of truss moment frames, main deficiencies concentrate on inadequate truss web, chord, and connections, especially when the truss' ability to carry vertical loads is compromised. Main seismic deficiencies in braced frames are associated with inadequate brace connections, inadequate foundations, or excessive drifts.

Many early (pre-1930) steel frame buildings had infill URM walls. The contribution of the URM walls for lateral load resistance was usually neglected in design when in fact, the masonry provides much of the lateral resistance of these buildings. Because the steel framing provides vertical support for the floors and roof, sudden or complete collapse of these structures is not expected. These structures have performed generally well in avoiding collapses, particularly in comparison with concrete frames with infill walls. However, infill wall collapse presents a life-safety threat.

Wood Frame Buildings

Although most older wood frame structures were not designed for seismic demands, wood buildings of normal size and shape have performed well in past moderate earthquakes, due to their light weight, ductile connectivity, and redundancy, provided they are integrated with the foundation. Poor anchorage of walls to foundations has been the primary reason for poor seismic performance of wood frame structures.

Nonstructural Elements

It is well known that nonstructural elements can be a threat to life safety. Field inspections after the 1971 San Fernando and 1972 Managua earthquakes showed that poor performance of parapets, concrete panels, nonbearing walls, partitions, and suspended ceilings were responsible for the highest number of fatalities.

METHODOLOGY

Seismic Evaluation Process

The U.S. Federal Emergency Management Agency (FEMA) describes in its prestandard 310 [1] a three-tiered approach to assess the seismic performance of buildings. The first tier shall allow the engineer to identify potential deficiencies of the building to resist seismic forces by means of a simple screening process. The characteristics of the building can be assessed by building inspection, by evaluation of building-type specific checklists supported by simple calculations. Upon the identification of deficiencies by Tier 1, the standard gives direction for more detailed Tier 2 analysis, commonly involving finite element analysis. The Tier 2 analysis is recommended if the Tier 1 screening found deficiencies in either the superstructure, the foundation, or nonstructural elements. The Tier 2 analysis typically involves linear elastic finite element analysis, static or dynamic, with either true material characteristics, if known, or with conservative assumptions of material characteristics given by FEMA 302/303 [2,3] or FEMA 356 [4]. For complex buildings, FEMA recommends to conduct a Tier 3 analysis, which evaluates the seismic behavior by means of nonlinear finite element analysis, such as push-over or time-history analysis.

A FEMA Tier 1 evaluation on the PSNS buildings was conducted in 1999 by others. Thirty-two buildings were selected for Tier 2 analysis. For most of the buildings, a Tier 2 analysis by means of linear finite element analysis was sufficient. Tier 3 analysis was deemed appropriate for two buildings where interaction with soil was present along the length of one side for their full 5-story height. Their lateral force resisting systems relied upon permanent soil anchors. The first step of the Tier 2 analysis was the collection of data. If as-built drawings were not available, such drawings had to be reconstructed to an acceptable accuracy for the analysis. Material characteristics were either taken from the drawings or estimated from a lower bound for the types of materials at the time of construction, or evaluated by testing, dependent on the required precision for the analysis. The analysis typically modeled the structures as three-dimensional grids of beam and shell elements reflecting all structural and nonstructural elements that contributed to the seismic response. Each model was subjected to a pseudo-dynamic lateral force that was determined from the site-specific response spectra, which reflected 2/3 of the peak accelerations during the standard Maximum Considered Earthquake (MCE) defined as ground motion having a 2 percent probability of exceedance in 50 years at the site. Because all buildings were at the same site, they were all evaluated for the same soil Type D subjected to the same response spectra. All buildings considered for this paper were evaluated for a performance level of Life Safety. The Life Safety performance level allows for damage of the building during the design earthquake while ensuring that the chance for the loss of human life is low.

Building Categorization

The buildings were categorized in building types to allow for interpretation and comparison of the findings from the seismic evaluation. The building categorization process was complicated not only because of the inherent uniqueness associated with each building system but also because the lateral resisting system of many buildings consisted of a combination of at least two building classes. In addition, many buildings consisted of multibuilding structures, in which substructure components had completely different lateral force resisting systems and construction materials. All these led to the definition of project-specific building categories for the PSNS buildings rather than adopting either the FEMA 310 [1] or FEMA 276 [4] classification system. Some buildings were classified in two separate building categories if their multibuilding characteristic allowed, increasing the total number of buildings within all categories to 34. The following sections provide detailed description of each PSNS-specific building category.

