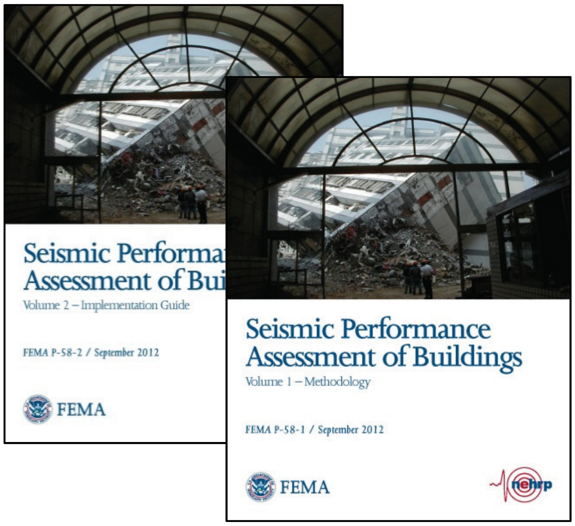

Background Document

FEMA P-58/BD-3.9.12

Fragility of Non-Structural Components

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Background Documentation

FEMA P-58 Background Documents are a series of reports documenting the technical background and source information for key aspects of the FEMA P-58 methodology and its implementation. These reports were developed over the course of the 10-year ATC-58/ATC-58-1 Projects funded under FEMA Contracts EMW-2001-RP-0056 and HSFEHQ-06-D-1105.

Background Documents were developed by consultants, serving at various levels within the project hierarchy, reporting the results of: (1) decisions on technical development protocols; (2) focused studies on the development of key aspects of the methodology; (3) documentation of recommended procedures; and (4) collection of available data for the development of structural and nonstructural fragilities. They were initially intended to serve as a record of the technical state-of-knowledge at the time they were produced, and as resources for the development of the eventual project reports. As such, they represent a snapshot in time, and may, or may not, match the technical content, recommended procedures, or data incorporated into the final methodology and its implementation.

This Background Document is intended for the purpose of providing supplemental knowledge to users of the FEMA P-58 methodology. Information contained herein has not been independently verified for accuracy as a stand-alone document, and may have been superseded in its final implementation within the methodology. Specifically in the case of certain nonstructural component fragilities, the NISTIR fragility classification numbering scheme was modified over the course of the project, and the fragility classification number assigned in this document might be different from numbers assigned in the final fragility database. Users of information in this document assume all liability arising from such use.

The fragility data contained in this report are based on the author's methodology for performing probabilistic assessment, and reflect the author's opinion on expected performance of nonstructural components constructed in accordance with new and existing seismic code requirements. These data were not consistent with the technical basis of the methodology, and the views expressed herein do not necessarily reflect the collective opinion of the ATC-58 Project Team. Data extracted from this report for implementation in the methodology have been substantially adjusted to account for these differences (e.g., peak ground acceleration adjusted to peak floor acceleration; and separation of ground motion dispersion from total dispersion). Readers are cautioned regarding the use of raw data from this document in FEMA P-58 seismic performance assessments.

Notice

Cover illustration – Primary resource documents for the FEMA P-58 *Seismic Performance Assessment of Buildings, Methodology and Implementation* series of products: FEMA P-58-1, *Volume 1 – Methodology*, and FEMA P-58-2, *Volume 2 – Implementation Guide*.

ATC 58
Fragility of Non-structural Components

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1.0 Introduction

This report provides seismic fragility functions for non-structural components found in common commercial facilities. The fragility functions are meant to be used for considering cost-effectiveness when doing performance based design and/or benefit cost analyses. The functions are applicable for estimating earthquake losses over a large quantity of similarly-installed components.

Section 2 presents the fragility functions for 17 classes of components. The development of the fragility functions for each of the 17 classes of component is cross referenced to separate sections in Appendices A.1 through A.17.

Section 3 addresses background issues covering code Importance factors, code design methods for equipment, general vulnerabilities of non-structural equipment, operational qualification, and guidelines for seismic design for piping, raceways, conduits and HVAC ducts.

Section 4 addresses inventory issues for non-structural components.

Section 5 summarizes available FEMA software that can address seismic performance of non-structural components.

Section 6 presents references.

Appendix A provides the derivations of the fragility functions for each class of components. Appendix A.18 addresses the use of empirical datasets for establishing fragility.

1.1 Key Findings

Fragility functions are provided for 17 classes of non-structural components. For most components, the functions are derived as a function of input peak ground acceleration or peak floor acceleration. For windows, the fragility functions are derived as a function of interstory drift ratios which depend on the style of building the windows are used in, but also listed in terms of PGA.

The fragility functions are presented with six parameters for each damage state. The six parameters are:

- Median acceleration (either PGA or floor ZPA) where 50% of the components would reach the damage state. The median acceleration value is based on M 6.5 to M 7.2 earthquakes in California; adjustments to alter the median value to factor in long duration subduction zone earthquakes or short duration earthquakes is discussed.

- Lognormal beta dispersion. The beta value is listed as a single value to include ground motion randomness and component uncertainty. The beta value is based on M 6.5 to M 7.2 earthquakes in California; adjustments to alter beta to factor for long duration subduction zone earthquakes is discussed.

- Life safety. Given the occurrence of the damage state, one of five life safety outcomes is provided: none, minor, major, life threatening if left untreated, instant fatality.

- Function. Given the occurrence of the damage state, one of three functional outcomes is provided: component remains functional; component is non-functional; component has an intermediate functional status.

- Short term repair cost. Given the occurrence of the damage state, the repair cost (as a proportion of replacement value) to make short term bypasses or emergency repairs to restore functionality.

- Long term repair cost. Given the occurrence of the damage state, the repair cost (as a proportion of replacement value) to make long term permanent term repairs to restore the component to its pre-earthquake condition.

1.2 Limitations

The findings in this report are meant for earthquake planning purposes. Nothing in this report should be construed as recommendations for any specific design. G&E Engineering Systems Inc., ATC, and the authors and reviewers of this report take no responsibility of any sort for any use, re-use, or derivative works based on the information in this report.

No warranty is offered with regard to the results, findings and recommendations contained herein either by the Federal Emergency Management Agency; the Applied Technology council, its directors, members or employees; G&E Engineering Systems Inc., its directors, employees of subconsultants. These organizations and individuals do not assumed any legal liability or responsibility for the accuracy, completeness or usefulness of any of the information, product or processes included in this publication.

It is intended that the data in this report can be used to evaluate the majority of commonly found non-structural components in a common building. Common buildings include: single family residential, multi-family residential, commercial (office, warehouse), hospitals, schools, and universities. Specialty installations (power, communication, water treatment, petrochemical, nuclear, etc.) are often designed to more rigorous seismic standards, and include more complex styles of equipment; with suitable attention, the fragility functions in this report can still be used for many items in such facilities, for items with similar quality.

By using suitable mathematical models, the fragility functions can be used to compute the chance of failure at any input ground motion level. If one does this computation, then one will show that there is chance of failure of almost any component should the design basis earthquake occur. Owners of facilities take on the entire responsibility for such failures. If the owner wishes to have very low or nearly zero chance of failure, then the owner must adopt more rigorous seismic standards than embedded in common codes like the UBC 1997, CBC 2001 or IBC 2006 or variations thereof such as used for critical care hospitals in California. If the owner truly wishes to have nearly zero chance of failure given the design basis earthquake, then the owner should adopt the standards used for U.S. commercial nuclear power plants, and recognize that the additional effort and cost will be involved.

Bias. Every attempt has been made to provide "unbiased" fragility functions, meaning that they should represent "reality". However, the author recognizes that some amount of bias has undoubtedly crept into the fragility functions, tending towards over-prediction of damage at lower input seismic motions. Most owners are interested in a "safe" design, especially when life safety or other economic factors are involved. Most engineers wish to have conservatism built into the design. Where bias has resulted in fragility levels that are too low, such bias will artificially increase benefit cost ratios and the perception of cost-effectiveness (perhaps not ideal for decision making); but also will increase actual safety levels to higher than assumed (perhaps a good thing).

Code and Function. By examining the fragility levels in Table 2-1, one observes that some equipment have serious damage states leading to functional impact with median failure rates of PGA = 0.50g or lower. For example, this is the case for elevators and suspended ceilings. The empirical data shows high damage rates at levels of shaking (PGA < 0.50g) equal to or less than assumed in design. This shows that there is some inadequacies in the current (or then current) codes, design practices, methods of installation, etc. It is not usually the intent of designers to tell owners that there is a high likelihood (perhaps 50% or higher) that the equipment will fail in the design basis earthquake. If the designer of a new facility truly wants high reliability that the equipment will function in the design basis earthquake, then the designer should adopt stricter codes and guidelines than the UBC 1997, CBC 2001, or IBC 2006; instead, we would recommend adoption of the ASME code for pressure pipe, IEEE 344 (or qualification by experience) for equipment with shaking-sensitive issues, and use of R=1 (or not much more than 1.5) when qualifying most types of equipment (i.e., nearly elastic design). Except where specifically addressed in this report, the fragility levels *do not* correspond to equipment specifically designed to these higher standards and guidelines.

The fragility functions represent the state of the authors' knowledge as of early 2009. As more earthquakes occur, and as more components are tested on shake tables, the state of knowledge will expand. No doubt, there can be improvements to the fragility functions in this report based on better and new information. Updates in the fragility functions will be needed to address component-specific installation issues and new developments.

1.3 Acknowledgements

This work was prepared under subcontract to the Applied Technology Council, Mr. Jon Heintz Project Manager.

The support and cooperation of ATC is gratefully acknowledged.

This report was prepared by G&E Engineering Systems Inc. by Mr. John Eidinger. Portions of the material presented herein were developed by Prof. Anshel Schiff. Mr. Bruce Maison provided valuable insight about performance based design.

Some of the findings reflect the observations made over the course of many post-earthquake investigations, beginning with the 1906 San Francisco California earthquake to as recent as the 2008 Wenchuan China earthquake. Bob Reitherman (1985) authored an excellent book on the seismic evaluation and upgrade of non-structural equipment, which has since been updated and re-published by FEMA (1994); it remains an excellent reference with a lot of practical guidance for many classes of non-structural components. ASCE's Technical Council for Lifeline Earthquake Engineering has been collecting and reporting on the seismic performance of equipment in lifeline facilities for more than 30 years. The information presented in this report builds upon the dedicated effort made by so many people to find out what "works" and what "does not work" after real earthquakes. The author expresses gratitude to the tireless efforts by Mr. Le Val Lund, Mr. Alex Tang, Mr. Curt Edwards, Prof. Anshel Schiff, Mr. Mark Yashinsky, Ms. Therese Elliott, Mr. Bill Elliott, Mr. Dennis Ostrom, Mr. David Lee, Mr. Del Shannon, Mr. Tom Cooper and many others who have participated in the collection of post-earthquake information about equipment and components used in utility and lifeline infrastructure.

1.4 Abbreviations and Definitions

A	Acceleration, horizontal median level to reach a damage state, g
ap	amplification factor for non-rigid components per code (normally 2.5)
ALA	American Lifelines Alliance
ASCE	American Society of Civil Engineers
ASME	American Society of Mechanical Engineers
Beta	Standard deviation of the natural log of a variable
BCA	Benefit Cost Analysis
BCR	Benefit Cost Ratio
C	Response Spectral Amplification relative to PGA
Ca	Free field horizontal peak ground acceleration, per code
CBC	California Building Code
DS	Damage State

F	Functionality code (= 0 functional, = 1 non-functional)
F, Fp	Force
FEMA	Federal Emergency Management Agency
g	Acceleration due to gravity = 32.2 ft/sec^2
G&E	G&E Engineering Systems Inc.
H	Height of block
HVAC	Heating, Ventilation and Air Conditioning
I, Ip	Importance factor (I for buildings, Ip for equipment)
IBC	International Building Code
km	Kilometer
LR	Long Term Loss Ratio (= 0 = 0% of replacement cost; = 1 = 100% or replacement cost)
LS	Life Safety code (=1 none, =2 minor, = 3 major, = 4 fatality if not treated, = 5 instant fatality)
M	Moment Magnitude
PGA	Peak Ground Acceleration, g. The maximum horizontal acceleration at the surface in the free field adjacent to a building.
PGD	Permanent ground deformation (inches)
PGV	Peak Ground Velocity (inches/second)
R, Rw, Rp	Code-based response modification factors (1 or higher) (R for buildings using strength design, Rw for buildings using working stress design, Rp for equipment)
RR	Repair Rate (per 1,000 feet)
SA	Value of Spectral Amplified (5% damping) portion of acceleration response spectra at the frequency of interest, g
SR	Short Term Loss Ratio (= 0 = 0% of replacement cost; = 1 = 100% of replacement cost)
UBC	Uniform Building Code
USGS	United States Geological Survey
V	Base Shear force
W	Width of block (narrowest direction); or Weight
Z	Code seismic zone factor. For Zone 4, Z = 0.4g
ZPA	Zero Period Acceleration, g. The maximum horizontal acceleration at a floor level on a multi-story building.

2.0 Fragility Functions

Table 2-1 presents the fragility data for a variety of non-structural components commonly found in commercial facilities.

Explanations of the fragility data in Table 2-1 are provided in Appendix A. Median A (g) is the zero period acceleration (ZPA) to damage 50% of the inventory. Beta Total is the lognormal dispersion parameter for the fragility curve, considering both uncertainty and randomness of the default assumed ground motion. Beta Equipment is the lognormal dispersion parameter for the fragility curve, considering only the uncertainty in the response of the equipment (wherever used in this report, except in Table 2-1 or otherwise specifically discussed, beta refers to Beta Total). By ZPA, we mean the peak horizontal acceleration input to the base of the equipment. If the equipment is mounted on a concrete slab at grade, then the ZPA is the same as PGA. If the equipment is located below grade, or at the roof level penthouse of a building, then adjustments need to be made to convert PGA to ZPA. For buried utilities, we recommend using PGV and PGD as the input predictor, and this is described in detailed in Appendix A.2.

In order to compute the probability of reaching a damage state, one must estimate the seismic demand. This report does not address how to compute the seismic demand, which will often include considerations such as magnitude of earthquake, distance of the facility to the causative fault, local soil amplification / deamplification, soil structure interaction, basin effects, fault rupture propagation, amplification of the motions through the structure to the location of the component, and component-structure interaction.

The user of the fragility data should be cognizant of which input motion that should be used. Specifically, should the input motion be the maximum horizontal motion in one direction, the maximum of two horizontal directions, or the orthogonal combination? There is no simple answer, and variations in these three can range by 20% or so. The following guidelines should be used.

- Where fragility data is based directly on tests, then the best predictor is the single axis motion that most severely loads the component.

- Where the fragility data is based on empirical observation, then the best predictor is the geometric mean of the two horizontal motions at the location of the component. This is true of the empirical data estimated the site motion using a single attenuation function that computes the geometric mean, and not based on a site-specific instrument.

- If the user is estimating motions using Shakemaps[1], then additional quirks are introduced. The Shakemap algorithms (as of 2008) show the highest acceleration from any of the three orthogonal instrument readings (including vertical). If the user can de-aggregate the three orthogonal Shakemap motions (such as by using the actual instrument recordings), then a better input motion be used.

- If the user selects the worst direction motion (time history orthogonal combination of two horizontal motions), then in general the fragility functions will tend to over predict damage.

- Usually, the variations in accelerations and fragility functions to account for these factors will result in damage estimates that are perhaps ±20% from ideal. For most practical cases, an estimate of damage that is within ±20% from ideal is well within the tolerances to make effective decisions about performance based seismic design. For example, if one uses the fragility functions in a benefit cost analysis[2], and finds the BCR to be 5, then the decision to upgrade is clearly a good one; similarly, if the BCR is 0.2, then the decision to upgrade is clearly a poor one; if the BCR is computed to be near 1, then the decision to upgrade / not upgrade is a close one.

Damage State	Median A (g)	Beta Total	Beta Equipment	Notes	Reference
Fire sprinklers – rate of functional damage per 1,000 feet				Field hung	A.1-4
Fire sprinklers - rate of functional damage per 1,000 feet				ASME Design	A.1-4
Buried utilities - damage per 1,000 feet				Based on PGV, PGD and type of pipe	A.2
Windows – edge cracking				144 Curves	A.3
Windows – glazing damage				144 Curves	A.3
Windows – major cracking without fallout				144 Curves	A.3
Windows – glass fallout				144 Curves	A.3
Suspended Ceilings - moderate damage	0.50	0.50	0.30	Wire hung	A.4-2
Suspended Ceilings - extensive damage	0.90	0.50	0.30	Wire hung	A.4-2
Suspended Ceilings - moderate damage	0.80	0.50	0.30	Add compression struts	A.4-2
Suspended Ceilings - extensive damage	1.30	0.50	0.30	Add compression struts	A.4-2
Elevators – Generic	0.40	0.50	0.30	Model 1	A.5.7
Elevators – No seismic design	0.35	0.50	0.30	Model 2	A.5.7
Elevators – With seismic design	0.90	0.50	0.30	Model 3	A.5.7
Escalators				Facility specific	A.5.5
Raised Floors – No Seismic Design	0.50	0.60	0.45		A.6-1

[1] Shakemaps are maps prepared by the USGS soon after earthquakes, computed using actual ground motion instrument data, coupled with regional soil maps and attenuation models to fill in motions at locations between the instruments.

[2] The methods to perform benefit cost analyses and to compute benefit cost ratios (BCRs) is not provided in this report. See FEMA (2006) for a comprehensive treatment of how to compute these values.

Damage State	Median A (g)	Beta Total	Beta Equipment	Notes	Reference
Raised Floors – Limited Seismic Design	0.70	0.60	0.45		A.6-1
Raised Floors – Basic Seismic Design	1.50	0.50	0.30		A.6-1
Raised Floors – Full Seismic Design	3.00	0.50	0.30		A.6-1
HVAC Ductwork - Building - extensive Damage	1.25	0.54	0.36	Rod hung	A.7-1
HVAC Ductwork - Building - complete Damage	1.88	0.54	0.36	Rod hung	A.7-1
HVAC Ductwork - Building - extensive Damage	2.38	0.54	0.36	With sway braces	A.7-1
HVAC Ductwork - Building - complete Damage	3.00	0.54	0.36	With sway braces	A.7-1
HVAC Ductwork - Penthouse - extensive Damage	0.50	0.54	0.36	Rod hung	A.7-1
HVAC Ductwork - Penthouse - complete Damage	0.75	0.54	0.36	Rod hung	A.7-1
HVAC Ductwork - Penthouse - extensive Damage	0.96	0.54	0.36	With sway braces	A.7-1
HVAC Ductwork - Penthouse - complete Damage	1.50	0.54	0.36	With sway braces	A.7-1
HVAC Vibration-isolated rotating equipment	0.50	0.50	0.30	No seismic design	A.7-2
HVAC Vibration-isolated rotating equipment	1.50	0.50	0.30	With seismic design	A.7-2
Bottom heavy equipment items	0.75	0.50	0.30	Unanchored	A.8-1
Evenly weighted equipment items	0.60	0.50	0.30	Unanchored	A.8-1
Top heavy equipment items	0.40	0.60	0.45	Unanchored	A.8-1
Bottom heavy equipment items	0.90	0.50	0.30	Poorly anchored	A.8-2
Evenly weighted equipment items	0.75	0.50	0.30	Poorly anchored	A.8-2
Top heavy equipment items	0.50	0.60	0.45	Poorly anchored	A.8-2
Well anchored equipment items	1.50	0.50	0.30	Standard anchors	A.8
Pendant Light, non-seismic	0.60	0.50	0.30		A.9-1
Pendant Light, with restrainer clips	1.10	0.50	0.30		A.9-1
Pendant Light, seismic design	1.50	0.50	0.30		A.9-1
Overhead cranes				Facility specific	A.10
Rigid Block, W/H = 0.33, unanchored	0.60	0.50	0.30	Toppling	A-11
Rigid Block, W/H = 0.33, common anchorage	1.50	0.50	0.30	Toppling	A-11
Rigid Block, W/H = 0.33, heavy anchorage	3.00	0.50	0.30	Toppling	A-11
Office workstation	1.00	0.50	0.30	Unanchored	A.12-1
Diesel Generators – seismically qualified	1.10	0.50	0.30		A.13-2
Diesel Generators - well anchored	0.60	0.50	0.30	Not in daily use	A.13-2
Diesel Generators - seismically vulnerable	0.25	0.60	0.45	Not in daily use	A.13-2
Electrical Cabinet - Unanchored	0.60	0.60	0.45		A.14-1
Electrical Cabinet - Nominally Anchored	1.00	0.60	0.45		A.14-1
Electrical Cabinet - Well Anchored	3.00	0.60	0.45		A.14-1
Communication Rack – Unanchored	0.20	0.60	0.45		A.14-2
Communication Rack - Flexible	1.00	0.60	0.45		A.14-2
Communication Rack – Well anchored	1.50	0.60	0.45		A.14-2
Top heavy equipment on rollers	0.40	0.50	0.30	Rolling	A.15
Top heavy equipment on rollers	0.60	0.50	0.30	Toppling	A.15
Mechanical equipment, no anchorage	0.60	0.60	0.45	Breaks attached pipes	A.16
Mechanical equipment, light anchorage	0.70	0.50	0.30		A.16
Mechanical equipment, heavy anchorage	2.00	0.50	0.30		A.16
Storage Rack – Loose items slide to floor	0.30	0.60	0.45		A,17-1
Storage Rack – Pallets slide to floor	0.70	0.60	0.45		A,17-1
Storage Rack – 27-11 design, Collapse, W < 0.40	0.90	0.50	0.30		A,17-1

Damage State	Median A (g)	Beta Total	Beta Equipment	Notes	Reference
Storage Rack – 27-11 design, Collapse, W > 0.60	0.60	0.50	0.30		A,17-1
Storage Rack – High seismic design, Collapse	1.50	0.50	0.30		A,17-1

Table 2-1 Fragilities for Equipment

There are many assumptions incorporated into Table 2-1. The discussions in the corresponding sections in Appendix A provide additional information as to the assumptions and limitations.

The fragility data in Table 2-1 should *never* be used for evaluation of a specific piece of equipment, and should *only* be used as approximations of seismic performance of a great number of pieces of equipment within a category, considering the usual range of installation configurations.

The fragility data in Table 2-1, coupled with suitable loss models, *could* be used as a first order estimate for purposes of benefit cost analyses and performance based design.

With few exceptions (noted in Appendix A), the fragility data in Table 2-1 is for the functional performance of the equipment. Unless otherwise indicated in the reference appendix, minor damage (stretching of anchor bolts, rocking without failure, sliding without failure) is generally not included in Table 2-1.

Life safety or injury implications of reaching the functional damage states is not included in Table 2-1. In most cases, the potential for injury is very slight, although exceptions occur.

The cost to procure and install equipment is generally not addressed in this report, although a few examples are provided in Appendix A.

In general, the time needed to make repairs should the above damage states be reached, will be proportional to the total damage, the total size and ramp-up of repair crews, and the priority assigned to making repairs to individual pieces of equipment. Therefore, this report makes no assertions as to "standard" repair times, as these variables are always case-specific.

The cost to make repairs should the above damage states be reached, is in general *not* proportional to the replacement value of the equipment. In some cases, the damage might cause functional outage of a $500,000 piece of equipment, and yet take only $200 for repair; in other cases, it might take $500,000 to repair. Appendix A discusses this in some detail. For application in a performance based design application, it should be the end-user that assigns the cost to make repair for each component, based on facility-specific application.

2.1 Life Safety, Function, Repair

Given the occurrence of any of the damage states, there will be additional impacts. We include four impacts: Life Safety, Function, Short Term Repair and Long Term Repair. All these functions are presented as deterministic, meaning that given the damage state occurs, then the impact occurs. This is an oversimplification, as given the damage state, occurs, there will be a range of possible outcome ranging from none to severe; such refinements are left to the end-user to implement on a case-by-case basis.

For Life Safety, the expected states are as follows. The meaning of the life safety states is the same as used n HAZUS (none, minor, major, possible fatality, instant fatality), with examples below.

- LS = 1 means that that is almost no chance for injury. For equipment that is located in normally non-occupied rooms, this is the default life safety value for many installations. For example, toppling of a housekeeping cabinet in a sump room would have almost no chance of injuring anyone, as the room is rarely occupied.

- LS = 2 means that there is a reasonable chance of causing minor injury or creating some modest egress impediment in normally occupied rooms. LS = 2 might be assigned to small lightweight components that might topple in normally occupied areas, such as lightweight countertop items that fall to the floor. Common injuries might be cuts.

- LS = 3 means that there is a reasonable chance of causing significant injury (or slight chance of causing fatality) should the component fail. For example, if a heavy sign over an occupied area falls, there is a reasonable chance that a significant injury will occur; or even a slight chance of fatality. A significant injury could include a broken bone.

- LS = 4 means that there is a significant chance of causing fatality should the component fail. If the person is not treated rapidly (trauma care, etc.), the person will die.

- LS = 5 means instant fatality. This will rarely be the case for damage to nonstructural equipment.

The number of people exposed to the life safety state will depend on specific application.

Functionality. Functionality means whether or not the non-structural component is functional. For a motor control center, this means whether the motor control center can continue to provide its primary function, given the damage state occurs, and prior to any post-earthquake repair efforts. Whether or not a building / facility will function is not addressed in the fragility data in this report, but instead would have to be assessed based on the accumulative damage to all structural and non-structural components, in consideration of redundancy and other factors.

- F = 0. Item remains functional.

- F = 0.5. Item has partial functionality. For most non-structural items, this damage state does not apply.

- F = 1.0 Item is non functional due to the damage. Loss of function due to loss of power is not included in this damage state, and would have to be factored into the overall assessment separately.

Short Term Repair. This is the cost (effort) to clean up the damaged component or make temporary bypass fixes to the component to restore functionality of the equipment. For example, should a battery topple and spill acid, then the short term cost would include the effort to clean up the site and remove / contain the acid spill, plus whatever actions are taken to restore power to the system For a suspended ceiling, the short term repair cost might be the removal / cleanup of fallen tiles.

The short term repair cost is presented as a faction of the replacement value of the item. Depending on application, it might be better to provide the short term repair cost as an absolute dollar amount. For example, replacing a battery for a small generator (value $20,000) might be $200 (0.01), or for a large generator (value $500,000) might be $250 (0.0005) as the actual labor involved is the same for both cases, and the size of the battery is similar in both cases. Since this report is written to be as generic as possible, a ratio is provided, and it is recommended that the end user refine the ratio for the actual piece of equipment used. In developing the ratios, we imply a certain amount for labor costs ($130 per hour), which represents the direct labor costs, overhead, benefits, small parts and equipment charges. In this report, we do not distinguish between higher cost areas of the country (say New York City) versus lower cost areas (say Alabama). The baseline costs are geared to large cities in coastal California (San Francisco or Los Angeles), and the user may adjust the costs up or down based on regional factors provided in HAZUS.

Long Term Repair. This is the cost to make permanent fixes to the equipment to restore the equipment to its pre-earthquake condition and functionality. By "pre-earthquake condition" it is meant that the repairs will restore the damaged component to its pre-earthquake condition, recognizing the intent of then-current (i.e., at the time of the earthquake) codes. For example, if a battery rack fails and batteries topple and spill acid, the long term cost will be to remove the old equipment and install a new battery rack. The long term repair costs assume that the equipment was functionally damaged.

For most non-structural components, the damage state does not imply a 100% repair effort: for example, if a SCADA electrical cabinet topples and is non-functional, the short term repair cost might be to institute some type of bypass (say manual reading of telemetry data instead of automatic reading via SCADA), and the long term repair cost is to repair the damaged item.

Restoration time. In many loss estimation models, a "repair time" is provided that relates percentage of function available with time after the earthquake. In general, these functions are based on judgment as to the general state of affairs of the individual facility and to the region as a whole, coupled with assumed ramp up times of repair crews, availability of mutual aid, availability of spare parts and similar. For purposes of this report, we provide more basic information about the required effort for repair (= short term repair plus long term repair), but we make no assumption about prioritization of repairs or the availability of repair crews. Therefore, in order for a restoration time to be established, the end user needs to do the following:

- (A) Add up all the repair costs.

- (B) Assign how many people / parts / equipment will be available (this is application specific). Most often, this will be controlled by the availability of people, but sometimes equipment or parts will control. If one assumes the restoration is controlled by the availability of people, then if one assumes an average hourly cost of $130 per hour per person, then the number of repair-person-hours required is simple (a) / $130 per hour.

- Divide A by B. This will give the time needed to complete all repairs.

- To establish restoration of partial functionality, the end user needs to decide which items get repaired first, which get repaired second, and so on; and then assign functionality based on what percentage of function is restored as any given time after the earthquake.

2.2 Fragility, Damage Algorithms and Loss Estimates

The terms "Fragility", "Damage Algorithms" and "Loss Estimates" are often used interchangeably, and sometimes without precise meaning.

For purposes of this report, we define "fragility" as follows:

- Fragility. For a given input PGA (SA, PGV, PGD), what is the probability of reaching a particular damage state (individual items) or what is the expected repair rate (number of failures per 1,000 feet or pipe)

For purposes of this report, we define "damage algorithm" as follows:

- Damage Algorithm. For a given PGA (SA, PGV, PGD), what is the expected loss. Usually, the loss is represented as the long term repair cost, but the short term repair cost, number of casualties (none, minor, major, severe, life threatening, instant fatality), the expected function can also be derived.

The mathematics of fragility functions and damage algorithms is described in HAZUS. We use the same definitions here.

A Damage Algorithm is readily calculated from a Fragility Function as follows. For example, select a range of input PGAs (for many FEMA-sponsored benefit cost analyses, commonly set at PGA = 0.06g, 0.12g, 0.24g, 0.435g, 0.675g, 0.90g and 1.20g). Then, calculate the probability of reaching or exceeding each damage state at each of these PGA levels.

Thus, while ATC 58 provides various computational software programs, the end user can always plug in the fragility functions described in this report into EXCEL, customize any particular variable as suitable for the project at hand, and derive the same results. The software provided (for free) from FEMA does the exact same calculations.

2.3 Earthquake Duration

All fragility information presented in this report is for an assumed M 6.5 to M 7.2 crustal earthquake in California. This presumes strong shaking duration (PGA over 0.05g) between about 8 to 20 seconds. Adjustments to the fragility levels must be made for longer duration earthquakes (such as M 8 San Andreas or M 9 Cascadia Subduction Zone) or in locations where local soil conditions allow amplification of duration of motion.

It is beyond this scope of this report to provide duration-sensitive estimates of the hazard.

Long duration earthquakes. As a first order simplification, the fragility levels (PGA levels) presented in this report should be reduced by 5% for a M 8 event to 15% for a M 9 events for components with ductile failure modes. A more detailed adjustment could be done by examining the failure mode (brittle or ductile); for brittle, no adjustment should be made; for ductile the adjustment should consider the accumulation of damage over multiple cycles in order to assess the effect of duration on high strain – low cycle fatigue failure modes.

Short duration earthquakes. M 5.5 to M 6.4 earthquakes might produce only 2 to 10 seconds of strong ground shaking (or so). Therefore, the fragility levels in Appendix A will tend to over predict damage for equipment with duration-sensitive (ductile) failure modes. For example, unanchored equipment on countertops may slide less in short duration events than in longer duration events, even with the same peak PGA and PGV values. As a first order simplification, the fragility levels (PGA levels) presented in Appendix A should be increased by 5% (M 6) to 10% (M 5.5) events for components with ductile failure modes.

2.4 Uncertainty and Randomness

The terms uncertainty and randomness are often used in conjunction with assessing the probability of failure. In this report, we define these terms as follows:

- Randomness. The variation in the free field ground motion. Commonly, it is assumed that there is nothing that the seismologist or engineer can do to reduce randomness. In practice, we accept a certain amount of randomness into the design process.

- Uncertainty. The variation in response of a specific piece of equipment that can be eliminated if the engineer resolves all the variables with accurate information. For example, uncertainty is introduced by not knowing the precise amplification of ground motion to the location of the piece of equipment by using simplified code formulae (but could be reduced with more accurate structural models); not knowing the precise mass of the equipment; not knowing the precise yield strength of the material, etc. In concept, all these uncertainties can be reduced to nearly zero if the time and expense and expertise is taken to collect / develop more accurate information. In practice, we accept a certain amount of uncertainty into the design process.

Throughout this report, where we list "beta", we mean a lognormal standard deviation. A single beta is used to factor in randomness (in the ground motion) and uncertainty (in the performance of the equipment).

For most damage states, we imply beta (ground motion) of about 0.40 and beta (equipment) of about 0.30, or a total beta of 0.50. The total beta is computed as follows: beta (total) = SQRT (beta (ground motion)**2 + beta (equipment)**2).

For damage states where we have increased uncertainty, we imply beta of 0.45 for the equipment, or a total beta of 0.60.

For Cascadia Subduction Zone earthquakes, beta (ground motion) is likely higher than 0.40, and the beta (total) values in this report should be increased accordingly.

For small magnitude earthquakes ($M < 6$), beta (ground motion) is likely higher than 0.40, and the beta (total) values in this report should be increased accordingly.

For cases where the design engineer does not have an accurate inventory (no field survey of actual equipment), then the true beta is so large as to make for huge chance of error. For example, it is typically not reasonable to apply a fragility for anchored equipment for equipment that is actually not anchored or vice versa; if one does so, then "garbage-in = garbage-out".

It is becoming "in-vogue" to use the terms "aleatory" and "epistematic" to describe "randomness" and "uncertainty", respectively. Aleatory is derived from the Latin word *aleatorius*, and means "depending on the throw of the dice, random". Epistemic is derived from the Greek word *episeteme*, meaning "knowledge". The use of aleatory and epistemic was made popular by Prof. Allin Cornell. In this report, we use common English words, and we call beta (earthquake) to describe all things "random" and beta (equipment) to describe all things "uncertain". Both the estimates of the earthquake motion and the equipment response have random and uncertain features, and the use of infrequently-used Latin and Greek words instead of English words has as much merit if the reader understands what they mean.

2.5 Form and Tails of the Distribution

The form of the fragility distribution functions is selected as lognormal. Why lognormal? Two reasons:

- First. It is computationally convenient. By convenient, we mean that whatever the mean and beta, the probability density function is defined only for positive values of the independent variable (A, PGA or ZPA).
- Second. There is some evidence that some of variables in real world are "lognormally distributed". However, there is no such evidence available for the bulk of the equipment and components covered in this report. Various researchers have suggested using bifurcated normal distributions for similar equipment (Dennis Ostrom), and other than the computational requirement to ignore the distribution in the negative space for the independent variable, there is no reason not to use such a distribution.

In reality, some types of equipment might be expected to have a "bathtub" type probability distribution function, where as a few style of equipment might be very fragile at low A, and the rest fragile at only high A. To some extent, by categorizing equipment into two classes (non-seismic / seismic, or unanchored / anchored), this report tries to mimic the bathtub curve. Reality is much more complicated, so the functions in this report should be considered only first-order estimates.

Other forms of the distribution functions can be selected. This will somewhat complicate the processing of the data in a computation model, but such additional computation work is generally trivial in terms of actual clock time needed to run a model.

In this report, we have used a single beta to reflect uncertainty and randomness as to the occurrence of the damage state. Should the damage stage occur, then we provide deterministic consequences (LF, F, SR, LR). Therefore, the variability in these four consequences is the same as the variability in the damage state. In reality, these four consequences also have independent randomness and uncertainty characteristics. Given the desire to create a general-purpose performance based design tool, we do not think that there is a point to adding in independent variables for the four consequence states. However, for a particular application of equipment in a particular building, the site-specific issues will control over the consequence states, and these should be adjusted accordingly, either by altering the deterministic consequence (first order modification), or adding in a dispersion parameter as suitable for each consequence (second order modification).

Whichever form of the fragility function is used, there are issues related to the low-sided tail of the distribution. For example, for one San Francisco Bay Area lifeline operator, we saw no evidence of any damage to more than 20,000 pieces of equipment in the Loma Prieta earthquake, exposed to PGA varying from 0.03g to 0.15g. Some of this equipment is not anchored. If we applied the fragility functions in this report to these 20,000 pieces of equipment, then a few will be expected to be damaged, as there is non-zero chance of failure at these low g values (under 0.15g). To resolve this issue, a simple modification to the fragility function is to "cut-off" the distribution at low g values. Depending on the item, the low g value could be selected at either: 75% of the lowest level observed with damage for similarly-installed equipment; the point on the curve corresponding to 1 chance in 1,000 of failure; or the median fragility divided by 4. We have not provided these values in this report, but we recommend that for any practical application, that the end user implement such a low-level fragility cut-off, to avoid spurious findings.

At the opposite end of the fragility curve, the high sided tail, there is less need for a cut-off. The practical reason is that for most applications, we are not interested in discerning whether 80%, 90% or even 100% of an equipment class fails at a particular g level, as no doubt this will imply poor design. Therefore, the accuracy needed at the high end of the fragility function is not of practical interest in most applications.

3.0 Design Approach

3.1 Importance Factors

The International Building Code (IBC) and Uniform Building Code (UBC) performance goals have been formulated with the intent of protecting building occupants from life safety hazards resulting from earthquake damage, as opposed to attempting to ensure post-earthquake operability of facilities. Structures and buildings built to the IBC / UBC may still suffer significant damage after an earthquake, such that they may not be able to perform their function. In some cases, resultant damage to IBC / UBC code-built buildings may be so significant such that repair of the building is not economical, and total demolition is warranted.

In a similar manner, nonstructural components designed to the seismic requirements of the IBC / UBC are intended only to provide "life safety" assurance. Such design to the

IBC / UBC does not necessarily provide assurance of continued functionality during or after an earthquake.

The IBC / UBC recognizes that some facilities, such as Emergency Operations Centers, Hospitals and similar "Essential Facilities" should have better performance such that they are available for use immediately following an earthquake. In order to accommodate this goal, the IBC / UBC specifies somewhat higher design force levels and more rigorous quality assurance measures during the design and construction process. The adjustment in force level is typically obtained with the application of an occupancy Importance, or "I" factor.

However, increasing the seismic base shear formula by 25% to 50% by applying the code-specific Importance factors, I, does not necessarily provide an increased chance that the nonstructural component will provide continued functionality if one uses "design by rule" concepts embedded in the IBC for certain kinds of equipment.

