Campus: UCSF
Building Name: HSIR West
CAAN ID: 3009
Auxiliary Building ID: NA

UNIVERSITY OF CALIFORNIA

Date: 8/22/2019

FORM 1
CERTIFICATE OF SEISMIC PERFORMANCE LEVEL
☒ **UC-Designed & Constructed Facility**
☐ **Campus-Acquired or Leased Facility**

BUILDING DATA
Building Name: Health Sciences Instruction and Research West
Address: 513 Parnassus Ave., San Francisco
Site location coordinates: Latitude 37.7623 Longitudinal -122.4577

UCOP SEISMIC PERFORMANCE LEVEL (OR "RATING"): IV

ASCE 41-17 Model Building Type:
a. Longitudinal Direction: S1: Steel Moment Frame
b. Transverse Direction: S1: Steel Moment Frame

Gross Square Footage: 237,042
Number of stories *above* grade: 15
Number of basement stories *below* grade: 0

Year Original Building was Constructed: 1964
Original Building Design Code & Year: UBC-1961
Retrofit Building Design Code & Code (if applicable): NA

SITE INFORMATION
Site Class: C Basis: (Rutherford & Chekene, June 2012, 36)
Geologic Hazards:
Fault Rupture: No Basis: UCSF Presumptive Buildings – Geotechnical Assessment, Egan (2019)
Liquefaction: No Basis: UCSF Presumptive Buildings – Geotechnical Assessment, Egan (2019)
Landslide: No Basis: UCSF Presumptive Buildings – Geotechnical Assessment, Egan (2019)

ATTACHMENT
Original Structural Drawings: (Health Sciences Instruction and Research, Dr. Alexander G. Tarics, 7/25/1962, F0) or
Seismic Evaluation: (UC San Francisco Health Sciences Instruction and Research Seismic Evaluation and Conceptual Retrofit Design, Rutherford & Chekene, June 2012, Nonlinear Dynamic Modeling)
Retrofit Structural Drawings: NA. Nonstructural life safety upgrades were completed under project numbers M2632 and M2637.

Campus: UCSF
Building Name: HSIR West
CAAN ID: 3009
Auxiliary Building ID: NA

UNIVERSITY OF CALIFORNIA

Date: 8/22/2019

CERTIFICATION & PRESUMPTIVE RATING VERIFICATION STATEMENT

I, Maryann T. Phipps, a California-licensed structural engineer, am responsible for the completion of this certificate, and I have no ownership interest in the property identified above. My scope of review to support the completion of this certificate included both of the following ("No" responses must include an explanation):

a) the review of structural drawings indicating that they are as-built or record drawings, or that they otherwise are the basis for the construction of the building: ☑ Yes ☐ No
b) visiting the building to verify the observable existing conditions are reasonably consistent with those shown on the structural drawings: ☑ Yes ☐ No

Based on my review, I have verified that the UCOP Seismic Performance Level (SPL) is presumptively permitted by the following UC Seismic Program Guidebook provision (choose one of the following):

☐ 1) Contract documents indicate that the original design and construction of the aforementioned building is in accordance with the benchmark design code year (or later) building code seismic design provisions for UBC or IBC listed in Table 1 below.

☑ 2) The existing SPL rating is based on an acceptable basis of seismic evaluation completed in 2006 or later.

☐ 3) Contract documents indicate that a comprehensive[1] building seismic retrofit design was fully-constructed with an engineered design based on the 1997 UBC/1998 ***or later*** CBC, and (choose one of the following):

☐ the retrofit project was completed by the UC campus. Further, the design was based on ground motion parameters, at a minimum, corresponding to BSE-1E (or BSE-R) and BSE-2E (or BSE-C) as defined in ASCE 41, or the full design basis ground motion required in the 1997 UBC/1998 CBC ***or later*** for EXISTING buildings, and is presumptively assigned an SPL rating of IV.
☐ the retrofit project was completed by the UC campus. Further, the design was based on ground motion parameters, at a minimum, corresponding to BSE-1 (or BSE-1N) and BSE-2 (or BSE-2N) as defined in ASCE 41, or the full design basis ground motion required in the 1997 UBC/1998 ***or later*** CBC for NEW buildings, and is presumptively assigned an SPL rating of III.
☐ the retrofit project was not completed by the UC campus following UC policies, and is presumptively assigned an SPL rating of IV.

[1] A comprehensive retrofit addresses the entire building structural system as indicated by the associated seismic evaluation, as opposed to addressing selective portions of the structural system.