Category A: Combination of Steel Moment Frame or Truss Moment Frame with Steel Braced Frame (SMF/STMF + SBF)

The lateral resisting system consists of a combination of steel moment frames in one direction with steel braced frames in the orthogonal direction, with either flexible or stiff wood, steel deck, or insulating concrete roof diaphragms and either wood or composite deck mezzanines. Moment frames are either truss moment frames or frames relying on cantilevering action of frame columns. The foundations consists of either spread concrete footings or piles. Ten buildings were assigned to this category.

Category B: Combination of Steel Frames with Unreinforced Masonry Infill Walls (SBF/SF + URM)

The lateral resisting system consists of steel frames braced or unbraced with URM infill walls, with either wood or steel deck roof diaphragms, and wood or composite slab floor diaphragms or mezzanines. The foundations consist of either spread concrete or pile footings. Five buildings belong to this category.

Category C: Unreinforced Masonry Buildings (URM/URMA)

The lateral force resisting system consists of URM wall buildings, with either concrete, wood or built-up roof and floor diaphragms, supported on spread concrete footings or piles. Four buildings fell under this category.

Category D: Combination of Steel Frames and Reinforced Concrete Shear Wall, Precast Tilt-Up Panels, or Reinforced Masonry Shear Wall (SMF/SBFR + CSW)

These buildings are a combination of reinforced concrete/masonry wall-type buildings with steel frames, with either flexible or stiff diaphragms. The wall system encompasses either concrete shear walls, precast concrete tilt-up panels, or reinforced masonry walls. This category is perhaps the most diversified among the different building classes considered in this study. Six buildings were assigned to this category.

Category E: Reinforced Concrete Shear Walls (CMF + CSW)
Buildings within this category are composed of concrete frames and concrete shear walls, with either flexible or stiff diaphragms. Three buildings fell under this category.

Category F: Wood Frames (WMF + WSW)
Buildings within this category are composed of wood buildings, which rely on wood shear walls and wood frames for lateral load transfer. Four buildings belong to this category.

Category G: Unique Structures
Buildings within this category are unique structures with lateral load resisting systems dependent upon ground anchors that also retain earth. There were two buildings assigned to this category. These buildings are not included in the trends summarized in this paper because the structural systems are unique.

In this paper, the seismic load resisting system of a building was divided into four systems to better localize the deficiencies. The subsystems follow the typical load path from roof top into the soil.

- Horizontally Lateral Load Resisting Systems (HLLRS) collect the lateral seismic forces and spread them horizontally to the vertical load resisting systems. Examples for such systems are roof and floor diaphragms.
- Vertically Lateral Load Resisting Systems (VLLRS) direct the lateral seismic forces to the foundations. Moment frames, braced frames, and shear walls belong to this category.
- Foundations introduce the lateral seismic forces into the soil.

Cost Evaluation Procedure
Evaluation of retrofit costs is essential both for evaluating whether proposed upgrade concepts are cost-effective and for signaling those structural categories for which structural upgrading may not be feasible. Due to the limited number of buildings examined, together with their particular characteristics, interpretation, or extrapolation of the upgrading cost trends herein reported should be exercised with care.

In this study, upgrading costs include only direct structural rehabilitation costs. Indirect costs, such as those related to relocation of building occupants, increase in market value of rehabilitated building, and even items, such as painting or other architectural upgrades, are not included. For multibuilding-type structures, the cost evaluation was provided for selected substructure building components. Retrofit costs of small “lean-to” structures with a structural system that differs greatly from the main building were not included in the estimates.

The upgrading cost evaluation was performed accounting for the following variables.

- Building age
- Building size
- Structural system, on the basis of configuration of PSNS-specific building category, based on the current (unrehabilitated) building condition
- Upgrading task
- Building occupancy condition or building function
- Historic category

Three major categories of upgrading tasks were defined corresponding to the three seismic structural subsystems.

- Foundation upgrade
- Vertically Lateral Load Resisting System (VLLRS) upgrade
- Horizontally Lateral Load Resisting System (HLLRS) upgrade

Categorization as a function of building occupancy type was difficult to implement because many of the buildings served multiple functions. Nevertheless, three occupancy types were defined. For multi-use buildings, the occupancy type was defined up to the authors' best judgment. Types are shown in ascending order, depending on the number of people dwelling in the building.

- Low Occupancy (storage buildings)
- Medium Occupancy (shop buildings)
- High Occupancy (office, school and first response buildings)

The effect of historicity of buildings was also accounted for in the upgrading cost evaluation. Out of the 34 building components examined, 11 are registered as historic buildings. This means that any structural upgrading of this structures shall be performed so that the building's aesthetics is preserved.

The seismicity level was not accounted for as a major factor because all buildings were located on the same site. The performance objective was not considered because all the buildings were evaluated for the same performance objective level, i.e., Life Safety.