By specifying an I factor of I=1.25 or I=1.50, in effect the structure or nonstructural component is designed for 25% to 50% more seismic load than normal structures. Counterbalancing this I factor, the UBC / CBC / IBC codes allow the use of R, Rw or Rp factors to reduce the seismic load, and these factors can range from 3 to 8, depending on the style of construction. The R factors are meant to consider nonlinear / ductility capability of the component, and to offset other factors of safety that might be inherent in the design. For non-structural components, use of R values implies a level of distortion that might imply damage and increased drifts / displacements of the component. Applying R factors to pipes, ducts, cable trays, raised computer floors, suspended ceilings and other types of equipment is often incorrect, as the can be no ductile capability for some of the common joinery used in these components.

If it is the intent of the owner to have as little damage post-earthquake in its facility(ies) as possible, in order to provide continuous or nearly continuous service, the design professional must recognize that use of any R value greater than 1 will imply some type of yielding or possibly non-ductile behavior, and a corresponding reduction in equipment reliability.

It can be reasonable to adopt code-based R values, where the primary purpose of the design is to provide higher reliability for life safety or egress assurance. However, the designed must be aware that where the IBC and similar codes allow "design-by-rule" for non-structural components, that there is some unquantified chance that the component will fail, even when I is taken to be 1.25 (or higher). It is only when the designer specified the complete load path and verifies that this load path is suitable for the proscribed level of seismic loads, can there be assurance that the component will perform as intended.

However, for components that are needed for immediate post-earthquake function, R values should generally be set to 1 (i.e., to achieve elastic or near elastic support and anchorage performance, given the occurrence of a design basis earthquake.)

With these factors in mind, the designer could select criteria for specific projects that are intended to provide greater reliability for nonstructural components than would be obtained by straight application of the IBC / UBC / CBC and other similar standards, when such reliability is economically justified.

Depending upon the level of service required after an earthquake, each component should be assigned an "I or Ip" factor and "R or Rp" factor. Table 3-1 provides three groups of importance:

Equipment Component Class, I or Ip	R or Rp	Description	Performance Goal
Standard Ip=1.0	As provided by UBC / CBC code, but not to exceed 4	Storage racks, book shelves, desk top equipment not needed for immediate post earthquake operation, non-tunnel ventilation, suspended ceilings other than those over important operation areas, storage cabinets, housekeeping items (ladders), escalators and elevators.	Provide substantial life safety protection for major earthquakes likely to affect the site. Component may be damaged or non-functional and may not be economically repairable in the event of such an event.
Important, Redundant Ip=1.25	1.0	Battery racks for all redundant systems. Suspended ceilings over critical operational areas. Raised computer floors.	Provide substantial life safety protection for major earthquakes likely to affect the site. Provide reasonable assurance of near elastic performance during a major earthquake.
Critical, Non-Redundant Ip=1.5	1.0	Systems and components that provide essential services, including ventilation in tunnels, fire detection, fire suppression, flood control, communication systems with no redundancy and the failure of which results in an unacceptable level of service. All battery racks for non-redundant systems.	Provide substantial life safety protection for major earthquakes likely to affect the site. Provide high assurance of elastic performance during a major earthquake.

Table 3-1. System/Component Importance and Performance Goals

In areas of the United States where the 475-year motion has PGA of 0.20g or higher, then when using Table 3-1, the design basis ground motions need not be higher than the 475 year return period motion at the site. In areas of the United States where the 475-year motion has PGA of under 0.20g, then when using Table 3-1, the design basis ground motions need not be higher than 2/3 of the 2,475 year return period motion at the site. In areas of the United States where a substantial amount of the risk associated with the design basis earthquake is due to a M 8.5 to M 9 earthquake (coastal Oregon, Washington, California and Alaska, within 150 km of the leading seismogenic edge of an active subduction zone, for items that depend on ductile performance, the R values in the UBC / IBC should be reduced by 25% or by other rational method to account for long duration effects. In areas of the United States where a substantial amount of the risk associated with the design basis earthquake is due to a M 8 earthquake (within 25 km either side of the San Andreas fault, within 50 km of the New Madrid fault zone), for items that depend on ductile performance, the R values in the UBC / IBC should be reduced by 15% or by other rational method to account for long duration effects.

3.2 Design for Total Lateral Force

In order to achieve the reliability intended, nonstructural components and systems may be simply designed in accordance with Section 1632 of the 2001 California Building Code (CBC).

In using the following CBC equations:

$$F_p = 4.0 C_a I_p W_p \tag{32-1}$$

Alternatively, F_p may be calculated by the following formula:

$$F_p = \frac{a_p C_a I_p}{R_p}\left(1 + \frac{3h_x}{h_r}\right)W_p \tag{32-2}$$

Except that:

F_p shall not be less than $0.7 C_a I_p W_p$ and need not be greater than $4 C_a I_p W_p$ (32-3)

Where:

- C_a = is taken from Table 16-Q of the CBC. C_a represents the free field horizontal peak ground acceleration at the site.

- I_p is taken from Table 3-1 (above). The combination of C_a * I_p need not exceed the 84th percentile non-exceedance motion for the free field for the site, for the lower of the 475-year return period earthquake (higher seismic areas with PGA (475 year) $\geq$ 0.20g) or 2/3 of the 2,475-year return period earthquake (lower seismic areas with PGA (475 year) < 0.20g.)

- a_p is taken from Table 16-O of the CBC, or may be calculated considering suitable dynamic analysis procedures.

- R_p is from Table 16-O of the CBC, except = R_p = 1.0 for any item with I_p = 1.25 or higher unless the designer can show that the component will remain functional with R > 1.

Any nonstructural component and its supports may be designed using dynamic analysis and strength of material methods that are consistent with the ASME B31.1 code for pressure pipe.

The formula in equation (32-2) is known to almost always (but not always) over predict in-structure amplification. Recognizing this, new versions of the IBC code (2006) have reduced the "3" amplification factors to "2". However, the designer must be cognizant that the simplified building amplification procedure in equation 32-2, whether using "3" or "2" is a gross simplification, and introduces substantial uncertainty into the actual motion that a component will experience. For application of the fragilities in this report, it is assumed that the designer will not use equations (32-1, 32-2, 32-3), and instead calculate in-structure amplifications at the support point of components, using rational methods (considering three orthogonal directions of motion), when coupled with suitable margin for uncertainties in mass and stiffness of the structure, and component-structure mass interaction where significant. If the designer instead adopts equations 32-1, 32-2 and 32-3 (or their similar proxies in simplified codes), then additional margins (in some cases, non-conservative) and uncertainties are introduced, the extent of which is building specific, and not included in any of the fragility functions in this report.

3.3 Vulnerabilities of Equipment

The IBC and similar codes proscribe a seismic anchorage force for equipment using methods similar to those described in Section 3.2. However, the designer should be aware of the common vulnerabilities of equipment, and consider those in the process of component selection and design.

Some equipment components are inherently rugged and maintain structural integrity and functionality during and after earthquakes. For these components, little special seismic design needs to be paid to these elements. On the other hand, some equipment components are vulnerable to seismic hazards, and appropriate design measures to limit risks from these vulnerabilities should be implemented where practical. Below, vulnerabilities are described which can reduce the reliability of the equipment.

Horizontal Pump

- Design anchorage for seismic loads. Expansion anchors are not acceptable.
- Engine (or motor) and pump must be connected by a rigid base or skid.

- Sufficient slack and flexibility must be provided in cooling, fuel, and electrical lines.
- Avoid attaching heavy valves to pipe near pumps.
- Avoid seismic interactions of pumps with other components.
- Assure that all equipment installed near vital pumps will not impact the pumps during seismic excitation and that such equipment are securely anchored.

Vertical Pump

- Shafts with unsupported length greater than 20 feet, must be evaluated for seismic loads.
- The impeller drive must be supported within the casing.
- Design anchorage for seismic loads. Expansion anchors are not acceptable.
- Avoid seismic interactions of pump with other components.
- Assure that all equipment installed near vital pumps will not impact the pumps during seismic excitation and that such equipment are securely anchored.

Valves

- Neither the actuator nor the yoke should be independently braced to the structure or supported by the structure unless the pipe is also braced immediately adjacent to the valve to a common structure; unless a rational analysis is performed to demonstrate that the seismic induced forces in the actuator and yoke are acceptable.
- Sufficient slack and flexibility should be provided to tubing, conduits, or piping which supplies air, fluid or power needed to operate the valve.
- Valves should not be near surrounding structures or components which could impact the valve during seismic excitation.

Motor Control Centers (for new installations)

- Must be floor mounted NEMA type enclosure.
- Anchorage must be designed for seismic loads. At least two anchor bolts should be used per MCC section.
- Anchorage of the MCC must attach to base structural members (not sheet metal).
- Avoid excessive eccentricities when mounting internal components.
- Do not mount heavy or vibration sensitive components directly to sheet metal. Use structural frame metal. Vibration sensitive components may require qualification by test or similarity, if that component is essential to operation.

Control Panels and Instrument Racks

- Anchorage must be designed for seismic loads.
- All door latches must be secured with locking devices.
- Wire harnesses or standoffs should be installed on cable bundles to preclude large deformation of bundles.

Battery Racks

- Batteries should be supported on two-step or single tier racks which have x-bracing (new installations) or suitable anchorage/restraint to suitable load bearing floors and walls.
- Batteries should be restrained by side and end rails.
- Provide snug fitting crush-resistant spacers between cells.
- Racks must be anchored, and anchorage designed for seismic loads.

Above Ground Equipment Piping

- Provide sufficient flexibility at equipment connections and nozzles.
- Assure flexibility of pipe routed between buildings and at expansion joints, so that the pipe remains elastic given the design basis inter-building movement (no R value used in calculation)
- Assure that pipe has sufficient space to displace during seismic excitation without impacting other nearby fragile components or structures.

Diesel Engines and Generator Sets

- Diesels should be anchored directly to the structural floor, or mounted on a skid which is directly anchored to the structural floor. Vibration isolators should not be used; or satisfactorily snubbed. Components (batteries, day tanks, mufflers, electric panels, etc.) should all be seismically designed.

Vibration Isolated Equipment

- Equipment mounted on vibration isolators are vulnerable to damage in earthquakes. Vibration isolators for equipment essential to functionality of the system should not be used. "Snubbed" vibration isolators should only be used if the "snubbing" devices are approved by the engineer as meeting suitable strength and operational requirements.

3.4 Anchorage

A minimum factor of safety of four should be used for expansion anchors used for equipment anchorage. The factor of safety is defined as the average strength to failure from test divided by the nominal allowable. When installation of anchors is verified using special inspection, then nominal allowables may be increased as allowed by code (up to a factor of two), but in no case shall the allowable be higher than the average strength from test divided by two.

The following equipment can be considered as structurally and functionally rugged, and need be designed only for the minimum anchorage forces and the other recommendations in this document:

- Valves
- Engines
- Motors
- Generators
- Turbines
- Horizontal Pumps
- Vertical Pumps (limited unsupported shaft length)
- Hydraulic and Pneumatic Operators (limited yoke length)
- Motor Operators (limited yoke length)
- Compressors
- Low Voltage Transformers with anchored internal coils

3.5 Operational Qualification

The following equipment can be considered as structurally rugged, and need be designed for the minimum anchorage forces and the other recommendations in this document. In addition, if post-earthquake operability of this equipment is critical, operational seismic qualification should be addressed by a knowledgeable engineer. Operational seismic qualification may be based on test or experience with similar equipment.

- Air handling equipment and fans (without vibration isolators)
- Low and Medium Voltage Switchgear (< 13.8 kV)
- Instrumentation Cabinets
- Distribution Panels
- Solid State Battery Chargers
- Motor Control Centers
- Instrument Racks
- Batteries in battery racks (must be in seismically designed battery racks)
- Floor mounted inverters up to 5 kVA
- Chillers

3.6 Piping, Raceways, Conduits and HVAC Ducts

Earthquake restraints for above ground piping, raceway and conduit systems, and HVAC ducts as determined by the UBC, CBC, IBC and ASCE 7 codes and other industry guidance such as SMACNA (1991), are oriented to reducing life safety risk, by limiting the falling potential for these items. Post earthquake functionality of these systems is not assured by following the these codes, and in some cases, the code-mandated lateral brace support systems (where specified by install-by-rule) may increase the potential for functional failures. Restraint systems other than that required by these code may be used, if justified by the engineer. Issues to be considered in design of above ground piping, raceway, conduit and HVAC ducts are as follows:

- Plastic pipes should be braced laterally at intervals not more than twice that recommended by the manufacturer for vertical support. This can be relaxed if the stress / load in the pipe and pipe joints is shown to be satisfactory.

- Pipes (and raceways, conduit, ducts) that cross expansion joints between adjacent structures shall be provided with expansion fittings, multiple bends or other suitable provisions to ensure their capacity to sustain expected differential movements between the structures. This can be relaxed if the stress / load in the pipe and pipe joints (or cable tray, conduit, duct) is shown to be satisfactory.

- Special care shall be taken to ensure that small branch lines off pipe headers do not by virtue of their attachment to structures or equipment, act as the brace for the pipe header unless demonstrated by calculation to have suitable capacity for this service.

- At the option of the engineer, pipes that contain very hazardous materials (e.g. chlorine gas) shall be stress analyzed following provisions of the ASME code to ensure that stress levels in the pipes and attached components are within allowables. Any steel pipe commodity may be designed for seismic loading using the stress criteria in the ASME code; and other types of pipe, cable tray, conduit and ducts may be designed using similar strength-of-materials procedures.

3.7 Costs of Upgrade

Cost is a critical variable in reaching cost –effective seismic design. For example, if seismic upgrade to the IBC code for "life safety" (I=1.0) costs $4,000,000, and upgrade for "functionality" (I=1.25) costs $8,000,000, it is desired to know if the extra cost is "worthwhile".

It has been the author's experience over hundreds of similar efforts that the bulk of the net present value of the "benefit" of seismic upgrade is for earthquakes that occur once every 50 to 200 years. For rare events (say an earthquake that occurs once every 2,475 years), the net present value of benefit is generally a very small percentage of the overall benefit.

The variation in seismic hazard between specific sites (say higher seismic hazard coastal California versus lower seismic hazard New Madrid fault zone) precludes a simple statement like: "anchorage of transformers is cost effective, no matter where you are located". Instead, we can say that in higher seismic hazard areas, the end user should be more willing to pay for equal cost seismic upgrades (i.e., it is more cost effective to anchor equipment in San Francisco than in Memphis, assuming equal costs).

The HAZUS technical manuals provide regional cost adjustments for most places in the United States, and while these are becoming dated, they still provide a good first order estimate of the variation in building costs in the United States.

In Appendix A, we provide a few examples of costs for seismic upgrade for a few pieces of equipment. All costs are for San Francisco or Los Angeles areas. We note the year of the cost, so that the end user will need to adjust for inflation and other factors to apply that information to a specific project.

Most of the seismic upgrades addressed in Appendix A deal with anchorage of equipment. The actual cost to install suitable anchorage will be dependent on who does the installing (maintenance department, or outside contractor working under specification); and what level of quality control is desired (generic design, site-specific design, inspection by engineer, etc.). The author has observed costs that range from $10,000 to $160,000 for anchorage of a single power transformer. With such a wide variation in cost, a cost-effectiveness test should be done to see which design (and cost) is best. In this example, the $160,000 option includes a completely new foundation, designed to resist all effects of overturning, whereas the $10,000 option re-uses the existing foundation, but provides perhaps only 85% of the resistance.

Given these issues, we can make *no recommendation* as to a standard seismic upgrade cost for equipment. Instead, we recommend that the end user examine the costs for a range of upgrade alternatives (cheap and light anchorage, moderate and so-so anchorage, expensive and very good anchorage); then check the cost effectiveness for each alternative, and then select the upgrade solution that is most cost effective. The FEMA Benefit Cost Toolkit (2006) can be used to evaluate cost effectiveness.

4.0 Inventory for Non-Structural Components

4.1 What Are Non-Structural Components

We differentiate "non-structural" items into two groups: those associated with specific equipment items, and those associated with the building envelop itself. This report provides fragility curves for non-structural items including:

- Suspended ceilings
- Raised floors
- Roof-mounted HVAC equipment
- Floor mounted electrical cabinets (motor control centers, transformers, switchgear)
- Fire sprinkler distribution pipe system

- Mechanical equipment typically at grade level (chillers, heat pumps, boilers, furnaces, fans, water pumps)
- Horizontal tanks (pressure vessels)
- HVAC ducts and diffusers
- Communication equipment
- Cable trays and conduits
- Diesel generators
- Fuel tanks
- Kitchen equipment including stoves
- Laboratory equipment
- Large diagnostic equipment (MRIs, CT Scanners, X-ray)
- Filing cabinets
- Bookcases
- Desktop computers (CPU boxes, monitors)
- Wall mounted electrical cabinets
- Water or gas distribution pipes within buildings (hot water, cold water, waste water, natural gas, propane gas, oxygen or nitrogen gas)
- Outside utility pipes and conduits (buried), including water, wastewater, natural gas, power, communication)
- Light fixtures
- Emergency lighting

Building nonstructural items are defined to include:

- Appendages such as entrance canopies, overhangs, porches, parapets
- Roofed walkways
- Exterior nonbearing walls
- Exterior infill walls
- Veneer attachments
- Stairs and shafts
- Corridor partition walls
- Fire separation partitions

- Room-to-hallways doors
- Fire doors
- Curtain walls
- Structural fireproofing

This report does not address building non-structural items. Generally, these items are included with the damage estimated for the building structure as a whole. For cases where more refined analyses are indicated, the building nonstructural items could be separately evaluated, often using interstory drift as the predictive variable.

4.2 Inventory and Use of Fragility Information

One good way to address the inventory issue for an existing building is for the cognizant engineer to perform a field survey of all the equipment and note its style of construction. This effort would ideally include review of original specifications, knowledge of actual construction, knowledge of actual material properties, etc. All of this information may not be readily available in the practical world.

For new construction, it *should* be entirely within the engineer's purview to specify the type of equipment that should be installed, as well as all the corresponding seismic details. Of course, developing detailed specifications will require additional time and effort on the part of the engineer, and possible additional cost during procurement.

Some owners may balk at having to pay for the extra effort to develop accurate inventories and detailed design of non-structural components, especially if it is over and beyond code minimum. In such cases, it is entirely proper for the engineer to tell the owner that "you get what you pay for" and the engineer should warn the owner that the building / equipment may fail to perform reliably under the design-basis earthquake. In other words, the proper application of seismic design for many types of non-structural components should not be construed as being a "customary" level of service on the part of the engineer, unless the engineer is charged with developing a code certification report (under ASME, IEEE or other industry guidelines and standards) for each component. Industry documents such as those by NFPA, SMACNA, CISCA and similar, where "install-by-rule" provisions are made for seismic loads, should not be construed as providing functional reliability. Lacking such a level of effort, the engineer is not in responsible charge for seismic design for functionality of non-structural components.

If the engineer is not aware of the actual inventory of equipment in a building, then it is very likely that the fragility information in this report will be mis-used. For example, if the engineer does not know whether or not a piece of equipment is anchored, then the engineer cannot use the fragility information in this report. Further, the fragility information in this report makes certain assumptions about valuation, life safety impacts and functional impacts; if these assumptions do not match the situation in the actual building, the results will be wrong. Appendix A provides descriptions of damage to various types of equipment and components in past earthquakes, test programs, engineering judgment, with consequences on life safety, function and repair, all of which enters into the formulation of the fragility models. To correctly apply the fragility models, one needs to understand the assumptions inherent in their development, which often times implies knowledge of the inventory of what was damaged in past earthquakes (or shake table tests, etc.) and how that is relevant to the actual inventory of equipment / components being evaluated in an actual building. Any mis-match in actual inventory assumptions and fragility models presented in this report will result in errors.

5.0 FEMA Software

FEMA provides, for free, two major suites of analysis programs for evaluation of facilities for earthquakes or other natural hazards. These programs are HAZUS-MH (older versions called HAZUS), and a suite of programs for Benefit Cost Analyses.

The fragility information in this report can be used in either of these FEMA software packages.

The HAZUS software (ref. HAZUS-MH) includes a large number of default fragility functions for buildings, as well as complex facilities such as power plants. HAZUS also provides some default fragility information for selected nonstructural components.

The FEMA Benefit Cost Analysis software (ref. FEMA 2006) also includes a large number of default fragility functions for buildings (the same ones as in HAZUS), as well as a number of default fragility functions for non-structural components.

The author of this report participated in the development of many of the fragility functions in these software packages, and cautions the end user that *none* of the default fragility models in HAZUS or the FEMA BCA software should be used for an individual facility unless verified by a cognizant engineer that the assumptions are suitable for the specific facility being evaluated. The *same* caution is provided to the user for all the fragility functions provided in this report.

Incorporated into these software packages are default ground motion models for the entire 50 states, as well as default fragility models for a wide variety of buildings and non-structural components (equipment). Every applicant for FEMA grant money (for earthquake or other natural hazard mitigation and other similar programs) must demonstrate a benefit cost ratio greater than 1. FEMA provides these software tools as an acceptable (de facto, essentially required) method to perform the BCA. The applicant is always allowed to change the default hazard and fragility values, provided that the changes are supported by suitable engineering documentation.

In practice, FEMA will almost always accept use of the default fragility values as suitable for the BCA, except in unusual cases. For example, FEMA would not accept using a "unreinforced masonry building" fragility curve for application to a heavy post-and-beam timber building. As a matter of good public policy, it is frowned on trying to "game" the software to try to develop very high BCRs.

Table 5-1 provides the default fragility values incorporated in the FEMA non-structural software. It is stressed that these default values might not be applicable to all practical situations, and facility-specific fragility evaluations that factor in actual installations is always preferred, although not always practical.

Item	FEMA A As Is	FEMA A Upgraded
Generic bottom weight unanchored	0.75	1.50
Generic bottom weight poor anchored	0.88	1.50
Generic even weight unanchored	0.60	1.50
Generic even weight poor anchored	0.73	1.50
Generic top weight unanchored	0.40	1.50
Generic top weight poor anchored	0.49	1.50
Parapet walls URM extensive damage	0.40	1.10
Parapet walls URM complete damage	0.60	1.50
Racks – shelves	0.60	1.00
Generators on isolators	0.25	0.60
Elevators moderate	0.35	0.90
Elevators extensive	0.75	1.50
Fire sprinklers limited	0.25	0.52
Fire sprinklers widespread	0.50	1.00
Fire sprinklers extensive	0.75	1.50
HVAC fans	0.30	1.00
HVAC ductwork rod hung extensive	1.25	2.38
HVAC ductwork rod hung complete	1.88	3.00
HVAC ductwork rod hung in penthouse extens.	0.50	0.96
HVAC ductwork rod hung in penthouse compl.	0.75	1.50
Suspended ceiling wire hung moderate	0.25	1.50
Suspended ceiling wire hung extensive	0.50	>1.50
Suspended ceiling wire diagonals moderate	0.50	1.50
Suspended ceiling wire diagonals extensive	0.90	>1.50
Suspended ceiling comp struts moderate	0.80	1.50
Suspended ceiling comp struts extensive	1.30	>1.50
Electrical cabinets unanchored	0.60	3.00
Electrical cabinets poorly anchored	1.00	3.00

Table 5-1. FEMA Non-Structural Default Fragilities

6.0 References

ALA, Seismic Fragility Formulations for Water Systems, American Lifelines Alliance, ASCE and FEMA, April 2001. http://www.americanlifelinesalliance.org/Products.htm#Guidelines

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Appendix A

A.1 Fire Sprinkler Systems

A.1.1 Background

Recent earthquakes in California (1989 Loma Prieta, 1994 Northridge) have demonstrated that fire sprinkler systems nominally built to recent codes (1970 and later) have in some cases been damaged in earthquakes. This damage leads to three undesirable outcomes:

1. The damaged sprinkler system needs to be repaired.
2. The damaged sprinkler system becomes inoperative following the earthquake, leading to somewhat higher risk of fire spread within the sprinklered building. To date, there have been no known fire ignitions within buildings that suffered sprinkler damage.
3. The damaged sprinkler system releases large volumes of water, with attendant water damage to the building. This has happened at modern built critical facilities including telephone central switching offices (Pacific Telephone, Oakland, 1989 Loma Prieta earthquake) and hospitals (Olive View, Holy Cross Medical Center, Northridge, all in the San Fernando Valley, 1994 Northridge earthquake), where continued post-earthquake function is at a premium. The water damage caused very high direct damage losses (approaching the full value of the telephone equipment contents) at the telephone central switching office. The water damage was a main factor in the decision to evacuate hospital patients at the Olive View hospital.

The earthquake may also cause damage to the municipal water distribution system that serves the facility. Most sprinkler systems operate off of the pressure provided by the municipal water system. Thus, if the municipal water system fails, then some damage to the fire sprinkler system may be "hidden" until such time that the municipal water system is restored to service.

Damage to fire sprinkler systems has been prevalent in recent California earthquakes. After the 1971 San Fernando earthquake, it was assumed that this type of damage could be mitigated by the addition of sway braces to the fire sprinkler water pipes. The Northridge earthquake showed that this is not the case, as sprinkler system in this earthquake was often a function of the interaction of the complete fire sprinkler system with adjacent structural and non-structural systems, particularly suspended ceiling systems.

A rigorous method to design fire sprinkler systems so as not to fail in earthquakes would be to design them to the ASME Boiler and Pressure Vessel code, including all seismic provisions. This would ensure that seismic inertial effects do not overload pipe and pipe joints, and seismic anchor motion effects do not break pipes at building seismic isolation joints, or where the pipes are connected to other building elements (sprinkler heads at ceilings, etc.) To date (2009), this level of design has not been incorporated into the systems used at many essential facilities in California.

Two sets of damage algorithms are suggested, listed in Table A.1-4. The damage factors relate to the effort to repair the sprinkler systems. The "field hung" seismic design approximates sprinkler systems built in California to the 2006 UBC code, with no special consideration of seismic anchor motions. The "ASME" seismic design approximates sprinkler systems built with both inertial and anchor motion considerations, including sprinkler head assembly interactions with ceilings, and control of pipe joinery to keep pipe stresses within limits.

The Life Safety impact to damage to a sprinkler system is 1 (none). Indirect life safety issues (like control of fire spread should a fire occur) is not included. Short term repair cost to the damaged pipes is about $200 per building, including observations of leakage and closing the water shut off valve(s). Long term repair cost to the sprinkler system is about $1,000 for each repair.

Water leaks from broken sprinkler heads can cause $1,000 to $10,000 of water damage to the room fixtures below. Water leaks from broken sprinkler pipes (especially 4 inch diameter and larger) have been known to cause damage in communication centers approaching the complete value of the equipment in the affected areas.

A.1.2 Code of Practice

Fire sprinkler systems are installed in some buildings, but not all buildings. The purpose of this report is not to decide the appropriateness of fire sprinkler systems, but only to consider them in context with the earthquake environment.

The usual code of practice for the installation of fire sprinkler systems in the general building stock is the NFPA 13 (NFPA). As with any standard, NFPA is a consensus document, and its provisions do not necessarily guarantee a lack of damage in earthquakes. Each edition tends to build upon prior editions, including new lessons learned.

Earthquake protection provisions for fire sprinklers were first included in 1947, in what was then known as NBFU 13. The 1947 document provided the following guidance for earthquake protection (abbreviated):

- Sprinkler system piping should be installed in a manner to avoid damage from earthquakes.

- There should be one to two inches clearance where sprinkler pipes pass through floors and walls. The space between the pipe and the floor / wall should be filled with suitable fire resistant materials, which are essentially flexible.
- There should be flexible couplings in sprinkler risers, where needed.
- Feed mains and cross mains should be provided with some form of lateral and longitudinal bracing. Branch lines do not require bracing.

The 1950 edition of NFPA 13 was unchanged from NBFU 13, except for the name change. The 1951 edition of NFPA 13 included provisions to ensure that the lateral braces could withstand a force of 50% of the weight of the piping, valves and water; one longitudinal brace was required for each main, and lateral braces were to be located at intervals of 30 to 40 feet. This criteria remained essentially unchanged until 1971.

The 1971 San Fernando earthquake revealed some damage to fire sprinkler systems. The 1973 edition of the NFPA emphasized that seismic provisions were mandatory (not advisory) in areas prone to earthquakes. The 1980 edition of the NFPA further emphasized that the seismic provisions were mandatory.

The 1983 to 1994 editions of NFPA were refined with increased attention to earthquake protection resulting from a series of earthquakes: 1978 Santa Barbara, 1979 Imperial Valley, 1983 Coalinga, 1984 Morgan Hill and 1987 Whittier.

Following the 1989 Loma Prieta earthquake, the 1991 NFPA identified the need to restrain branch lines where movement could damage sprinklers through impact with other building features.

The 1994 Northridge earthquake produced the strongest levels of ground shaking to affect the largest inventory of buildings, in U.S. history. Even so, two-thirds of the buildings in the Northridge area were built prior to 1976, and very few were built to the provisions of the 1991 NFPA 13. A survey was done following the earthquake to assess the damage rates to sprinkler systems. There were approximately 3,300 sprinkler systems in the San Fernando Valley area at the time of the earthquake. It is estimated that perhaps 5% of these systems suffered at least some sort of observable damage; of that 5%, about one-third included damage to pipes resulting in water leakage; with the remainder suffering damage to pipe supports that did not result in water leakage.

One insurance research agency reported a total of 144 sprinkler leakage losses, combined from the 1994 Northridge, 1989 Loma Prieta and 1987 Whittier earthquakes. The gross loss was $30,000,000 ($1995), or $212,000 per loss. Much of the loss was presumably a combination of water damage and business interruption losses.

A.1.3 Factors Causing Damage to Sprinkler Systems

The following lists summarize the main engineering factors that can lead to damage to sprinkler systems. This list is indicative, and not all inclusive.

- Fire sprinkler pipes are mostly built with screwed fittings. High seismic forces and bending moments on these screwed fittings cause them to leak. If fire sprinkler systems were built with higher quality pipe and fittings (welded joints, designed to ASME code for seismic loading), then damage to fire sprinklers in even the largest earthquakes would be essentially nonexistent; however, this type of sprinkler system is only cost effective for new construction.
- The NFPA 13 standard requires that addition of lateral and longitudinal braces. By adding more braces to rod hung pipe systems, seismic loads are increased on the pipes themselves, resulting in higher potential for leakage at screwed fittings. The philosophy of the NFPA 13 standard is to try to "stiffen" the pipe system to prevent excessive displacement (beneficial), but occasionally has the negative impact of causing high stresses on weak pipe joints.
- By bracing pipes, pipe displacements are lowered. Lowering pipe displacements is beneficial, in that impacts with adjacent items are reduced.
- Laterally braced pipe systems are subject to high inertial forces and (sometimes) seismic anchor movements. These high forces must be resisted by the lateral braces. As there is often inadequate attention to detailing these braces, damage to braces often occurs, particularly at anchorages. Unless the damaged pipe supports also cause a loss of vertical load carrying capacity (extremely rare), damage to pipe supports is acceptable, even if not desired.
- Seismic anchor movements at sprinkler heads can be caused by forces with adjacent ceilings, which may be suspended. These forces can shear off or distort a sprinkler head, and they may cause excessive high bending moments on the screwed fitting between the riser and the pipe main.
- Seismic anchor movements between rigidly braced sprinkler pipes across a seismic joint between buildings may impose excessive loads on the pipe spanning across the seismic joint.
- Pipes which pass through penetrations at corridors or suspended (not-rated) ceilings may have inadequate space to allow pipe movements. When the pipe impacts the hard penetration, the resulting pipe loading may be high enough to cause leakage at screwed fittings.
- Items adjacent to fire sprinkler systems (notably suspended ceilings, HVAC ducts, lighting fixtures, electrical cable trays and conduit, storage racks, structural elements) will all move during an earthquake. With the exception of the structural elements, many of these commodities are flexible, and can impact the fire sprinkler system. While the structural elements may be quite rigid, often the

structural engineer during the design stage has no knowledge of the location of the sprinkler system piping, and interactions between the two may still occur.

- It is recognized that many fire sprinkler systems are installed by rule, rather than designed by analysis. In other words, a simple rule-based approach is incorporated into the NFPA standard, thereby eliminating the need for an engineer to calculate the true loading (pipe stress, pipe support forces, pipe movements) on the fire sprinkler pipe system. This approach leads to field installations which are less costly to install, but which may accidentally incorporate one or several "weak links". For most building installations, the licensed Structural Engineer is not required to approve the fire sprinkler system installation. Installation per NFPA rules does not guarantee that the seismic-induced stresses in the fire pipes (or supports) will be less than yield or any other rational ductility demand. Further, there is no provision in any current code to ensure that commodity interactions are reviewed for acceptance, after construction is completed.

A.1.4 Empirical Evidence

In the 1994 Northridge earthquake, 21 buildings at health care facilities suffered broken non-sprinkler related water lines, with most of the damage occurring to smaller lines (under 2.5" diameter) for which seismic bracing is not required. (Note: the bulk of the length of non-sprinkler pipe is also under 2.5" diameter, so the conclusion about bracing is not entirely clear).

Sprinkler line breakage occurred at 35 buildings, all in smaller "unbraced" branch lines (OSHPD 1994, CDMG, 1995). Much of the damage occurred at or near sprinkler heads, where interaction with ceiling systems was likely a significant contributor to breaking off sprinkler heads.

Number of buildings where specified damage occurred				
Damaged Item or Type of Damage	Hospitals		Skilled Nursing and Intermediate Care Facilities	
	Pre-Act Buildings	Post-Act Buildings	Pre-Act Buildings	Post-Act Buildings
Water Piping	8	5	2	0
Fire Sprinkler System	9	5	3	0

Table A.1-1. Non-Structural Damage In Buildings with Moderate, Extensive or Complete Structural Damage

Number of buildings where specified damage occurred				
Damaged Item or Type of Damage	Hospitals		Skilled Nursing and Intermediate Care Facilities	
	Pre-Act Buildings	Post-Act Buildings	Pre-Act Buildings	Post-Act Buildings
Water Piping	4	2	0	0
Fire Sprinkler System	7	5	6	0

Table A.1-2. Non-Structural Damage In Buildings with Slight or No Structural Damage

In the 1994 Northridge earthquake, there were a number of breaks in the sprinkler system at Holy Cross Medical Center. An exact count was not tabulated, but hospital staff estimated at least 20 sprinkler heads were broken. Municipal water supply was not lost at the hospital (at least initially), and this let substantial amounts of water to be released into the hospital, with corresponding water damage.

The hospital initiated a review of the sprinkler system, to determine what deficiencies caused this level of damage. It was originally thought that flexible sprinkler pipes were a contributing factor. Therefore, the seismic bracing of the fire sprinkler pipes was examined. A fire sprinkler consultant prepared a report describing the current condition. The report examined:

- checking of all sway bracing, per NFPA 12-3-5.3.5, both lateral and longitudinal
- checking of all hangers of fire sprinkler system
- checking of all piping for corrosion and strained areas due to earthquake damage
- checking of all sprinkler heads
- conducting a hydraulic test at 200 psi for 2 hours, per NFPA 13-2-1.3.2

A visual examination of the sprinkler system was performed from the basement (1st level) to the penthouse. The examination found that the sprinkler system was in very good condition, and the leaks that occurred during the January 1994 Northridge earthquake were caused by the rigid one-piece sprinkler head flanges. Vertical movement during the earthquake caused a drop and the head therefore tore through the ceiling, causing the head to go off. Other factors (sway bracing, etc.) were not found to be contributing to the actual damage. Mitigation taken by this hospital included installation of approximately 1,220 fire sprinkler heads and 401 new two-piece escutcheons.

A.1.5 Process Pipe

Almost every building will include an inventory of a variety of different types of process pipes. These pipes include hot water, cold water, steam, sanitary, fire sprinkler, medical gas and natural gas pipes. Most of these have very similar design features.

Silver (1988) and Stevenson (1989) conducted surveys of the performance of power plant piping in 29 past earthquakes between 1923 and 1985. The summary findings are listed in Table A.1-3. A total of 141 combined repairs were observed, based on a total of 1,200,000 feet of pipe. The statistics in Table A.1-3 suggest that less than 0.01% of all power plant piping and supports subjected to peak ground accelerations of 0.2g to 0.5g failed as a result of the earthquake.

Category	Failures	Damaged
Piping (Above Ground)		
Seismic Anchor Movement	15	0
Corrosion	7	0
Interaction	3	60
Non-welded Joints	36	10
Pipe Supports	11	29
Internal Equipment	15	19

Table A.1-3. Piping Damaged and Failure in Power Plants Based on Worldwide Survey of 29 Earthquakes

Allowing that the 141 combined failures and damages were on a total of 1,200,000 feet of pipe, we arrive at an above ground piping repair rate of:

$$n = \frac{141}{1200} = 0.12 \text{ repairs per thousand feet, with PGA} = 0.20\text{g to } 0.50\text{g.}$$

The damage statistics in Table A.1-3 covers a large variety of piping, with most damage to metal pipes with screwed fittings, or attributable to corrosion. This damage excludes damage to pipe supports, which occurs on at least as common basis, but does not usually impact service of the pipeline.

The common "design-by-rule" criteria of including sway supports at regular intervals, as promoted in recent codes like NFPA 13, IBC 2006, etc., does not seem to make much sense when examining the statistics in Table A.1-3. Assuming that damage to supports is secondary, most building owners will be mostly interested with damage to pipes that result in loss of water (gas, or other contents). These failures are mostly attributed to corrosion (7 instances, 22% of total), seismic anchor motions, such as imposed movements between buildings, (15 instances, of 25% of total), or overloads at joints in the pipe (36 instances, 59% of total). Interaction between pipes and adjacent items (3 instances, 5% of total) is the fewest cause of pipe failure. By adding sway braces, such as recommended by IBC 2006, will increase pipe frequency and reduce pipe deflection. Reducing pipe deflection will reduce the failure rate due to interaction (5% of total) but likely increase bending moments in the pipe, resulting in increasing the rate of the other failure modes. In other words, there might be no clear benefit of adding sway braces using "rule based assumptions", as these braces tend to:

- Increase pipe frequency, likely from under 1 hertz (or so) to 5 hertz (or so), thereby moving "up" the spectral acceleration on the pipe often by a factor of 2 to 4.