CERTIFICATION SIGNATURE

AFFIX SEAL HERE

Maryann T. Phipps	President
Print Name	Title
S2995	6/30/2020
CA Professional Registration No.	License Expiration Date
Maryann T. Phipps	8/22/2019
Signature	Date

REGISTERED PROFESSIONAL ENGINEER
MARYANN T. PHIPPS
No. 2995
EXP. 6/30/20
STRUCTURAL
STATE OF CALIFORNIA

Maryann T. Phipps

9/4/2019

Estructure, (510) 235-3116, 1144 65th St Suite A, Oakland
Firm Name, Phone Number, and Address

Table 1: Benchmark Building Codes and Standards

Building Type[a,b]	Building Seismic Design Provisions	
	UBC	IBC
Wood frame, wood shear panels (Types W1 and W2)	1976	2000
Wood frame, wood shear panels (Type W1a)	1976	2000
Steel moment-resisting frame (Types S1 and S1a)	1997	2000
Steel concentrically braced frame (Types S2 and S2a)	1997	2000
Steel eccentrically braced frame (Types S2 and S2a)	1988[g]	2000
Buckling-restrained braced frame (Types S2 and S2a)	[f]	2006
Metal building frames (Type S3)	[f]	2000
Steel frame with concrete shear walls (Type S4)	1994	2000
Steel frame with URM infill (Types S5 and S5a)	[f]	2000
Steel plate shear wall (Type S6)	[f]	2006
Cold-formed steel light-frame construction—shear wall system (Type CFS1)	1997[h]	2000
Cold-formed steel light-frame construction—strap-braced wall system (Type CFS2)	[f]	2003
Reinforced concrete moment-resisting frame (Type C1)[i]	1994	2000
Reinforced concrete shear walls (Types C2 and C2a)	1994	2000
Concrete frame with URM infill (Types C3 and C3a)	[f]	[f]
Tilt-up concrete (Types PC1 and PC1a)	1997	2000
Precast concrete frame (Types PC2 and PC2a)	[f]	2000
Reinforced masonry (Type RM1)	1997	2000
Reinforced masonry (Type RM2)	1994	2000
Unreinforced masonry (Type URM)	[f]	[f]
Unreinforced masonry (Type URMa)	[f]	[f]
Seismic isolation or passive dissipation	1991	2000

Note: This table has been adapted from ASCE 41-17 Table 3-2. Benchmark Building Codes and Standards for Life Safety Structural Performed at BSE-1E.

Note: UBC = *Uniform Building Code*. IBC = *International Building Code*.

[a] Building type refers to one of the common building types defined in Table 3-1 of ASCE 41-17.

[b] Buildings on hillside sites shall not be considered Benchmark Buildings.

[c] not used

[d] not used

[e] not used

[f] No benchmark year; buildings shall be evaluated in accordance with Section III.J.

[g] Steel eccentrically braced frames with links adjacent to columns shall comply with the 1994 UBC Emergency Provisions, published September/October 1994, or subsequent requirements.

[h] Cold-formed steel shear walls with wood structural panels only.

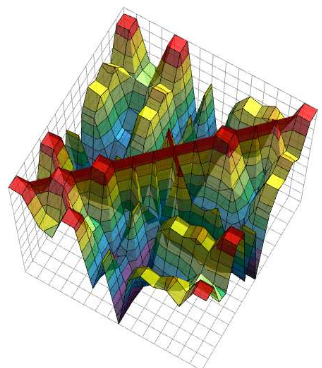
[i] Flat slab concrete moment frames shall not be considered Benchmark Buildings.

UC San Francisco Health Sciences Instruction and Research

Seismic Evaluation and Conceptual Retrofit Design

Volume 1

June 2012



Prepared for:
University of California, San Francisco
Capital Programs & Facilities Management
P.O. Box 0894
San Francisco, CA 94143

Prepared by:

Rutherford & Chekene
55 Second Street | Suite 600
San Francisco, CA 94105
415.568.4400

Joe Maffei, Ph.D, S.E. | Principal
Lawrence Burkett | Design Engineer
Andreas Schellenberg, Ph.D, P.E. | Design Engineer

2009-049S

EXECUTIVE SUMMARY

The Health Sciences Instruction and Research (HSIR) complex includes two large laboratory buildings on the Parnassus Campus of the University of California San Francisco. The buildings are designated Health Sciences East, 16 stories tall, and Health Sciences West, 15 stories tall. Adjacent to each laboratory building is a reinforced concrete mechanical tower which houses air supply and return ducts. The laboratory buildings are linked by a central elevator tower with a reinforced concrete core. The complex was constructed in 1963 and provides approximately 430,000 ft^2 of laboratory space for the School of Medicine.