The seismic upgrade costs were calculated for the buildings to meet the Life Safety performance level at 2/3 of the Maximum Considered Earthquake (MCE). The main upgrading cost descriptor used in this paper was the cost per square foot. This descriptor was used per building, per building group, and per upgrading activity.

OUTCOME OF SEISMIC BUILDING EVALUATION

Identified Seismic Deficiency and Possible Upgrades

The seismic deficiencies identified during the evaluations were consistent with those highlighted in the overview of seismic vulnerability of building systems. Major deficiencies were found in all areas of the seismic load resisting systems, i.e., in the horizontal and vertical load resisting systems, as well as in the foundations. Flexible building systems, such as those of Categories A, B, D, and F that do not rely on shear walls, yield excessive transverse drift during an earthquake. Whereas, buildings that rely on shear walls, such as those in Categories C and E, often lack continuous seismic load paths, shear walls of insufficient strength, or exhibit excessive torsional eccentricity. Thus, repair efforts often focus on completing and strengthening the seismic load paths and on stiffening flexible building systems. Additionally, the overturning moment of vertical load resisting systems can result in failure of the foundations, where such systems are concentrated rather than distributed over the building footprint, such as in buildings with braced frames or shear walls. Lighter building systems, such as wood buildings, often lack adequate strength within the connections between load resisting elements.

Common seismic retrofit measures proposed to eliminate deficiencies include adding or replacing entire load resisting systems, strengthening of deficient elements, and upgrading the connections between load resisting elements. Foundations are strengthened by enlarging the footings or by adding micropiles. Other proposed measures reduce the seismic load by reducing building weight by replacing heavy concrete roof panels with light weight metal decks or removing obsolete cranes. Masonry elements that pose a possible falling hazard, such as unreinforced parapets, unrestrained infill walls, or cladding, have to be restrained.

In the case of URM structures or steel frame structures infilled with masonry, inadequate tiebacks of masonry to frame, inadequate out-of-plane bending strength, and inadequate in-plane shear strength of the masonry were the prime deficiencies. Diaphragms (floor and/or roofs) were often found to have insufficient in-plane shear strength.

Although not designed for earthquake, all of the buildings were presumably designed for lateral loading due to wind. Therefore, buildings of light weight construction (wood, steel with corrugated cement asbestos board siding) were designed for a lateral load equivalent to a greater percentage of building weight than buildings of heavy construction (concrete, URM, framed with masonry infill walls).

The choice of retrofit measures is often limited due to functional requirements of the building or due to the building being categorized as historic. For example, unreinforced masonry, which is part of a historic building's appearance, cannot be replaced and can only be strengthened on the face that is not visible. Likewise, braces can only be added where they do not impact the appearance of the historic building. More details on the identified major deficiencies of the seismic load resisting systems of the buildings and considered upgrade measures for each building type are summarized in Tables 1 and 2.

Table 1 Major Deficiencies of Load Resisting System for Each Building Category

A	- Inadequate diaphragm connections to vertical force resisting system - Overstressed moment and truss moment frame elements - Inadequate moment and truss moment frame connections - Overstressed vertical and horizontal braces - Inadequate brace connections - Pile foundations prone to uplift - Spread footings inadequate to resist sliding, settlement, and overturning
B	- Inadequate roof and diaphragm connections to URM walls - Overstressed frame elements and connections - Overstressed vertical and horizontal braces and connections - Very slender and overstressed URM infill walls - Pile foundations prone to uplift - Spread footings inadequate to resist sliding, settlement, and overturning
C	- Deficient floor and roof diaphragms - Inadequate floor and roof diaphragm connections to URM walls - Overstressed, slender URM walls - URM walls not dowelled to foundation
D	- Inadequate connection of tilt-up panels to roof, foundation, and between each other - Overstressed frame elements and connections - Overstressed vertical and horizontal braces and connections - Pile foundations prone to uplift - Spread footings inadequate to resist sliding, bearing, and overturning
E	- Insufficient concrete diaphragm chord reinforcement - Inadequate connectivity between URM infill walls and structure - Overstressed shear walls, columns, and URM infill walls - Shear walls prone to overturning
F	- Inadequate connectivity between roof diaphragms and wood frames - Overstressed wood shear walls and frame components - Inadequate connectivity between frame elements and footings - Insufficient footing uplift and overturning resistance