- Reduce the ability of pipe to accommodate differential imposed seismic anchor motions. For example, a pipe with 30 feet of unsupported length can take 4 times more differential end movement as compared to a pipe with 15 feet of unsupported length, all other factors remaining the same.

- As a pipe become stiffer (more supports), it has less ability to absorb thermal growth and contraction, and will undergo more severe stress reversal cycles. This will promote high cycle fatigue failures at a higher rate.

- Corrosion failures occur in earthquakes as the additional stress due to seismic-induced pipe bending overcomes the remaining capacity of the corrosion-caused thinner wall. By adding many lateral braces, pipe frequency increases, leading to higher spectra acceleration imposed on the pipe, leading to additional corrosion-related failures.

Unless a stress analysis of the pipe is conducted, and shown that the seismic-induced pipe stresses remain within pipe allowable capacities for the actual pipe arrangement, there can be no assurance that adding sway braces per IBC 2006 improves pipe performance. By adding sway braces per IBC 2006 / NFPA 13, there is less chance that the pipe will fall on the floor, and that there will be fewer pipe interaction failures.

Given these issues, we suggest the damage algorithms listed in Table A.1-4. The repair rate listed (Field Hung) are extrapolated from the findings in Table A.1-3. The repair rate for ASME code assumes non-nuclear application, and assuming quality control that might be uncertain, and lack of ongoing inspection / preventative maintenance (in other words, assuming that if it is not broken, do not fix it).

It remains dubious that the addition of away braces using design-by-rule presents a reliable way to reduce the pipe failure rate to nearly zero. Without a doubt, the best way to assure pipe functional performance is to design the pipe by ASME rules, consistent with dead load, thermal, seismic, hydrodynamic, fatigue, corrosion and related issues. A good design will accommodate these issues by proper pipe material selection, specification of shop and field connection details, and suitable quality control.

As the design-by-rule approach for adding sway braces ignores most of these issues, it cannot be asserted that adding sway braces will make a net improvement to pipe functional performance. Instead, other types of seismic retrofit (pipe replacement with better pipe material, or engineered support placement allowing for three directional state of inertial stresses. anchor motions, thermal and other operating loads), can reduce the potential of seismic pipe failure to much lower repair rates, as listed in the right-most column. The values in the left column in Table A.1-4 are for representative for steel pipe with screwed fittings; rates may be 2 to 4 times higher for copper pipes.

Peak Ground Acceleration, g (at surface)	n, Repairs per 1,000 Feet, Field Hung	n, Repairs per 1,000 Feet, design to ASME Code for PGA = 0.50g
0.10	.01	~.0
0.20	.08	~.0
0.30	.10	~.0
0.40	.14	~.005
0.50	.16	~.01
0.60	.20	~.015
0.70	.24	~.02
0.80	.30	~.025
0.90	.34	~.03
1.20	.46	~.04

Table A.1-4. Process Pipe Damage Algorithm (Leaks)

A.1.6 Loss Estimate Formulation

Then the number of pipe repairs is n * (Length / 1,000). Use n from Table A.1-4. If one assumes a Poisson distribution of damage, then the probability of one or more repairs (leading to leak) is (1-exp(-n*L)) where L = actual length (feet) / 1000.

If the actual design per ASME rules is for a different PGA than 0.5g, then adjust the value n to be about 0.01 per 1,000 feet at the actual design level PGA. If the quality of pipe design is such that at the design level PGA that the pipe is expected to remain within ASME limits (no more than 20% higher than steel yield, and all pipe joinery is ductile), then n can be taken as essentially 0 at the design level PGA, assuming no corrosion. For pipes with known flaws (corrosion, quality control, etc.), adjustments have to be taken to reflect the actual in-situ condition of the pipe.

An intermediate solution would be to install the pipe per NFPA 13, and then have a cognizant engineer field-inspect the installation for the potential of adverse sprinkler head interactions, and then take suitable mitigation steps to eliminate the interaction potential. This solution would reduce n (left column in Table A.1-4) by perhaps a factor of 3 or so.

Not included in Table A.1-4 is damage to pipe supports or to pumps or control equipment. Damage to pipe supports is not considered to have a material impact to life safety or functional impact to a building, and many times goes unrepaired. If the owner wishes to estimate the damage to pipe supports, as a first order estimate, assume one visually damaged support for each leaking pipe (this ratio could vary by a factor of 5 in either direction).

A.1.7 Mitigation

Seismic mitigation may consist of seismically upgrading the existing fire sprinkler system. Further improvement to the fire sprinkler system could be achieved by extending the area covered by sprinklers to include all floors.

Deficiencies in the system should be rectified in the remediation phase. If the owner wishes to upgrade to meet proscriptive NFPA guidelines, then sway bracing should be added where required. Pipes that cross expansion joints or other areas subject to differential displacements should be capable of absorbing these displacements without breaking. The sprinkler head – suspended ceiling (or other interferences) design should be carefully checked for interaction potential, to ensure that sprinkler heads are not broken during strong ground shaking.

If the seismic upgrade includes elimination of all seismic anchor motion weaknesses and sprinkler head interaction weaknesses, coupled with other operational issues, then the bulk of the seismic vulnerabilities will be eliminated. If sway braces are added to metal pipe with screwed joints, then there may be no improvement, and possibly even reduction in safety, unless the induced bending moments and axial forces on the screwed fittings are showed to be reduced below their non-leak capacity.

A.1.8 References

NFPA 13, Standard for the Installation of Sprinkler Systems, National Fire Protection Association, Quincy, MA (1947 - 2007 editions).

OSHPD, Report to the Building Safety Board on the Performance of Hospital Buildings in the Northridge Earthquake of January 17, 1994, Sacramento, CA, 1994.

CDMG, The Northridge Earthquake of 17 January 1994, Special Publication 116, California Division of Mines and Geology, Sacramento CA, 1995.

Silver, M.M., et al, *Recommended Piping Seismic Adequacy Criteria Based on Performance During and After Earthquakes*, prepared for the Electric Power Research Institute by EQE Inc., NP-5617, January 1988.

Stevenson, J.D., "Use of Bounding Spectra to Demonstrate Seismic Margin at Low Seismicity Sites," in *Symposium on Current Issues Related to Nuclear Power Plants Structures, Equipment and Piping with Emphasis on Resolution of Seismic Issues in Low Seismicity Regions*, EPRI NP-6437-D, 1989.

A.2 Buried Utilities

Buried utilities include water, sanitary sewage, natural gas, communication and power systems. Each of these systems will generally have a meter. Each facility will usually own and be responsible for the piping from the meter to the buildings; whereas the utility lifeline will own and be responsible for the meter and the pipes / conduits in their system.

Damage to buried utilities has been prevalent in many earthquakes, on both the customer and utility sides of the meter. Buried pipeline damage is caused either from the effects of ground shaking, usually correlated with peak ground velocity (PGV), or some form of permanent ground deformation (PGD). The PGD may be caused by liquefaction, landslide, surface faulting or other factors, and may manifest itself as settlement or lateral spreads.

Buried utilities owned by building owners are usually not networked; in other words, if one pipe fails, the flow to the affected building is lost. Buried pipes in utility systems are often networked, and a single break in the utility's buried pipes may or may not result in cut off of supply to the individual building. It is beyond the scope of this report to describe network analyses of utility lifeline infrastructure, but is obviously important for the facility owner to be able to forecast how long the utility's services may be lost after an earthquake.

Many existing buildings are situated on soils that can experience varying amounts of settlement in earthquakes. For heavier or more modern buildings so-situated, many of the structures will have been built on pile foundations; these buildings will experience little if any settlement. At large facilities with many buildings, smaller or lighter buildings may be built on spread footing foundations, which can experience varying amounts of settlement due to earthquake-induced liquefaction and/or compaction. Some of the at grade floor slabs of pile supported buildings may also settle.

Observed settlements in the Northridge area in 1994 at a large hospital campus ranged from 3 inches to reportedly more than one foot, and average was in the range of 4 to 5 inches. At each building on piles, this settlement caused differential motions for adjacent buried utility lines. Discussion with plant staff indicated that most of the buried utility lines broke at the entrances to the buildings: cast iron water pipes and sewers broke; some cables in conduits were sufficiently stretched to cause pull out at terminal blocks.

The question remains for the facility owner to assess the performance of the buried utility lifelines from the meters into the buildings.

The following describes the development and usage of fragility curves for various kinds of buried pipe. These fragility curves are geared towards water pipes. They can also be used for buried sewer and natural gas pipes. They can be used as a first order (conservative) proxy for buried low voltage (under 12 kV) electrical conduits use for power and communication. Except as noted. they should not be used for buried high voltage cables or for pipelines that have been designed using ALA (2005) or similar seismic guidelines or standards, for which seismic performance should be considerably more robust.

For buildings at sites prone to differential settlement (seismic or otherwise), some type of mitigation is usually recommended where buried pipes enter the building. There can be residual vulnerability at building-to-soil transitions, and there will always remain some chance of pipe failure at locations away from the building-ground transition locations. For new installations, the owner could consider the use of HDPE pipe with fusion welded joints for buried sewer and gas line installations; alternate materials which provide reasonably good seismic performance in areas prone to ground deformations include welded steel pipe (minimum wall thickness and corrosion protection required). Unrestrained cast iron, PVC, concrete, clay and asbestos cement pipe should not be used for buried utility lines at locations subject to differential settlements (or landslides, lateral spreads or surface faulting), but could be used where soils are otherwise firm. Ductile iron or other types of pipe with restrained or chained joints can provide suitable capacities to withstand permanent ground deformations if properly designed.

A.2.1 Factors That Cause Damage to Buried Pipes

The following subsections describe the factors that lead to damage to buried pipe in earthquakes.

A.2.1.1 Ground Shaking

Ground shaking refers to the transient soil deformations due to seismic wave propagation. It affects a wide area and can produce well dispersed damage.

The level of ground shaking at a pipeline location can be measured in terms of peak horizontal ground velocity (PGV).

A.2.1.2 Landslides

Landslides are the permanent deformation of soil mass, which can be very damaging to buried pipe. These produce localized severe pipe damage. More landslides will occur if the earthquake occurs in the rainy winter season. Some landslides will be small and displace only a few inches. Some landslides may involve 100,000 cubic yards of soil or more, over many feet of distance, and will damage entire areas of pipelines.

The amount of landslide movement is measured in terms of permanent ground displacement (PGD).

A.2.1.3 Liquefaction

Liquefaction is a phenomenon that occurs in loose, saturated, granular soils when subjected to long duration, strong ground shaking. Silts and sands tend to compact and settle under such conditions. If these soils are saturated as they compact and settle, they displace pore water, which is forced upwards. This upward pressure on the pore water causes two effects. First, it creates a quick condition in which the bearing pressure of the soils is temporarily reduced. Second, if the generated pressures become large enough, material can actually be ejected from the ground to form characteristic sand boils on the surface. This displaced material in turn results in further settlement of the site.

Lateral spreading is a phenomenon that can accompany liquefaction - induced settlements. On many sites, the layers of liquefiable materials are located some distance below the ground surface. If the site has significant slope, or is adjacent to an open cut, such as a depressed stream or road bed, liquefaction can cause the surficial soils to flow downslope or towards the cut. Lateral spreading can be highly disruptive of buried pipelines.

Seismic soil liquefaction has the potential to occur in certain soil units, and can result in permanent ground deformations. Heavy concentrations of breaks will occur in areas of liquefaction-induced lateral spreading. The orientation of the pipe relative to the ground movement can affect the amount of damage [O'Rourke and Nordberg].

The amount of liquefaction movement is measured in terms of permanent ground displacement (PGD).

A.2.1.4 Settlement

Pipe breaks will occur due to relative vertical (differential) settlements at transition zones from fill to better soil, and in areas of young alluvial soils prone to localized liquefaction. Breaks can also occur where pipes enter tanks or buildings.

The amount of settlement movement is measured in terms of permanent ground displacement (PGD).

A.2.1.5 Fault Crossings

Localized permanent ground deformations occur in surface fault rupture areas. Damage to segmented pipes (e.g., cast iron pipe having caulked bell-and-spigot joints) will be heavy when crossing surface ruptured faults. Butt welded continuous steel pipes may sometimes be able to accommodate fault crossing displacements, up to a few feet.

The amount of fault offset movement is measured in terms of permanent ground displacement (PGD).

Continuous butt-welded steel pipelines are less prone to damage if they are oriented such that tensile strains result from the fault displacement. This is because tensile deformation takes advantage of the inherent ductility and strength of the steel, whereas compressive deformation promotes pipe wall wrinkling and accumulation of high local strain.

The angle of the pipeline-fault crossing has a major impact on pipeline response for orientations that promote tension. For continuous ductile pipelines that cross strike slip faults, the performance will improve as the angle of the pipeline-fault intersection increases, for cases where the pipe can displace the surrounding soil consistent with the assumptions outlined by (ref. Newmark and Hall).

For segmented pipelines subject to tension, the optimal angle of the fault crossing depends on joint characteristics. This angle depends upon taking maximum advantage of both the pullout and joint rotational capacities of the joints. Leaded joint couplings appear to be able to take only 1 to 2 inches of fault displacement before failure. Extra long restrained couplings can take up to about a foot of fault displacement (O'Rourke and Trautmann).

For both segmented and continuous pipelines, it is advantageous to avoid bends, tie-ins, and local constraints close to the fault. This allows the pipeline that crosses the fault additional length over which to distribute the imposed strains resulting from the fault offset.

Burial depth is also a factor at fault crossings. The shallower the burial, the less overburden, and hence less frictional resistance by the soil on the pipe. The lower the frictional resistance, the easier the pipe will be able to deform or buckle upwards in fault crossings, without causing severe damage. For example, a pipeline with 3 foot overburden can sustain about 4 times the fault displacement as compared to a pipeline with 10 feet of overburden.

Two failure modes occur when a pipeline is deformed in compression: the pipeline may buckle as a beam, or it may deform by local warping and wrinkling of its wall. Buckling can occur across fault crossings, either due to fault creep or sudden fault offset. Pipe wrinkling failure occurs in thinner walled pipes in high frictionally restraint soil conditions.

A.2.1.6 Continuous Pipelines

Continuous pipelines are pipes having rigid joints, such as continuous welded steel pipelines. Continuous pipelines built in accordance with modern codes of practice have generally performed better in past earthquakes than those constructed with other methods (Newby).

Experience has shown that some pipelines constructed before and during the early 1930s did not benefit from the same quality controls that prevail today. For example, the 1933 Long Beach earthquake caused over 50 breaks in high-pressure gas pipelines in welded joints. In every instance, the breaks in large diameter lines were discovered at welds that lacked proper penetration or bond with the body of the pipe. Poor welds have also contributed to failures of more modern (1960s vintage) welded steel pipelines using arc-welding procedures.

Experience has also shown that welded pipelines with bends, elbows, and local eccentricities will concentrate deformation at these features, especially if permanent ground deformations develop compression strains. Liquefaction-induced landslides during the 1971 San Fernando earthquake caused severe damage to a 49 inch diameter water pipeline at nine bend and welded joints (O'Rourke and Tawfik, 1983).

A.2.1.7 Segmented Pipelines

A jointed pipeline consists of pipe segments that are connected by relatively flexible (or weak) connections (e.g., a bell-and-spigot cast iron piping system). They typically can fail in three ways: excessive tensile and bending deformations of the pipe barrel; excessive rotation at a joint; or pullout at a joint (Singhal). Segmented pipe with somewhat rigid caulking such as Portland cement cannot tolerate much relative movement before leakage occurs. Pipes with flexible rubber gaskets can generally tolerate more seismic deformations.

A.2.1.8 Appurtenances and Branches

Experience has shown that pipeline damage tends to concentrate at discontinuities such as pipe elbows, tees, in-line valves, reaction blocks, and service connections. Such features create anchor points (rigid locations) that will promote force/stress concentrations. Locally high stresses can also occur at pipeline connections to adjacent structures (e.g., tanks, buildings and bridges), especially if there is insufficient flexibility to accommodate relative displacements between pipe and structure. This was reported as the reason for most of the damage to service connections of water distribution piping during the 1971 San Fernando earthquake.

A.2.1.9 Age and Corrosion

Age and corrosion will accentuate damage, especially in segmented steel, threaded steel and cast iron pipes.

Older pipes appear to have a higher incidence of failure than newer pipes. Pipe damage due to the 1987 Whittier Narrows earthquake (Los Angeles area) showed an increasing trend of earthquake pipe breaks vs. age of pipe (Wang). Similar trends have been documented for the 1989 Loma Prieta earthquake for steel pipe (Eidinger 1998).

Age effects are possibly strongly correlated with corrosion effects, due to the increasing impact of corrosion over time.

Corrosion weakens pipe due to the effective decrease in material thickness as well as creating stress concentrations. Screwed / threaded steel pipes appear to fail at a higher rate than other types of steel pipes. Some cast iron pipes have also experienced higher incidences of corrosion failure (Isenberg 1978, Isenberg 1979, Isenberg and Talyor).

A.2.2 General Form of Pipeline Fragility Curves

Fragility curves for buried pipe are expressed as a repair rate per unit length of pipe, as a function of ground shaking (peak ground velocity, PGV) or ground failure (permanent ground deformation, PGD).

The development of fragility curves for buried pipe is currently (2009) primarily based upon empirical evidence, tempered with engineering judgment, and sometimes by analytical formulations.

Empirical evidence means the following: after an earthquake, data is collected about how many miles of buried pipe were experienced what levels of shaking, and how many pipes were damaged (broken or leaking) due to that level of shaking.

Most of the empirical evidence we have prior to 1989 is for the performance of small diameter (under 12 inches) cast iron pipe. This is because cast iron pipe was the most prevalent material in use in water systems for earthquakes that occurred some time ago (like San Francisco, 1906). More recent earthquakes (Loma Prieta 1989 and Northridge 1994) have yielded new empirical evidence for more modern pipe materials, like asbestos cement, ductile iron and welded steel pipe. Still, as of 2009, we do not have a complete empirical database for all pipe materials under all levels of shaking.

Most empirical evidence documented in the literature shows pipe fragility in terms of a repair rate per unit length of pipe. We adopt this format, using the following description of fragility: repair rate per 1,000 feet of pipe.

A pipe repair can either be due to a complete fracture of the pipe, a leak in the pipe, or damage to an appurtenance of the pipe. In any case, these repairs require the water agency to perform a repair in the field.

The pipe repairs predicted using the fragility curves are for repairs in buried pipe owned by the water agency. This includes the pipe mains in the street, pipe laterals that branch off the main to fire hydrants, and service connections up to the meter owned by the water agency.

Buried pipe from the water agency's meter up to the customer's structure may also be damaged. This pipe is usually very small diameter (under 1 inch diameter for water pipes to residential structures, under 2 inches damage to most commercial structures, up to 6 inches in diameter (or larger) for some heavy industrial water users), and is generally the responsibility of the customer for repair. If this pipe breaks, then water will leak out of the water main until someone shuts off the valve at the service connection.

A.2.3 Backbone Pipeline Fragility Curves

Sections A.2.3.1 and A.2.3.2 compile the previous historical earthquake data into two pipe damage databases, one for wave propagation damage and another for ground failure damage. Statistical analyses are then performed to estimate vulnerability functions. The complete empirical datasets are provided in (ALA, 2001)

The fragility curves relate overall pipe damage measures to relatively simple demand intensity descriptions. The functions are entirely empirical, based on reported damage from historical earthquakes. Damage is expressed in terms of pipe repair rate defined as the number of repairs divided by the pipe length exposed to a particular level of seismic demand. Two separate types of pipe damage causing mechanisms are considered: seismic wave passage, and earthquake induced ground failure.

Wave passage effects refers to the transient vibratory soil deformations caused by seismic waves generated during an earthquake. Wave passage effects cover a wide geographic area and affects pipe in all different types of soil. Strains are induced in buried pipe because of its restraint within the soil mass. In theory for vertically propagating shear waves, peak ground strain is directly proportional to peak ground particle velocity (PGV), and therefore PGV is a natural demand description.

Ground failure effects are the permanent soil movements caused by such phenomena as liquefaction, lurching, landslides, and localized tectonic uplifts. These tend to be fairly localized in geographic area and potential zones can be somewhat identified *a priori* by the specific geotechnical conditions. Ground failure can be very damaging to buried pipe because potentially large localized deformations can develop as soil masses deform and move relative to each other. Such deformations can cause fracture or pull-out of pipe segments embedded within the soil. Permanent ground displacement (PGD) is used here as the demand description. It is recognized that the PGD descriptor ignores the variation in the amount of ground displacement and the direction of ground displacement relative to the pipeline; if the analyst is interested in this kind of detail, then site-specific analytical methods should be used instead of area-wide vulnerability functions.

A.2.3.1 Wave Propagation Damage Database and Fragility Functions

The damage considered for the fragility functions presented in this section is that caused by seismic wave propagation only (no ground failure effects). The empirical dataset used to establish these functions includes 164 data points from 18 earthquakes. Many damage statistics cited were in different formats that necessitated adjustments in order to make them more consistent. The data points are included in (ALA 2001); the data that is used for statistical analysis is described below.

Several aspects about repairs as reported in damage surveys warrant discussion. The first deals with accuracy of repair records that are used as the basis of damage estimation. Detailed damage survey compilations are performed typically by a third party, some time after the water system is restored. Repair records by field crews are commonly used to ascertain damage counts. The main objective of the field crews is to restore the water system to service as rapidly as possible after the earthquake, and understandably, accurate documentation of damage is of secondary importance. Hence, there are inaccuracies in the damage estimates (omission of repair records, vague descriptions of what was damaged, multiple repairs at a single site lumped to one record, etc.). Unfortunately, this uncertainty is inherent to all damage surveys, is likely to vary significantly from earthquake-to-earthquake, and is impossible to quantify.

Very often little or no differentiation of damage severity was included in the damage surveys. The damage was reported in the survey if, and only if, a repair crew actually performed some type of pipe repair at a particular site. If a repair crew repairs a pipe one day after the earthquake, and the same location is repaired again five days after the earthquake, then it is counted as two repairs (the same pipe can be damaged after it is initially repaired once full system pressures are applied, due to continued soil movements, etc.). When the repair crew makes the repair, some type of damage report is developed by the utility. When possible, these damage reports are reviewed by engineers to decipher the cause and type of damage, but often the repair record provides no information as to sort of damage, or such scant information that engineering interpretation may be incorrect. For purposes of system-wide hydraulic analysis, it would be useful to be able to differentiate whether the repair was a "small leak" or a "major failure". A small pipe leak allows the continued system operation thus having relatively low repair priority, whereas a major failure of a pipe requires the local system shut-down, no water can flow, and merits higher priority for repair.

The interpretation of repair records leading to numbers of damaged pipes varies from earthquake-to-earthquake, and exactly what was included in the damage counts is not always clear. Repairs can be to a variety of system components including in-line elements (e.g., pipe, valves, connection hardware) and appurtenances (e.g., service laterals, hydrants, air release valves). Some surveys counted damage to in-line elements for use in repair rate calculations; other surveys included damage to utility-owned service laterals up the utility-customer meter; some surveys included damage to service laterals up the customer house. Pipe damage data that is for damage only to the main pipe is useful for ascertaining relative vulnerability of different pipe materials. However, for level of effort estimates required to restore the water system to its pre-earthquake condition (e.g., crew man-days), then all damage requiring field work ought to be included. Table A.2-1 illustrates the effect of counting repairs to customer service laterals (portion of service pipe from water main to customer utility meter). The surveys shown suggest that the ratio of service lateral repairs to pipe repairs can vary widely, and the numbers of service repairs can even exceed the numbers of pipe repairs (in one of four cases reported; but note that in Japan, the length of service laterals can be quite long, whereas typical U.S. water utilities own only a few feet of service lateral up to the meter connection). If both pipes and service laterals can be repaired during the same site visit then the service damage counts may not be that important, but vice-versa if each requires a separate repair trip. It is recognized that most damage statistics for U.S. earthquakes exclude most damage to service laterals on the customer side of the meter; and that customers have often hired their own private contractors to make service line repairs, at the customer's sole expense. However, water utility staff have occasionally repaired a service line on the customer's side of the meter, and in these instances, the damage would be included in the raw database.

Earthquake	Number of Service Lateral Repairs	Number of Main Pipe Repairs	Ratio Service:Pipe
1995 Hyogoken-nanbu (Kobe) (Shirozu, et al, 1996)	11,800	1,760	6.7:1
1994 Northridge[1] (Toprak, 1998)	208	1,013[2]	1:4.8
1989 Loma Prieta (Eidinger, et al, 1995)	22	113	1:5.1
1971 San Fernando (NOAA, 1973)	557	856	1:1.5
Notes 1. Numbers of field repair records. 2. Includes repairs to hydrants.			

Table A.2-1. Reported Statistics for Main Pipe and Service Lateral Repairs

From a water utility's point of view, the data in Table A.2-1 can be considered as follows. First, calculate the damage to the main pipes, using the vulnerability functions presented in Section A.2.3.3. Second, allow for an additional 20% in terms of number of damage locations to account for damage to service laterals, up to the point of the utility / customer meter. Some type of refined analysis for very long service laterals would be required if these laterals exceed, on average, about half the width of streets.

From a customer's point of view, we recommend adopting the same approach as used by the water utility, except that the selection of the fragility curve should be for the service lateral. Additional damage to the service lateral due to building damage (such as failure of cladding / bricks that impacts the ground over the buried lateral; or differential displacement between the building and the ground) must be added to the total damage estimate.

Most of the empirical data are from four earthquakes: Kobe, Northridge, Loma Prieta and San Fernando. For these four earthquakes respectively, the repair counts were based on number of repairs to:

- Kobe: in-line components and appurtenances
- Northridge: in-line components and hydrants
- Loma Prieta: in-line components and appurtenances
- San Fernando: in-line components

Earthquake	Number of Data Points	Percentage
1995 Hyogoken-nanbu (Kobe)	9	11%
1994 Northridge	35	43%
1989 Loma Prieta	13	16%
1971 San Fernando	13	16%
Other Earthquakes (8)	11	14%
Totals	81	100%

Table A-2.2. Earthquakes and Data Points in Screened PGV Database

The most common material in the database is cast iron (38 points) followed by steel (13), asbestos cement (10), ductile iron (9), and concrete (2). Another 9 points have both cast and ductile iron pipe combined. In terms of pipe diameter, the database contains mostly those sizes associated with distribution main systems, only 8 points were identified as specifically for large diameter pipe (> 12 inch). (See Section A.2.4.7 for further analysis of the database to consider pipe diameter).

Demand used is peak ground velocity (PGV). However, different definitions exist for PGV, e.g., average of the peak horizontal values (from orthogonal directions at a point), geometric mean (square-root of the product of the peak horizontal values), or the peak value from either horizontal direction. Since the intended use of the pipe vulnerability functions is for the loss estimation from possible future earthquakes, it is natural to base them on the geometric mean PGV since this is the quantity typically estimated using modern attenuation relationships (e.g., Sadigh and Egan, 1998). The geometric mean is usually close to the average and is less than the peak of the two directions. Also, some demands were reported in terms of Modified Mercalli Intensity (MMI) or peak ground acceleration (PGA), and for these conversions were made based on Wald et al. (1999). The variability in PGV values from these different methods is probably moot considering the scatter of repair rate when plotted against PGV as shown below.

Other adjustments to the raw data include elimination of data points that were duplicates, contained permanent ground displacement (PGD) effects, or included damage from multiple earthquakes. Judgment was used in these assessments and some errors may be present in the screened database because of misinterpretation of the sometimes vague descriptions contained in the sources. Some sources provided multiple damage statistics for same earthquake, and such duplicate points were eliminated. Several earthquakes had reported repair rates much greater than the others and the source did not specifically indicate whether PGD effects were present. These were judged to include PGD effects and were eliminated. One earthquake had an aftershock of similar intensity as the main shock and the repairs for that earthquake were eliminated.

The database exhibits substantial scatter in plots of repair rate versus PGV. To better discern a causal dependency, PGV ranges were assigned, and repair rates were lumped into the various “bins” according to their associated PGV values. There is a clear trend of larger repair rate with increasing PGV thus suggesting pipe vulnerability functions based on PGV are viable. Two different models were formulated as follows.

Linear (Median) Model. Repair rate RR (repairs per 1000 feet of pipe), is a straight line function of PGV (inches per sec):

$$RR = a \bullet PGV$$

where, a = the median slope of the data point set, and an individual data point slope is taken as the repair rate divided by its associated PGV. Coefficient a = 0.00187 for the data set having all 81 points. The line defined by this model has the property of having equal numbers of points above it and below it. It is one description of central tendency that is not sensitive to data outliers. A two parameter liner model (RR = 0.01427 + 0.001938 * PGV) has a higher slope, reflecting the influence of the high repair rate of outliers.

Power Model. Repair rate is a function of PGV:

$$RR = b \bullet PGV^{c}$$

where, b and c = coefficients set using the standard linear least squares method on log(PGV), and b = 0.00108, and c = 1.173 for the data set having all 81 points.

Both models are about the same especially when considering the scatter in the data points. The models fit the trend of increasing repair rate according to PGV. The data point scatter is large, and the bounds on the variability in term of 84^{th} and 16^{th} percentile lines constructed so that respectively 68 and 13 of the data points fall below. Two-thirds of the points lie between the bounds. The upper bound slope of 0.00529 is 2.8 times the Median Line slope, and the lower bound slope of 0.00052 is 0.28 times, thus indicating a confidence interval for the vulnerability function. The range is relatively large having a factor of 10 between the bounds (= 2.8/0.28). If a single lognormal standard deviation were to be applied, beta would be 1.15.

Additional analyses were performed to assess the influence of pipe material, pipe diameter and earthquake magnitude. For different pipe materials, relative vulnerability was explored by computing Linear models for each material and taking the ratios of the slope coefficients (parameter a). Ductile iron and steel pipe were found to be less vulnerable than cast iron, by less than a factor of two; and asbestos cement was the best performer. These trends are not in keeping with conventional thinking which ranks brittle materials such as cast iron or asbestos cement more vulnerable than ductile materials such as steel or ductile iron by more than a factor of three (e.g., NIBS, 1997). Moreover, statistical tests (Wilcoxon rank-sum) on pairs of material types (e.g., CI versus DI) could not accept the hypothesis (at a 5% significance level) that the individual data point slope populations differ (an exception was between CI and AC). This suggests that the deviations in the Linear Model slope coefficients from different materials could be from sampling error rather than differing statistical populations. In a similar manner, analyses were carried out to assess the effect of pipe diameter, but with only eight data points for large diameter pipe, results did not show much difference in relative vulnerability versus either distribution pipe or small diameter pipe. Finally, duration of strong motion shaking during an earthquake could intuitively have an effect on pipe damage due to cumulative cyclic damage (more cycles of deformation leading to more damage). Earthquake magnitude is a surrogate for the duration of strong shaking, but the magnitudes of the earthquakes in the database were mostly in the range of 6 to 7, and hence no meaningful statistical assessment of a duration effect could be made, even if it is intuitively reasonable to assume that there is such an effect.

The Linear Model can be compared to several others: HAZUS brittle pipe (NIBS 1997, Eguchi et al. 1983 cast iron pipe, Eidinger 1998 cast iron pipe, and Toprak 1998 cast iron pipe). The HAZUS model is that used in the FEMA U.S. national loss estimation methodology. The Eguchi model is one of the earliest which segregated wave propagation from ground failure damage (demand here was converted from MMI to PGV using Wald et al. (1999) equation). The Toprak model represents a recent model based on sophisticated GIS analysis of Northridge pipe damage. The Linear Model and Toprak models agree favorably, and yield repair predictions less than either the HAZUS, Eidinger or Eguchi models.

A.2.3.2 PGD Fragility Functions

The damage considered for the fragility functions presented in this section is that caused by permanent ground deformations (wave propagation effects are masked within the more destructive effects of PGDs). The database contains 42 points from four earthquakes, and liquefaction ground failure is the predominate mechanism (Table A.2-3).

Earthquake	Number of Data Points	Percentage	Ground Failure Type
1989 Loma Prieta	12	28 %	Liquefaction vertical settlement
1983 Nihonkai-Chubu	20 (note 1)	48 %	Liquefaction lateral spread
1971 San Fernando	5	12 %	Local tectonic uplift
1906 San Francisco	5	12 %	Liquefaction lateral spread
Totals	42	100 %	
Note 1. Excludes 14 data points for gas pipes which are listed in database but not used in statistical analysis.			

Table A.2-3. Earthquakes and Number of Points in PGD Database

Material types include asbestos cement (20 points), cast iron (17), and cast iron and steel mixed (5). The diameters are mostly those sizes associated with distribution main systems with only 5 points specifically identified as from large diameter pipe (> 12 inch). It is of interest to note that cast iron gas pipes were reported (Hamada, et al, 1986) to have a trend of higher repair rates than the weaker asbestos cement water pipes in the Nihonkai-Chubu quake because gas leaks were detected much more accurately (implying that many water pipe leaks go undetected). Hamada et al. (1986) did not report the types of joints used in the asbestos cement or steel pipe.

Statistical analysis of the database was carried out in a similar way as that described above for the wave propagation data. The repair rates are about two orders of magnitude greater than those for wave propagation thus indicating the extreme hazard that PGD poses for buried pipe. Even for PGDs up to 5 inches, the repair rate is about 2 repairs per 1000 feet. In the context of post-earthquake water system performance, a system-wide average of only 0.03 "breaks" per 1000 feet of pipe is assigned a serviceability of 50% using the HAZUS methodology, where 100% serviceability corresponds to the pre-earthquake condition. (HAZUS assigns 20% of wave propagation repairs as "leaks", and 80% of ground failure repairs as "breaks.") Hence, those portions of water systems that experience ground failure are likely to be mostly inoperable immediately after the earthquake. Also, the repair rates are somewhat insensitive to PGD value as an order of magnitude increase in PGD only produces a factor of roughly 2 to 3 increase in numbers of repairs.

Both Linear and Power models were fitted to the data. The Linear model has coefficient, a = 0.156, and for the Power model, b = 1.06, c = 0.319. The Power model is a better overall fit to the data (ALA, 2001). However, for relatively small PGDs (which are still quite damaging), it could yield some under-prediction when compared to the median of the data points in this range.

The power model's bounds on the variability in term of 84^{th} and 16^{th} percentile curves have 35 and 6 of the data points fall below, respectively. About two-thirds of the points lie between the bounds. The upper bound is 2.0 times the Power Model, and the lower is 0.45 times the Power model, thus indicating a factor of 4.4 times between the 16^{th} and 84^{th} percentiles. If a single lognormal standard deviation were to be applied to the Power Model, beta would be 0.74.

The Power Model is compared to others: HAZUS brittle pipe (NIBS, 1997), Eidinger (1998) for cast iron pipe and the Harding Lawson model for cast iron pipe (Porter et al, 1991). The HAZUS model is that used in the FEMA U.S. national loss estimation methodology. The median Power Model yields larger repair rates higher than HAZUS, but lower than the Harding Lawson or Eidinger models.

A.2.3.3 Recommended Pipe Fragility Functions

Table A.2-4 provides the recommended "backbone" pipe fragility functions for PGV and PGD mechanisms. These functions can be used when there is no knowledge of the pipe materials, joinery, diameter, corrosion status, etc. of the pipe inventory; and when the evaluation is for a reasonably large inventory of pipelines comprising a water distribution system.

For a simplified evaluation of customer-side service line pipes, when the type of pipe, quality of construction and other design factors are unknown, we would recommend using the functions in Table A.2-4, multiplied by two. The reasons to multiply by two are as follows: service laterals tend to use small diameter pipe which tend to be more affected by corrosion (especially steel pipe); laterals have a relatively high rate of appurtenances (pressure regulators, meters, gate valves, branches) per foot of pipe as compared to water mains. Even with this factor of two increase, the repairs do not include additional damage caused by building-ground differential settlements, which could be added accumulatively as follows: Additional pipe repairs = PGD, ≤ 4, where PGD is measured in inches of settlement or lateral spread. If the service line was installed with flex couplings to absorb settlement PGD, then the additional repairs due to building-ground differential displacement should be = 0.05 PGD, ≤ 2, where PGD is measured in inches of differential settlement (this assumes that the couplings are ineffective for lateral spread). If the service line was designed to meet the requirements of the ALA (2005) for imposed PGDs, then the probability of failure should be small and assigned by the cognizant design engineer.

Hazard	Vulnerability Function	Lognormal Standard Deviation, β	Comment
Wave Propagation	RR=0.00187 * PGV	1.15	Based on 81 data points of which largest percentage (38%) was for CI pipe.
Permanent Ground Deformation	RR=1.06 * $PGD^{0.319}$	0.74	Based on 42 data points of which largest percentage (48%) was for AC pipe.
Notes 1. RR = repairs per 1,000 of main pipe. 2. PGV = peak ground velocity, inches/second .PGD = permanent ground deformation, inches 3. Ground failure mechanisms used in PGD formulation: Liquefaction (88%); local tectonic uplift (12%)			

Table A.2-4. Buried Pipe Vulnerability Functions

A.2.4 Pipe Fragility Functions – Considerations for Analysis

The user can use the fragility functions in Table A.2-4 to predict damage to buried pipes due to ground shaking, liquefaction and landslide. Table A.2-4 should be used if the user has no knowledge of pipe materials, pipe joinery, pipe diameter or soil corrosivity. However, this could produce significantly uncertain results, and may not be suitable for loss estimation or design purposes. Considering these issues, we provide the user with more refined fragility functions in the following sections.