The architecture and structural and mechanical engineering were innovative and unique for the era in which the buildings were designed. The two laboratory buildings are steel moment-frame structures made of large built-up sections. Interior girders span 93'-7" to keep the interior of the buildings free of columns. The two mechanical towers and one elevator tower use steel concentric braced frames embedded in concrete walls. The buildings are founded on the bedrock, using drilled piers at deeper bedrock and footings and rock anchors where the bedrock is closer to the ground surface.

A unique aspect of the buildings relates to the seismic separation joints, 16 inches wide, that occur between the laboratory buildings and the adjacent mechanical and elevator buildings. There are five joints in total. The joints represent a major seismic vulnerability in that relative motion of the structures is likely to damage utilities and systems that cross or are located within the joints.

OBJECTIVES

The University has two objectives that provide the impetus for this seismic evaluation and conceptual retrofit design. The first is a seismic code-compliance requirement triggered by the cumulative value of renovations conducted over the life of buildings. Because the cumulative cost of improvements has now reached 25% of the replacement cost of the buildings, the California Code of Regulations requires a review of the seismic performance of the building structure and nonstructural components. Part of this review includes a request by the University that Rutherford & Chekene classify the structures according to the recently revised University of California rating system for existing buildings.

A second objective for the University is to define measures to minimize the potential disruption to research activity in the event of an earthquake. As a hub of campus innovation, the work taking place in the buildings is critical to UCSF's mission and identity, representing $50 million in annual research funding. To address this objective, we conduct a detailed evaluation of the potential seismic deficiencies of the buildings, including their systems and contents, and propose a range of retrofit schemes to provide earthquake resilience to the facilities.

Our seismic evaluation of the Health Sciences buildings considers the effects of earthquake shaking on:

- The structure.
- Building utilities and systems, including mechanical, plumbing, fire protection, electrical, information technology, window glazing, elevators, and exiting.
- Laboratory contents, including equipment, casework, and partitions.

The overall seismic resilience of the complex depends on an interaction of the performance of all of the systems, which we quantify using advanced probabilistic and performance-based methods.

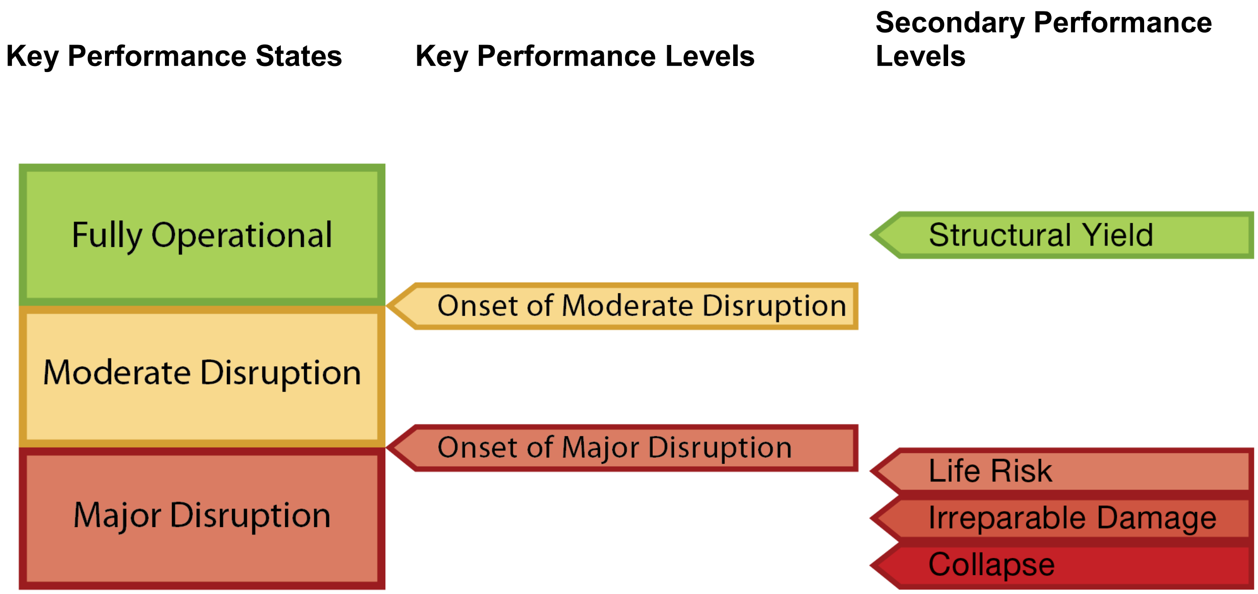


Figure 1: Chart of project-specific performance states and levels used in the HSIR evaluation.