Table 2 Considered Seismic Upgrade Measures for Each Building Category

A	- Upgrading of existing moment frame components and connections - Addition of vertical and horizontal braces - Addition of entire new vertical load resisting systems, such as super frames - Improvement of roof, floor, and mezzanine connections to braced frames - Reduction of weight by removing obsolete cranes and by replacing heavy precast concrete roof elements with metal deck - Foundation retrofit at upgraded bracing locations - Enlargement of spread footings and conversion of spread footings into pile foundations
B	- Addition of concrete shear walls - Shotcreting of URM infill walls - Bracing of URM parapets - Tie cladding to structure - Addition and replacement of both vertical and horizontal braces - Foundation retrofit at upgraded bracing locations - Construction of shear wall grade beams - Footing enlargement - Addition of micropiles
C	- Strengthening of floor and roof diaphragms - Addition of shotcrete overlay to URM walls - Provide connectivity between retrofitted diaphragms and retrofitted walls - Construction of wall grade beams - Bracing of URM parapets
D	- Addition and replacement of both horizontal and vertical braces - Improve connectivity between shear walls to floors and roof - Improve connectivity among tilt-up panels and between panels and roof and floors - Dowel shear walls and tilt-up panels to grade beams - Addition of concrete shear walls or shotcrete overlays - Strengthening of moment and truss moment frames - Conversion of footings into pile/pile cap foundations - Construction of grade beams for tilt-up panels, new shear walls, and shotcrete overlays - Foundation retrofit at upgraded bracing locations
E	- Upgrade roof and floor diaphragms - Addition of concrete shear walls or shotcrete overlays - Upgrading of connections between retrofitted shear walls and diaphragms - Construction of grade beams for shear walls and shotcrete overlays - Enlargement of footings at shear wall locations - Addition of micropiles
F	- Diaphragm retrofit through plywood sheets and blocking - Addition of wood shear walls - Upgrade of connections between diaphragm and shear walls - Strengthening of wood frames - Construction of grade beams or footing enlargement for new wood shear walls - Addition of micropiles

Cost Evaluation of Seismic Upgrade

Effect of Building Age

Earthquake design requirements have grown increasingly stringent since first put into practice in the early fifties. The majority of the buildings recommended for further seismic evaluation (23 of 32) was designed and constructed prior to the formulation of earthquake regulations. Thirteen buildings were associated with World War II era, eight associated with World War I, and two at the dawn of the twentieth century.

Figure 1 shows a comparison of upgrading costs per square foot as a function of building age for all buildings. Buildings have been arrayed according to the PSNS-specific building categories. Historic buildings are shown with solid symbols. Consistent with twentieth century building seismic design knowledge evolution, Figure 1 shows that upgrading costs increase with building age. Results suggest the increase is linear and also independent of whether the building is historic or not. The figure also shows that the upgrading cost required to bring about half of the shipyard buildings that were built in World War II to Life Safety compliance does not differ much from that associated to the newest buildings on site.