A.2.4.1 Fragility Curve Modification Factors

The fragility curves in Table A.2-4 are "backbone" fragility curves, representing the average performance of all kinds of pipes in earthquakes. Throughout Appendix A.2, there are many discussions as to how various pipe types might behave in earthquakes. Tables A.2-5 and A.2-6 present our summary recommendations as to how to apply the fragility curves in Table A.2-4 to particular pipe types. By diameter, small means 4 inch to 12 inch diameter, and large means 16 inch diameter and larger. Tables A.2-5 and A.2-6 are for pipelines installed without seismic design specific to the local geologic conditions. To apply Tables A.2-5 and A.2-6, the pipe fragility functions in Table A.2-4 are adjusted as follows:

$$RR = K_1(0.00187)PGV \text{ (for wave propagation)}$$

$$RR = K_2(1.06)PGD^{0.319} \text{ (for permanent ground deformation)}$$

Pipe Material	Joint Type	Soils	Diam.	K_1
Cast iron	Cement	All	Small	1.0
Cast iron	Cement	Corrosive	Small	1.4
Cast iron	Cement	Non corr.	Small	0.7
Cast iron	Rubber gasket	All	Small	0.8
Welded steel	Lap - Arc welded	All	Small	0.6
Welded steel	Lap - Arc welded	Corrosive	Small	0.9
Welded steel	Lap - Arc welded	Non corr.	Small	0.3
Welded steel	Lap - Arc welded	All	Large	0.15
Welded steel	Rubber gasket	All	Small	0.7
Welded steel	Screwed	All	Small	1.3
Welded steel	Riveted	All	Small	1.3
Asbestos cement	Rubber gasket	All	Small	0.5
Asbestos cement	Cement	All	Small	1.0
Concrete w/Stl Cyl.	Lap - Arc Welded	All	Large	0.7
Concrete w/Stl Cyl.	Cement	All	Large	1.0
Concrete w/Stl Cyl.	Rubber Gasket	All	Large	0.8
PVC	Rubber gasket	All	Small	0.5
Ductile iron	Rubber gasket	All	Small	0.5

Table A.2-5. Ground Shaking - Constants for Fragility Curve

Pipe Material	Joint Type	K_2
Cast iron	Cement	1.0
Cast iron	Rubber gasket	0.8
Cast iron	Mechanical restrained	0.7
Welded steel	Arc welded, lap welds (large diameter, non corrosive)	0.15
Welded steel	Rubber gasket	0.7
Asbestos cement	Rubber gasket	0.8
Asbestos cement	Cement	1.0
Concrete w/Stl Cyl.	Welded	0.6
Concrete w/Stl Cyl.	Cement	1.0
Concrete w/Stl Cyl.	Rubber Gasket	0.7
PVC	Rubber gasket	0.8
Ductile iron	Rubber gasket	0.5

Table A.2-6. Permanent Ground Deformations - Constants for Fragility Curve

A.2.4.2 Cast Iron Pipe Fragility Curve

The cast iron fragility pipe curve includes the following considerations:

- If the cast iron pipe is located in soils with uncertain corrosive soil conditions, set K1 = K2 = 1.0. This reflects that the bulk of the empirical dataset is governed by cast iron pipe with either cement or lead type joints.

- If the cast iron pipe is in corrosive soils, the damage rate should be higher than if the pipe is in non-corrosive soils. Unfortunately, the bulk of the empirical database does not provide us with information as to soil corrosiveness. We make the engineering judgment that a small diameter cast iron pipe in corrosive soil is about 40% more damage susceptible than the best fit curve from the empirical database, and that cast iron pipe in non-corrosive soils is about 30% less damage susceptible than the best fit curve from the empirical database. This translates to a factor of 2 difference between cast iron pipe in corrosive versus non-corrosive soils (1.4 / 0.7 = 2.0).

- If the cast iron pipe uses rubber gasketed joints (occasional use by some water utilities), assume about 80% of the damage rate for ground shaking and about 80% the damage rate for ground deformation. This reflects that gasketed pipe of all types (AC, DI) have lower damage rates than cement or lead jointed cast iron pipe (more common construction in older cast iron pipe), and factors in the relative earthquake vulnerability of rubber gasketed cast iron pipe.

- The K1 constants in Table A.2-5 can be multiplied by 0.5 for cast iron pipe with 16 inch diameter and larger.

- K2 for restrained CI pipe is set about 30% lower than regular cemented joint CI pipe. There is limited length of restrained CI pipe in use, so there is no empirical data available to confirm this trend. Based on engineering judgment, the restraint offered by bolted joints should provide some extra ability of CI pipe to sustain PGD before being damaged.

A.2.4.3 Asbestos Cement Pipe

The asbestos cement pipe corrections factors K1 and K2 include the following considerations:

- The Loma Prieta earthquake showed that AC pipe in the epicentral area of the earthquake (with rubber gasketed joints and 8 foot to 13 foot pipe segments) had better seismic performance than would have been anticipated by using older empirical models at least in areas subject only to ground shaking.

- The empirical data for rubber gasketed asbestos cement pipe (Loma Prieta 1989, Northridge 1994) differs considerably from previously reported empirical data for asbestos cement pipe in Haicheng or Mexico City (O'Rourke and Ayala). One explanation is that the AC pipe damage in those earthquakes used predominantly cemented joints instead of rubber gasketed joints. Cement joints limit flexibility of the pipe. This factor is considered in differentiating the damage algorithm for AC pipe into two: one for rubber gasketed pipe (better than Cast Iron pipe), and one for cemented joint pipe (similar as for Cast Iron pipe).

- AC pipe in areas subject to settlements (PGDs) have had high damage rate (such as in Turkey, 1999). The K2 factors of 1.0 (cemented) or 0.8 (rubber gasketed) reflect little reduction from the backbone fragility curve for AC pipe.

A.2.4.4 Welded Steel Pipe

The welded steel pipe fragility curve should include the following considerations:

- If the steel pipe is in corrosive soils, the damage rate should be higher than if the pipe is in non-corrosive soils. We make the engineering judgment that a small diameter steel pipe in corrosive soil is about 50% more damage susceptible than the best fit curve from the empirical database, and that small diameter steel pipe in non-corrosive soils is about 50% less damage susceptible than the best fit curve from the empirical database. This translates to a factor of 3 difference between welded steel pipe in corrosive versus non-corrosive soils (1.5 / 0.50 = 3.0). Adjustment for corrosion should be applied only when there is no corrosion protection measures taken and the pipe is in corrosive or moist soil; corrosion measures might include a suitable coating system with sacrificial anodes.

- Note that for steel pipe with corrosion protection which includes suitable coating and sacrificial anodes, or suitable coating with impressed current, the use of correction factors for corrosion may not be suitable.

- Corrosion is an age related phenomenon. Relatively new (under 25 years of age) steel pipe in corrosive soil environments will not be as affected as older steel (over 50 years old) pipe in the same environment. Similarly, corrosion will not play as big a role if special corrosion protection is included in the design. For these cases, use K1 = 0.3 for small diameter welded steel pipe.

- The factor of 3 increase in repair rate is representative of corroded pipe based on 1971 San Fernando, 1983 Coalinga and 1989 Loma Prieta earthquake experience.

- If age is not an attribute that will be available in a limited effort loss estimation study, we recommend that an average corrosion factor of 2 be used when steel pipe is located in corrosive soils. For this case, use K1 = 0.6 for small diameter welded steel pipe.

- The repair rates are decreased for steel pipe having nominal diameters greater than or equal to 12 inches. The 1989 Loma Prieta empirical evidence indicates a repair rate diameter dependency (Eidinger, 1998). Other studies (Sato and Myurata, O'Rourke and Jeon) have also reported lower damage rates for large diameter pipes. Important factors may include: quality of construction; fewer lateral connections; and alignments possibly in better soils. Considering these factors, we have included a diameter dependency for large diameter pipes as follows: repair rates are reduced by 75%. The reduction for repair rates for large diameter pipe probably reflects a number of factors:

- There are few service connections attached to large diameter pipe.
- Corrosion effects on large diameter pipes (which can lead to small pin hole leaks) are not as pervasive for large diameter pipes as for small diameter pipes.
- There are fewer bends and tees in large diameter pipes (stress risers).
- Large diameter pipes have thicker walls to contain an equal amount of pressure, and are hence stronger.
- Large diameter pipes may be installed with better care.
- It is easier to weld large diameter pipes than small diameter pipes.
- Soil loads, as a function of pipe strength, are lower for larger diameter pipe given the same depth of soil cover.

A.2.4.5 Compare Cast Iron, Asbestos Cement and Ductile Iron Pipe

The following observations are made from the empirical evidence from the Northridge (1994) earthquake:

- Ductile Iron pipe had the lowest damage rates at the lowest PGVs.
- AC Pipe had similar damage rates as DI pipe, projected to be the lowest damage rate at PGVs over 14 inches/second.
- Cast Iron pipe had the highest damage rates.

Based on the complete dataset (all earthquakes), vulnerability functions are fitted through data for specific types of pipe. The following models are found for pipe damage due to ground shaking:

- Cast Iron pipe. RR=0.00195 * PGV. Damage rates are 104% (=195/187) of the average. (RR = 0.00195 * PGV for cast iron pipe based on only CI datapoints).

- Ductile Iron pipe. RR=0.00103 * PGV. Damage rates are 55% (=103/187) of the average. (RR = 0.00103 * PGV for DI pipe based on only DI datapoints).

- Asbestos Cement pipe. RR=0.00075 * PGV. Damage rates are 40% (=75/187) of the average. (RR = 0.00075 * PGV for AC pipe based on only AC datapoints).

A.2.4.6 Other Pipe Materials

At the current time, there is insufficient empirical evidence to describe performance for many classes of buried pipe. For example, we have not yet had an earthquake that has severely tested large quantities of PVC pipe with rubber gasketed joints. Similarly, we have not done comprehensive studies to examine the performance of various classes of service lines (copper, steel, various types of plastic, etc.) One should make some recommendation as to how to treat the various classes of pipe.

We make the following recommendations:

- Ductile Iron. Use the cast iron damage algorithm (unknown soil conditions), but scaled by about 0.50 based on the empirical evidence in the 1994 Northridge earthquake. Note that some ductile iron pipe networks include cast iron appurtenances, making them the weak link.

- Welded Steel Arc Welded X Grade. By "X" grade, it is meant welded steel pipelines installed to the general quality controls and design procedures commonly used for oil and gas pipelines, or designed by ALA (2005) with direct computation of pipe stress due to imposed PGV and PGD. Joints are generally butt welded. Use the cast iron damage algorithm (unknown soil conditions), but scaled by 0.01.

- Concrete with Steel Cylinder. These are generally large diameter pipes, typically 24" to 60" in diameter. Three typical pipe joints are used: lap welds of the internal steel cylinder; cemented joints; and Carnegie (rubber gasket) joints. The thin wall of the internal steel cylinder is usually designed to take between one-third and two-thirds of the hoop tension. The limited data available for this type of pipe, coupled with the thin wall and eccentric welds of the internal cylinder, suggest a base rate curve about equal to the average of the empirical dataset. Allowing for the lack of empirical evidence available at this time, and noting that at least one of these pipes 60-inch diameter failed in the 1989 Loma Prieta earthquake at low g levels, it is difficult to establish K1 or K2 constants with much certainty. The approach taken is to set K1 and K2 as somewhat lower than 1.0.

- Riveted Steel. Use about 2 times the arc welded steel damage algorithm. If the rivets are stronger than the base pipe, use the base pipe algorithm.

- Steel, Rubber Gasket. Use the arc welded steel damage algorithm, but scaled by 1.2.

- PVC, Rubber Gasket. Use the asbestos cement damage algorithm (rubber gasket). The rationale is that segmented pipe having similar joint qualities should have similar seismic performance. On a engineering judgment basis, plastic PVC pipe with rubber gasketed joints is somewhat better than similar AC pipe, due to plastic's better tensile strength capability, but is somewhat worse than AC pipe due to longer segment sections, thereby increasing the joint pullout demands. Lacking empirical evidence, we assume equivalent pipe properties. In practice, the relative capacity of rubber gasketed AC vs. PVC pipe is likely to be strongly correlated to the relative insertion depths for the specific installations (a shorter installation depth leads to a weaker pipe).

- Service Laterals. For service laterals of unknown style of construction, and smaller than 2.5-inch diameter, use the base fragility curve times two; plus add in additional damage should there be building-ground differential settlements or lateral spreads as indicated in Section A.2.3.3.

A.2.4.7 Effect of Pipeline Diameter

Various researchers over the past 20 years have considered that the diameter of the pipe has some bearing on the capacity of the pipe to withstand the effects of earthquake without damage.

The strong diameter trend (bigger diameter = much lower damage rate) shown for Northridge data (cast iron pipe) does not show up for Loma Prieta data (bigger diameter = about the same damage rate, possibly a slight decrease). The Loma Prieta data also shows an increasing damage rate with increasing diameter for asbestos cement pipe. The question is why? and how should the fragility curves account for this behavior.

To answer "why?", ideally we would like an explanation which is based on strength of mechanics principles.

A possible explanation of the reasons that small diameter pipe have shown higher damage rates in at least some earthquakes is that the small diameter pipes were located in the worst soil areas, and constructed with the lowest quality control. If these explanations are true, then the diameter effect seen in the Northridge dataset may not be true for another water system.

Tables A.2-7 and A.2-8 present damage data for the combined cities of Kobe, Ashiya and Nishinomiya for the 1995 Kobe earthquake (after Shirozu et al). These tables suggest no particular diameter dependency for common diameter distribution pipes (4" to 12" diameter); a higher rate for very small diameter pipe (≤ 3" diameter, uncommon in the United States except for service laterals); and a moderately lower damage rate for larger diameter pipes (16" and larger). Tables A.2-7 and A.2-8 make no distinction for pipe diameter versus level or type of seismic hazard, so it is possible that the larger diameter pipes were located in areas with less shaking or less ground failure. For Table A.2-8, the total number of repairs were 915 (ductile iron pipe) and 611 (cast iron pipe).

Pipe diameter	Repairs	Length (km)	Repair Rate per km
≤ 75 mm	505	266.1	1.898
100 – 150 mm	1,317	1,423	0.926
200 – 250 mm	412	439.9	0.937
300 – 450 mm	283	362.6	0.783
≥ 500 mm	87	169.5	0.513

Table A.2-7. Pipe Repair, 1995 Kobe Earthquake, By Diameter, All Pipe Materials

Pipe diameter	Repair Rate per km, Cast Iron Pipe	Repair Rate per km, Ductile Iron Pipe	Ratio, DI to CI
≤ 75 mm	2.600	1.029	0.40
100 – 150 mm	1.860	0.486	0.26
200 – 250 mm	1.687	0.545	0.32
300 – 450 mm	0.850	0.480	0.56
≥ 500 mm	0.301	0.061	0.20

Table A.2-8. Pipe Repair, 1995 Kobe Earthquake, By Diameter, CI and DI Pipe

In conclusion, at this time, there is not enough empirical evidence to prove that there will always be a diameter effect for all pipe materials for any water system. However, the empirical evidence strongly indicates that some relationship does exist, and that the largest pipes (over 12 inch diameter) have damage rates that are lower than common diameter distribution pipes (4 inch to 12 inch diameter), and that service laterals (pipes usually under 2.5 inch diameter) have a substantially higher damage rate than common distribution pipes (4 inch to 12 inch diameter).

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A.3 Windows

The design of windows generally does not include explicit factors for seismic design. In Section A.3, we examine the potential seismic performance of different types of windows in different types of buildings.

Section A.3 also examines a possible seismic mitigation to existing windows, namely the installation of transparent film. One purpose of the film is to limit the potential debris generated by broken windows in future earthquakes. By so limiting the debris, there should be some reduction of broken glass induced injuries or fatalities to people.

Section A.3 examines the basis of window damage to develop fragility curves for the glass windows, both in "as-is" condition and possible "upgraded" condition. Glass windows come in a variety of styles, and are installed in a variety of buildings. Both the style of window and the type of structure are important factors in assessing the fragility of a window.

Considering these factors, Section A.3 provides fragility curves as follows:

- Fragility curves for 4 damage states: edge cracking, glazing damage, major cracking of the glass (without fallout), and glass fallout.
- Basic fragility curves for 8 styles of windows. These include 7 styles of single pane annealed glass windows (the most fragile type in common use), and 1 style of fully tempered single pane glass windows. Fragility curves for annealed double pane windows, laminated windows, insulating glass windows, heat tempered windows are not provided, in that these windows compromise a small portion of the total window inventory; and these windows are generally more rugged than single pane annealed windows. The basic fragility curves are based on interstory drifts, so that they are independent of the type of building they are installed in.
- For each of the 8 styles of windows, fragility curves are provided that relate the damage states for individual windows in 9 types of buildings. These fragility curves are based on free field peak ground acceleration (PGA). These fragility curves factor in both the innate fragility of the window and the imposed interstory drifts of different types of buildings.
- For each style of window in each type of building, fragility curves are provided both for the "as is" window and the "upgraded" window.
- The total number of PGA-based fragility curves provided are: 4 x 8 x 9 x 2 = 576.

A.3.1 Background

There are three main characteristics of windows that influence their behavior under seismic motions. These are as follows:

1. Type of glass. This includes annealed, laminated, heat strengthened, tempered and insulating glass units. The glass can be of varying thicknesses, ranging from about 1/8" (thinnest, used only in small windows), 1/4" (common, used for windows of moderate dimensions, such as 5 feet x 6 feet), to 1/2" or thicker.
2. Type of window frame. This includes wood sash, or aluminum, steel and vinyl frames. Attachment of the glass to the window frame can be made with putty or with gaskets ("dry glazing"). A variable in the installation is how much clearance is left between the glass pane and the surface of the frame.
3. Type of structure. Structures which exhibit large interstory drifts will impose higher stresses on windows than structures which exhibit small interstory drifts. For example, regular shaped concrete shear wall buildings and precast concrete tilt up buildings undergo small interstory drifts, and therefore will cause less damage to windows. As another example, steel moment frame buildings, buildings with soft first stories, and buildings with torsional irregularities will undergo large interstory drifts.

A.3.2 Type of Glass

The most common type of window is a single sheet of glass fixed to the structure of the building through aluminum, steel, vinyl or wood frames. The most common type of glass is an annealed pane of glass. The significant engineering characteristics of this type of glass are that it is quite strong in compression, but weak in tension.

There are other types of glass that can be used in windows. These include: laminated glass, tempered glass, and insulating glass units (IGU). Laminated glass is usually two plies of annealed glass, bonded together with a polyvinyl butaryl (PVB) layer. Tempered glass is glass that has undergone a heat treatment process which prestresses the outer skins of the glass, causing a net pre-compression of the glass. Laminated glass could use tempered panes. IGUs are made from two panes of glass, separated by an air tight space; each pane could be annealed or tempered.

The following abbreviations are used:

- A = Annealed (standard glass)
- L = Laminated
- T = Tempered
- IGU = Insulating Glass Unit

A.3.3 Type of Framing

The most common framing materials are aluminum, steel or vinyl (in more modern buildings) and wood sash (in older buildings). The glass panes are set in the windows using either dry glazing (using rubber gaskets in aluminum glazing) or "wet" glazing, using putty in aluminum, vinyl, wood or steel frames. For dry glazing, the clearances between the glass pane and the metal (vinyl) frame will vary from the minimum 1/4" to 3/8" needed to set the glass in the frame (more common), to as much as 1" in modern "wide mullion" type frames (less common). For wet glazing, the putty sealing the glass pane to the window could be relatively flexible (more common), or hard (less common). For wet glazing, the clearances between the glass pane and the wood / metal frame will usually vary between 1/4" to 3/8".

The following abbreviations are used for the more common types of glazing:

- AS = Aluminum frame, soft putty
- AH = Aluminum frame, hard putty
- AGN = Aluminum frame, gasketed, narrow clearance
- AGW = Aluminum frame, gasketed, wide clearance
- WS = Wood sash, soft putty
- WH = Wood sash, hard putty

At the present time, there is insufficient test data for vinyl frames. The use of vinyl frames is becoming more popular, owing to its lower cost versus aluminum frames. The engineering attributes of vinyl frames are probably similar to aluminum frames, with regard to earthquake loading. Therefore, for purposes of this analysis, no distinction is made between aluminum and vinyl frames.

A "fixed" window is a window which cannot be opened. Other types of windows, such as awnings, double hung, single hung, casement and sliding windows, may behave with somewhat higher seismic capacities than fixed windows, owing to their inherent larger flexibility. For purposes of this report, all windows are assumed to be "fixed".

A.3.4 Type of Building

The amount of loading on the window is highly correlated to the level of interstory drift in the building. Therefore, the fragility curves / damage algorithms used in performance based design must consider the type of structure where the window is located.

Most wood frame buildings use plywood or other type of shear walls, and are not usually exposed to considerable interstory drift. However, some wood frame buildings have irregular framing systems, and are exposed to considerable interstory drift along one side of the building.

Buildings which have often had higher glass breakage rates than average include single story commercial shopping malls, which often have one side of the building almost entirely glass plane (the customer side of the building). These types of buildings can undergo considerable torsion, which induces large interstory drifts along the weak side of the building. Buildings with extra tall first floors (soft story) have experienced a higher rate of glass damage in the soft story floor than on other floors.

Under very high levels of earthquake excitation, glass damage becomes incidental to other types of damage in buildings. Once a building is at the "complete" damage state, the building has undergone such severe deformations that the structure itself is compromised. Glass damage in this type of building is common, but likely incidental to the purpose of glass retrofits.

To summarize: the benefits gained from seismically retrofitting (or installing seismically-robust) glass windows in a building are mostly accrued from earthquakes which are severe enough to damage fragile un-retrofitted windows, but not severe enough to heavily damage the structure itself.

There is a wide variety of structural systems used in the general building stock. A first order estimate is provided below.

- Single story wood frame. Perhaps 60% of all buildings.
- Single story temporary classroom, mobile home, manufactured housing. Perhaps 10% of all buildings.
- Two or three story concrete shear wall. Perhaps 5 to 10% of all buildings.
- Two or three story concrete frame. Perhaps 5 to 10% of all buildings.
- Two or three story reinforced masonry. Perhaps 5 to 10% of all buildings.
- Three to eight story steel moment frame. Perhaps 1% of all buildings.
- Two story unreinforced masonry. Perhaps 1% to 2% of all buildings.
- The remaining inventory is comprised of light metal buildings, precast concrete tilt up buildings, steel moment frame buildings with unreinforced masonry walls, steel braced frame buildings, and multi-story buildings.

A.3.5 Type of Window Used by Type of Building

To forecast the damage potential of glass windows, it is necessary to make assumptions about the type of windows used in each type of building. The following table presents a rough estimate of the actual types of windows used public school buildings in California of varying structural systems and vintages.

	Glazing System		
Structural System	Pre 1960	1960-1980	Post 1980
Wood Frame	A-AS = 20% A-AH = 10% A-AGN = 30% A-AGW = 5% A-WS = 15% A-WH = 15% L-T-IGU-All = 5%	A-AS = 10% A-AH = 5% A-AGN = 50% A-AGW = 10% A-WS = 10% A-WH = 5% L-T-IGU-All = 10%	A-AS = 10% A-AH = 5% A-AGN = 50% A-AGW = 20% A-WS = 0% A-WH = 0% L-T-IGU-All = 15%
Temporary Classroom	A-AS = 20% A-AH = 10% A-AGN = 30% A-AGW = 5% A-WS = 15% A-WH = 15% L-T-IGU-All = 5%	A-AS = 10% A-AH = 5% A-AGN = 50% A-AGW = 10% A-WS = 10% A-WH = 5% L-T-IGU-All = 10%	A-AS = 10% A-AH = 5% A-AGN = 50% A-AGW = 20% A-WS = 0% A-WH = 0% L-T-IGU-All = 15%
Concrete Frame, Reinforced Masonry, Concrete Shear Wall, Steel Braced Frame, Precast Concrete Tilt up	A-AS = 30% A-AH = 15% A-AGN = 40% A-AGW = 10% A-WS = 0% A-WH = 0% L-T-IGU-All = 5%	A-AS = 12% A-AH = 6% A-AGN = 60% A-AGW = 12% A-WS = 0% A-WH = 0% L-T-IGU-All = 10%	A-AS = 5% A-AH = 0% A-AGN = 55% A-AGW = 25% A-WS = 0% A-WH = 0% L-T-IGU-All = 15%
Steel Moment Frame	A-AS = 30% A-AH = 15% A-AGN = 40% A-AGW = 10% A-WS = 0% A-WH = 0% L-T-IGU-All = 5%	A-AS = 10% A-AH = 5% A-AGN = 60% A-AGW = 15% A-WS = 0% A-WH = 0% L-T-IGU-All = 10%	A-AS = 5% A-AH = 0% A-AGN = 55% A-AGW = 25% A-WS = 0% A-WH = 0% L-T-IGU-All = 15%
Unreinforced Masonry	A-AS = 20% A-AH = 10% A-AGN = 10% A-AGW = 0% A-WS = 25% A-WH = 25% L-T-IGU-All = 5%	not applicable	not applicable

Table A.3-1. Glazing Systems By Building Type

Abbreviations:

First letter(s) = type of glass. A = annealed, L = laminated, T = tempered, IGU = insulating glass unit. Last letters = type of framing system. A = aluminum frame, W = wood sash, S = soft putty, H = hard putty, GN = gasketed with narrow clearance, GW = gasketed with wide clearance.

A.3.6 Window Damage Modes

Earthquakes impose two types of loads on windows:

- "In plane" loading refers to earthquake loading posed by interstory drift. In plane loading is the primary cause of failure of windows in earthquakes.
- "Out of plane" loading refers to the self-weight inertial loads on a window. Most small to medium size windows can accommodate their own self weight for out of plane loading, a requirement for transport and wind loads.

For purposes of this report, it is assumed that window damage is correlated to the amount of interstory drift and corresponding in-plane loading applied to the window.

The following four damage states are established for windows. For each damage state, the life safety, functionality, short term and long term repair costs are described. The actual fragility levels are listed in subsequent tables, depending on window type and building type.

1. Under very low levels of loading, the putty or dry gaskets in most window frame systems tend to isolate the glass from the structural forces, and the window is not loaded to a significant degree. There is no chance of glass pane damage. LS=1, F=0, SR=0, LR=0.
2. Under moderate levels of loading, the window frame system is racked sufficiently to allow the glass pane to bear directly against the aluminum frame or wood sash; the glass pane takes some of the seismic loads. There is no chance of window fallout, but some slight amount of edge damage to the window may occur. This edge damage will be generally hidden from view, within the glazing pocket in aluminum frames, under the putty in wood sash frames. The slight amount of edge damage could, in rare cases, adversely affect the air tightness of the window, or reduce the strength of the glass to resist subsequent wind forces or projectile forces. It is rare that building owners will repair this type of damage, although the useful lifetime before the window is eventually replaced could be reduced. LS=1, F=0.1 (mostly functional), SR=0, LR=0.02 (2 in 100 windows are replaced).
3. As the level of drift increases, and the glass panes are seriously pushed up against the framing systems, the principle tensile stresses in the glass increase to a point where the glass fractures. The locations of glass fracture will depend upon: the strength of the glass pane; the location of support blocks used in fit up of the

glass pane; hardness of the window frame (wood is more forgiving, allowing the glass pane to sustain higher loads before fracture; steel frames are less forgiving, causing stress risers and early fracture of the glass pane); and whether the glass pane has had beveled edges (less common, stronger) or flat edges (more common, weaker). For fully tempered glass, the initial crack will generally lead to complete window fallout, in a "dicing" pattern. For annealed glass, the initial fracture does not lead to immediate glass fallout. As the earthquake causes the building to dynamically reverse direction, the glass is exposed to the opposite type of loading, and the glass will eventually fracture at multiple locations. With repeated cycles of interstory drift, the glass will eventually be cracked in many planes, allowing pieces (shards) of the glass to fall out of the window. At the end of the earthquake, some of the glass window still is held in place in the window frame, but a significant percentage (5% to 75%) may have broken out of the window frame and lie either on the interior or exterior of the building. LS=2, F=0.7 (a few windows remain functional to keep out weather), SR=0.05 (clean up glass shards), LR=1.00 (window is replaced).

4. When the glass does fracture and fall out, the distance that the glass will move outwards from the building depends upon the height of the glass above the ground level. After the 1978 Miyagi-ken-oki earthquake in Japan, Sakamoto et al (1984) did field investigations to assess the broken glass coverage as a function of height of the original window, versus the observed horizontal distance away from building that the glass fell. The maximum horizontal distance of glass fallout was found to be about 50% of the height of the window above the ground; on average, about 33% of the height above the ground. LS=3, F=1.0, SR=0.06 (clean up glass shards), LR=1.00 (window is replaced).

A.3.7 Empirical Evidence in Past Earthquakes

The following sections provide brief summaries of glass damage in some recent earthquakes. To keep this report brief, no attempt has been made to be exhaustive in presenting the empirical evidence. Except where stated, the empirical evidence is based upon the reconnaissance efforts done by the author of this report.

A.3.7.1 Mexico City, 1985

After the 19 September 1985 Mexico City earthquake, a working group was organized to study the implications of glass damage in buildings. Over 300 buildings were investigated. The findings were as follows (Evans and Ramirez 1989, Goodno and Wolff, 1989):

- About half, or more, of commercial buildings receiving some sort of structural or envelope damage during the earthquake can expect some glass damage. Approximately one-quarter of these buildings can expect serious glass damage.

- Those buildings experiencing large amounts of drift (more flexible buildings) received three to four times as much serious glass damage as buildings not experiencing large amounts of drift.
- Buildings with complex or irregular configurations received twice as much structural and serious glass damage as those with regular configurations.
- Buildings adjoined by other buildings from 25% to 75% as tall, and subject to pounding, received twice as much serious glass damage as buildings with either much lower or higher adjoining buildings.
- Smaller window glass areas received less serious glass damage than larger glass areas. Large glass windows received three times as much serious damage as small glass windows, other factors being equal. Glass windows with edge dimensions less than about 5 feet or windows with areas less than about 20 square feet appear to receive the least amount of serious glass damage.
- Vertical glass shapes received twice as much serious glass damage as horizontal or square shapes.
- The more flexible glazing systems (metal frames) received twice as much serious glass damage as the more rigid systems.

A.3.7.2 Loma Prieta, 1989

In the downtown San Francisco area, levels of shaking were about 0.06g to 0.10g. One building, the Pacific Gas and Electric headquarters buildings (32 stories tall, steel moment frame, soft story), exhibited no window damage from floors 2 through 32; however, some windows (full height) on the first floor were broken with glass fallout. There were no injuries from the glass fallout. This damage pattern was similar throughout the area.

In the Nob Hill area of San Francisco, the building inventory is primarily wood frame residential construction, dating back from 1910 to 1920. Ground shaking was about 0.10g. No windows were broken sufficiently to cause glass fallout in the area, including hundreds of buildings.

In the Marina District of San Francisco, where ground shaking was about 0.20g to 0.25g, including amplifications from soft soils, there was window damage with glass fallout, but primarily to buildings which suffered significant structural damage and/or collapse. Buildings which were not in the extensive or complete damage states had little glass window fractures or fallout.

A.3.7.3 Northridge, 1994

In the Simi Valley area, where ground shaking was 0.30g to 0.50g or above, there was glass damage to many storefront windows in commercial "mall" type shopping centers. Within one or two days after the earthquake, some of the damaged windows (those with large cracks or those with glass fallout) had been replaced by plywood panels (temporary measure). In the shopping malls with glass damage, about 25% of the windows were damaged. Glass window damage to nearby residential structures (wood frame and mobile home) was much lower.

In the Northridge area, there was little glass damage to mid rise steel moment hospital frame structures built past 1960, where ground level shaking was about 0.40g.

For equivalent building materials, it has been found that there is systematically more window damage to larger buildings than to smaller buildings. For example, a 12,000 square foot single story wood frame shopping mall plaza is more likely to suffer slight to moderate damage than six single story wood frame 2,000 square foot residential buildings, all other factors being equal. This is particularly true at higher PGA levels (0.20g and above).

A.3.7.4 Kobe, 1995

In the city of Kobe, Japan, ground shaking was on the order of 0.40g to 0.50g. Most undamaged modern high rise buildings exhibited no window pane breakage or fallout. Many buildings had single floors which "pancaked" due to failure of the structural system; on the pancaked floors, all windows were broken with glass fallout; on other floors, essentially no windows were broken.

Glass damage in wood frame structures was severe, but usually only when the drifts in the structures were high enough to cause partial or complete collapse of the buildings.

A.3.8 Test Data on Windows

A.3.8.1 Dry Glazed, Narrow Mullion

Behr (Behr et al, 1995) conducted seismic testing of different types of glass commonly used in dry glazed, narrow mullion aluminum window frames.

Glass types included in the tests represented the five most common types of glass windows used in storefronts, based on a survey with 25 glass installation companies in 1992. According to the survey, 1/4" annealed monolithic glass constitutes the vast majority of architectural glass in existing storefront wall systems. For newer installations, IGUs are becoming more common, and can be the material of choice in the future due its superior energy performance attributes. Fully tempered and annealed glass (1/4") are generally used when safety glazing is required by law, usually when glass panels are within 18 inches of the ground or within 4 feet of a doorway. In addition, annealed laminated glass is commonly used when security of the building envelop is a significant concern. According to the survey, dry-glazed, narrow mullion systems are the most common glazing configuration used for architectural glass in storefront applications.

From the testing, Behr identified a number of damage states for glass, which are renamed below to match the typical naming convention used in loss estimation efforts. The following describes the damage states.

- No damage. The window survives the seismic loading with no damage.
- Slight damage. Glass edge damage. The glass pane survives the seismic loading. A portion of the perimeter of the glass pane embedded in the glazing, usually near the corners, has spalled in small pieces or flakes or has suffered small cracks (less than 1 inch long) due to interaction with the framing system. The total length of edge damage is usually under 2% of the entire perimeter of the glass pane. The framing system may be gouged by 1/4" annealed monolithic or laminated glass. Often, this damage is unnoticed after the earthquake, unless the framing system is removed. This form of damage is thought to contribute to longer term subsequent damage to the glass panel due to subsequent thermal stresses or stresses due to wind loading. This type of damage has the potential of allowing water vapor to infiltrate the glass panel. On average, few building owners will repair this type of damage. On average, this damage state represents about 5% loss of the replacement value of the window.
- Moderate damage. Glass translations and gasket failures. The interaction of the glass panel with the framing system has caused the glass panel to translate and rotate within the framing system. At the end of the earthquake, the glass translation may be noticeable, especially if there is engraving or lettering on the glass. The usual translation and rotation of the panel is less than 1 inch; however, if the translation is sufficient, the gasket system that holds the glass pane in the framing system may become compromised. If the gasket material is compromised, the water and air tightness of the glass window would be compromised. Some owners will elect to repair the damage, which entails deglazing, repositioning the glass pane, repair of any damaged gasket material, and reglazing the glass panel - a labor intensive effort. On average, this damage state represents about 15% loss of the replacement value of the window. Glass translation less than 1/8" inch is not considered significant.
- Extensive damage. At the extensive damage level, the glass panel has suffered cracks which extend into the visible part of the glass. Major crack / shard patterns are formed. Cracks are usually 3 to 18 inches long. Shards may be formed (a shard is a piece of glass which is cracked along its entire perimeter). At the extensive damage state, the shard is still in the window frame; in the complete damage state, the shard has fallen out. The damaged glass pane remains in the frame. The window will have to be replaced after the earthquake. For annealed and laminated annealed glass, reaching the extensive damage state does not mean that there will be fallout. For fully tempered glass, reaching this damage state is the same as the complete damage state, as significant fracture of tempered glass almost always leads to complete pane fallout. On average, this damage state represents 100% loss of the replacement value of the window.

- Complete damage. At the complete damage level, the glass panel has suffered severe cracks and major parts of the glass pane have fallen out. On average, this damage state represents 100% loss of the replacement value of the window.

Key results for annealed monolithic glass in narrow mullion aluminum frames:

- At a drift ratio of 0.024, no glass suffered large cracks and there was no glass breakage. However, in 9 of 9 tests, the windows all suffered glass edge damage and some amount of damage to the glazing.
- At a drift ratio of 0.047, 11% of the glass windows suffered large cracks and glass fallout; 56% of the glass windows suffered large cracks but no significant glass fallout; and 33% of the glass windows suffered minor to moderate damage (edge cracks and glazing damage).

A.3.8.2 Dry Glazed, Wide Mullion

A series of 72 tests were performed on 5 feet wide x 6 feet tall glass panes (Pantelides and Behr, 1994). These 72 tests covered 8 types of glass panes, covering a range of annealed monolithic glass, annealed laminated glass and fully tempered glass.

One set of tests was performed on 9 panes of 1/4" annealed monolithic glass, including a 0.004 inch thick polyethylene terephthalate (PET) adhesive film made by Madico Inc. of Woburn, Massachusetts. The film was not anchored to the curtain wall framing members, but was positioned on the glass to leave a 5/8" wide unfilmed border around the perimeter to simulate a retrofit film installation on in situ glass. Unanchored window film installations are most common in current industry practice.

The glass panes were set in aluminum frames. The aluminum frames include a glazing pocket. The glass pane is set in the glazing pocket, resting on neoprene glazing gaskets. The glass included 1/2" bite (bite = distance of insertion into the glazing pocket) in each glazing pocket. The distance between the glass edges and the aluminum frame was 1", which is very generous (hence the term "wide mullion"). It is generally felt that glass held in wide mullion systems is more capable of withstanding interstory drift than glass held in narrow mullion systems.

The combined aluminum frame and glass system was tested using static displacement controlled tests, to simulate in-plane racking caused by interstory drifts. While the applied displacements were monitored, the observed story drifts represent the total drift in the floor, of which the glass panes represent only about half the story height. Actual drift ratios on the glass panes were not recorded.

Each glass pane was tested in increments of ever increasing interstory drift. The Phase I tests applied drift compatible with a 15 story steel moment resisting frame undergoing significant vibrations. The maximum displacement applied was 2.95 inches over a frame height of about 12 feet, representing a interstory drift ratio of 0.020; this was applied for

5 cycles. To account for damage from lower drift levels, the loading also included 70 displacement cycles covering a range of lower interstory drift ratios.

No glass pane was damaged sufficiently in the Phase I tests to cause sufficient glass breakage to cause glass fallout.

A Phase II test program was then conducted to a maximum interstory drift of 0.0175, repeated for 600 cycles (10 sets of 60 cycles each); plus 600 cycles at interstory drift ratios of 0.0068, 0.0041 and 0.0027. Glass damage at the end of these tests were recorded. Glass fallout damage was noted in many of the glass pane types at the end of these tests. Note that the Phase II tests did not apply drifts higher than in the Phase I tests; instead, the Phase II loading can be considered representative of applying a major earthquake about 8 times to a single glass window; and then recording the damage at the end of the 8th earthquake.