Our study team includes our in-house structural and geotechnical experience and consultants with mechanical, electrical, information technology, and cost expertise. The consultants have specific knowledge of the various utilities, equipment, and systems of the Health Sciences buildings.

Methodology and Performance States

To describe the expected damage and disruption to the Health Sciences buildings following an earthquake, we define three potential post-earthquake performance states

- *Fully Operational:* Only minor disruptions to building function. The buildings should receive a green tag inspection posting allowing the building to be occupied after the earthquake.
- *Moderate Disruption:* Some damage that affects the continuation of research for days or weeks. No irreplaceable research is lost. The building could receive a yellow tag posting, allowing some short term occupancy shortly after the earthquake.
- *Major Disruption:* Casualties, irreparable structural damage, or other significant damage in which some essential research is irrecoverably disrupted.

In our performance-based analysis, we evaluate the probability that the buildings enter or exceed at least once in a given time period (e.g., the next 30 years) each of these performance states given a range of potential levels of earthquake shaking that can be observed at the site. We evaluate the likelihood of experiencing any of the performance states for the existing buildings (without retrofit), and also for the buildings with retrofit measures implemented.

In our analysis, we use the boundaries between the performance states as key performance levels—Onset of Moderate Disruption and Onset of Major Disruption. We also evaluate secondary performance levels of interest—Structural Yield, Life Risk, Irreparable Damage, and Structural Collapse. The performance states and levels are illustrated in Figure 1.

PROJECT-SPECIFIC PARAMETERS AND FRAGILITY FUNCTIONS

To estimate the annual probability of exceeding a given post-earthquake performance state, we identify the most appropriate variables that indicate the various manifestations of seismic performance. These engineering demands parameters (EDP*s*) are structural analysis output that we correlate with damage to the building structure, utilities, or laboratory contents. Given the unique properties of the Health Sciences structures, we define a total of 16 EDPs to track critical building response parameters. The EDPs include measures of peak and residual interstory drift ratio, peak floor acceleration, motion of separation joints, elongation of building column splices, and shear force in the mechanical towers.

For each of the EDPs, for each system affected by the EDP, we create and validate project specific fragility functions that relate the probability of each performance level or worse is occurring given that the value of the EDP is observed. For example, a fragility function will relate the probability of Major Disruption from laboratory contents damage given the peak floor acceleration at the worst floor of the building. The curves are developed both for the structure and systems in their existing state (without retrofit), and also for the case with various retrofit measures implemented. For example, if laboratory contents are anchored according to an "enhanced resilience" protocol, the probability of Major Disruption for a given floor acceleration is greatly reduced.

SEISMIC HAZARD

To characterize the potential for earthquake shaking at the site, we carry out a site-specific seismic hazard analysis, using our geotechnical characterization of the site, data from the United States Geological Survey, and Next Generation ground motion prediction equations. For reviewing code compliance, we develop site-specific uniform hazard spectra and use spectrally matched ground motions.

For evaluating the seismic risk and resilience of the buildings, which uses advanced probabilistic methods, we define the seismic hazard in a state-of-the art evaluation that uses:

- Conditional mean earthquake response spectra
- Mean return periods ranging from 70 to 5000 years
- "Dispersion-appropriate" ground motion sets

SEISMIC EVALUATION AND POTENTIAL DEFICIENCIES

Our structural evaluation of the buildings is based on a detailed examination of component capacities and governing behavior modes. The evaluation of structural components considers the numerous potential governing behaviors for the structural elements. For example, a tension flange connection of the steel moment frame can be governed by nine possible behavior modes of bolts or other connection parts in shear or tension.

The evaluation is done in conjunction with an explicit nonlinear dynamic computer model of the structures. The analysis uses 49 ground motion records and explicitly includes the modeling of a large number of potential nonlinearities in both the laboratory buildings and the stair/mechanical towers. The model includes the potential for yielding and elongation at column splices. The computer models for various schemes of retrofitting applied to the structure include viscous dampers according to the conceptual retrofit designs.

Our team evaluates building utilities and systems using thorough surveys and descriptions of each of the systems involved, including mechanical, plumbing, fire-protection, electrical, information technology, window glazing, elevators, and exiting. We evaluate laboratory contents based on detailed surveys of two representative laboratories, and on inventories (from the purchasing department) of laboratory equipment.

The primary seismic vulnerabilities identified in our analyses include:

- Potential damage to utilities and systems at the separation joints
- Potential damage to laboratory contents and building equipment from floor acceleration
- Potential damage to utilities and systems from building deformations
- Potential yielding and residual deformation at column splices at the lower stories

A number of structural elements are shown by our evaluation to have good seismic resistance and deformation capacity, including most of the main structural elements and connections of the steel moment frames.