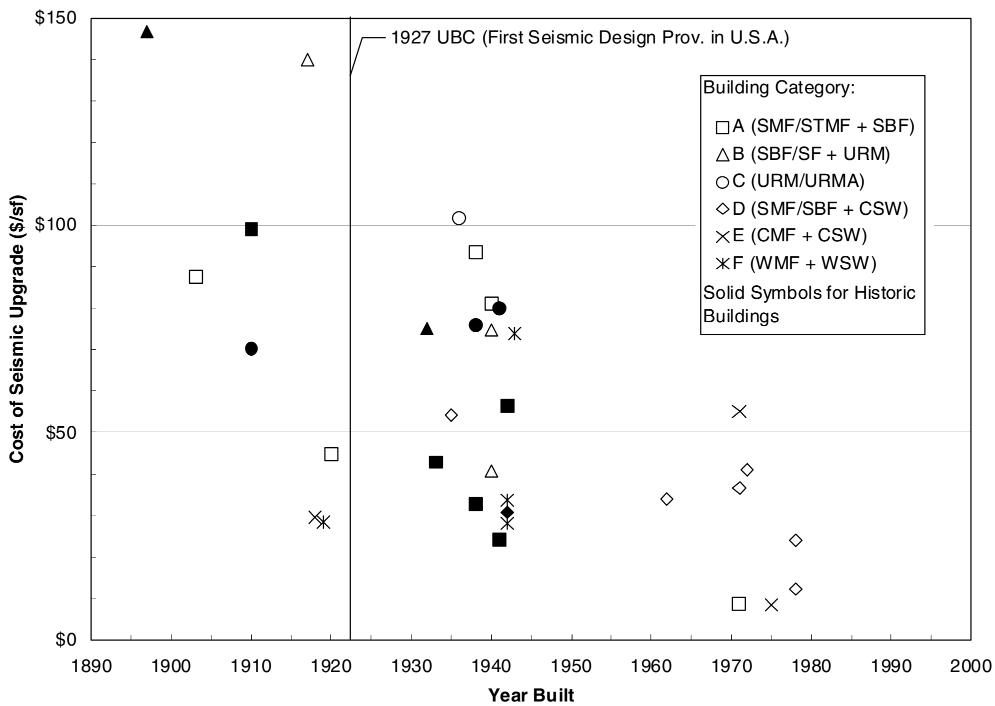

Figure 1 Total Mean Upgrade Cost per Square Foot versus Building Age

Effect of Building Size

Figure 2 shows upgrading costs per square foot as a function of building size. Buildings have been arrayed per PSNS-specific building types. Solid symbols indicate historic buildings. At first sight, the data suggests that upgrading costs are higher for smaller buildings. However, if the two most expensive buildings (built in 1897 and 1917, respectively) are treated as outliers, results show that upgrade costs on a square footage basis are relatively independent of building size.

Results also show that upgrade costs per square footage basis for historic buildings do not seem to vary with respect to those of nonhistoric buildings. This observation is rather interesting because historic buildings are often stereotyped as expensive upgrading projects.

Effect of Building Category

Figure 3 shows mean upgrade costs per square foot for all building categories. The figure shows that Categories B and C buildings are the most expensive building types to repair on a square footage basis. The high upgrade costs for these two categories are not surprising. URM wall buildings have consistently performed unsatisfactorily during earthquakes in the past, requiring masonry walls to be strengthened over large areas. Old steel braced frame and steel moment frame buildings require significant foundation and superstructure upgrade work. The plots also confirm that the least costly building type to seismically retrofit is wood framed (Category F). On a broader scale, the plot confirms the inference that the cost of seismic retrofit for buildings of greater mass (Categories B and C) is higher. Thus somewhat paradoxically, structures conventionally thought to be most durable are more vulnerable to earthquake hazards if not designed for seismic loads, whereas light weight structures regarded as less permanent are found to be inherently more earthquake-damage resistant.