A Phase III test program was then conducted. Each Phase III test repeated the Phase I and Phase II tests, beginning with an initial interstory drift of 0.0205. Thus, the maximum Phase III drift ratios were about 0.04.

When the 1/4" glass pane with film did crack, the effect of the film was as follows. At lower levels of drift, the film tended to keep smaller shards from falling out of the window that would otherwise have fallen out. As drift increased, and the glass crack pattern extended over long lengths, the effect of the film was to contribute to fall out of the entire pane out of the window, as the broken section pulled out the entire glass pane.

When fully tempered laminated glass broke, the entire pane would tend to fall out. At failure, each glass pane exhibited a dicing pattern, typical of fully tempered glass; the fragments were held together by the interlayer. When both layers broke, the window acted as a "wet carpet", and would fall out if subsequent racking caught an edge of the aluminum mullion on the edge of the broken glass, thus forcing the window out of the frame.

Key results for annealed monolithic glass in wide mullion aluminum frames:

- At a drift ratio of 0.020 (end of Phase I tests), no glass suffered large cracks and there was no glass breakage. In some tests, the windows suffered glass edge damage and some amount of damage to the glazing.
- At a drift ratio of 0.020 (end of Phase II tests), about 95% of the glass windows suffered large cracks with glass fallout. Note that this test is representative of glass performance after about 8 major earthquakes. If major earthquakes occur on a 100 year time horizon, the glass damage reflects the cumulative damage to a window after 800 years. It is not reasonable to expect that glass windows will stay in place for 800 years. For this reason, the Phase I drift ratio is more indicative of glass window damage in a single earthquake, and the Phase II drift ratio is indicative that window damage is cumulative (i.e., glass damage is sensitive to the

duration of the earthquake). This suggests that a glass window may have the same withstand capacity for a short duration local magnitude 6 earthquake (high PGA, high drift, one or two cycles of maximum loading) versus a long duration distant magnitude 8 earthquake (moderate PGA, moderate drift, many cycles of maximum loading).

- At a drift ratio of 0.039 (end of Phase III tests), 100% of the glass windows suffered large cracks and glass fallout. Note that at the end of Phase III tests, a single glass window had undergone the equivalent of about 18 major earthquakes, followed by a 19th earthquake with the building beginning in a severely racked position.

Key results for annealed monolithic glass in wide mullion aluminum frames with PET film:

- At a drift ratio of 0.020 (end of Phase I tests), no glass with film suffered large cracks and there was no glass breakage. Minor damage was about equivalent as for annealed windows without the film.
- At a drift ratio of 0.020 (end of Phase II tests), about 40% of the glass windows with film suffered large cracks with glass fallout.
- At a drift ratio of 0.039 (end of Phase III tests), 50% of the glass windows with film suffered large cracks and glass fallout.

A.3.8.3 Putty Glazed, Wood Sash and Aluminum Frame

A series of 39 tests were performed by Bouwkamp (1960) to examine the performance of windows in school buildings. The types of windows considered in this test program were: sash material, clearance between glass and sash, hardness of putty used to connect the window to the sash.

Windows were 4 feet wide by 2 feet high; 4 feet wide by 4 feet high, and 4 feet wide by 8 feet high. These were thought to be the most common window sizes used in schools. The windows were mounted three types of frames: aluminum frame, steel frame, or wood sash. The window panes were attached to the frames either on all four sides, or sometimes on only the top (head) and bottom (sill) sides.

The glass panels were standard commercial glass, ranging from 1/8" to 1/4" in thickness. The glass to pane clearances were between 1/8" and 3/16".

The putty used to attach the glass panes to the sashes were specified as either "hard" of "soft". Soft putty was considered representative of recent (1950s) installations, made with standard commercial putty, namely Fuller's Steel DAP 1012. Hard putty was considered representative of old installations, made with a hard putty substitute, a gypsum-celite mixture.

Loading was applied to the windows by means of statically increasing in-plane load, and monitoring the resulting drifts. Loading was stopped once the glass suffered large cracks, which generally occurred before there was glass fallout.

Key findings from the tests were as follows:

Type of Frame	Type of Glazing	Drift Ratio when Major Cracks Form in Glass
Aluminum	Soft Putty	0.030
Steel	Soft Putty	0.034
Aluminum	Hard Putty	0.0064
Wood Sash	Soft Putty	0.051

Table A.3-2. Drift Ratios from Test at Glass Breakage

A.3.9 Fragility Curves

Fragility curves are developed for purposes of performance based design. The fragility curves are primarily based upon the seismic testing, accounting for the empirical evidence of damage to windows in past earthquakes.

A.3.9.1 Damage States

Five damage states are defined. Replacement value is defined as the cost to remove the glazing, insert a new glass window, and reglaze the window. The replacement value is a portion of the cost of an entire new window, less the material value of the frame. Earthquake damage to the window frame itself is usually not sufficient to warrant frame replacement.

1. No damage.
2. Slight damage. Window suffers edge cracking, but is not noticeable. Loss = 5% of window replacement value.
3. Moderate damage. Window suffers edge cracking, some noticeable translation, some damage to glazing material. Many windows will not be repaired, but some will. Loss = 15% of window replacement value.
4. Extensive damage. The window has cracked. Cracks are often 3 inches in length, but can extend the length of the window. For annealed monolithic and annealed laminated glass, the window remains in the pane without significant glass fallout. Minor glass fallout is possible (less than 1 square inch of window). For fully tempered glass, the Extensive damage state immediately leads to essentially complete glass fallout. Loss = 100% of window replacement value.
5. Complete damage. The window is cracked, and a part or all of the window has fallen out. For annealed glass, glass fallout is usually between 25% and 75% of the area of the pane. Loss = 100% of window replacement value.

Replacement value is defined as the cost to remove the glazing, insert a new glass window, and reglaze the window. The Loss ratios above represent the cost to replace the window to its pre-earthquake condition. Other losses (casualties, functional down time, etc.) are not included in the above Loss ratios.

A.3.9.2 Fragility Curves - Versus Drift

Tables A.3-3 through A.3-10 provide the fragility curves for eight common types of windows. For each type of window, there are four fragility curves, representing the four damage states. The values provided are interstory drift ratios at which 50% of the windows would reach the given damage state. The lognormal dispersion parameter for all fragility curves is 0.64, which incorporates the uncertainty in the window capacity and the ground motion. These fragility curves were developed by comparison with observed test failure rates (Section A.3.8) and the empirical evidence (Section A.3.7).

The "as is" fragility curve represents the existing window. The "upgraded" fragility curve represents the same window, with the addition of PET film to one side of the glass pane, leaving a unfilmed perimeter between the glass and the window frame.

As can be observed in the fragility curves, the addition of the film has no impact on the slight and moderate damage states (edge damage, glazing damage). Testing of filmed windows by Behr confirmed this finding. The "extensive" damage state fragility is just slightly higher with film than without film. This reflects that the tests found that filmed windows break at just about the same drift levels as unfilmed windows, but that filmed windows tend to retain small shards (under 1 inch square or so) better than unfilmed windows. The addition of film is shown to substantially improve the complete damage state for annealed monolithic windows. Actual tests showed that filmed windows will still have complete pane fallout once the glass cracking is substantial enough to cause more than half of the pane to become disconnected from the frame; the large weight of the broken glass is enough to pull the entire pane out of the frame.

Damage State	Drift Ratio - As Is	Drift Ratio - Upgraded
Slight	0.010	0.010
Moderate	0.018	0.018
Extensive	0.030	0.040
Complete	0.050	0.070

Table A.3-3. Annealed Monolithic Glass, Aluminum Sash, Soft Putty A-AS

Damage State	Drift Ratio - As Is	Drift Ratio - Upgraded
Slight	0.012	0.012
Moderate	0.024	0.024
Extensive	0.040	0.050
Complete	0.100	0.140

Table A.3-4. Annealed Monolithic Glass, Aluminum Frame, Dry Glazing, Narrow Mullion A-AGN

Damage State	Drift Ratio - As Is	Drift Ratio - Upgraded
Slight	0.003	0.003
Moderate	0.005	0.005
Extensive	0.006	0.007
Complete	0.009	0.014

Table A.3-5. Annealed Monolithic Glass, Aluminum Sash, Hard Putty A-AH

Damage State	Drift Ratio - As Is	Drift Ratio - Upgraded
Slight	0.012	0.012
Moderate	0.024	0.024
Extensive	0.060	0.070
Complete	0.120	0.200

Table A.3-6. Annealed Monolithic Glass, Aluminum Frame, Dry Glazing, Wide Mullion A-AGW

Damage State	Drift Ratio - As Is	Drift Ratio - Upgraded
Slight	0.015	0.015
Moderate	0.030	0.030
Extensive	0.050	0.060
Complete	0.125	0.200

Table A.3-7. Annealed Monolithic Glass, Wood Sash, Soft Putty A-WS

Damage State	Drift Ratio - As Is	Drift Ratio - Upgraded
Slight	0.004	0.004
Moderate	0.006	0.006
Extensive	0.008	0.009
Complete	0.012	0.018

Table A.3-8. Annealed Monolithic Glass, Wood Sash, Hard Putty A-WH

Damage State	Drift Ratio - As Is	Drift Ratio - Upgraded
Slight	0.020	0.020
Moderate	0.030	0.030
Extensive	0.180	0.190
Complete	0.180	0.200

Table A.3-9. Fully Tempered Glass, Aluminum Frame, Dry Glazing, Wide Mullion T-AGW

Damage State	Drift Ratio - As Is	Drift Ratio - Upgraded
Slight	0.020	0.020
Moderate	0.035	0.035
Extensive	0.120	0.130
Complete	0.200	0.280

Table A.3-10. Laminated Glass, Aluminum Frame, Dry Glazing, Narrow Mullion

A.3.9.3 Fragility Curves - Versus PGA - Methodology

To apply the above fragility curves for a specific building, it is not uncommon to free field peak ground acceleration (PGA) as the input parameter.

The drift fragility curves in Section A.3.8.2 are converted to PGA fragility curves using the following procedure.

- Establish the drifts to reach varying damage states for the building. For a unreinforced masonry (URM) building, it takes a drift of 0.010 to reach the slight structure damage state, 0.018 to reach the moderate structure damage state, etc.
- Establish the free field PGAs necessary to achieve these structure drifts. For a URM building, the PGA needed to reach a drift of 0.010 is 0.18g. Due to degradation of the building structural capacity with increasing drift, the relationship between drift and PGA is nonlinear.
- The design professional can perform a building-specific analysis to convert building drift to PGA; or even establish drift by floor level. Such an analysis will always be preferred to the simplified conversions done herein.

Following this example for a A-AS window in a URM (Table A.3-5), the A-AS fragility curve, using PGA, is as follows:

Damage State	Drift Ratio As Is	PGA As Is	Drift Ratio Upgraded	PGA Upgraded
Slight	0.010	0.18 g	0.010	0.18 g
Moderate	0.018	0.25	0.018	0.25
Extensive	0.030	0.32	0.040	0.38
Complete	0.050	0.44	0.070	0.56

Table A.3-11. Annealed Monolithic Glass, Aluminum Sash, Soft Putty A-AS (PGA)

Interpretation of the fragility curves in Table A.3-11 is as follows:

- At a PGA of 0.18g, 50% of all A-AS windows in a URM will be in the slight damage state or worse damage state, assuming that the URM building remains standing. This implies that URM building will have undergone a maximum interstory drift ratio of 0.010.
- At a PGA of 0.44g, 50% of all A-AS windows in a URM will be in the complete damage state (glass fallout), assuming that the URM building remains standing. The remaining 50% of windows will be in the extensive, moderate or slight damage states, with a very small chance that some windows may be undamaged. This implies that URM building will have undergone a maximum interstory drift ratio of 0.050 and remains standing.

Since there is a reasonable chance that part of the URM building will have collapsed at a PGA of 0.44g, all windows within that collapsed part will also be completely broken. The fragility curves in Table A.3-11 do not take this into account. For purposes of only performing the window film upgrade, the effectiveness of the upgrade must be adjusted to account for the benefits accrued in the extensive and complete damage states for the window, only for those times that the building itself has not reached the complete damage state with partial or complete collapse.

A.3.9.4 Fragility Curves - Window Type A-AS

Tables A.3-12 and A.3-13 provide the fragility curves for the annealed single pane glass windows in aluminum frames with soft putty glazing (A-AS), for 9 classes of structures. The structures are assumed to be designed to California building codes commonly in place from 1970 through 1990 (any age for non-retrofitted URM buildings).

The fragility curves indicate that mobile home / manufactured housing / temporary classroom type structures are the most vulnerable to glass damage. This is because these types of structures are very flexible, and can have considerable drift. The large drifts damage the glass. If the specific mobile home structure is more rigidly constructed, then this will not be the case.

The next most vulnerable group of windows are those in unreinforced masonry structures. As the URM buildings are damaged, they undergo large drifts, leading to glass failures.

Building Type	Slight (Edge Damage)	Moderate (Glazing)	Extensive (Cracks)	Complete (Glass Fallout)
URM	0.18 g	0.25 g	0.32 g	0.44 g
Precast Tilt up	0.31	0.44	0.63	0.85
Reinf. Masonry	0.32	0.45	0.74	1.05
Concrete Frame	0.17	0.64	0.89	1.16
Conc. Shear Wall	0.53	0.70	0.95	1.19
Steel Braced Fr.	0.53	0.74	1.05	1.34
Steel Moment Fr.	0.33	0.51	0.77	1.09
Mobile Home	0.18	0.23	0.31	0.39
Wood Frame	0.54	0.74	0.92	1.23

Table A.3-12. Fragility Curves - Window Type A-AS - By Building Type - As Is

Building Type	Slight (Edge Damage)	Moderate (Glazing)	Extensive (Cracks)	Complete (Glass Fallout)
URM	0.18 g	0.25 g	0.38 g	0.56 g
Precast Tilt up	0.31	0.44	0.74	1.07
Reinf. Masonry	0.32	0.45	0.89	1.35
Concrete Frame	0.17	0.64	1.02	1.43
Conc. Shear Wall	0.53	0.70	1.07	1.42
Steel Braced Fr.	0.53	0.74	1.20	1.63
Steel Moment Fr.	0.33	0.51	0.93	1.42
Mobile Home	0.18	0.23	0.35	0.48
Wood Frame	0.54	0.74	1.08	1.45

Table A.3-13. Fragility Curves - Window Type A-AS - By Building Type - With Film Upgrade

Inherent in these fragility curves is the usual dispersions for building quality. If applied to an actual building, then the fragility levels will increase if the actual building has no irregularities (torsion, soft story, etc.) or was designed to higher than UBC standard quality levels (Z=0.40g, I = 1.0). The fragility levels will decrease if the actual building includes substantial irregularities.

A.3.9.5 Fragility Curves - Window Type A-AGN

Tables A.3-14 and A.3-15 provide the fragility curves for the annealed single pane glass windows in aluminum frames with narrow mullion aluminum frames with dry glazing (A-AGN), for 9 classes of structures.

Building Type	Slight (Edge Damage)	Moderate (Glazing)	Extensive (Cracks)	Complete (Glass Fallout)
URM	0.20 g	0.28 g	0.38 g	0.74 g
Precast Tilt up	0.35	0.54	0.74	1.40
Reinf. Masonry	0.38	0.62	0.89	1.81
Concrete Frame	0.51	0.76	1.02	1.83
Conc. Shear Wall	0.57	0.82	1.07	1.78
Steel Braced Fr.	0.58	0.89	1.20	2.07
Steel Moment Fr.	0.37	0.64	0.95	1.90
Mobile Home	0.19	0.27	0.35	0.60
Wood Frame	0.60	0.83	1.08	1.79

Table A.3-14. Fragility Curves - Window Type A-AGN - By Building Type - As Is

Building Type	Slight (Edge Damage)	Moderate (Glazing)	Extensive (Cracks)	Complete (Glass Fallout)
URM	0.20 g	0.28 g	0.44 g	0.98 g
Precast Tilt up	0.35	0.54	0.85	1.84
Reinf. Masonry	0.38	0.62	1.05	2.42
Concrete Frame	0.51	0.76	1.16	2.36
Conc. Shear Wall	0.57	0.82	1.19	2.25
Steel Braced Fr.	0.58	0.89	1.34	2.66
Steel Moment Fr.	0.37	0.64	1.09	2.55
Mobile Home	0.19	0.27	0.39	0.77
Wood Frame	0.60	0.83	1.45	2.07

Table A.3-15. Fragility Curves - Window Type A-AGN - By Building Type - With Film Upgrade

A.3.9.6 Fragility Curves - Window Type A-AH

Tables A.3-16 and a.3-17 provide the fragility curves for the annealed single pane glass windows in aluminum frames with hard putty glazing, for 9 classes of structures.

Building Type	Slight (Edge Damage)	Moderate (Glazing)	Extensive (Cracks)	Complete (Glass Fallout)
URM	0.10 g	0.13 g	0.14 g	0.17 g
Precast Tilt up	0.15	0.22	0.25	0.30
Reinf. Masonry	0.14	0.22	0.26	0.32
Concrete Frame	0.22	0.32	0.35	0.43
Conc. Shear Wall	0.30	0.38	0.41	0.51
Steel Braced Fr.	0.28	0.36	0.40	0.51
Steel Moment Fr.	0.13	0.19	0.22	0.30
Mobile Home	0.08	0.14	0.15	0.17
Wood Frame	0.23	0.38	0.41	0.50

Table A.3-16. Fragility Curves - Window Type A-AH - By Building Type - As Is

Building Type	Slight (Edge Damage)	Moderate (Glazing)	Extensive (Cracks)	Complete (Glass Fallout)
URM	0.10 g	0.13 g	0.15 g	0.22 g
Precast Tilt up	0.15	0.22	0.27	0.38
Reinf. Masonry	0.14	0.22	0.28	0.42
Concrete Frame	0.22	0.32	0.37	0.55
Conc. Shear Wall	0.30	0.38	0.45	0.62
Steel Braced Fr.	0.28	0.36	0.44	0.64
Steel Moment Fr.	0.13	0.19	0.24	0.42
Mobile Home	0.08	0.14	0.16	0.21
Wood Frame	0.23	0.38	0.44	0.66

Table A.3-17. Fragility Curves - Window Type A-AH - By Building Type - With Film Upgrade

A.3.9.7 Fragility Curves - Window Type A-AGW

Tables A.3-18 and A.3-19 provide the fragility curves for the annealed single pane glass windows in aluminum frames with wide mullions, with dry glazing, for 9 classes of structures.

Building Type	Slight (Edge Damage)	Moderate (Glazing)	Extensive (Cracks)	Complete (Glass Fallout)
URM	0.20 g	0.28 g	0.50 g	0.86 g
Precast Tilt up	0.35	0.54	0.96	1.62
Reinf. Masonry	0.38	0.62	1.20	1.96
Concrete Frame	0.51	0.76	1.29	2.10
Conc. Shear Wall	0.57	0.82	1.30	2.01
Steel Braced Fr.	0.58	0.89	1.49	2.36
Steel Moment Fr.	0.37	0.64	1.25	2.22
Mobile Home	0.19	0.27	0.43	0.68
Wood Frame	0.60	0.83	1.34	2.01

Table A.3-18. Fragility Curves - Window Type A-AGW - By Building Type - As Is

Building Type	Slight (Edge Damage)	Moderate (Glazing)	Extensive (Cracks)	Complete (Glass Fallout)
URM	0.20 g	0.28 g	0.56 g	1.34 g
Precast Tilt up	0.35	0.54	1.07	2.50
Reinf. Masonry	0.38	0.62	1.35	3.33
Concrete Frame	0.51	0.76	1.43	3.17
Conc. Shear Wall	0.57	0.82	1.42	2.96
Steel Braced Fr.	0.58	0.89	1.63	3.53
Steel Moment Fr.	0.37	0.64	1.42	3.51
Mobile Home	0.19	0.27	0.48	1.01
Wood Frame	0.60	0.83	1.45	2.91

Table A.3-19. Fragility Curves - Window Type A-AGW - By Building Type - With Film Upgrade

A.3.9.8 Fragility Curves - Window Type A-WS

Tables A.3-20 and A.3-21 provide the fragility curves for the annealed single pane glass windows in woos sash with soft putty glazing, for 9 classes of structures.

Building Type	Slight (Edge Damage)	Moderate (Glazing)	Extensive (Cracks)	Complete (Glass Fallout)
URM	0.23 g	0.32 g	0.44 g	0.89 g
Precast Tilt up	0.39	0.63	0.85	1.68
Reinf. Masonry	0.44	0.74	1.05	2.19
Concrete Frame	0.57	0.89	1.16	2.16
Conc. Shear Wall	0.64	0.95	1.19	2.07
Steel Braced Fr.	0.66	1.05	1.34	2.44
Steel Moment Fr.	0.44	0.77	1.09	2.30
Mobile Home	0.21	0.31	0.39	0.70
Wood Frame	0.69	0.92	1.23	2.07

Table A.3-20. Fragility Curves - Window Type A-WS - By Building Type - As Is

Building Type	Slight (Edge Damage)	Moderate (Glazing)	Extensive (Cracks)	Complete (Glass Fallout)
URM	0.23 g	0.32 g	0.50 g	1.34 g
Precast Tilt up	0.39	0.63	0.96	2.50
Reinf. Masonry	0.44	0.74	1.20	3.33
Concrete Frame	0.57	0.89	1.29	3.17
Conc. Shear Wall	0.64	0.95	1.30	2.96
Steel Braced Fr.	0.66	1.05	1.49	3.53
Steel Moment Fr.	0.44	0.77	1.25	3.51
Mobile Home	0.21	0.31	0.43	1.01
Wood Frame	0.69	0.92	1.34	2.91

Table A.3-21. Fragility Curves - Window Type A-WS - By Building Type - With Film Upgrade

A.3.9.9 Fragility Curves - Window Type A-WH

Tables A.3-22 and A.3-23 provide the fragility curves for annealed single pane glass windows in wood sash with hard compound glazing, for 9 classes of structures.

Building Type	Slight (Edge Damage)	Moderate (Glazing)	Extensive (Cracks)	Complete (Glass Fallout)
URM	0.12 g	0.14 g	0.16 g	0.20 g
Precast Tilt up	0.18	0.25	0.28	0.35
Reinf. Masonry	0.18	0.26	0.30	0.38
Concrete Frame	0.29	0.35	0.40	0.51
Conc. Shear Wall	0.34	0.41	0.49	0.57
Steel Braced Fr.	0.32	0.40	0.48	0.58
Steel Moment Fr.	0.16	0.22	0.27	0.37
Mobile Home	0.11	0.15	0.16	0.19
Wood Frame	0.30	0.41	0.47	0.60

Table A.3-22. Fragility Curves - Window Type A-WH - By Building Type - As Is

Building Type	Slight (Edge Damage)	Moderate (Glazing)	Extensive (Cracks)	Complete (Glass Fallout)
URM	0.12 g	0.14 g	0.17 g	0.25 g
Precast Tilt up	0.18	0.25	0.30	0.44
Reinf. Masonry	0.18	0.26	0.32	0.50
Concrete Frame	0.29	0.35	0.43	0.64
Conc. Shear Wall	0.34	0.41	0.51	0.70
Steel Braced Fr.	0.32	0.40	0.51	0.74
Steel Moment Fr.	0.16	0.22	0.30	0.51
Mobile Home	0.11	0.15	0.17	0.23
Wood Frame	0.30	0.41	0.50	0.74

Table A.3-23. Fragility Curves - Window Type A-WH - By Building Type - With Film Upgrade

A.3.9.10 Fragility Curves - Window Type T-AGW

Tables A.3-24 and A.3-25 provide the fragility curves for fully tempered single pane glass windows in aluminum wide mullion frames with dry gasket glazing, for 9 classes of structures.

Building Type	Slight (Edge Damage)	Moderate (Glazing)	Extensive (Cracks)	Complete (Glass Fallout)
URM	0.26 g	0.32 g	1.22 g	1.22 g
Precast Tilt up	0.47	0.63	2.28	2.28
Reinf. Masonry	0.54	0.74	3.03	3.03
Concrete Frame	0.68	0.89	2.90	2.90
Conc. Shear Wall	0.74	0.95	2.72	2.72
Steel Braced Fr.	0.79	1.05	3.24	3.24
Steel Moment Fr.	0.55	0.77	3.19	3.19
Mobile Home	0.25	0.31	0.93	0.93
Wood Frame	0.77	0.92	2.69	2.69

Table A.3-24. Fragility Curves - Window Type T-AGW - By Building Type - As Is

Building Type	Slight (Edge Damage)	Moderate (Glazing)	Extensive (Cracks)	Complete (Glass Fallout)
URM	0.26 g	0.32 g	1.28 g	1.34 g
Precast Tilt up	0.47	0.63	2.39	2.50
Reinf. Masonry	0.54	0.74	3.18	3.33
Concrete Frame	0.68	0.89	3.03	3.17
Conc. Shear Wall	0.74	0.95	2.84	2.96
Steel Braced Fr.	0.79	1.05	3.39	3.53
Steel Moment Fr.	0.55	0.77	3.35	3.51
Mobile Home	0.25	0.31	0.97	1.01
Wood Frame	0.77	0.92	2.80	2.91

Table A.3-25. Fragility Curves - Window Type T-AGW - By Building Type - With Film Upgrade

A.3.9.11 Fragility Curves - Building Structures

The fragility curves for the 9 classes of buildings, which correspond to the fragility curves for the windows, are provided in Table A.3-26.

Building Type	Slight	Moderate	Extensive	Complete
URM	0.08 g	0.12 g	0.23 g	0.35 g
Precast Tilt up	0.11	0.25	0.63	1.07
Reinf. Masonry	0.10	0.26	0.74	1.35
Concrete Frame	0.29	0.51	0.89	1.56
Conc. Shear Wall	0.26	0.49	0.95	1.54
Steel Braced Fr.	0.24	0.48	1.05	1.78
Steel Moment Fr.	0.13	0.33	0.77	1.90
Mobile Home	0.14	0.18	0.31	0.60
Wood Frame	0.38	0.69	1.23	1.79

Table A.3-26. Fragility Curves - Building Structures

A.3.10 Glass Window Retrofit Options

There are three general classes of glass window retrofit options:

1. Add film to existing single pane glass windows. The film is applied as an adhesive. A rough cost of installing film is about $3.50 (costs in Section A.3 are in year 2000 dollars) per square foot of window. (The actual cost of the film can be under $0.50 per square foot - the remaining cost is labor for installation; total installation costs can vary widely, possibly as low as $1.00 per square foot). Once the glass breaks, the film acts as an adhesive to keep broken pieces of glass together, and hence will reduce the amount of falling debris generated. Under severe damage, the large broken sections of glass can still fall out of the window.
2. Replace the existing single pane with double pane - laminate glass windows. A rough cost of installing glass laminate windows is about $7 per square foot. Double pane laminated windows are extremely resistant to glass fallout even in severe earthquake motions.
3. Replace the existing single pane with fully tempered glass windows. A rough cost of installing glass laminate windows is about $9 per square foot. A fully tempered glass window is manufactured with composite materials. Once the glass breaks, it forms into many individual "glass balls", which do not have rough edges (similar to car glass windows). These pieces still generate debris, but individual pieces of debris are very small; large sharp shards do not occur often.

A.3.11 References

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A.4 Suspended Ceilings

Suspended ceilings are commonly found in commercial buildings. The are usually constructed of a grillwork of metal (sometimes plastic) runners (often called t-bars due to their shape) with inserted acoustic tiles, lights, HVAC returns, and sometimes, fire sprinklers.

The distance between the structural ceiling above and the suspended ceiling below is commonly between 1.5 feet to 6 feet, sometimes more.

The minimum-cost style of construction, common through the 1980s, was to hang the suspended ceiling from the structural roof above, using vertical wires. Low cost of installation was a major factor, and a $3 installation cost per square foot for a suspended ceiling was not uncommon.

Empirical evidence of ceiling performance in past earthquakes has shown that they are regularly damaged. As just one example of many, the suspended ceiling over the engineering offices of Southern California Edison in Rosemead was substantially damaged with many falling tiles due to the 1987 Whittier Narrows earthquake. The debris from the damaged ceiling was the primary cause to keep the office shut for two days, while clean up was performed. Many, many other examples abound, including failure of the suspended ceiling at the Burbank airport in the Northridge 1994 earthquake. There have not been reported incidents of fatalities due to falling suspended ceilings.

Given the poor performance of suspended ceilings, the UBC (and later the IBC) codes and related guideline documents have required improvements. The first round of improvements required that diagonal tension wires be installed (concept: the diagonals wires could take some seismic forces); and that heavier items be separated wired to the roof (so that if they become dislodged, they do not fall entirely onto occupants below). However, earthquakes that have occurred since such diagonal wires were installed have shown that the imposed tension into the diagonal wires led to uplift of the ceiling, still leading to damage. Given this, the next round of code-mandated upgrade was to include vertical compression struts at the locations where the diagonal tie wires were located, thus providing a more complete load path. Until recently, this was conceived as a "robust" seismic design for suspended ceilings.

At essentially facilities like nuclear power plants, the suspended ceiling systems require design-by-analysis, whereby the actual seismic loads in the ceiling are computed, and the design includes a complete lateral load path that accommodates both strength and stiffness issues. For designs like these (rarely used in commercial buildings or hospitals), the ceiling system can be much more rugged than if installed per modern code (IBC) proscriptive design-by-rule methods.

Damage to lightweight suspended ceilings rarely if ever causes casualties, but causes a mess than must be cleaned up; this can affect restoration time to regular service levels for a facility.

Given major damage to a suspended ceiling, the clean up time is usually about one to two days once repair efforts begin. This could extend the downtime for a facility that is otherwise not affected by the earthquake.

The usual retrofit concept for these ceilings is to add diagonal tension wires, and sometimes compression posts at the t-bar intersections. Combined, these upgrades can increase the transverse stiffness of the ceiling system, as the retrofitted system behaves in a truss-like manner. Less usual retrofit concepts might include adding hold-down clips on tiles to prevent them from popping out.

The cost to perform these retrofits varies according to individual installation requirements and access issues.

As a benefit of performing this upgrade, the reduced potential for ceiling movements will put less load on sprinkler heads, which should help to reduce the potential for breaking these items, and ensuing water damage.

Recessed lights are assumed to be components of suspended ceilings.

A few vendors of ceiling components have conducted shake table tests of their hardware. Where such information exists, a ceiling-specific fragility is appropriate.

A.4.1 Tests of Suspended Ceilings

A variety of test programs have been performed to examine the seismic performance of suspended ceilings. The following is adapted from Badillo-Alamaraz et al (2007).

- ANCO tests, 1983. Common damage to a suspended ceiling with intermediate-duty runners and lay-in tiles included buckling and detachment of runners from edge walls. For this particular installation, the addition of vertical struts did not reduce the level of damage to the ceiling.

- Rihal and Granneman (1984) showed that for a particular ceiling, that addition of vertical struts and diagonal wires reduced the vertical displacement and dynamic response of the ceiling.

- Armstrong World Industries (1993) tested their specific hardware that was designed for seismic zones 2A, 3 and 4 per UBC 1988, including in-structure amplification. The conclusion from these tests was that their system met the intent of UBC Zone 4 design for nonstructural components in essential facilities.

- Yao (2000) tested a ceiling system to evaluate the impact of adding diagonal sway wires. Testing revealed that installing sway wires at 45°, as recommended by CISCA (1992), did not produce a significant reduction in the seismic vulnerability of the ceiling system.

- Armstrong World Industries conducted additional shake table tests (Badillo-Almaraz et al, 2006, 2007). The ceiling system tested included the Armstrong Prelude 15/16 exposed tee system, compression posts with diagonal wires. Lay-in tiles were either snug fitting or undersized (easily popped out). The tests also examined the ceiling performance with the addition of retainer clips (to keep the tiles in place, but not commonly used in commercial installations); with and without vertical compression posts. Conclusions included; undersized tiles increases damage; addition of vertical posts may increase or decrease damage; addition of restrainer clips reduces damage; damage to the ceiling as a whole increases should the attachment of the ceiling at its edges fail; damage at beam connections leads to popping out of tiles; the use of compression posts reduces minor damage modes (a few tiles popping out), but does not influence major damage modes.

A.4.2 Fragility Based on Tests

Based on the test program (Badillo-Almaraz 2006, 2007), Table A.4-1 shows the input peak floor acceleration to reach four damage states:

- DS1. 1% of tiles pop out. Area below should remain functional with only minor clean up (less than 1 hour effort). No material life safety consequence. Repair cost is to re-insert the ceiling tiles.

- DS2. 10% of tiles pop out. Area below should remain functional with only minor clean up (less than a few hour effort, depending on the size of the ceiling). No material life safety consequence. Repair cost is to re-insert the ceiling tiles.

- DS3. 33% of tiles pop out. Area below should remain functional after clean up (20 manhour effort for 10,000 square foot area). Minor risk of injury. Damage to grid components. Repair cost is to re-place damaged components, re-insert undamaged tiles. Repair cost may approach 50% of value of ceiling when including clean up, re-order parts and installation effort.

- DS4. Partial or complete failure of suspension system. Grid members are damaged and need to be replaced. Edge attachments damaged. In most cases, minor risk of injury. Gross failure and dropping of the ceiling is considered rare, or essentially impossible if components have safety wires on them; but should gross failure occur, then the cost to repair will usually be 100% of the value of the ceiling, plus the clean up effort.

System	DS1 (Minor)	DS 2 (Moderate)	DS 3 (Major)	DS 4 (Complete)
1. Undersized tiles	0.70	0.90	1.05	1.25
2. Undersized tiles with clips	1.25	>1.25	>1.25	1.10
3. Normal sized tiles	0.85	1.05	1.60	1.25
4. Normal sized tiles with clips	1.00	1.60	1.60	1.05
5. Normal sized tiles with no compression struts	0.60	1.00	1.35	1.00

Table A.4-1. Fragility Levels (A), horizontal input acceleration

In Table A.4-1, the beta dispersion parameter from varies between about 0.05 to 0.20, reflecting only the variation between test repeats (i.e., with certain input motion). Beta would be reduced to between 0.04 to 0.09 if the input parameter was spectral acceleration. If one were to use the information in Table A.4-1 for a specific project, is a single beta model is used, then in California use beta = 0.4 for DS 1, DS 2, and DS3; or beta = 0.5 for DS 4.

Before using Table A.4-1 for a specific application, the end user would have to confirm that the actual installed suspended ceiling used the same components as those used in the test program, and configured and installed with the same level of quality control. For most cases, this will not be practical.

Examination of DS1 in able A.4-1 suggests that at PGA = 0.2g or below, one would expect no tile pop outs. However, in practical observations of suspended ceilings in many earthquakes, we do regularly see DS 1 at PGA = 0.15g. For example, a few tiles popped out of a suspended ceiling at a BART passenger rail station in a recent M 4.2 earthquake, (local site PGA = 0.15g). This can be explained by assuming that the hardware used in the test program is more robust that that used in many existing facilities.

A.4.3 Fragility for Performance Based Design

For most installations, the popping out of a few tiles is of little consequence. Therefore, we consider only "moderate" and "extensive" damage states.

In Table A.4-2, by "Moderate Damage" it is meant that there will be a few displaced ceiling tiles / light fixtures causing localized damage, and some of these might fall, for a ceiling covering about 1,000 square feet; there may be impacts with fire sprinkler heads. LS=1 (none), F=0 (roof system remains sufficiently intact to allow continuous operation in room below), SR=0.02 (minor cleanup), LR=0.01 (perhaps a tile or two needs to be replaced per 1,000 square feet).

By "Extensive Damage", it is meant that perhaps 30% or more of the ceiling tiles will be damaged, there will be damage to runners, t-bars, edge rivets, there will be gross distortion / rotation of inset light fixtures and HVAC components, there will be interactions with some fire sprinkler heads (ceiling distortions at sprinkler heads commonly more than 1 inch). LS=2 (minor injury, but possible none), F=0.9 (roof system is sufficiently damaged with sufficient dropped or hanging components as to disrupt functional use of the same below until the damaged components are cleaned up), SR=0.50 (major cleanup), LR=1.0 (replacement).

	Wire Hung		With Compression Struts	
	PGA (g)	β	PGA (g)	β
Suspended Ceilings			Add compression struts	
Moderate damage	0.50	0.50	0.80	0.50
Extensive damage	0.90	0.50	1.30	0.50

Table A.4-2. Suspended Ceilings

In Table A.4-2, we assume that safety wires are attached to all heavy components. Tiles are not necessarily snug. Quality of construction and cumulative damage over time is average.

Should the designer select the same ceiling system as was used during test (such as in Table A.4-1), then it is recommended to not use Table A.4-2, and instead use the fragility data for the particular components selected.

For special situations where the ceiling uses design-by-analysis; uses special components (like hold down clips); or is otherwise designed to remain elastic at the design-level earthquake motion, the fragility levels will be higher, possibly substantially higher. In such cases, we recommend that the design professional develop fragility information to match the specific installation issues.

The damage to potential sprinkler failures and subsequent flooding is not included in Table A.4-2.

A.4.4 Mitigation

In private industry, suspended ceilings are usually not seismically retrofitted even in seismically very active areas, unless potential debris would likely cause a significant loss of critical function, such as for:

- ceilings located over emergency operations centers
- ceilings whose movements will break fire sprinkler heads
- ceilings located over plant operators using computers, and who must make real time critical decisions
- ceilings over hospital corridors and emergency rooms / operating theatres

Upgrades of ceilings over non-critical locations, like employee lunchrooms, storage closets, etc. are often lower priority.

The following assumptions are made for suspended ceilings in the "upgraded" condition.

- The basic upgrade strategy for suspended ceilings is to add taught diagonal wires to t-bars; to screw HVAC filters and recessed lights into the t-bars; to attach the outer edge of the t-bars into adjacent walls (two of four sides of the ceiling system); to install compression struts to resolve the forces from diagonal wires; and to add loose wires to heavy equipment items (recessed lights, filters, etc.).

- These types of upgrades reduce the lateral flexibility of the ceilings, and generally meet the life safety intent of the 2006 IBC code.