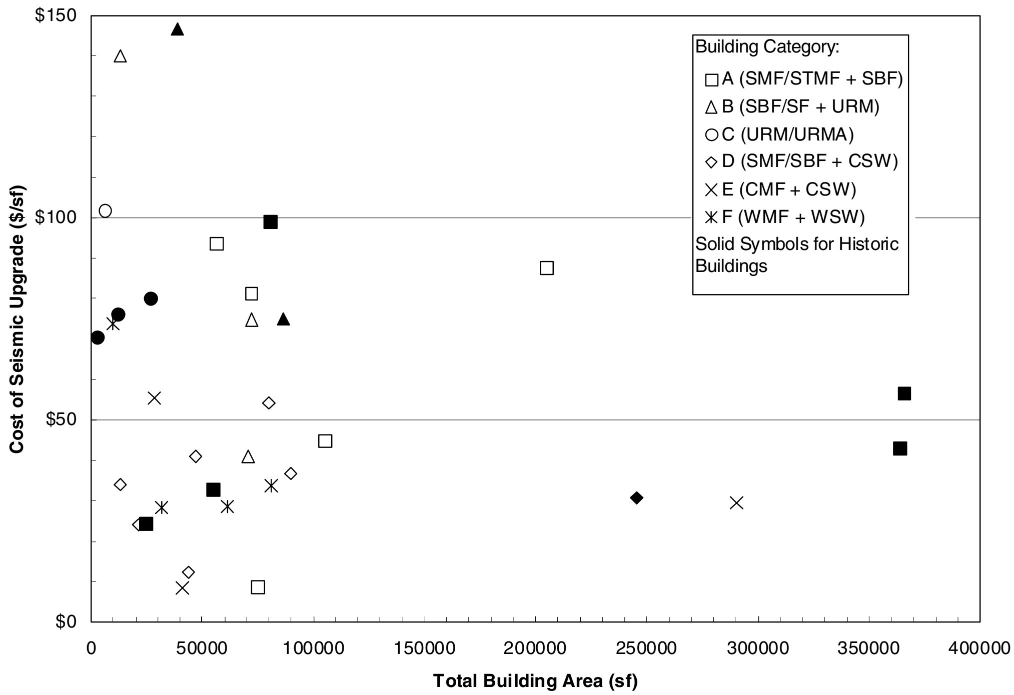

Figure 2 Total Mean Upgrade Cost per Square Foot versus Total Building Area

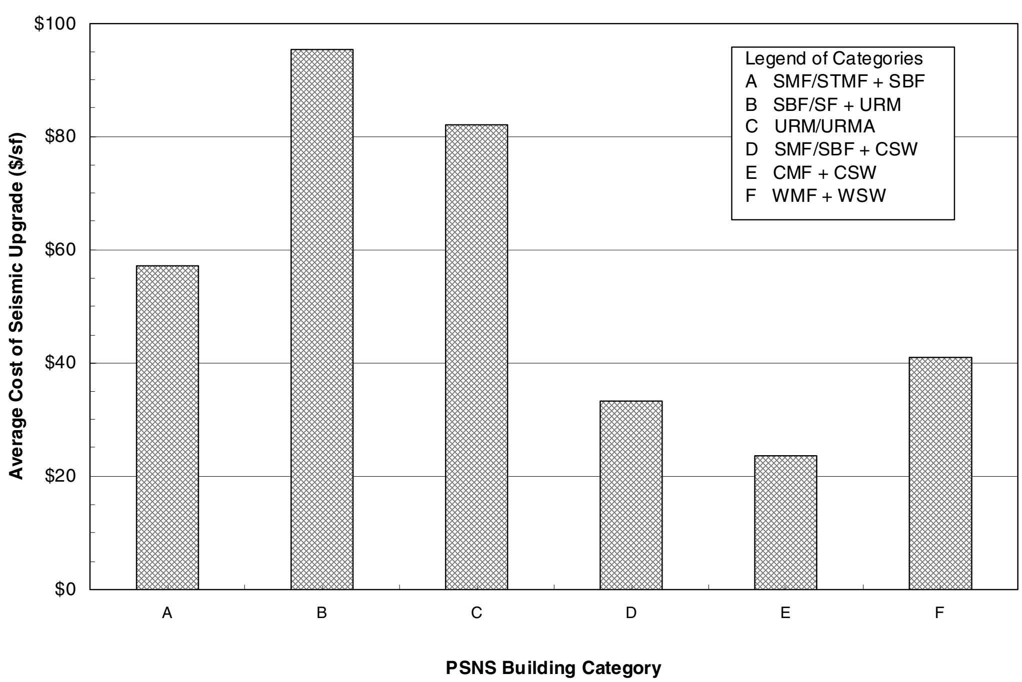

Figure 3 Upgrade Cost Comparison per Building Category

Effect of Building Category on Upgrade Activity
Figure 4 shows mean upgrade costs per square foot for all building categories per upgrading activity. The figure shows that foundation and superstructure upgrade costs in Categories B and C are highest than in any other building category. The high upgrade cost for Category B buildings is expected. Proper seismic response of steel braced frames and steel moment frames relies heavily on foundation systems that are far more efficient against overturning than the spread footings that were typically found in the considered buildings. This explains why Category B buildings are the only ones to display mean foundation upgrading costs that are higher than those associated with the lateral load resisting system repair.

The figure also shows that superstructure upgrade costs in Category C buildings are highest among all building categories. This is because this system's repair concentrates above ground, in this case through URM wall shotcreting. Figure 4 also indicates relatively consistent foundation upgrade costs for Categories C, D, E, and F, which are essentially shear wall-type buildings.

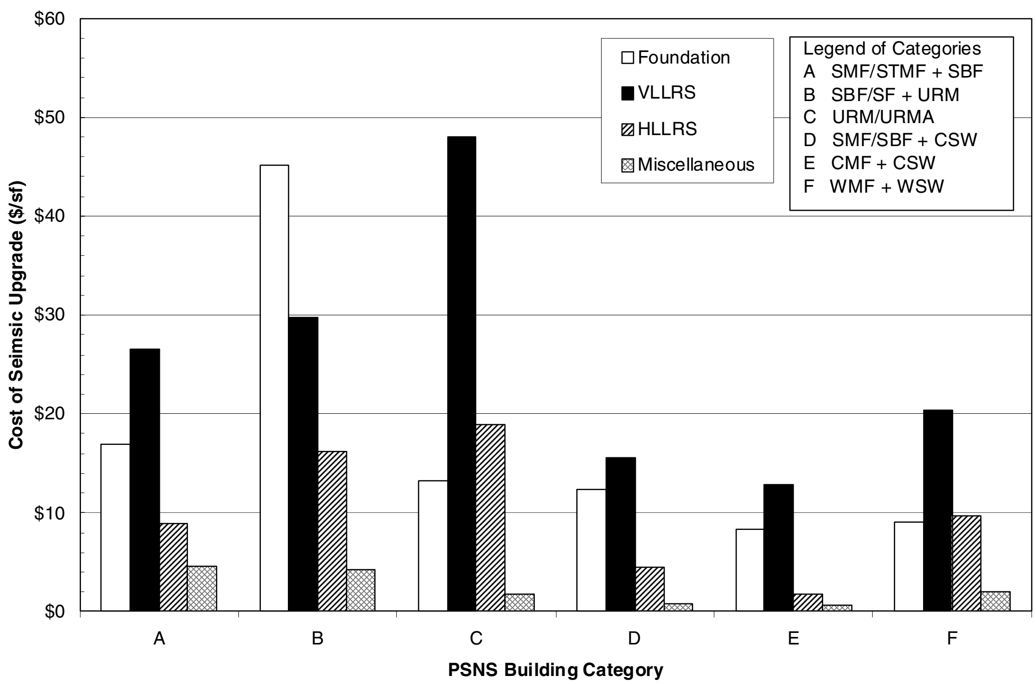

Figure 4 Average Upgrade Cost per Building Category

Effect of Building Occupancy
Figure 5 shows mean upgrade costs against building use. Solid symbols indicate historic buildings. The building use has been categorized according to the building occupancy whereby storage and warehouses have low occupancy; shops have medium occupancy; and offices, schools, and buildings for first response, such as fire stations, have high occupancy. The scatter in the results suggests that mean retrofit cost of buildings is independent of the use. A seismic deficient building with higher occupancy bears a higher risk for causalities during an earthquake. It seems, therefore, appropriate to invest in upgrade of higher occupancy buildings first in order to optimize personal safety for the available budget.

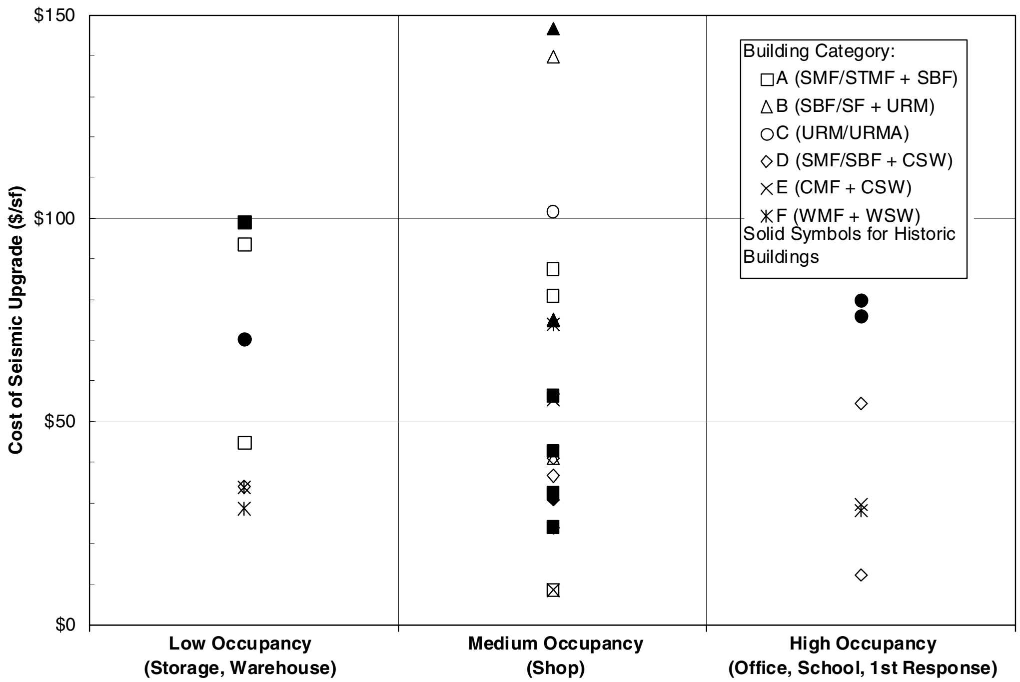

Figure 5 Total Mean Upgrade Cost per Square Foot versus Building Occupancy

DISCUSSION OF BENEFIT OF SEISMIC EVALUATION BY SCREENING OR ANALYSIS

The seismic evaluation of the 32 buildings by screening (Tier 1) was successful in identifying the structural elements in need of seismic upgrade. However, the screening could not quantify the level of deficiency of the structural elements, as this is not in its scope. If deficiencies have to be quantified, due to the need for either a cost estimate or a design for the seismic upgrade, then further analysis is unavoidable. Experience with the more aged of the subject 32 buildings suggests that, once deficiencies are found by the screening procedure, retrofits designed to fully comply with the Prestandard requirements for Life Safety performance will have significant cost impact. If funding for upgrade is not available, further detailed evaluation may not be useful.