- However, the seismic performance of such retrofitted ceilings has been less than expected. It has been observed that such ceilings still allows considerable movement, and panels can still drop out (albeit, these panels are light enough that serious injuries are not likely). Damage to interior sprinkler heads is not totally mitigated.

To reduce fire sprinkler head-to-ceiling interaction, it is not uncommon to widen the hole in the ceiling tile around the sprinkler head (1" annulus space) and/or brace the sprinkler pipes to reduce pipe motions to below the level needed to damage the sprinkler head.

Even for ceilings upgraded to IBC 2006, damage can be expected.

If the owner wishes a "immediate performance" capability of the suspended ceiling, then the design of the ceiling should avoid "install-by-rule" practices, such as those described above. Instead, the ceiling should be designed using rational strength-of-materials end engineering mechanics methods and should be stamped by the Structural Engineer. The design will have a positive load path for all lateral and vertical forces; factor in the true strength of t-bar elements and their connections. For such evaluations, the response modification factor "R" should be taken as 1 unless the engineer can demonstrate that the specific load-carrying elements have suitable post-yield cyclic capability to withstand overloads. All possible interactions between the ceiling and its elements (including fire sprinkler pipes) much be designed in a rational manner. It would not be uncommon for the cost to design and install such a ceiling to be between 5 to 25 times more than a simple wire-hung suspended ceiling.

Where upgraded, the common strategy is to design and install per CISCA Zone 3-4, ASCE 7-02 9.6.2.6.2.2. This means that the upgraded ceilings will have seismically qualified t-bar systems, compression struts, lateral wire braces and/or panel clips as suitable to limit panel pop-outs or t-bar disassembly at the selected design level PGA. For California, the design level might be selected as the 475-year return motion, but not less than PGA = 0.4g the Los Angeles or San Francisco areas. For coastal Oregon, the design level might be PGA = 0.3g, given that while this might be closer to a 2,475 year motion in many areas in the Willamette Valley, it is prudent to select a higher PGA to reflect that the design-basis earthquake might be a M8.5 to M 9 event (in other words, select a somewhat higher PGA as a proxy for duration effects; see Section 2.3 for further discussion about earthquake duration).

A.4.5 References

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A.5 Elevators

Elevators have been found to be vulnerable in earthquakes. The 1971 San Fernando earthquake provided evidence that counterweights in cable traction elevators shook out of their guide rails, endangering occupants. In response, legislation was enacted to require all elevators in California to be retrofitted to shut down when the elevator is shaken in an earthquake. This shutdown requirement includes those elevators in hospitals, even though those elevators may be needed for patient transport and emergency response.

However, even with the lessons learned from the 1971 San Fernando earthquake, elevators in reconstructed facilities still lacked sufficient seismic design details to prevent damage in strong shaking. For example, the Holy Cross Medical Center in the 1994 Northridge earthquake, having been rebuilt after the 1971 earthquake, still had one of four elevators suffering extensive damage (counterweight loosened and hit car); and the track rails for the remaining three elevators all suffered some out-of-alignment damage.

With these factors in mind, the following report outlines the overall issues dealing with elevators in earthquakes. Considering these factors, a fragility model is described in Section A.5.7. Much of the following information is adapted from information provided by Anshel Schiff (2008, 2009).

A.5.1 Key Findings

- Loss of electric power could cause large numbers of elevator occupants to be trapped and require first responders to remove them from the elevators.

- The tripping of seismic switches would require large numbers of elevators to be inspected by elevator mechanics before being put back into service and cause long restoration times.

- The slow restoration of elevator function in critical facilities, such as hospitals, is a potential life-safety issue.

- Structural damage to elevators is possible in some instances. However, structural damage is only one of several reasons why elevators do not operate after earthquakes.

- Some measures that could mitigate the effects identified above are discussed.

A.5.2 Past Performance

Based on survey reports, damage rates are low with about 200 elevators damaged in the 1989 Loma Prieta earthquake and 700 in the 1994 Northridge earthquake. The estimated inventory of elevators in service at the time of these earthquakes was about 50,000 (greater Los Angeles Area) or 30,000 (greater San Francisco Bay Area). The proportion of elevators that experienced PGA = 0.25g or higher was likely about 3,300 elevators (1994 Northridge, mostly in the San Fernando Valley area) or perhaps 2,000 or so (western San Jose, parts of Santa Clara). As a first order estimate, the repair rate is about 21% (1994 Northridge, PGA > 0.25g), or 10% (1989 Loma Prieta, PGA > 0.25g).

We must consider these repair rates as speculative, as in both earthquakes, the way the data was collected was by contacting a few elevator maintenance companies; largely, they do not provide detailed information about damage, in part for contractual reasons. Also, we do not have precise breakdowns of styles of damage that require a "repair", but presume it to mean that a service call was required.

In earthquakes, injuries to people in elevators are extremely rare and can be counted on one's fingers. The trapping of people in elevators has not been a documented problem, but this is probably due to luck and circumstances.

Further compounding the issue is that the above repair rates are based on reports from companies that service only about half of the elevators in the area and completeness of the reports that were received is questionable. Because damage reports did not indicate details about the cause of the damage and buildings where damage occurred were not identified, damage could not be correlated to shaking intensity.

Following the 1971 San Fernando earthquake, public policy changed and damage reports became public documents. It is speculated that the information in damage reports could be used by "ambulance chasers" resulting in increased liability exposure and a drop in report submittals.

The most common types of significant damage were counterweights coming out of their guide rails, fouled ropes or governor cables, cable damage to door operating mechanisms, and movement of equipment or control cabinets. The largest risk of severe injury is when safety systems are overridden (probably by building engineer) and the elevator put into service after a counterweight came out of its guides. This can result in the counterweight striking the elevator cab. (This has happened with unoccupied elevators.)

From an emergency response perspective, the loss of electric power causes the elevator to stop and trap occupants, which usually requires rescue personnel for extraction.

In a large earthquake in a major population area in California, it is speculated that demands on elevator mechanics to inspect elevators in buildings with tripped seismic switches will exceed their ability to respond in a timely manner. Traffic congestion will exasperate the problem. This will even be a problem for critical facilities, such as hospitals, that are given high priority for service.

A.5.3 Background on Elevator Industry

The elevator code is a very proscriptive code so that for elevators in low-rise structures the code effectively provides the design using parts from elevator supply companies.

The industry has a two-tiered structure in which one tier consists of large elevator companies with engineering staffs that provides the elevators in high-rise structure as well as other structures. The second tier consists of smaller companies that provide elevators for low-rise structures. These smaller firms often have limited engineering capability.

Large elevator companies also generally provide maintenance service for elevators. There are many small, independent elevator maintenance companies.

California was the first jurisdiction to adopt seismic elements into its elevator code and the national code followed several years latter. Recently California adopted the ANSI 17.1 code with some modifications. Unlike the ANSI Code, when the seismic elements were introduced into the California code, they required some retrofit to improve seismic performance of existing elevators, but there was a procedure that allowed this provision to be circumvented.

A.5.4 Background on Elevators

Other than for anchorage, the code generally uses a maximum of 0.5 g horizontal acceleration for linear performance of structural elevator components. Because earthquake induced motions in buildings can exceed this level, two seismic safety devices were incorporated into elevators, a counterweight-derail detector and a seismic switch. As the name indicates, the derail detector sense this conditions and initiates a shut down procedure.

The seismic switch is a device that senses acceleration and shuts down the elevator if the acceleration exceeds the trigger level. If the counterweight-derail detector triggers, the shut down procedure is modified. While it is thought that it is primarily horizontal acceleration (or perhaps interstory drift) that initiates damage to elevator components, the acceleration sensed by the switch is orientated in the vertical direction and the level is set at PGA = 0.15g, well below the expected damage threshold for horizontal motions.

The seismic switch serves two functions. It will shut down the elevator if the level of the vertical motion would indicate that horizontal motion would approach the damage threshold. It also is configured to shut down the elevator early so that it can be stopped and the doors opened before it is stopped for some other reason. The seismic switch sensor is orientated in the vertical direction to sense the arrival of the P wave, which precedes the arrival of the damaging S waves. If the site is some distance from the epicenter, there will be a short time before the structure response to the S wave build and the arrival of the P wave, so there may be an opportunity for the elevator to stop at a floor, open the doors and allow passenger to leave the elevator and avoid being trapped. It is also hoped that this will occur before there is a loss of power that would immediately stop the elevator and trap the passengers. It should be noted that because of the confinement in a small space, the general fear of many of elevators, and the added concerns after an earthquake, being stuck in an elevator after an earthquake holds a level of terror for many.

The performance of the seismic switches can be problematic in that extraneous non-earthquake vibrations, such as nearby construction or truck traffic vibrations can cause a false trigger that disrupts elevator service. A more serious problem is that the switch may trip in a critical facility, such as a hospital, that needs the elevator to provide critical services. After a severe earthquake, hospitals will have extraordinary demands placed on them. The impact on the ability of a hospital to provide service, when they will have to transport patients between floors using the stairs may be very severe. The switch is designed to trip below the damage threshold, so that even though the switch tripped, there may be no damage. The intent of the code was to have the elevator checked out by an elevator mechanic prior to being put back into service after the seismic switch trips. When this requirement was initiated, hospitals and buildings with many elevators typically had a resident elevator mechanic on site. With the switch from electrical mechanical controls to solid state electronic controls is thought to have increased system reliability, resident elevator mechanics are now rare. It may take hours for a mechanic to get to the hospital, even if it is his or her first service call. For example, after the Loma Prieta earthquake, it took over two hours to go from Stanford University to Los Altos Hills (local area PGA was about 0.20g), which is normally an eight-minute drive.

The elevator code committee has developed a procedure that provides guidance for a non-elevator mechanic to safely put elevators back into reduced-speed service without an inspection by an elevator mechanic. Unfortunately, it has not been able to get approved for various reasons. It seems to some code committee members that such a procedure would provide a high degree of safety. It must be realized that the loss of elevators in a hospital is itself a potential life-safety issue. It should be noted that some medical facilities do provide some elevators with emergency power, so that the disruption of elevator service due to the action of the seismic switch will be the main cause for the loss of service.

Elevators are commonly divided into two classes, direct plunger lift hydraulic elevators and all others (including traction machines). Two and three-story buildings typically use hydraulic elevators. For buildings less than about 7 stories, hydraulic elevators cost less then traction machines. The actuator on a direct plunger lift hydraulic elevator may extend below the foundation of the building. It should be noted that hospitals in California typically have tractions elevators and seismic switches.

It should be noted that the dynamic response of a building can amplify the ground motion. Peak horizontal acceleration at the top of moderate height buildings has typically exceeded the peak horizontal ground motion by a factor of 1.5 and sometimes by a factor of 4 (for base isolated buildings, roof level horizontal motion will usually be 1/3 to 1/2 of the free field horizontal motion). Thus, in most buildings (non-base isolated), horizontal accelerations will exceed the 0.50g used in the design of elevators, whenever local PGA exceeds about 0.20g or so.

Some elevators are provided with a means to lower the elevator to the nearest floor after power is lost and open the doors. It would appear that this capability is rather uncommon. In tall buildings some elevators will be provided with emergency power.

An elevator mechanic would be able to mechanically lower a stopped elevator to a landing from the machine room. He or she would then be able to open the doors at the landing to free trapped passengers.

A.5.5 Vulnerability of Elevators

Injuries in earthquakes related to elevators are extremely rare. Damage to elevators has been very small compared to the large number of elevators in service, although there has not been a study to relate damage to ground motion intensity or building characteristics. Extensive disruptions can be expected from the action of seismic switches and the loss of power. Restoration of service after a seismic switch has triggered requires that the elevator be inspected by an elevator mechanic. This will overwhelm the available resources and transportation delays will exasperate the situation. Generally fire service personnel have been trained to extract people who are trapped in elevators. With proper training elevator doors can be opened without damaging the doors and people can be extracted. In general, local building "engineers" will not know how to get elevator doors open without causing expensive damage.

Beyond the loss of power or shutdown of elevators due to seismic switch actuation, there are other physical damage modes. These include derailment of the counterweights, impact of the counterweight into the car, tangling of ropes (traction powered), and other types of damage. The influence of building damage on elevator damage cannot be entirely ruled out, including that imposed by interstory drift.

As noted above, it is thought that direct damage to elevators (such as structural damage) is not the most common cause leading to repair. Rather, the disruption of service associated with the loss of power and delays in restoring service that requires the inspection of an elevator mechanic is the major problem. The lack of restart procedures for critical facilities, such as hospitals, could have a significant operational impact.

The timing, location and size of the earthquake event could result in large numbers of people to be trapped in elevators. While in most circumstances people being trapped does not present a life-safety issue, the demands on elevator mechanics and fire fighters will result in significant damage to elevator doors that are pried open by people who are nor familiar with methods to open doors and a potential hazard if not executed properly, as discussed below.

Two lifeline interactions can affect the performance of elevators. The loss of power will have a major impact on elevator performance. Some high voltage power system components have been demonstrated to be very vulnerable to earthquake damage, as demonstrated in the estimated extent and duration of power disruptions in the earthquake scenario. Although there is an improved voluntary standard (IEEE 693) that should improve power system performance as qualified equipment finds its way into service, this will be a 20 to 30-year process. The disruption of high voltage system elements, such as power transformers, at a given location can impact a large area distant from the power system substation. Should the affects of power system disruptions be nearly instantaneous, then the loss of power can eliminate the benefits of seismic switches that allow passengers to quickly exit an elevator. The timing of the loss of power is critical, as a 5 to 10 second delay would allow elevators to stop, open doors and avoid trapping passengers. Unfortunately, the current state of knowledge prevents accurate prediction of the times of power disruptions within the first 20 seconds or so after the earthquake.

For the disruption of power to trap people in elevators requires several factors to come into play. The earthquake must be close to a location that has many elevators, such as a large city. The earthquake must occur at a time when elevators are being used. The disruption of power must occur within 5 to 10 seconds after the shaking starts, so that the action of the seismic switch to stop the elevator and open the doors has been disrupted.

Historically, we have only three earthquakes that have impacted major metropolitan areas: 1971 San Fernando, 1989 Loma Prieta, and 1994 Northridge earthquakes. The San Fernando earthquake occurred at 6:01 in the morning. The Loma Prieta earthquake occurred a few minutes before the World Series was to start at about 5:00 pm. The Northridge earthquake occurred at 4:30 am. While the number of people trapped in elevators in these earthquake is not known, and the timing of the loss of power relative to the start of shaking is not know, the lack of press reports of people being trapped in elevators suggests that large numbers were not trapped. The timing of these events suggests a large factor that may have contributed to the few people trapped in elevators.

Delays in transportation due to traffic congestion will probably significantly impact timely service to elevators after earthquakes. There are numerous examples in which fire service personnel were unable to negotiate traffic even with flashing lights and sirens.

For escalators, the common design practice is to bolt the top of the escalator to the upper level floor, and allow the bottom of the escalator to float (no horizontal anchorage to lower floor). It has not been unknown for actual installations to have the escalator bolted to both levels, making the escalator structure (commonly a steel truss) to become part of the building's lateral load resisting system, possibly with unknown and undesired consequences. Therefore, fragility of escalators should be made on a facility-specific basis, coupled with field inspection to confirm proper installation. If the escalator was designed to behave elastically at PGA = 0.5g, and installed properly, then the median fragility is likely to be substantially greater than PGA = 1.0g; however, if the escalator was installed to act as part of the building structural system, the fragility should be computed consistent with the principles of structural mechanics, in consideration of joint building-escalator interaction.

A.5.6 Mitigation Issues

Potential mitigation actions are listed below in approximate order of their importance, ease of implementation, and estimate of the success of their implementation.

Restoration of Service after Triggering without Inspection in Hospitals and other Facilities where Disruption is a Life-Safety Issue (Highest Priority)

The ANSI 17.1 committee has considered (but not yet adopted) proposed guidelines for restarting elevators that are stopped due to the action of a seismic switch. These proposed guidelines are thought to provide a reasonable degree of safety for all concerned and should be implemented for facilities where the loss of elevator service is a demonstrated life-safety issue.

As noted above, elevators are vital to the operation of hospitals and there are currently no procedures for restoring service without an elevator mechanic that conforms with the intent of the code. Normal elevator maintenance service is more or less uniformly distributed over time and the number of service personnel in an organization reflects this situation. After an earthquake there is a sudden demand to inspect many elevators at once, so that elevators mechanics will probably not be available in a timely manner. There service can be delayed traffic congestion.

It should be noted that elevator companies feel strongly that only elevator mechanics should enter an elevator hoist way because of the dangers that are involved. Experience shows that when there are long delays in restoring service, a building “engineer” can be very resourceful in getting the system to operate, but their resourcefulness may go beyond their knowledge of the safe operation of elevator systems and result in additional damage or injuries. For this reason informed guidance provided by the industry would be useful if it could be accompanies by proper training.

Since many large hospitals will provide emergency power to some elevators, the delay in seismic switch trip restoration will be the most likely cause of disruption. A procedure has been developed by the ANSI A17.1 national elevator committee that can could be used, even thought it has not been officially approved. There is an organization in California, which could promulgate such a procedure (OSHPD), if they could be so motivated.

Extricating People from Elevators Trapped Due to Power Disruption

The parameters (timing, size and location) of the earthquake suggest that there is a high probability of having many people trapped in elevators due to the loss of power caused by the earthquake (during normal business hours). Or far fewer in the early morning hours or on weekends, depending on facility. Elevator mechanics and fire fighters, who would be knowledgeable about opening elevator doors will probably not be available. The procedures are not complex and an effort to educate building engineers would largely solve this problem.

The stopping of an elevator due to the loss of power will be a random event and for most structures the elevator will not be stopped at a floor landing. Few buildings have blind sections of shaft (no access to elevator doors) and it should not be difficult to locate the position of an elevator, as occupants would be shouting or banging on the walls of the cab. With some training the building engineer can be taught how to open the outer and inner door without damaging the elevator so that passengers can be removed from elevators without having to leave through the ceiling access door. There is one serious hazard. If the floor of an elevator is over 4 feet above the floor on which the door is opened, access to the shaft will exposed. Thus, a person who jumps from the elevator could loose their balance and fall into the shaft. Thus, in this situation it is best to open the door on the upper floor to provide egress from the elevator. An educational program would address this issue.

Code does not Require Key Parts to be Tested or Qualified

The code covers structural elements, such as rails, rail supports, and anchorage. Some important elements, such as roller guides are not governed by specific design criteria and may not be designed for impact loading. Elevator parts suppliers supply these parts to the second tier companies and even first tier companies. The first tier suppliers may have design specification or test the parts that they get, but this might be generally beyond the capability of second tier companies. The elevator business is very competitive, and it would not be surprising if some of these parts are manufactured with uncertain quality control.

Detailed Reports of Damage to Local Elevator Jurisdictions

Although it would be desirable to reinstate the limited disclosure of damage reports, this would appear to reside with the judicial system and sunshine laws. Mandating a damage report be submitted to elevator authorities has good and bad features. A simple one-page report with a fill-in-the-blank format could be completed in about two minutes and could provide needed information. The elevator inspectors could use the report to verify that repairs have been made. The reports could be summarized and the results passed on to the code committee so that deficiencies could be identified and addressed.

At a meeting of elevator maintenance companies after the Loma Prieta earthquake a complaint that was voiced by several companies was that some building owner refused to authorize a repair, thinking that that it was not needed. The maintenance company would like to have the owner sign a release to eliminate their liability, but it is a competitive business and there was a concern that the owner would switch companies. Having to document the damage may eliminate occasional exaggerated damage reports and assure that repairs are made. There is a downside about liability exposure, but damage is seldom associated with personal injury so it is not clear if this is a major issue. Elevators are not designed to be damage-free in earthquakes; rather, they are designed to provide a safe, cost-effective vertical conveyance. Thus, some damage can be expected, as demonstrated by the use of seismic switches.

Seismic Switch Trigger and Central Triggering to Shut Down Elevators

Seismic switches provide a safety function and a means for stopping elevators before they are otherwise stopped to prevent passengers from being trapped. Because of their safety functions, switches cannot be eliminated. For the purpose of stopping elevators and allowing occupants to exit requires that the elevator site be located some distance from the epicenter so that propagation times allow time to stop the elevator after the arrival of the P wave. With modern strong-motions systems in California, it is possible that the signal to initiate a shut down could come from a central system. This could increase the time for the elevator system to stop, open the doors and lockout, as the electronic signal would have a shorter propagation time than the P wave. This has the potential to eliminate some problems and create others, depending on how it is implemented. If the central system replaced the local seismic switch, local false triggers would be eliminated. As noted above the switch cannot be eliminated. The central system could reset the elevator without inspection if the seismic switch trip was not due to an earthquake. To provide a faster trip signal, the central trigger would have to sense the earthquake, calculate its location and magnitude and quickly send out the signal to shut down quickly. The local trigger associated with the elevator in a building would have to estimate the likely local intensity by using an attenuation relationship and the relative locations of the epicenter and the site. It may be possible for the central control system to identify which elevators should be triggered and send out the appropriate code that the building site can identify. If this generates a circular action area surrounding the epicenter, there could be many false triggers. There may be more sophisticated methods of central trigger selection where a computer at the site could assess the magnitude and locations data to determine if a unit should be triggered. There are other options. It must be emphasized that because of safety reasons, and reliability, the seismic switch cannot be eliminated.

Mutual Aid

Today's competitive environment means that organizations retain only the personnel they need to meet normal workloads. As a result, the organization will probably not be able to respond to a severe event in a timely fashion. Major California utilities, such as large power and communications companies have large service areas that stretch for hundreds of miles. Because an earthquake only impacts a small portion of their system, they can draw on remote resources to aid in restoration. Even with this ability, they also have mutual aid agreements with other utilities to get additional assistance. Large service organizations affiliated with the major manufacturers may be able to draw on assets outside the damage area, but this will not be available for the smaller independent maintenance companies. This may be an advantage in going with the larger company and the issue may be relegated to one of commercial interest. However, very long delays in elevator inspection and restoration will have a societal impact and may be an issue appropriate for consideration in the analysis of the earthquake scenario. While many of the features of elevators are uniform and differences in rail supports and the like can be observed, car construction may require some familiarity with the specific manufacturer. Mutual aid is complex as there are financial issues of who pays and other issues such as workers compensation.

Structural Mitigation

A possible mitigation for some elevators is to install stronger cages to prevent the counterweights from swinging loosely, as well as additional bracing for both the counterweights and the rails, which limit bending of rail track alignments which can prevent the elevators from traveling.

Observed damage. (multiple earthquakes). The most consequential are counterweights becoming dislodged form their guide rails. Another damage mode is fouling of the ropes. Newer designs may include non-snag provisions, so when the ropes swing around, they do not get hung up. It was reported in a recent earthquake in 2008 (detail unconfirmed) that a rope broke.

In the 1994 Northridge earthquake, there was damage to the four elevators at the Holy Cross Medical Center. For one elevator, the counterweight was loosened sufficiently to allow it to swing and bump into the car, causing extensive damage. The track rails for the other elevators suffered some out-of-alignment damage (which prevents the elevator from traveling). The site motion is estimated to have been PGA = 0.60g.

Singh et al (2002) examined the structural damage modes for counterweights, rails, roller guides and bracket supports in elevators. For a specific elevator, the chance of structural damage was estimated to be about 10% at PGA = 0.8g.

Code Issues

The national elevator code (A17.1) is an evolving standard. Its earthquake provisions were originally listed as optional, and then mandatory in subsequent application. In California, the earthquake provisions have been mandatory since about 1990. The California code allows the cognizant engineer to decide that should the building envelop sustain heavy damage, that no special upgrade is needed for the elevators. When used, the upgrade was to install seismic triggers to halt movement of the elevator; and also tie bars between guide rails to limit their potential for separation to retain the counterweight.

The National code does not require retrofitting. The California code does require retrofit under certain circumstances.

The mitigation upgrades might include the strengthening of guide rails in the elevator shafts; reinforcement of guide rails, rail splices and bracketing to the supporting structure; and upgrades to limit deflections to l/1667. Counterweight cages can be reinforced to prevent excessive movements of the weights. These upgrades can be effective in reducing the potential for elevator structural damage in future earthquakes, and should also be adequate to keep the elevators functional in strong ground shaking (barring damage to the building that induces damage to the elevator; loss of power; or shutdown activation switches).

A.5.7 Fragility Model

If one is trying to evaluate the likely performance of an individual elevator due to specific earthquake motions, it is required to examine the structural elements of the elevator, the interaction of the elevator with the building, the operational attributes of seismic switches, the likelihood of loss of offsite power, the availability of emergency onsite power for the elevator, and the availability of trained people to respond to elevator outages. All these factors bear some role in estimating the likelihood that an individual elevator will be operable during or after earthquakes.

If one is trying to make a forecast as to the number of elevators that will be out of service after an earthquake, then a macro-fragility model (Model 1) is suggested below.

Model 1. All elevators, without consideration of style of construction, including the influences of building interaction, seismic switches, and availability of power.

- Median PGA = 0.4g, with beta = 0.5.

In other words, 50% of elevators will require some type of repair response, given ground shaking = 0.4g. Most of the time, the response will be due to loss of offsite power. For common electric grids, there is about a 50% chance of offsite power at PGA = 0.35g. This means that the above fragility implies that at least some elevators will have backup power, and some will have structural damage. The use of this fragility model should be limited to examination of several hundred to thousands of elevators over areas of many tens of square miles, and it cannot be used for evaluation of individual facilities.

When using Model 1, the most likely consequences are: LS=1 (none), F = 1 (no operation), SR = 0.02 (service call), LR = 0. In Model 1, we recognize that there will be a few elevators with structural damage, but the consequences are for the most common case where the elevator stops due to loss of power or activation of the seismic switch.

Model 2 below is similar to Model 1, but presumes that the elevators are older / have no specific seismic detailing for structural members, but some contain seismic switches.

Model 2. Elevators without specific seismic design for PGA = 0.5g.

- Structural damage (rails, tangling of ropes, etc.) or outage due to operation of switches or failure of offsite power. PGA = 0.35g, beta = 0.5. In a survey of damage to elevators from the 1994 Northridge earthquake, the median fragility level was PGA = 0.31g (elevators built prior to 1971) or PGA = 0.39g (elevators built post-1971). This survey included about 100 elevators, and was heavily weighted with elevators at hospitals. Elevators at hospitals tend to be about 2 to 6 times larger than those in residential or office facilities, and this may influence the fragility level. One would like to believe that elevators built to ANSI 17.1 seismic provisions would have a higher fragility level, but there is (as yet) insufficient evidence to confirm this from empirical evidence.

- Major damage (counterweight impacts cage). One chance in ten, given occurrence of structural damage.

In Model 2, the consequences are the same as in Model 1 when the failure is due to loss of power / switch actuation. In Model 2, when the damage is severe structural (about 10% of the time), then the most likely consequences are: LS=2 (minor), F = 1 (no operation), SR = 0.02 (service call), LR = 0.50; a more serious life safety condition can occur, but is not likely.

Model 3 is geared towards prediction of damage for elevators that have been designed per ANSI 17.1 with seismic switches and designed for 0.5g loads.

Model 3. Elevators with specific seismic design for PGA = 0.5g. This damage state excludes outages due to activation of seismic switches or loss of offsite power.

- Structural damage (rails, tangling of ropes, etc.). PGA = 0.90, beta = 0.5. Note: this value is based largely on judgment, as comprehensive studies of damage to ANSI 17.1 designed-and-installed elevators are not yet available.

- Major damage (counterweight impacts cage). One chance in ten, given occurrence of structural damage.

In Model 3, the consequences are the same as in Model 1 when the failure is due to loss of power / switch actuation. In Model 3, when the damage is structural (about 10% of the time), then the most likely consequences are: LS=2 (minor), F = 1 (no operation), SR = 0.02 (service call), LR = 0.50; a more serious life safety condition can occur, but is not likely.

There is some evidence that traction elevators have more serious damage than hydraulic elevators, as the traction elevators have ropes that can get tangled. As a first order estimate, fragility levels for structural damage to hydraulic elevators could be assumed to be PGA = 0.10g higher than the comparable values for traction elevators.

We do not recommend that any of the above elevator fragility values in Section A.5 be used for purposes of performance based-design of specific elevators; in these cases, the design professional should select the fragility level consistent with the specific design and construction attributes appropriate for the specific project at hand. The elevator fragility values in Section A.5 are considered suitable only for loss estimates of large inventories of elevators that have similar attributes as those in the San Jose area (circa 1989) and Northridge area (circa 1994).

A.5.8 Elevator Performance in San Andreas M 7.8

In this section, we make a forecast of functionality of elevators due to a San Andreas M 7.8 in Southern California, using the Model 1 fragility model.

In southern California, within 100 km either side of the San Andreas fault, excluding the City of Los Angeles, there are estimated to be about 100,000 conveyance devices that include passenger and freight elevators, moving walkways, and escalators. There are about 60,000 elevators (freight and passenger). In the City of Los Angeles there are about 26,000 conveyance devices. Assuming that elevators make up the same percentage, this would mean that there are about 16,000 elevators in the City of Los Angeles.

Thus, in the City of Los Angeles, if all power was suddenly disrupted in a San Andreas M 7.8 event, and from 5% to 10% of the elevators were occupied at the time, about 800 to 1,600 elevators would be stopped containing one or more passengers. A repair effort will be needed to move cars and open doors to allow trapped people to get out of cars. Perhaps 90% of all impacted elevators these will be restored to service once power is restored and a repair call is made to re-set the seismic switch; for the remaining 10%, some type of physical damage might need to be repaired. This simple application of the fragility model, is about as precise as one can make, not knowing the specific styles of construction for individual elevators.

A.5.9 References

ASME A17.1 Elevator & Escalator Code.

IEEE 693, Recommended Practices for Seismic Design of Substations, 2005.

Schiff, Anshel, Assessing the Impact of a M 7.8 Southern San Andreas Fault Earthquake: Elevators, January 18, 2008.

Schiff, Anshel, Personal Communication, 2009.

Singh, M. P., Roldova, Suarez, L.E., Seismic behavior of railed-counterweight systems of elevators in buildings, Technical report MCEER-02-0002, MCEER, Buffalo, NY 2002.

A.6 Raised Floors

Raised floors are also known as sub-floor, computer floor, and access floor. These names are interchangeable. The term "raised floor" is used in this document. Any floor system that is constructed on a concrete floor with the intention of creating space under the equipment frames for routing cables, pipes, ducts, and directing airflow (environmental control system) can be classified as a raised floor.

The practice of installing raised floor began when mainframe computers were installed in large companies. It provides a clean and esthetically pleasing environment for mainframe computers and makes routing cables (both power and data cables) between equipment frames easier. In the mean time, it also provides a more uniform cool air distribution to the equipment.

In the late 1980's, telephone companies, control centers of utility companies, and large PBX system for banks, financial companies and emergency operation centers commonly installed their equipment on raised floors.

There is no industry standard governing the performance of raised floor. The Ceiling and Interiors Systems Construction Association (CISCA) publishes "Recommended Test Procedures for Access Floors" (2007) that establishes tests methods for concentrated loads, ultimate load, rolling load, stringer load, pedestal axial load and pedestal overturning moment. However, these tests do not set performance standards. Manufacturers' published specifications are test results based on the test procedures of CISCA.

Almost all manufacturers of raised floor systems do not provide an "off the shelf" seismic raised floor for direct equipment mounting. All designs are for vertical load and some horizontal loading for equipment rolling. However, there are a few manufacturers do market special designs, such as active control raised floor, base isolated raised floor, etc. The fragility information in this report does not deal with these special floor systems.

A.6.1 Background

NTT (Nippon Telegraph and Telephone) of Japan started research programs and testing of equipment installed on raised floor in early 1980s for the purpose of reducing equipment damage due to strong ground motions. In North America, Bell Lab also developed installation practices and methods for raised floor installation in seismically active regions. Some equipment manufacturers in North America had initiated their own design efforts in this application.

Most of the information of the seismic raised floor applications is collected from the telephone industry.

There are many large raised floor manufacturers globally. Most of them offer both product and installation services. Some of them provide customers with custom designs to meet their requirements. There are no off the shelf "seismic raised floor" products. Some of the "seismic raised floor" products are for personnel safety purposes.

In general, most of the raised floors in the market are structurally designed to handle both vertical load and rolling load. However, the available floor tiles are designed to handle various environments and the function of the facility.

A.6.2 Components of Construction

Some raised floor systems have stringers connecting the pedestals forming a grid of squares. The stringers (spandrels) do add strength to the raised floor to resist horizontal loading. The connection details of the stringers determine the level of horizontal loading performance.

There are many accessories that are supplied by manufacturers to dress up a finished product. The accessories include electric grounding hardware, access ramps, rails, dress panels, etc.

Floor tiles are commonly made of either wood, steel sheet, or cast aluminum. The most common size is 24 inches by 24 inches (600 mm x 600 mm). The surface material depends on the application. For office environment, the surface might be laminated with carpet; for general equipment area, it might be laminated with PVC; for electronic equipment area, it might be laminated with antistatic material to eliminate static discharge to the equipment; for medical application, the tiles might be designed to meet application-specific requirements.

For raised floor without stringers the four corners of the floor tile will be resting on the pedestal heads.

A.6.4 Mitigation

For a common non-seismically-designed raised floor, the floor tiles are 2 feet x 2 feet, and are supported on metal stands. Each metal stand typically supports 4 tiles, and the tiles are typically connected to the stand with a single screw. The stand rests directly on a concrete floor deck below, with an adhesive applied between the stand bottom and the structural floor below. Normally, the stand is not bolted or screwed to the structural floor below. Not all of the floor tiles are positively screwed to the columns stands. The height of the stands are commonly between 6 inches to 18 inches, averaging about 12 inches. The fragility information assumes an average height floor.

For this type of floor, as part of a basic seismic retrofit program, all essential equipment that is bolted (or screwed) to a floor tile must be verified as having their tiles screwed to all supports; and that the combined system (equipment to tile to stand to structural floor) can withstand laterally applied seismic loads (usually the 475-year motion for the site). In some cases, lateral bracing can be added below the tiles to provide superior lateral resistance capability. Heavy equipment might require direct anchorage to the concrete floor below; or improvements to the equipment-to-tile and tile-to stand and stand-to-floor connections. Note that the seismic upgraded here is meant to protect the valuable equipment, and not to prevent damage to the floor system itself, except as an indirect benefit.

An intermediate level of mitigation might include additional of angle stops on the tops of the tiles to prevent equipment from sliding laterally atop the raised floor. This type of mitigation is lower cost than the above described mitigation, and provides less improvement. No fragility information is provided for this upgrade strategy.

A more robust seismic upgrade of the raised floor would be to make the entire raised floor a seismically-designed system, meant to have no damage under the design basis earthquake, and that all equipment is attached either to the raised floor or the structural floor below. Such an upgrade would imply no damage to the raised floor, no dislodging of tiles, no unintended rolling, rocking or toppling of equipment. This will require assessment and upgrade as needed of the tiles and stringers and pedestals for all seismic loads, and will generally cost more to implement than the basic seismic upgrade described above.

A.6.5 Fragility

Table A.6-1 lists fragility functions for a standard non-seismically-designed raised floor, as outlined in A.6.4. This function assumes a pedestal higher of between 9 to 15 inches, equipment on the raised tiles that is not anchored directly to the structural floor below the raised floor, and no positive cross bracing of the floor itself. Pedestals may be standing directly on the underlying structural floor below without positive anchorage (could have adhesive). Should any of these assumptions not apply to the particular raised floor, then the fragility function should be changed.

For raised floors, the damage state is as follows:

- Non-seismic condition. Panels pop out. Unanchored equipment displaced 12 to 48 inches. Equipment anchored to panels only topples. Some pedestals tilted. Floor deforms more than 0.5 inches laterally at several locations.

- For the basic upgraded condition, the equipment is positively anchored to the tiles and the tiles are positively restrained; or the equipment is positively anchored to the structural floor below. Some damage may still occur to non-critical portions of the raised floor.

Should the damage stage occur, the consequences are assumed as follows:

- Life Safety. No impact or possibly minor injury (LS 1 or possibly LS 2) due to impact of rolling / toppling equipment or pop-out of panels or distortion of the floor.

- Functionality. Given occurrence of the damage state, the equipment is assumed to be non functional (tear out or wires or more serious), F=1.

- Short Term. The short term repair effort is to reset the popped out tiles, relocated the rolled / damaged equipment. This is set at 1% of the value of the entire floor / equipment system, but in practice is meant to be about $400 (assume $125 per hour for labor, overhead, small parts and equipment). If the entire floor / equipment value is $1,000,000, then the short term repair cost ratio is SR = 0.0004. For installations with less or more value, the short term repair effort will vary.

- Long Term. The long term repair cost implied modest damage to the floor and some equipment movements that caused functional damage to the equipment. It is assumed that perhaps 10% of the equipment will be functionally damaged due to rolling or rocking or toppling, but the actual repair cost will depend greatly on the actual types of equipment in pace, and their sensitivity to damage given movement. The cost to repair the damaged floor system is assumed to be 5% of the entire value. Therefore the total long term repair cost is 5% (floor) and 10% (equipment) or 15% of total valuation. For installations with less or more value, the long term repair effort will vary.

Case	A	Beta	LS	F	SR	LR
1. Non seismic	0.50	0.60	1	1	0.0004	0.15
2. Limited seismic	0.70	0.60	1	1	0.0004	0.15
3. Basic seismic	1.50	0.50	1	1	0.0004	0.15
4. Full seismic	3.00	0.50	1	1	0.0004	0.15

Table A.6-1. Fragilities – Raised Floor

A.6.6 References

I-Kwang Chang, Shih-Chi Liu, and Haresh C. Shah, Seismic Support of Electronic and Computer Equipment on Raised Floors.

Schiff, Anshel, Personal Communication.

Tang, Alex K., Seismic Considerations of Raised Floor Application for Telecommunication Network Facilities, US-Japan Workshop on Earthquake Disaster Prevention for Lifeline Systems, Oct 1992.

CISCA, Recommended Test Procedures for Access Floors, 2007.

A.7 HVAC

Heating, Ventilation and Air conditions (HVAC) equipment is often found in basements, grade level, and often atop buildings or in penthouses. In larger multi-story facilities, sometimes HVAC equipment is located at intermediate floor levels.