The evaluation (Tier 2) yielded some understanding of the magnitude of seismic deficiencies when subjected to the actions of the criteria ground motion. The analysis quantified the seismic deficiencies in terms of required strength over expected strength (demand versus capacity ratios – DCRs) for each member of the seismic load resisting system. It did not establish a sufficient basis for priorities and strategies to implement seismic improvements. When the evaluation was complemented with concept retrofit designs that were accompanied with estimates of the construction cost for their implementation, full appreciation for the magnitude of building deficiency was obtained.

In a number of cases, the cost or retrofit was estimated to approach the cost of replacement. If funding is available, but the seismic evaluation estimates upgrade costs are high, it might be worthwhile to consider demolishing and rebuilding the structure, especially if functional improvements of the building are due. This finding is not new. Following the devastating effects of the 1971 San Fernando earthquake in several Veteran Administration (VA) hospitals, the VA undertook a program aimed at evaluating the seismic condition of 280 buildings and 26 hospitals all over the United States and at determining ways to strengthen deficient structures (Lefter [6]). The majority of the buildings, constructed from the turn of the twentieth century to 1972, had not been originally designed for earthquake loads. In many instances, the VA determined it was more appropriate to abandon obsolete buildings than to reinforce them.

In its examination of costs of reinforcing existing Los Angeles school buildings and construction of new schools to meet earthquake codes, a study conducted at MIT by Larrabee and Whitman [7] concluded there were some similarities between the retrofit cost evaluation trends displayed by the VA buildings and the Los Angeles school buildings that were evaluated. The study recommends that when the retrofit cost of a school building exceeds 70 to 80 percent of the cost of a new building, the school should be replaced.

The evaluation offered no definitive valuation as to the capability to withstand more frequently expected earthquakes of lesser intensity. Evaluation to a lower standard may serve as a useful tool in targeting deficiencies to be corrected in the first stage of a multistage program, especially if such upgrades were identified as integral to the seismic upgrade planned to resist a major earthquake. In this manner, a priority for upgrade measures could be established. Immediate life-safety improvements could be realized at a relatively low threshold cost with the implementation of intermediate retrofit.

The improvement of life safety is but one facet of the problem to those charged with the stewardship of large facilities. It has been learned with experience at other large essential facilities that seismic upgrades are most cost-effective when combined with other projects having the objective to improve functional capability. Thus, rather than competing for attention, functional improvement becomes an enabler for seismic improvement. This broad perspective can be the basis for a comprehensive program to sustain, refurbish, and modernize the buildings.

Potential strategies for facilitating the economical improvement of life safety versus seismic hazard, given in the order thought to be most cost-effective, are as follows.

- Demolish obsolete buildings that no longer efficiently serve the scope of the facility. If the building to be demolished is historic, document the historic features as appropriate and preserve in a manner suitable to be enjoyed by future generations, such as implementing historic architectural features into the new buildings.
- Develop space immediately adjacent to structure to be upgraded with a new structure sufficiently strong to laterally support the existing one in book-end fashion.
- Perform seismic upgrade in conjunction with an enabling project that improves the functional capability of the facility.
- Complete upgrade in phases so that relatively economical improvements that make some improvement of life safety can be implemented early.

CONCLUSION

Thirty-two buildings were subjected to a seismic evaluation by means of the two-tiered procedure according to FEMA 310 [1]. The Tier 2 analysis confirmed the deficiencies identified by the relatively simple Tier 1 screening, thereby confirming that the screening procedure is a powerful tool to qualify the seismic deficiencies of the buildings. The Tier 2 analysis quantified the deficiencies in terms of strength of the seismic load resisting system elements. However, the evaluation did not provide a satisfactory basis for the development of upgrade priorities and implementation strategies without supplementing the Tier 2 analysis with studies on upgrade concepts with affiliated cost estimates. Moreover, the conducted evaluation did not offer alternative solutions that consider partial upgrades or less severe performance goals to optimize upgrade works for the available funding.

The evaluation of the building stock considered suggests that the cost for seismic upgrade can be substantial, to the extent of surpassing the cost of that for new construction. The costs for upgrade are mainly a function of building age and are rather proportional to the building area. Contrary to popular belief, whether a building within the same age group is listed as historic or not, seems not to influence the upgrade costs. Similarly, a larger building does not seem to be less expensive on a square foot price base than a smaller building. Complete seismic upgrade of large buildings, however, may become impossible under limited funding. Old buildings with any amount of unreinforced masonry walls and old steel braced or moment frame buildings were the most expensive to upgrade. Unreinforced masonry walls need substantial work to be strengthened and secured, while steel braced frames and moment frames require significant foundation work to withstand newly developed overturning moments.

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