HVAC equipment covered in Section A.7 includes two categories: rotating equipment (centrifugal fans and similar) that are mounted on vibration spring isolators, and the ductwork that moves the chilled / heated air to various locations in the building.

A complete HVAC system will also include a variety of other equipment, including: gas or oil-fired steam / hot water boilers, chillers, motor driven pumps, filters, louvers, fans, blowers, programmable controllers (mounted in floor-supported or wall-mounted metal cabinets), water tanks (1,000 to 100,000 gallon size), hot and chilled water pipes. Most of this equipment is included in other sections in this report (see mechanical equipment, electrical cabinets, process pipes, etc.)

A.7.1 HVAC Ducts

Lightly supported HVAC ducting is vulnerable to earthquake damage. This is primarily due to the relatively light weight construction materials used, as well as limited interlocking capability for many types of duct systems. One way to limit ducting damage is to upgrade the ducts with continuous pipe. Another way to limit damage is to substantially support the ducts to limit lateral displacements and hence damage at the duct joints. It is not unknown that "duct tape" is used to seal joints, often very effectively.

Several sets of HVAC duct fragility curves are provided (Table A.7-1). Sets are for HVAC ducts located throughout a building; and more sets are for HVAC ducts located in a penthouse location atop a building:

1. HVAC ducts - rod hung - throughout building
2. HVAC ducts - rod and sway braces - throughout building
3. HVAC ducts - rod hung - penthouse
4. HVAC ducts - rod and sway braces - penthouse

One set of fragility curves provided are for flexible ductwork, hung primarily by rod hangers. Another set of fragility curves is for the typical mitigation provided for this type of ducting, namely the addition of lateral seismic braces. In this case, the ducts will undergo much less damage in comparable motions, although some leakage and duct buckling will still occur, and limited ductwork repair may still be needed.

By "slight" damage, it is meant that a few ducts in the facility will leak, (many leaks may not be found for some time), and the leaks are easily repairable. By "moderate" damage, it is meant that several ducts will leak, and some may be pulled apart (per 1,000 feet of duct). By "extensive" damage, it is meant that there will be numerous duct system leaks, making the system effectively inoperable, some duct will be sufficiently broken to require placement. By "complete" damage, it is meant that there will be widespread duct system damage, many supports will be broken.

Table A.7-1 provides the fragilities for the HVAC distribution ducts. By "slight" damage, it is meant that a few ducts will leak, (many leaks may not be found for some time), and the leaks are easily repairable. By "moderate" damage, it is meant that several ducts will leak, and some may be pulled apart. By "extensive" damage, it is meant that there will be numerous duct system leaks, making the system effectively inoperable, some duct will be sufficiently broken to require placement. By "complete" damage, it is meant that there will be widespread duct system damage, many supports will be broken. These descriptions are for 1,000 feet of duct.

While there has been damage to ductwork in past earthquakes, we are unaware of any situations where ductwork has reached the "complete" damage state. The PGAs listed are geared towards free field horizontal PGA levels. For ductwork located at the roof level (or in roof-level penthouses), amplified motion will usually occur, and the PGA levels in Table A.7-1 should be reduced by 40% to 50% (or so) if the hazard is measured using PGA.

In Table A.7-1, we have included fragility estimates for "rod hung" and "with sway braces". The "rod hung" column is based on common installations observed in commercial facilities, mostly prior to 2000, and assuming essentially no explicit design for seismic loads. While we show "with sway braces" as having increase fragility levels, this is entirely speculative, and assumes reasonably good seismic detailing (expansion joints across seismic expansion joints, good interlocking duct connections, etc.) as well as installation of lateral sway braces where needed to limit deflection and keep stresses in the ducts / deflections in the duct joints within acceptable levels.

The value of beta (0.54) reflects that on average there is more uncertainty about duct installation than for other common pieces of equipment where beta = 0.50.

Note that in Table A.7-1, we use PGA, and not A. This is done to highlight that damage to HVAC equipment at roof levels (atop the roof or hanging from the roof, or in a penthouse at the roof level) have been observed to be sustain damage whereas no damage was observed in other lower elevations of the building.

	Rod Hung		With Sway Braces	
	PGA (g)	β	PGA (g)	β
HVAC Distribution			Add Supports	
Slight (not at roof level)	0.38	0.54	0.79	0.54
Moderate (not at roof level)	0.75	0.54	1.59	0.54
Extensive (not at roof level)	1.25	0.54	2.38	0.54
Complete (not at roof level)	1.88	0.54	3.00	0.54
Slight (at roof level)	0.19	0.54	0.40	0.54
Moderate (at roof level)	0.38	0.54	0.80	0.54
Extensive (at roof level)	0.50	0.54	0.96	0.54
Complete (at roof level)	0.75	0.54	1.50	0.54

Table A.7-1. HVAC Duct Distribution System

Consequences.

Slight. LS=1 (none), F=0 (Ducts remains sufficiently intact to allow continuous flow of forced air), SR=0.00 (no short term emergency action), LR=0.01 (perhaps a minor repair is done for a leak).

Moderate. LS=1 (none), F=0.1 (Ducts remains sufficiently intact to allow continuous flow of forced air for 90% of the service area), SR=0.00 (no short term emergency action), LR=0.02 (some minor repairs for a leaks).

Extensive. LS=1 (none), F=0.5 (Ducts remains sufficiently intact to allow continuous flow of forced air for 50% of the service area), SR=0.01 (temporary fans / chillers as suitable for portions of the building), LR=0.10 (some duct replacements).

Complete. LS=1 (none), F=0.0 (Ducts have sufficient damage to make the forced air system inoperable), SR=0.02 (temporary fans / chillers as suitable for portions of the building), LR=0.25 (some duct replacements).

A.7.2 HVAC Rotating Equipment

Table A.7-2 provides fragilities for rotating equipment on vibration isolators. See Table A.18-1 for damage rates to centrifugal fans in past earthquakes.

The fragility levels of the rotating equipment does not include functional failures due inertial / impact overload on the equipment itself. It is usually assumed that such equipment is already "vibration" tested to 1g or much higher forces under shipping or normal operation modes, and that seismic-induced inertial loading does not control. For critical facilities (such as emergency generators for nuclear power plants where the residual risk of failure must be nearly zero), qualification for seismic induced loading is required either by test or analytical qualification; such level of effort for qualification for common important commercial facilities (including water treatment plants, hospitals, emergency operations centers, etc.) is not required nor normally done. Where such extra qualification effort is done, then the fragility level could be set by selecting A and beta so that the probability of failure at the ZPA qualification level is one (if brittle mode) or two (if ductile mode) standard deviations below the median; the end user must use judgment in selecting the number of standard deviations, based on the residual margin left at the qualification level.

A common retrofit for spring-mounted rotating equipment is to install suitably large and strong seismic lateral stops. These "stops" allow for perhaps 1/8" free vibration of the equipment (actual gap size varies per installation) in lateral directions, but "stop" further lateral movement. This can be achieved by use heavy angles with anchorage, or various types of proprietary "load-rated" hardware. The designer should verify that the "load rating" is suitable for the actual installation, considering both simplified static horizontal load (V=0.xW) as well as energy impact (potential energy greater than imposed kinetic energy) considerations. With well-designed lateral stops, displacement-induced failure modes should be eliminated; we set A= 1.5g. It has been observed that "load rated" stoppers made of brittle materials have failed in past earthquakes, and these should have a lower median capacity.

	A (g)	β	A (g)	β
HVAC Vibration Isolated Equipment			With seismic design	
Fails	0.50	0.50	1.50	0.50

Table A.7-2. HVAC Rotating Equipment

A.8 Office Equipment

A typical office will include a variety of equipment commonly encountered in offices, hospitals and laboratories. This includes:

- Desk top computers
- Desk top computer monitors
- Desk top printers and copiers
- Floor-standing copiers
- Refrigerators, washers, dryers, vending machines
- Desks, credenzas
- Wall hangings (prints, etc.)
- Measuring and observation equipment
- Instruments used in examination rooms (scales, microscopes, etc.)
- Liquid storage bottles used for IVs
- Televisions in hospital rooms
- X-ray machines
- CT scanners
- Shadowless lamps
- Operating tables
- Autoclaves
- Sterilized water devices
- Hemodialysis machines
- Anesthesia instruments
- ICU monitors and recording / analysis devices
- Electrocardiographs
- Biochemical automatic analysis devices
- Respirators (automatic artificial breathing machines)
- Electroencephalographs
- Electromyographs
- Respiratory function diagnostic devices

- Ultrasonic diagnostic devices
- Bathtubs used for treatment
- Dental diagnosis and treatment equipment
- Thermometers, beakers, lab glassware
- Magnetic Resonance Image (MRI)
- CT Scanner (Cat scan)
- X-Ray Equipment

These types of equipment can be further classified by how they are anchored, as well as whether they are relatively "top heavy", "bottom heavy" or "evenly weighted". Fragility curves are provided in that represent three common unanchored configurations (Table A.8-1), as well as three poorly anchored conditions (Table A.8-2).

A single fragility curve is provided in Table 2-1 for anchored items, which makes the assumption that the anchorage is robust, and generally more robust than needed for most equipment items, but standardized to be uniformly applied in practice.

The equipment fragility curves in Table A-1 should be multiplied by two-thirds for items located at floors 40 feet or higher from the ground, in buildings which are 60 feet in height or higher.

Almost all buildings contain a number of pieces of office equipment. This equipment is used to provide information for various users. Without this equipment, data retrieval is slowed, hampering delivery of functional services of the service (or patient care at hospitals).

There are a variety of types of small to medium-sized medical equipment in hospitals. Similar types of equipment are commonly used at water treatment plant laboratories, University and private research laboratories, and similar facilities.

For hospitals, this equipment may include items emergency rooms, laboratories, x-ray, surgery, central laboratory, ICU, PCU, OB. This equipment is used to provide various diagnosis, monitoring and treatment functions. Loss of this equipment leads to reduced ability to provide patient care, as well as potential life safety implications.

During the 1994 Northridge earthquake, a number of computers and files were lost (fell on floor, damaged), which required about two days to repair or replace.

Damage in the 1994 Northridge earthquake at hospitals included shifting of the MRI and linear accelerator equipment, causing outages; repair costs were about $6,000 ($1994) each. Other medical equipment was lost (broken), and services had to be curtailed until replacement equipment was procured. In strong shaking, shelf mounted items can slide and fall, causing both loss of service as well as financial loss for equipment replacement. In very strong shaking, larger pieces of equipment will topple, causing both loss of service as well as significant financial loss.

Common mitigation strategy is to anchor / restrain / brace all essential office and medical equipment. The design basis is to provide anchorage / bracing which can withstand lateral forces of about 100% of the item's weight (V = 1.0 W), although various codes might allow lower levels of force at the ground level, and require higher levels of force at penthouse levels.

In application, it should be recognized that some of the most vulnerable office equipment are unanchored computer CRT monitors. These should be considered top heavy items. When these monitors are exposed to ground shaking much above 0.30g, the monitors begin to rock violently; much above 0.40g, there is a reasonable chance that they will have rocked and slide sufficiently to either tip over, or directly fall off the desk to the floor. A large percentage of monitors that fall on floors will break. Other considerations in upgrades are the addition of retaining bars for shelving, if contents are not to spill (especially important in a laboratory or pharmacy); and addition of latches or locking mechanisms in file cabinets, to prevent drawers from sliding open and causing large overturning tendencies. Unanchored items, such as horizontal-oriented desktop computers boxes, can be considered "bottom heavy" items, and will fall only if they slide sufficiently to slide off the desktop.

The technical basis of the fragility curves in Table A.8-1 for unanchored items is described in Section A.18. For Table A.8-2, the quality of the anchorage system is assume to be "poor". For example, sheet metal screws into a gypsum board (non-load bearing) wall, or other similar style non-engineered restraint systems that are "better than nothing" but not necessarily well designed.

Table A.8-1 provides the fragility curves for existing unanchored equipment. The median capacities for equipment located at the upper floors of taller buildings should be adjusted, to reflect the increased accelerations (typical) experienced in higher elevations of such buildings.

Existing Unanchored	A (g)	β	A (g)	β	A (g)	β
Office Equipment	Top Heavy Items		Bottom Heavy Items		Evenly Weighted Items	
Tip Over, Slide, Fall on Floor	0.40	0.60	0.75	0.50	0.60	0.50

Table A.8-1. Office Equipment, Unanchored

Some equipment may already have some type of anchorage. Some of these anchorages may not have been designed and installed to meet very high earthquake forces. Table A.8-2 provides the fragility curves for these "partially" anchored equipment items.

Partially Anchored	A (g)	β	A (g)	β	A (g)	β
Office Equipment	Top Heavy Items		Bottom Heavy Items		Evenly Weighted Items	
Tip Over, Slide, Fall on Floor	0.50	0.50	0.90	0.50	0.75	0.50

Table A.8-2. Office Equipment, Partially Anchored

In the upgraded condition, most items should not fail in the earthquake (the standard anchorage system will have lots of over capacity). A reasonable fragility curve for the upgraded items is a median PGA of 1.5g; with dispersion β of 0.5.

Given that the item slides enough tap fall off a countertop, or an item topples, the common consequences are as follows (these should be adjusted for site-specific considerations):

- LS = 1 or 2 (either no injury or minor injury will be the most likely consequence, assuming anyone is nearby).

- F = 0.5. this implies that about half the time, the fallen object will be broken, and half the time, the object will remain serviceable.

- SR = 0.01. We assume it takes 1% of the replacement value to pick it up and restore it to service, or pick it up and dispose of it should it be broken.

- LR = 0.5. The long term repair cost is zero if the item is functional, or 100% (replacement) if it is not. In other words, LR = F. Depending on the item, the user should alter these default values to match the specific conditions.

In some cases, addition of "high-friction" mats under the equipment can constitute cost effective solutions. This has been done for cases where equipment sliding is thought to occur at low g levels (hard plastic on stainless steel countertops, for example). In such cases, the additional of friction mats will increase the fragility level, but usually not as high as the case of providing a more robust positive anchorage system.

A.9 Suspended Pendulum Lights

Pendulum lights are a common type of lighting fixture. Common installations include the following components:

- A horizontal light fixture. Lighting may be direct (shines downwards) or indirect (shines upwards, reflected off ceiling). The light fixtures may be 4-feet, 8-feet or 16-feet long.

- Pendant Support Rods / Cables. The horizontal light fixture is hung from the ceiling above using two or more support rods or cables.

- At the top, the support rods / cables may be attached to the ceiling above using a variety of connection hardware, commonly with an anchor bolt into a concrete floor above. The rod / cable may be attached to the top anchorage in a rigid (for rods) or flexible (for rods or cables) type assembly.
- At the bottom, the rod / cable may be attached to the light fixture in a variety of types of mechanisms. These mechanisms allow for dead load transfer of the light fixture to the rod under normal conditions, and are also designed to allow assembly / disassembly of the light fixture. For example, the bottom mechanism can be designed to allow easy disassembly of the light fixture, if the rod / cable is rotated relative to the light fixture. While this type of mechanism provides good characteristics for ongoing maintenance, it can allow for unexpected disassembly of the light fixture from the rods / cables under large earthquake motions, when the light fixture swings laterally through large arc movements.

Possible failure mechanisms of the light fixture include: high rotation-induced disassembly of rod systems; fatigue failure of threaded rod-type pendulum supports; pullout of the rods / cables from the upper connection to the ceiling above. The latter two failure mechanisms are less prevalent than the disassembly failure mode.

Other failure modes, such as pop-out of the lamp, can occur, but are not considered in this report. If lamp pop-pop occurs, it will go dark, and possibly fall on the floor. Common actions by humans (duck and cover) are sufficient to preclude material risk of injury from this and the above-described failure modes.

A series of shake table tests were performed to examine the performance of one manufacturer's light fixtures, both for non-seismic installations and with seismic retrofits. The light fixtures tested included assemblies with two or three supports, light fixture lengths of 4 feet, 8 feet or 16 feet, rod / cable lengths of 12 inches, 18 inches and 24 inches. Supports were either rigid rods or cable systems.

Depending on configuration, the fundamental mode period of most the light fixture assemblies was between 1 and 2 seconds. Rigidly supported light fixtures had periods as short as about one-third second.

In their non-seismic design condition, 13 of 14 fixtures "failed" at normalized peak PGA ground motions ranging from 0.3g to 0.8g; none failed below 0.3g. A failure was the disassembly of the lower connection point of one or more rods / cables from the light fixture. If this occurred on a two-support system, then one end of the light fixture would drop down, supported by the other support. If this occurred on a three-support system, then the light fixture would not "dangle down" unless the other two rods were not well spaced, or if they failed also. The normalized median failure PGA was 0.62 g, with a lognormal standard deviation (beta) of 0.47.

In their seismically-retrofitted design condition, 4 of 4 fixtures survived maximum applied normalized PGA ground motions ranging from 0.4g to 0.8g.

With this in mind, damage algorithms for pendant lights are provided in Table A.9-1.

PGA (g)	As is, P (failure)	Upgraded, P(failure)	Replaced, P(failure)
0.04	0.00%	0.00%	0.00%
0.06	0.00%	0.00%	0.00%
0.08	0.00%	0.00%	0.00%
0.12	0.02%	0.00%	0.00%
0.16	0.19%	0.03%	0.00%
0.24	2.08%	0.38%	0.01%
0.32	7.73%	1.60%	0.09%
0.435	22.09%	5.65%	0.67%
0.55	39.39%	12.33%	2.43%
0.675	56.65%	21.59%	6.24%
0.8	70.18%	31.66%	12.04%
0.9	78.25%	39.62%	17.83%
1.0	84.26%	47.15%	24.28%
1.2	91.84%	60.26%	37.88%

Table A.9-1. Damage Algorithms for Suspended Light Fixtures

Recommended fragility levels are 0.60g (non-seismic), 1.10g (with retainer clips), or 1.50g (seismic design).

Given that the damage state occurs, the common consequences are as follows (these should be adjusted for site-specific considerations):

- LS = 1 or 2 (either no injury or minor injury will be the most likely consequence, assuming anyone is nearby).

- F = 0.5. this implies that about half the time, the dangling object can be put back into service, and half the time, is will suffer electrical failure.

- SR = 0.01. We assume it takes 1% of the replacement value to pick it up and restore it to service, or pick it up and dispose of it should it be broken.

- LR = 0.5. The long term repair cost is zero if the item is functional, or 100% (replacement) if it is not. In other words, LR = F. Depending on the item, the user should alter these default values to match the specific conditions.

The following notes apply for Table A.9-1.

- Typical support installations are 12", 18" or 24" long, with 2 (most) or 3 (some) rod or cable-type supports with clip-type arrangements, with 4-foot to 16-foot long lights.
- Failure mode occurs (generally) when the light swings sideways sufficiently to "unhook" the clips holding the rod (or cable) to the light.
- A single failure of one cable / rod is assumed to allow part of the lamp to drop to a lower level.
- A failure could cause complete damage of the lamp fixture (50% of the time), requiring replacement.
- It is assumed that the upgrade for an existing lamp involves the installation of suitable retainer clips on the cables / rods supporting the lamp fixtures.
- It is assumed that a replaced lamp fixture will have been seismically qualified for no disassembly of the lamp from rods or cables, at PGA = 0.4g, with high confidence.
- Failure at very high g levels reflects potential interaction with nearby items, pull out of anchors from the roof, or unexpected attachment failures.
- All values are suitable for lights installed in one story buildings. Additional amplification / deamplification is required for lights supported from higher floors in multi-story buildings, or below grade.

A.10 Overhead Cranes

Overhead cranes are commonly used in large maintenance yards, and where needed to lift heavy equipment.

The cranes can be of various sizes with varying lift capacities. Most common cranes used in these facilities will have lift capacities of 10 tons or less.

The cranes are composed of the following main parts. There are two parallel steel rails the length of the building, atop of which the sliding crane rolls. These rails can be supported on reinforced concrete corbels or sometimes on steel columns. The main seismic vulnerabilities are as follows:

- The crane could "jump" off the rails due to strong ground shaking with local building / crane rail amplification. This is not likely to occur for ground motions under 0.25g. Vertical motions at the crane / wheel interface need to exceed 1g (considering local amplifications / resonance) to lift the wheel off the rail. Once the wheel is lifted off the rail, continuing lateral motions of the crane / building / rail interface could cause derailment.

- The only credible chance that there could be serious derailment (and subsequent chance of falling) is if the crane has sufficient space to move laterally before being stopped by the adjacent building. This is not likely when the two crane rails are supported on rigid concrete building systems which tend to move in phase with each other.

- One set of crane rails might be supported on a stiff building, and the other set supported on a tall moving steel frame. The combined differential motions between the wheels on the building-supported track and the wheels on the at-grade supported track may be sufficient to cause derailment, possibly with crane toppling. This was observed to occur for a crane at a PGA of 0.30g to 0.40g.

- Other failure modes: the building could collapse, hence bringing down the crane; functional failure (loss of electric power supply). These failure modes are not considered herein.

The usual low cost strategy to mitigate potential crane risks is to install suitable stops / hold down devices between the crane wheels and the rail. This can be done at one location, coupled with operating instructions to park the crane at that location when the crane is not in use.

It is speculative to establish a crane fragility level to capture all possible inventories of cranes. Instead, we recommend that the user establish a fragility for the specific crane, consistent with strength of materials concepts.

A common overhead "crane" issue is seen in small pumping plants with joist- or truss-type roof systems. At these pumping plants, no permanent crane is installed. Instead, maintenance staff often place a timber 6x6 (or similar) spanning between the joists or trusses, to provide a simple lifting system. These timbers are totally unattached to the building system, and can slide around under strong (>0.35g) ground shaking. Depending upon the overhangs provided, these timbers could slide sufficiently to slide one end off a roof joist or truss, thus dropping the timber down onto the equipment below. Depending upon the nature of the equipment, a falling 6x6 timber could do damage to hydraulic lines, etc. A simple solution is to have this 6x6 timber placed on the floor of the pumping plant, and only placed into the joist / truss system when the lifting system is needed for maintenance purposes.

A.11 Rocking and Toppling of Unanchored Blocks

Many of the fragility models described in this report are based on the seismic performance of unanchored equipment. It is useful to examine the theoretical basis for the sliding, rocking and toppling of rigid blocks.

Theoretical models have been developed to estimate the acceleration amplitude needed to topple a rigid block having an evenly distributed mass and a particular slenderness ratio, assuming a perfect sine-wave or cosine-wave input (Markis, 1998, 1999).

The key findings of interest to practical equipment installations are as follows.

- Assume a piece of unanchored equipment with height H and width W and high frequency (but still has a rigid body frequency). Depending on the form of the input acceleration time history, it may move with the ground (no uplift and no sliding); it might slide; it might rock; or it might slide and rock. If the magnitude of rocking is sufficient, the block might topple (overturn). The propensity of the rigid block to take on any of these five modes depends on the W/H ratio, the coefficient of friction, the magnitude of the input acceleration, the energy absorbed at each uplift / down cycle, and the frequency and timing characteristics of the input acceleration.

- Near field motions (with velocity pulses) do not particularly cause more overturning or toppling of unanchored equipment due to long-period velocity pulses. This is because the period of potentially destructive velocity pulses (2-3 seconds or so) is quite a bit longer than the typical period of unanchored equipment (well under 0.5 seconds).

- Toppling of smaller blocks is more sensitive to PGA, whereas toppling of larger blocks is more sensitive to PGV. In other words, a smaller block will overturn due to the higher frequency acceleration pulses, whereas a larger block will tend to overturn due to the longer period velocity pulse.

- Overturning can be thought of as occurring in one of two modes. Mode 1 has the block rocking, and then on the impact of the down-movement the block proceeds to fall over on the next uplift cycle. Mode 2 has the block rocking sufficiently on the first rocking cycle that it overturns without any down-movement impact.

- The effect of concurrent vertical acceleration is negligible.

Assume that a rigid block has a base friction level greater than W/H. Say for a component W = 1 foot, H = 5 feet, then the required coefficient of friction to initiate rocking rather than sliding is 0.20, which is true for almost all common installations (except perhaps stainless steel on smooth plastic). Then, for input acceleration over 0.20g, the seismic-induced overturning moment will exceed the static deadweight restoring moment, and the block will begin to rock.

Tall and slender blocks will tend to have multiple cycles of rocking as compared to short and wide blocks, unless they topple on the first rocking cycle. When W/H exceeds about 1.4, the rotational inertia of the block upon falling back to its original position is generally too small to create a rocking uplift in the opposite direction. From a practical point of view, it would require another input pulse of sufficient intensity to get the short-wide block rocking multiple times, whereas a single input pulse is sufficient to get a tall slender block to rock multiple times.

Damage to an unanchored piece of equipment could initiate upon initial sliding (visualize tearing of attached wires); sufficient sliding (visualize a counter-top oven sliding far enough to reach and then overhang the ledge of the counter top and then fall onto the floor); rocking without toppling (visualize tearing of wires due to rocking movements or high frequency impacts at the end of the rocking cycle that damages frequency-sensitive equipment); or outright toppling. Even in the toppled condition, the author has observed maintenance staff at a water treatment plant to lift back a toppled motor control center, and with all equipment within remaining functional. While, the author would not recommend that toppling is a safe condition for design for most items, the reader should recognize that depending on the item involved, toppling might not result in functional damage (for example, the toppling of a football off a shelf will not damage the football).

For idealized input motions, the peak acceleration needed to topple a common rigid block is about PGA = (W/H)(1+0.2*(f eq/f block)). In other words, if W=H, then PGA = 1g plus a factor which is related to the frequency of the input motion, f eq, as compared to the frequency of the rigid block, f block, (within practical frequency ranges). In other words, if the frequency of the driving input motion is nearly zero (static condition), then PGA needed to topple the square block is just barely over 1g. For bookcases, where W/H is commonly about 0.33, it would take about 0.33g to topple the case under static conditions, and perhaps 0.6g (considering the variable nature of input pulses) to topple the bookcase under realistic conditions. Thus, the recommended fragility level of 0.6g (median) for an evenly weighted bookcase assumes a certain W/H (about 0.33) and an input driving frequency that is higher than the bookcase's rigid body frequency. Clearly, a bookcase could topple at PGA = 0.34g is just the right input motion occurs, or a bookcase might rock but remain upright at PGA > 0.8g, if the dynamic input motion is just right to counterbalance the effects of rocking. For practical conditions, if the floor is "soft" (like carpeting, rubber pads, etc.), more energy is absorbed on impact at each rocking cycle, and that can also impact the tendency to topple, unless the item topples at the first rocking cycle. It should be noted that for very high input accelerations at very high frequencies, the above formulation for toppling does *not* apply; for example, applying a 2g amplitude sine wave input at 40 hertz to a 2 hertz rigid block (W=H) will not initiate rocking, whereas dropping the input driving frequency of 5 hertz might topple the same rigid block.

If the designer is interested in determining the "precise" toppling fragility level for a specific piece of equipment, then the item can be modeled using a general purpose nonlinear structural analysis program, suitable nonlinear elements used to represent the bounding of the item, and then using suitable input acceleration motions. This has been tried before and compared to shake table results (Hucklebridge, 1975), and it has been consistently found that trying to get precise correlation of shake table results with analytical model results is amazingly difficult (either in intensity of uplift movement, or timing of uplift movement); but reasonable matches of generally uplift cycles can be duplicated. Such efforts are usually limited in accuracy by the precise modeling of the impact forces and energy loss at each rocking cycle, and minor variations in assumptions of these properties at the first uplift cycle can drastically change the timing and amplitude of future uplift cycles.

Once the decision has been made to anchor (restrain) the unanchored block, the fragility level needed to topple the block is set at 1.50g. The selection of 1.50g is arbitrary, as the designer can select a suitably strong anchorage to prevent toppling at any desired level of input motion. For performance based design, as long as a median capacity to toppling of 1.50g is reached, it is likely that the residual chance of toppling is so small as to make additional robust anchorage to have rather small marginal benefits. Of course, this depends on the importance and value of the equipment.

Over the past 70 years or so, the nominal code-provision for anchorage of non-structural components in Zone 4 California has varied from as low as F = 0.08W (large storage racks) to now as much as F = 3.6W (site PGA = 0.6g, Ip = 1.5, flexible equipment). For most ground supported equipment, a formula of F = 1.62W is sufficient for elastic design (R = 1) of components at sites with a 475 year PGA = 0.55g or so, and where the equipment has low damping. A lot of equipment installed in Zone 4 California from 1960 to 1990 is anchored assuming F = 0.3W to F = 0.45W.

If the weakest element in the anchorage system is a drilled-in expansion anchor bolt, then the code-style design might begin with F=0.3W, but if a factor of safety of 4 against average pullout is implied in the anchor bolt selection, then the actual capacity is F = 1.2W (median failure rate). If the equipment is rigid (first mode much above 15 hertz or so), then A (median) = 1.2g *at the initiation* of rocking (after the anchorage has failed), and the A (median) for toppling would need to be somewhat higher, depending on the frequency of the input motion versus the rocking frequency of the component.

The issue is further complicated by the fact that the code often allows the factor of safety on the anchorage to be reduced via "special inspection", so that the design allowable becomes average strength divided by two (not four). The issue is then further complicated by the fact that some designers neglect the prying action on anchor bolts, and many existing installations have improperly factored in the effects of edge distance, anchor bolt spacing, and the like. On top of these uncertainties, the failure of an anchor bolt usually has no impact of equipment functionality, if the subsequent motion during the earthquake is sufficiently small that the item no longer tends to rock or topple or slide in its then fully-unanchored or semi-unanchored condition.

Given all these factors, we assume that if the actual piece of equipment is designed at grade level for F = 0.5W, and that the actual anchorage system selected actually has a factor of safety of 2 against average ultimate capacity, that the A (median) is 1.50g for a function-impairing damage state, for "standard" anchorage. For well anchored electrical cabinets, where the design factors in location in the building, good design (prying action, bolt edge distances considered, some ductile capacity of the anchor system) and good quality control, the A (median) is set at 3.00g. This is to say that the residual chance of a toppling-caused functional failure becomes exceedingly rare for very-well designed anchorage.

Analytical results by Markis (1999) show that if the anchorage system is installed, and it fails (i.e., the anchorage was under designed for the actual motion), then the rigid block actually has an increased chance of toppling. This somewhat counterintuitive result is plausible, as the energy released into the rigid block by the failed anchorage can aggravate the tendency for the block to rock and then topple, under certain frequency ranges.

Code based "Rp" values for ductility of non-structural components are largely based on committee judgment or guesswork and should not be considered as having been proved to ensure functionality. Many types of equipment will functionally fail once they yield, for example those with moving parts and tight tolerances; or wrinkling of sheet metal parts when dimensional stability is required, etc., and in these conditions, one cannot assume R greater than 1 for anything by life safety assurance (and even that is uncertain).

The potential energy absorbed by common anchorage systems (expansion anchors, many types of hold-down clips) can be dwarfed by the kinetic energy of the inertially-excited massive equipment, so that the equivalent damping provided by the yielding anchorage item is small, and does not support using Rp much over 1, even for functionality reasons. Often, the "savior" in using the code-based Rp values is that the anchorage is overdesigned by a factor of 4, so the net design is reasonably okay. However, if the designer adopts Rp = 3 to 8, and then designs non-ductile anchorage systems with little actual margin, then if a design-basis earthquake occurs, then failure of the anchorage system is the likely outcome, and if the earthquake time history after anchorage failure has the right properties, toppling can ensue.

If the designer wishes a reliable design for the design basis earthquake without anchorage failure, then it is best to use Rp = 1, calculate the prying actions and edge distance limitations correctly, and then use an anchorage system that is to remain elastic under the design-basis earthquake; for anchor systems (expansion or epoxy), a highly reliable design can be achieved this way by setting the design allowable for the bolt at 50% of the average test data, and having good quality control throughout the design and construction process. For initial design, the selection of a 0.75-inch diameter anchor bolt is not much more costly than a 0.375-inch diameter anchor bolt, so the cost impact of this design approach is nearly trivial; in a retrofit situation, the extra benefit of robust anchorage might not always be cost effective.

Loss of functional operability of an anchored component (where anchorage does not fail) due to inertial loading is rare, and not covered in this report. For nuclear power plants, functional damage modes are avoided by having the component qualified by shake table tests or similar procedures. The extra cost to implement this type of qualification is not thought to be cost effective for almost all kinds of commercial buildings, including water treatment plants, emergency operations centers, fire stations and similar "essential" facilities. Should the owner of an essential facility desire to include operability as part of the design basis, then the owner could: a) implement U.S. commercial nuclear power plant requirements (potentially increasing the design and procurement cost by 100% or more); b) retain cognizant engineers to select commercial grade equipment that is more likely to be operational at the design basis earthquake level, but accept a residual risk of equipment operational damage. To get the "most bang for the buck", option (b) will usually be preferred. For example: experience from the 1994 Northridge earthquake showed that evacuation of hospitals was more likely caused by inadvertent release of water from sprinkler heads, or gross damage to HVAC or diesel generators due to poor vibration anchors, than the inadvertent actuation or failure of a solenoid switch.

At a thermal power plant in the 1989 Loma Prieta earthquake, there was continued operation during and after the earthquake, but the plant was ultimately shut down accidently by the inadvertent failure of a solenoid switch; the plant operator had known about the sensitivity of the switch and had spares, so simply replaced the failed switch. It is doubtful that the extra cost for initial design to procure only seismically-qualified equipment throughout the plant could be justified by an additional two-hour outage (and $100 spare part) of the plant once every 100 years or so. For example: say the extra cost to procure seismically operationally-qualified equipment = $5,000,000 at time zero. Say the cost of a 2 hour outage is loss of power sales for 2 hours, or 500,000 kilowatts x 2 hours x $0.10 per kilowatt = $100,000, one time over 100 years. Assume a discount rate of 7%. Then the benefit cost ratio is: $100,000 (future loss in current dollars) * 0.01 (chance per year) * 13.8 (present value of $1% over 50 year lifetime at 7%) / $5,000,000 (extra capital cost) = 0.0028. In other words, the extra capital cost can be no more than $13,800 in order to "break even" in this simple example. Clearly, it will cost much more than $13,800 to procure all equipment in a large power plant to the operational requirements of a U.S. commercial nuclear power plant.

A.11.1 References

Markis, N., Zhang, J., Rocking Response and Overturning of Anchored Equipment under Seismic Excitations, report prepared for PG&E, 1999.

Markis, N., Roussos, Y., Rocking Response and Overturning of Equipment under Horizontal Pulse-Type Motions, PEER 98/05, Berkeley, 1998.

Hucklebridge, Art, and Clough, R.W., Seismic response of uplifting frames, Journal of the Structural Division, ASCE, Vol. 104, No. ST8, pp 1211-1229, 1978.

A.12 Office Work Stations

Almost every commercial and residential building includes a number of office work stations.

At most facilities, most office work stations are unanchored. Figures A.12-1 and A.12-2 show two common workstations. These were tested atop a shake table (UBC, 1997). These were tested using a suite of five earthquake time histories (Kobe 1995, Loma Prieta 1989, Northridge 1994, Landers 1992, Simulated), with peak horizontal input motions from 0.20g to 1.42g. The motions represent recorded motions from M7± events within 5 km of the causative fault.

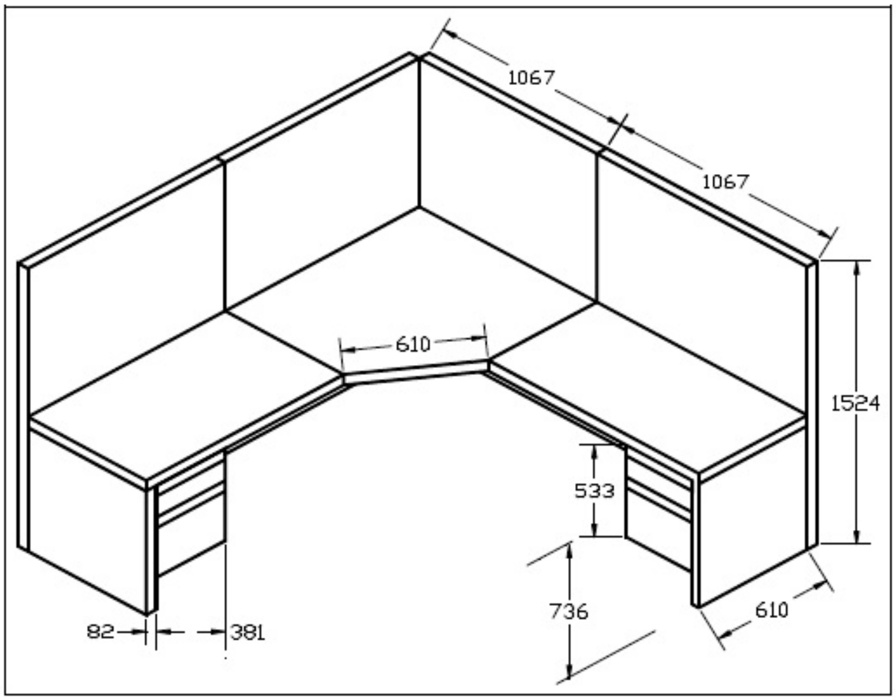

Figure A.12-1. Workstation 1 (dimensions in mm)

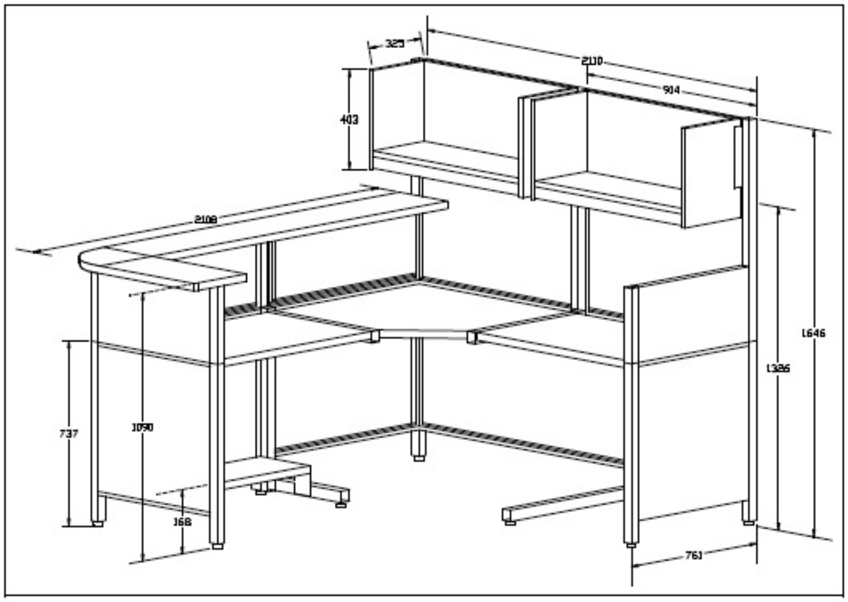

Figure A.12-2. Workstation 2 (dimensions in mm)

The workstations themselves suffered little if any damage, although workstation 2 moved a bit more than workstation 1 owing to lower friction between the legs and the underlying carpet.

Equipment	Office Workstation	
Case	A	Beta
1. Unanchored	1.0	0.50

Table A.12-1. Fragilities – Office Workstations

In Table 12-1, the damage stage is "minor", meaning minor disassembly of the partition walls of the workstation. The median A value would likely be lower for the similar workstation atop a hard floor surface (wood, tile, etc.) where substantial sliding would begin once the lateral force exceeds the coefficient of friction, and the extent of sliding could be estimated depending on the number and amount of exceedances, using a "sliding black" analogy commonly used for estimating landslide movements.

Given that the damage state occurs, the common consequences are as follows (these should be adjusted for site-specific considerations):

- LS = 1 or 2 (either no injury or minor injury will be the most likely consequence, assuming anyone is nearby).

- F = 0. this implies that most of the times, the slightly damaged workstation remains function (i.e., major disassembly does not occur).

- SR = 0.01. We assume it takes 1% of the replacement value to move the workstation back into its original position, and to adjust shelf or wall units to their original upright condition.

- LR = 0.05. The long term repair cost is assumed to be 5% of the replacement value, assuming that some minor part is bent and needs to be replaced.

A.12.1 References

University of British Columbia, Shake Table Testing of Office Work Stations, Department of Civil Engineering, British Columbia, 1997.

A.13 Emergency Generators

Emergency generators are used to provide backup power at facilities in the event of loss of off-site power.

For many buildings (including emergency operations centers, pump stations, etc.), there will often be only one emergency generator. For larger facilities (such as multi-building hospitals), the electric load may be large enough to justify multiple emergency generators, although not all generators may be designed to carry the load for all parts of the facility.

Even if all generators work, not all parts of the facility will be served by the backup power, as often the emergency power system is often limited to supply power to just a very few functions of the facility.

To assess the impact of damage to the emergency generators, one also needs to be able to estimate the potential loss of off-site utility power. Table A.13-1 provides a damage algorithm of loss of utility off-site power immediately after the earthquake. For most urban centers, major electric utilities have been able to restore power to close to 100% of their customers within 1 day after a moderately damaging earthquake, or up to 3 days after a major damaging earthquake.

Peak Ground Acceleration, g (at surface)	Probability of Loss of Off-Site Power
0.10	1 %
0.20	13 %
0.30	38 %
0.40	60 %
0.50	76 %
0.60	86 %
0.70	92 %
0.80	95 %
0.90	97 %
1.20	99 %

Table A.13-1. Loss of Off-site Power

The fragility of the diesel generator itself will depend upon its construction. Most diesel failures in the past, after earthquakes, have been due to one of several causes:

- The diesel-generator set falls off its vibration mounting, causing binding, severe of fuel or other lines, or other damage.
- Vibration-induced failure of exhaust systems, allowing exhaust to be vented into the same room with the diesel generator. For outdoor generators, this damage mode should not shut down the generator. For indoor generators, this damage mode will often require shut down of the generator.

- Battery failures, caused by failure of battery racks or movement of batteries within the racks.
- Other, including non-specific relay failures, fuel contamination, etc.

Under non-earthquake emergency conditions, emergency backup power diesels have been found to fail about 3% of the time (results from tests of several hundred emergency generators at water utility pump stations). If they are tested once every two weeks to one month period, then it is typical that a reasonably maintained diesel will not work, upon demand, once per year. However, if the diesel is maintained in operation almost daily, reliability is much higher.

We simplify the failure modes into failure / non-failure of an individual emergency diesel generator set (Table A.13-2). Two fragility curves are provided: one fragility curve is for a diesel generator that has well designed anchorage (direct anchorage to foundation, or heavily snubbed anchorage with isolators), well designed batteries and battery rack, and well designed ancillary equipment. The other fragility curve is for a diesel generator mounted on vibration isolators, or has any other significant seismic vulnerability.

For sites with multiple generators, seismic mitigation could include the additional of suitable transfer switches to allow emergency power from any generator to be used for any part of the facility. This will greatly improve reliability of the backup power system, as well as possibly provide extra power to allow lighting of more areas of the facility.

Common seismic mitigation is that all generators and associated tanks and electrical equipment should be fully anchored or snubbed.

For emergency generators, the damage state is as follows:

- Generator displaced laterally off vibration isolator supports or generator fails to start. Repair times about 2 to 8 hours after maintenance crew is mobilized.

Equipment	Generator	
Case	A	Beta
1. Vulnerable	0.25	0.60
2. Upgraded	0.60	0.50
3. Qualified	1.10	0.50

Table A.13-2. Fragilities – Emergency Generators

Given that the damage state occurs, the common consequences are as follows (these should be adjusted for site-specific considerations):

- LS = 1 (no injury will be the most likely consequence).
- F = 1. The generator does not start up or does not produce power.

- SR = 0.01. It takes 1% of the replacement value make minor adjustments to repair the damage. The damage could be a failed battery; dropping off the isolators; failure of fuel supply; failure of exhaust system; etc.

- LR = 0.05. The long term repair cost is based on the assumption that the damage was due to a failed isolator system. The cost reflects the installation of a new isolation system.

The fragility values in Table A.13-2 are for emergency generators that are *not* operated under full load on a daily (or near daily) basis. The fragility for daily-operated generators would be higher.

Should the emergency generator be seismically qualified per IEEE 344 (or similar for non-nuclear application) for PGA = 0.40g, then the fragility level should be at least PGA = 1.10g with beta = 0.5, which implies about a 2% to 3% failure rate at PGA = 0.40g.

A.13.1 References

IEEE 344, Recommended practice for seismic qualification of Class 1E equipment for nuclear power generating stations, 1975, updated 1987 and 2004.

A.14 Electrical Cabinets and Communication Equipment Racks

Electrical cabinets include low voltage switchgear (commonly 34.4 kV and below), including transformers, circuit breakers, disconnect switches, meters, relays and the like. The cabinets are usually made of heavy gage steel, internally reinforced with angles, and with suitable cut-outs for instruments and doors.

These cabinets are usually mounted on concrete floors. In some cases, the cabinets will rest on a "housekeeping pad" of concrete (perhaps 2 to 4 inches high). The purposes of the housekeeping pad is to keep minor flooding from affecting the equipment inside the cabinet.

Three kinds of seismic installations are commonly observed at commercial facilities. The cabinet rests directly on the floor, with no anchorage of any sort; the cabinet rests on the floor with a few small anchors; or the cabinet rests on the floor with robust anchorage / restraint.

For the unanchored / lightly anchored conditions, the issue is that with sufficiently high seismic forces, that the overturning moment will exceed the dead weight resistance (or limit resistance offered by limited anchorage), resulting in toppling of the panel. Complete toppling of such panels has been occasionally observed in past earthquakes at sites with PGA > 0.4g.

The potential to topple an unanchored panel depends upon the mass distribution in the cabinet, as well as the height-to-width ratio (higher H/W, the lower the PGA needed to topple), and the resistance offered by any conduits attached to the panel. Unanchored panels with heights of 6 feet and widths of 18 inches, with no top-level restrained conduits, are more fragile than unanchored cabinets with base width of 3 feet, and with several restrained conduits.

The fragilities in Table A.14-1 are for three ranges of electrical cabinets. These fragilities do not include damage to items within. Items in the cabinets are often seismically robust, but it is possible that there can be unanchored transformers within; unanchored batteries within; mercury switches that will incorrectly actuate due to shaking; solenoid valves that will incorrectly actuate due to shaking. Field interior inspection (taking off panels taking proper precautions if the cabinet is energized) is required to verify anchorage, and presence of interior unanchored items.

Communication equipment within a facility may include a variety of items. The general model presented here does not provide fragility curves by item. Instead, it is assumed that there are a variety of rack-mounted or desk mounted items and backup power supply that are required to make land-line-based analog phone and internet-protocol systems operate.

Radio-type communication equipment includes rack and desktop-mounted equipment. Radio communications often rely on off-site radio repeater stations. Loss of backup power at either the facility or the repeater station will hamper the system. Loss of power could result from loss of off-site utility power, failure of on-site backup supplies, or depletion of battery power (typically within 8 hours).

For communication and equipment racks (Table A.14-2), the damage states are as follows:

- Communication Racks. Racks topple leading to general equipment failure. Localized equipment failures within the racks (dislodging of cards, etc.) are not considered here, but likely under 1% of components.

For the "Unanchored" state, we refer to equipment racks are unanchored, with height-to-width ratio of about 4. For "flexible", we refer to racks that are anchored into the floor, but whose mass and stiffness properties allow it to bend sideways easily. This suggests amplified motion under strong ground motion, and also large enough displacements to potentially damage attached cables; also, it is rather common to observe equipment in these racks that are unrestrained (modems, monitors, switches, etc. that can slide off leading to potentially damaged cables. For "well anchored" we mean an anchored communication rack that is stiff, with no unrestrained components (could have lateral braces to provide stiffness). The "1.50 g" fragility level for the upgraded rack reflects that communication equipment is sometimes less rugged to intense ground shaking, with panel boards sliding out of slots and similar issues. If the engineer is aware of the seismic capacity of the components, then a higher median for the "upgraded" condition might be justified. However, for almost all practical situations, a value of 1.50g would imply a very low chance of component failure.

Equipment	Electrical Cabinets	
Case	A	Beta
1. Unanchored	0.60	0.60
2. Nominally anchored	1.00	0.60
3. Well anchored	3.00	0.60

Table A.14-1. Fragilities- Electrical Cabinets

Equipment	Communication Racks	
Case	A	Beta
1. Unanchored	0.20	0.60
2. Flexible	1.00	0.60
3. well anchored	1.50	0.60

Table A.14-2. Fragilities- Communication Racks

For cases where the equipment is located in elevated floors (penthouses) in multistory buildings, the fragility values would need to be adjusted down. For cases where the equipment is located in a sub-grade basement, the fragility should be increased (or conversely, the input motion decreased) by perhaps 25% or so.

Given that the damage state occurs, the common consequences are as follows (these should be adjusted for site-specific considerations):

- LS = 1. No injury will be the most likely consequence.

- F = 1. The equipment does not function.

- SR = 0.01. It takes 1% of the replacement value to implement some type of emergency work-around to achieve emergency level of function. Depending on the situation, there may be no emergency work around available until the long term repair is implemented.

- LR = 0.25. The long term repair cost assumes that most of the time, the bulk of the equipment (75%) is not damaged, and that the remaining 25% of the equipment needs to be replaced.

A.15 Equipment on Rollers

Many types of equipment are built on caster roller assemblies, to allow easy relocation within the facility. These include:

- Test bed equipment
- Equipment carts
- Beds
- ICU Beds
- Neonatal incubators
- Neonatal intensive care units (NICU incubators)

A test program was conducted to study the effect of varying type of roller (caster) conditions for incubators, typical hospital beds and ICU hospital beds (Mizuno, 1986). Unlike non-caster roller mounted items, these types of equipment can be found in three "anchorage" conditions: all casters locked; some casters locker; all casters unlocked.

The experimental tests observe that in practice, toppling of equipment on rollers occurs at accelerations about a third-higher than the theoretical value that initiates rocking, for typical items that rest on the floor without wheels, or have casters which are all locked. If two of four casters are locked, then the input acceleration needed to achieve toppling is about double or higher that for the "all-four-locked" condition, double if the locked casters are locked on one side, possibly higher if the casters are locked diagonally. Items with high width-to-height ratios, such as general hospital beds, are generally resistant to toppling (see discussion for fragility, below). Items with low width-to-height ratios, such as NICU incubators with heavy attached equipment items, are more prone to toppling. ICU beds, with narrower widths, have intermediate resistance to toppling.

To establish a suitable fragility curve for toppling of equipment on wheels, one can refine the analysis to account for each piece of equipment width to height ratio, and the usual practices used at the facility for locking casters. This approach appears too detailed for design, as the locking of casters is operational and outside the usual ability of the designer to enforce, so we make the assumption that there will be a variation in width-to-height ratios and caster locking practice, and suggest a fragility curve that appears reasonable for initial loss estimation purposes for all such equipment. The fragility curve would have to be adjusted to reflect actual inventory. With respect to damage and losses caused by rolling of equipment (blocking passageways, pullout of cables, etc.), the fragility does not include this damage state.

The fragility listed (A = 0.60g, beta = 0.50) reflects a top heavy weighted item with no locked wheels with H/W ratio of about 3, but to be somewhat conservative, assume this to be the case for tall, moderately top heavy items with two of four casters locked. As a first order estimate, for items on rollers with larger or smaller H/W ratios, multiply A by 3 * W/H (actual) (in other words, if W/H (actual) = 2, then A = 0.60g * 3 * 2 = 3.6g, or essentially rugged (cannot topple). A further adjustment should be made if the item on rollers is extremely top weighted, even weighted or bottom weighted, using the normalized ratios of fragilities for unanchored non-wheeled items.

For items on wheels that topple, the consequences could range from some type of injury (incubators, ICU beds) to damage (roller cabinet with computer monitor on top shelf) to spillage of books (library carts), etc. The LS, F, SR ad LR values will depend on the contents, and would have to be supplied by the cognizant engineer based on the particular application; no default value makes sense for all applications.

Observations in past earthquakes shows that items like refrigerators have often rolled across rooms (substantial distances) where PGA > 0.40g. Depending on the item, this may involve only minor effort to push the item back into place if there is no concurrent toppling or tearing of copper pipes. The tendency of the item to roll is a function of the coefficient of friction of the wheels on the floor (hard surface versus carpeted surface, for example, would impact the friction level). No fragility function is provided for the rolling mode, as in general this is not a major issue as the consequences are usually limited. While one can imagine a situation where rolling equipment might pose an egress risk, in most cases people can roll the equipment away, so this is a largely a trivial issue. The rolling of equipment on raised floors is addressed in Section A.6. The rolling of "housekeeping" items into nearby functional equipment can be an issue where the functional equipment has very sensitive parts that can realistically by damaged by impacts; in such cases, the mitigation is to remove the rolling equipment; chain it to wall, or other low cost housekeeping-type strategy.

A.15.1 References

Mizuno H., Iiba, M., Yamaguchi, N, Okano, H, Shaking Table Tests on Earthquake Resistance of Medical Equipment, Report of the Building Research Institute, No. 108, January 1986.

A.16 Mechanical Equipment

Mechanical equipment is typically located at grade level, and includes pressure vessels, fuel tanks, chillers, heat pumps, boilers, furnaces, fans, water pumps.

Most of this equipment has a relatively low center of gravity (low H/W ratio), and barring excessive displacements, is essentially rugged to vibratory shaking.

Depending on the nature of the equipment, functional failure can lead to modest inconvenience (loss of air conditioning), to serious life safety (loss of a liquid oxygen tank and system in a hospital is essential for certain medical services; patients on respirators require oxygen, without which they will die if not rapidly hooked up to a small portable oxygen tank).

Three fragility curves are provided. For unanchored low center-of-gravity equipment, we assume that it will begin to slide at about 0.5g (steel on concrete), and that by 0.6g it will have slid sufficiently to break attached pipes, resulting in loss of function; beta is set at 0.60, reflecting that damage might have a moderately high chance to occur at lower g levels. For lightly anchored systems, we mean a vertical tank or other similar component with limited / light anchorage (say designed to reach anchorage limit at about V = 0.15W; in other words, includes code "R" values). The median value is set at 0.70g, reflecting that the common damage mode is damage to attached pipes once the anchorage breaks and the equipment is free to rock or slide; beta is reduced to 0.50. By "well anchored", it is meant a similar component, but with the anchorage system designed for essentially elastically calculated seismic forces. For all these damage modes, LS = 1 (assumes the room / location is normally unoccupied), F = 1 (assumes breakage of pipes or conduits making the equipment inoperable), SR = 0.005 (isolate the leaking pipe), LR = 0.02 (repair of attached pipe / conduit and re-anchor). Consequential impacts (loss of medical gas, loss of service form the mechanical equipment, etc.) is not included, and must be evaluated separately based on the specific condition.

For specific pieces of equipment where the damage could involve toppling in occupied areas, the fragility levels may be lower and the consequences more severe.

A.17 Storage Racks

A variety of facilities will include large storage racks. These can be commonly found at retail outlets (Home Depot, Costco, etc.), and warehouses. Smaller storage cabinets are covered under A.8.

Common large storage racks are from 8 to 14 feet high, and 6 to 12 feet long. Depending on location, many (or most or all) of all the racks may be totally unanchored. Individual storage rack assemblies might be self standing, placed in long rows, back-to-back to adjacent similar racks. Some racks might have four columns and three or four shelves; some racks might have two columns oriented to provide shelves in a cantilever arrangement.

What is common is almost all large storage racks is that the rack assembly has multiple shelves that can be re-positioned to fit the customer's storage needs. To allow for the shelves to be re-positioned, the shelf-to-column attachments use a variety of "pin-and-hook" connectors, easily popped out for assembly by adjusting a light latch locking mechanism. Under seismic loading, these "pin-and-hook with latch" connectors have the requirement of being able to support dead loads, as well as seismic-induced reversible bending moments.

Over the past 30 years or so, common design practice was to design the racks for seismic loads assuming V = 0.X W, where W was taken as the dead load plus one-half the maximum imposed shelf contents, and X was set based on common code equivalent lateral force concepts, adjusted for the level of the design basis earthquake. For a self-standing rack, V=ZIC/Rw (W). Say Z = 0.4 (zone 4), I = 1 (regular importance), R = 6 (braced) or 8 (moment frame) and W = dead weight plus half the contents, say about 0.55W. then, for a moment frame rack, V = 0.076W (moment frame) or V = 0.10W (braced frame). The basis for the 50% factor on the contents portion of W stems from a UBC code case (27-11), which in turn stems from work done at URS (Chen et al 1980). This code case allowed base shears from 30% to 50% less than those the UBC code for other similar non-structural components, in part by relying on the back-to-back attachment of adjacent 4-legged racks which would provide more redundancy in the load path, and factor in that some racks might not be fully loaded. During the early 1990s, there was some effort on the part of the SEAOC to require increases in the base shear values for storage racks, spurred on, in part, by the observation that some heavily loaded storage racks had been damaged in the 1987 Whittier and 1989 Loma Prieta earthquakes. As not uncommon with code case changes, there was resistance to increasing the base shear requirement (increasing rack cost, and dispute as to what caused the rack failures in the 1987 and 1989 earthquakes) and concern that it might lead to costly retrofit of existing racks.

Given that the design of many racks in current use (2009) was predicated on V = 0.076W to 0.10W, using working stress rules, then we would not be surprised to see at least some of these racks collapse should they be heavily loaded and exposed to ground motions with PGA = 0.40g or higher. In fact, several such heavily loaded racks collapsed in the 1994 Northridge earthquake (see Bachman, 2005 for examples).

Today (2009), a concerned owner with a site right next to an active fault who wished to have high reliability might select a more robust design for similar storage racks in high seismic areas. Say PGA = 0.66g, I = 1.0, C = 2.5 and Rp = 4, then V = 0.412W, or about 500% more seismic load than allowed by UBC code case 27-11.

Some storage racks may have previously been damaged, often by impact by lift trucks. These can leave the columns twisted and buckled, which seriously compromises the seismic safety of the racks. The median PGA fragility to collapse damaged racks is lower than for undamaged racks.

The recommended least-cost mitigation for floor standing racks is to install a suitable number of anchor bolts from the rack to the floor, generally at least 4 bolts per rack, sometimes 8 bolts per rack, depending on the style of floor base plate. For racks with valuable items that can fall to the floor and be damaged, the racks should be upgraded with suitable shelf restraints (light slats / cords / or similar system). Mitigation could also be done by removing heavily loaded items, or by structurally upgrading the structural frames to have higher capacities / ductility to resist lateral loads.

Due to lack of available floor space, a series of lightweight storage racks could be installed in a "double decker" fashion. These might have been installed without explicit provision for seismic loading. The second story of storage racks rest atop the first story of racks. In some places, the second story racks might be attached only to light gage grates with very limited capacity friction clamps; and the grate in turn is lightly clamped to the lower level racks; these connections can take very limited lateral loading before failure. The racks are made of lightweight sheet metal, interconnected by limited capacity bolted channels. Under strong ground motions, they will try to act like cantilevers, attracting bending moments according to their relative stiffnesses. The racks might not be anchored to the floor, but might be able to resist high lateral loading through the frame action of the limited capacity bolted channels. The retrofit scheme for these multi-level racks might be to install horizontal cross members between individual racks with sufficient seismic capacity to make the entire rack system behave as a continuous structural system for the design basis earthquake. Second story racks that are connected lightly on the grate system should be positively anchored to a suitable lateral load resisting structure. The fragility of multi-story racks should be developed based on their unique characteristics, and is not covered by this report.

Rack components that have been damaged by impact by lift trucks should be repaired. Shelf restraint systems can be added to limit fallout of fragile components from the shelves.

Rather than retrofit existing racks, if the cost becomes too high, it might be more cost effective to replace the old weak racks with new, seismically-designed racks.

Given the above issues, Table A.17-1 lists fragility information for heavy storage racks. By "heavy", we mean storage racks that are carrying at least 30% of their rated weight. Two damage states are provided: rack contents slip out onto the floor, and rack suffers sufficient structural deformation as to allow a portion of contents to fall onto the floor, or general structural collapse. Should a rack be known to be carrying nearly 100% of its rated weight (or even more), then downward adjustment to A should be made.

For the "Loose contents slide to floor", we assume that there are no restrainer system to contain the contents, and that the items are loose are relatively lightweight individually. The median A is 0.25g. This mode of failure is commonly observed in supermarkets or warehouses, beginning to be observed at PGA = 0.15g, widespread at PGA = 0.30g, and extensive at PGA = 0.50g. LS = 2 meaning that there is some small chance of minor injury of people are nearby, but likely infrequent. SR = 0.01, meaning the cost to pick up the items and replace them onto the shelves. LR = 0.20, reflecting that some items will be damaged when they fall (use value of rack and contents together). In specific situations, SR and LR will vary, based on the nature of the contents. The meaning of functionality for s storage rack is that the rack can perform its function (hold contents), not for functionality of the fallen items or the warehouse as a whole.

For palletized items that are restrained to wood pallets, the coefficient of friction is higher, so the A is higher. If the item slides but does not fall, we do not consider that as a failure. LS=3, meaning that serious injury could occur if people are nearby, with some chance of fatality. SR= 0.02 reflecting that more effort is needed to restock the items (lift trucks) and LR = 0.30 reflecting that the fallen item might have a higher chance of damage. However, if the items are rolls of paper or plastic, LR might be as low as 0.01, whereas if the item is a transformer, LR might be as high as 0.90.

Damage states 1 and 2 are precluded if suitable shelf restraints are installed.

Damage states 3 and 4 are for structural collapse of the storage rack, assuming design to UBC 27-11 for Zone 4. The high A value for DS 3 reflects that lightly loaded racks (weight under 40% of rated level) have rarely been observed to fail. The A level for DS 4 reflects that many of the highest loaded racks at warehouse stores in zones with very high levels of ground shaking in Northridge earthquake collapsed.

Damage stage 5 is for racks designed for the design level earthquake for fully-loaded racks, with a complete lateral load resisting system, with items prevented from sliding off the shelves, and for limited ductility demand on cold formed members or joints that is shown to be able to withstand without gross failure. Generally, this implies a design for V = 0.30W or higher.

Damage states 3, 4 and 5 should be adjusted lower if there is damage to the existing columns, or if the shelves / bracing have been compromised to limit their seismic capacity.

For cases where the rack is unanchored, a rack toppling failure mode should be checked, in addition to the failure modes in Table A.17-1 (see A.11) and the lower capacity considered.

Damage State	A	Beta	LS	F	SR	LR
1. Loose contents slide to floor	0.30	0.60	2	0	0.01	0.20
2. Palletized contents slide to floor	0.70	0.60	3	0	0.02	0.30
3. Structural collapse, W<0.40	0.90	0.50	4	1	0.02	0.70
4. Structural collapse, W>0.60	0.60	0.50	4	1	0.02	0.70
5. Structural collapse, high design	1.50	0.50	4	1	0.02	0.70

Table A.17-1. Fragilities – Storage Racks

A.17.1 References

Bachman, R., Editor, Seismic Considerations for Steel Storage Racks Located in Areas Accessible to the Public, prepared by NIBS, FEMA 460, September 2005.

Chen, C.K., Scholl, R.E. and Blume, J.A., Seismic Study of Industrial Storage Racks, report prepared for the NSF and for the Rack Manufacturers Institute and Automated Storage and Retrieval Systems, URS/ John A Blume, San Francisco, 1980.

UBC Standard No. 27-11, 1991.

A.18 SQUG and Empirical Databases

A.18.1 SQUG

SQUG is the acronym for the "Seismic Qualification Utility Group". SQUG was formed as a group of U.S. commercial nuclear power plant owners to address an unresolved safety concern about the seismic qualification of mechanical and electrical equipment that provide safety-related functions in operating commercial nuclear reactors in the United States.

In a nutshell, the issue was as follows. Several nuclear power plants were constructed in the United States prior to the adoption of rigorous safety requirements for seismic qualification of equipment, per IEEE 344 (1975). In December 1980, the Nuclear Regulatory Commission (NRC) issued a request to all operating reactors, called Unresolved Safety Issue A-46 (USI A-46), "Seismic Qualification of Equipment in Operating Reactors". For nuclear power plants then under construction, it was mandatory that all safety related equipment be qualified under IEEE 344 (1975) or related standards, and hence were not covered by USI-A46. For nuclear power plants then under operation that were covered by USI A-46, each could re-examine all of its equipment under IEEE 344 and make upgrades as needed; however, this was likely to be a costly effort.

In response, a group of utilities got together to develop a generic response to this unresolved safety question, in a manner that was likely to be less costly to the utility owners, while still satisfying the NRC. Hence the name "SQUG".

The concept of SQUG was as follows. The empirical evidence of actual performance of various classes of mechanical and electrical equipment was collected. This data was then collated into a database. Some of the equipment failed, and some of the equipment performed well. The database was sorted and queried to establish classes of equipment, with suitable characteristics, that always survived actual earthquakes. This good-performing equipment then became the technical basis to prove that similarly constructed and installed equipment in operating nuclear plants was also qualified, subject to verification that there were no other possible failure modes, such as "systems interaction", whereby the equipment could still be damaged due to adverse interaction with the movement or other type of impact from nearby items.

In 1987, the NRC issued generic letter 87-02 (1987) that outlined how the NRC was going to accept the use of the above approach to seismically qualify equipment.

To summarize, a group of equipment was considered to have an adequately low chance of failure in earthquakes, if it was exposed to an earthquake with PGA under 0.30g, and if the equipment had all the characteristics of similarly installed equipment that had successfully underwent large ground motions. The key aspects to meet this criteria were: only equipment at selected older nuclear power plants could use this approach for qualification; all others had to use IEEE 344; the site PGA had to be under 0.30g; the equipment had to be anchored; the equipment could not contain parts (such as solenoid switches) that might or might not have been the same as in the experience database. There are other limitations in the use of empirical database for nuclear power plants, and these are not discussed herein.

For purposes of this report, the question arises: how can one use (re-use) empirical evidence of how electrical and mechanical equipment have performed in real earthquakes, in order to establish equipment fragilities.

The natural inclination is to use the collected database from SQUG and then sort the data in a manner as to extract fragility parameters for various classes of equipment. Table A.18-1 provides one such summary from an empirical dataset. However, the database is not in the public domain, and access to it requires fees and possibly other restrictions. For those interested in access to the database, please contact Mr. Robert Kassawara (1-650-855-2775) at the Electric Power Research Institute (EPRI 2003). The data in the database includes one record per item or group of similar items of equipment at one location, with photographs and catalog information for that item, as well as notes dealing with observations about the earthquake, including information from public sourses such as EERI, USGS and CDMG.

Examining Table A.18-1, one observes that there were reported that out of about 200 spring mounted centrifugal fans, 26 failures have been reported in past earthquakes. The data in Table A.18-1 is not so detailed as to distinguish among the quality of spring isolation systems with or without suitable seismic design. The damage algorithm rate in Table A.18-1 has been estimated for this dataset as under 1% for PGA < 0.1g, 5% for PGA ~0.2g, and 30% for PGA > 0.35g; the dataset does not distinguish between roof-level and basement level installations. The author has observed failures of spring-mounted items at roof levels of two-story hospitals (site PGA = 0.15g to 0.2g, roof level ZPA = 0.4g). It would seem reasonable to set A = 0.5g for a simple spring-isolated item, with the damage state being that the item displaces sideways sufficiently to either fall off one or more isolator units, or to break attached pipes that do not have the ability to take large (~2 inch or more) lateral imposed anchor motions. If the item reaches this damage state, this report assumes it is non-operable (could break the attached pipe). The common repair is to re-support the item (sometimes on wood blocking as an interim measure), and reconnect the attached pipes. It is not common to have damage to the actual rotating equipment once it is re-leveled, power and pipes restored, although this cannot be ruled out. LS=1, F=1, SR = 0.02, LR = 0.10.

Generic Equipment Category & Number of Examples from EPRI Database	Probability of Equipment Damage Requiring Prolonged Repair or Replacement			
	Overall Failure Rate	MMI VII	MMI VIII	MMI IX
Gas-Fired Steam/Hot Water boilers	Sample size: 24 Failures: 2 Overall failure rate = 8%	Sample size too small to allow estimates by shaking intensity		
Packaged Circulating Water Chillers	Sample size: 60 Failures: 1 Overall failure rate = 1 - 2%	0.5%	2%	7%
Motor-Driven Pumps (horizontal, 10 – 200 horsepower)	Sample size: 270 Failures: 9 Overall failure rate = 3%	0.5%	2%	10%
Centrifugal Fans (Including spring-mounted with or without seismic design)	Sample size: ~200 Failures: 26 Overall failure rate = 13%	0.5%	5%	30%
Air Handlers (coils, filters, louvers, fans)	Sample size: 70 Failures: 11 Overall failure rate = 15%	4%	10%	15%
Programmable Controllers (Mounted in free-standing cabinets, anchored or unanchored)	Sample size: 60 Failures: 7 Overall failure rate = 12%	3%	5%	30%
Vertical Steel Tanks (1,000 – 100,000 gallons, anchored or unanchored)	Sample size: 220 Failures: 22 Overall failure rate = 10%	1%	5%	25%
Hot & chilled water piping (per in-building runs of 1,000 feet, excluding failures due to shifting equipment)	Twenty-three E.Q.-induced leaks out of an inventory of ~200 miles of piping (Sample from steam power plants only) Overall failure rate = 2% per 1,000-ft of line	0.1%	3%	20%

Table A.18-1. Earthquake Performance Data

A.18.2 Can One Rely on Empirical Evidence?

The collection of data after an earthquake as to the performance of "what worked" and "what did not work" is an expensive and time-consuming process. Open-source and public / industry funded organizations such as EERI, ASCE / TCLEE, USGS and CDMG have performed many valuable post-earthquake reconnaissance efforts, and their findings are usually published and available for zero or nominal cost. Private companies (like structural engineering consultancies) sometimes perform their own reconnaissance efforts, and may selectively release some of their findings to their clients, or publish them in journals.

However the data is collected, what is needed to establish fragility information is the following:

- The level of shaking at the site of a particular piece of equipment
- The observed performance (failure / non-failure) of the equipment

In other words, for a particular level of shaking (PGA, PGV, PGD, SA, etc.), what is the numerator and what is the denominator, for equipment of similar characteristics.

Issues in using this type of information include:

- What is the sample size? As the sample size gets larger, the confidence in the statistics gets better.
- What are the characteristics of the equipment? Is it fair to group all electrical cabinets into one class of equipment, even though the metal enclosures have different designs; some are anchored; some are unanchored; some include heavy equipment (transformers); some are almost empty (very light weight); etc.
- When collecting the information, the field engineer may or may not be cognizant of all the important characteristics of all types of equipment. For example, the ASCE TCLEE Earthquake Investigation Committee requires its team members to undergo training once per year, so that they can understand the workings of various types of equipment found in different types of lifelines. Even so, no one person understands all equipment in all types of facilities. The teams sent out by EERI, USGS, CDMG, universities and private consultancies often have expertise in a few areas, but not all the areas.
- It has often been the case that one post-earthquake investigation team will report "there was no significant lifeline damage" in the earthquake. However, a second earthquake investigation team will go out, spend more time interviewing lifeline operators, and come back with a report showing "widespread lifeline damage".

Given these issues, how does one answer the central theme: "Can we rely on empirical evidence?" Obviously, we want to say "yes", but under what conditions can we be confident?

Porter et al (2008) has examine a portion of the SQUG database. Johnson et al (1999) has also examined a version of this database with 3,919 pieces of equipment from 123 sites in 23 earthquakes. Fragilities were developed based on this data. The fragilities are based on classes of equipment, but do not distinguish whether or not the equipment was anchored at the time of the earthquake. Unless the data is re-sorted and the fragilities re-developed by style of installation (anchored / not anchored, etc.) then the predictive power of the published fragility information is limited to a forecast of the level of damage to an inventory of equipment that matches the inventory of equipment in the underlying database.

The author of this report has prepared databases of equipment used by many lifeline operators, with more than 20,000 records covering more than 40,000 pieces of equipment. Some of this equipment has already been exposed to large earthquakes, and most to small earthquakes. The author has also visited many facilities after past earthquakes (1986 North Pam Springs, 1989 Loma Prieta, 1992 Big Bear, 1994 Northridge, 1995 Kobe Japan, 1999 Izmit, 2000 Napa, 2001 Peru, 2001 Bhuj India, 2002 Denali, 2003 San Simeon, 2008 Wenchuan, and participated with ASCE TCLEE on reviews of many other earthquakes.

The fragility data presented in this report reflects the observations in all of these post-earthquake reconnaissance efforts. As future earthquakes occur, perhaps new observations will be made, thereby requiring revision to the fragilities in this report.

Item	This report, unanchored or on springs A, g	This report, anchored or restrained A, g	Porter (2008) A, g	MCEER (1999) A, g
Air compressor	0.5	1.5	2.2	2.5
Air handling unit	0.5	1.5	1.4	1.9
Batteries in rack			3.0	2.5
Battery charger			4.2	2.0
Chiller	0.6	2.0	0.7	2.1
Control panel	0.6	3.0	2.3	2.3
Distribution panel	0.6	3.0	3.4	2.8
Engine Generator	0.25	0.6	1.6	2.0
Fan			1.4	1.6
Horizontal pump			2.6	3.0
Low voltage switchgear	0.6	3.0	1.2	1.3
Motor control center	0.6	3.0	1.8	1.5
Motor generator	0.25	0.6	1.8	1.5
Transformer			1.3	1.6
Valve			4.5	4.0

Table A.18-2. FEMA Non-Structural Default Fragilities

In Table A.18-2, Porter assigns beta to be 0.6 (all classes except motor generators, with beta = 0.4); whereas MCEER assigns beta to be 0.4 (all classes except air handing units, chillers, fans, which are assigned 0.5). In Table A.18-2, A (Porter) is the geometric mean peak horizontal acceleration applied at the base of the equipment. To convert the geometric mean to the single direction A, one needs to reduce the geometric mean A by about 15%. Observations about differences in values for A are as follows:

- About 50% of engine generators in the epicentral area of the 1994 Northridge earthquake failed to start. We do not have accurate data for every engine set as to its installation type. The recommended fragilities (0.25g or 0.6g) are both less than the values by Porter or MCEER by a wide margin.

- This report uses A = 1.0g for nominally anchored electrical cabinets and 3.0g for well anchored electrical cabinets. Porter and MCEER provide A between 1.2g and 3.4g for similar classes of equipment. Should the Porter work be de-aggregated for actual anchorage / vibration isolator installations, then likely there will be better agreement.

- The beta in this report is commonly 0.5, but sometimes larger. The beta by Porter is commonly 0.6, or by MCEER is commonly 0.4. Using a single beta to represent all randomness and uncertainty, the MCEER value of 0.4 appears too low for purposes of performance based design, even if it is supported by the datasets.

- The scatter of data in the real world often shows that some installations have few / no damage, while some installations have very high rates of failure. Fitting a lognormal fragility curve through such a dataset can be done, but likely ignores the actual causes of failure through significant mis-matches of the underlying inventory. Therefore, the resulting fragility curve will only be useful for predicting damage to an inventory of equipment that has the same mis-matches as in the empirical database; as the mis-matches in the empirical database are generally unknown, the resulting fragility curve may have no predictive power for a particular type of equipment at a particular building installation.

- The high values of A or some items (4.5g for valve, for example) is not based on actual observation of when 50% of the items fail; instead, it is simply an artifact of observing a low data rate (2% or so) at a lower level g, selecting a beta, and then back-calculating the median A to fit the data at lower left tail of the assumed failure distribution.

Consider as an example of the concern of using the Porter or MCEER fragilities. Consider the base of battery chargers. Normally, these are solid state (or nearly so) equipment mounted in an electrical box that is anchored to the floor. The A for Porter / MCEER is 4.2g / 2.0g respectively, both fairly rugged. The author would agree that any installation with good anchorage is nearly always rugged; however, the author has observed unusual installation conditions, including: unanchored battery chargers resting within or atop electrical panels; in these installations, the unanchored charger would fail if it slides and falls down, perhaps at A = 0.5g or so. Therefore, the end-user cannot use the Porter / MCEER fragilities unless the end user can also verify that the actual installations match the actual inventory in the underlying empirical evidence used in the Porter / MCEER formulations.

A.18.3 References

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