RETROFIT MEASURES AND COST

To address the potential seismic deficiencies identified in our analysis and to improve the seismic resilience of the Health Sciences buildings, we identify a number of possible retrofit measures. Table 1 summarizes the retrofit measures and the cost of each. Subsequently, we organize the retrofit measures into logical combinations to create four overall retrofit schemes.

The individual retrofit measures for the structure include column splice retrofitting, adding viscous damping devices across the joints between the mechanical towers and the laboratories, and a more extensive scheme of adding damping devices in frames throughout the towers.

Retrofit measures for the utilities and systems of the building include anchorage of equipment and provisions for flexibility between floor levels or across separation joints. We define three levels at which this work can be done. The first level, designated U0, is required for compliance with Chapter 34 of the California Building Code. We also define a targeted loss-reduction level and an enhanced resilience level. The increasing levels of retrofitting differ mainly in the number of equipment items and systems that need to be addressed. The window glazing and elevator systems do not need retrofitting.

For the laboratory contents, we similarly define three different levels at which retrofitting can be carried out, designated as C1, C2, and C3. The levels differ mainly in how many types of items are to be anchored, and in whether partitions used to anchor items are to be evaluated and retrofitted or are left without evaluation and accepted as a potential source of damage risk.

The costs presented here are total construction costs. The estimates include abatement of asbestos, but do not include soft costs such as occupant surge during construction, design costs, or the University's project management costs.

Table 1: Construction cost and disruption of individual retrofit scopes.

Structural Retrofit	Utilities and Systems Retrofit	Contents Retrofit
S0: CBC Chapter 34 Minimum (No retrofit required) **$0**	U0: CBC Chapter 34 Minimum Temporary disruption to utilities **$548,000**	C0: CBC Chapter 34 Minimum (No retrofit required) **$0**
S1: Column Splice Levels 5.5 & 7.5 Localized disruption in 16 locations **$1,677,000**	U1: Targeted Loss Reduction Temporary disruption to utilities **(Additional) $1,417,000**	C1: Targeted Loss Reduction Minor disruption **$713,000**
S2: Mechanical Tower Dampers Exterior work, minimal disruption **(Additional) $2,105,000**		C2: Life Safety + High Value Minor disruption **(Additional) $411,000**
S3: Laboratory Damper Frames Phased disruption ¼ floor per installation, must coordinate with architecture **(Additional) $12,397,000**	U3: Enhanced Resilience Temporary disruption to utilities **(Additional) $2,972,000**	C3: Enhanced Resilience Minor disruption **(Additional) $213,000**

Retrofit Schemes and Costs

Based on logical combinations of retrofit scopes for the Health Sciences structures, utilities, and laboratory contents, we define four overall retrofit schemes, with estimated construction costs, as shown below:

- Scheme 0: $548,000 (Chapter 34 Minimum Code Compliance)
- Scheme I: $6,460,000
- Scheme II: $10,056,000
- Scheme III: $22,453,000

The costs for each retrofit scheme are assembled from the costs of individual retrofit scopes, which are shown in Table 1. The retrofit scope for each scheme is identified in Table 2 to Table 5. All of the retrofit schemes can be constructed with the building being occupied, with small areas—approximately 1/4 to 1/2 of a laboratory floor—being vacated at a time.

The retrofit schemes most consistent with the University's seismic performance objectives for the Health Science laboratories are Schemes II and Scheme III. Scheme III includes all of the retrofit measures shown in Table 5 and provides a cost-effective solution given the value of research conducted in the buildings. Scheme II provides a seismic performance level somewhat below that of Scheme III but can be implemented with less architectural disruption and at a lower cost.

Retrofit schemes 0 and I would be suitable to pursue, possibly as interim measures, if the University is not able to obtain full funding for Retrofit Scheme II. Retrofit Schemes I and II make a marked improvement in the expected seismic resilience to the Health Sciences facilities, and bring the building up to a Level III (Good) seismic rating, per the UC system. However, the additional cost of Retrofit Scheme III can be justified when considering the volume and importance of the research activities that take place in the Health Sciences buildings.

Retrofit Scheme 0 represents the minimum scope of work required to achieve full compliance with the requirements of Chapter 34 of the California Building Code. Our evaluation shows that the buildings in their current condition already meet the Chapter 34 requirements as related to the structure and the laboratory contents; only certain utilities and systems need to be retrofitted to meet this requirement. Specifically, the scheme provides flexible couplings for hot water and chemical plumbing above the separation joint at the exit pathway to the mechanical stairs. Adequate hazardous chemical storage should be verified as part of this retrofit scheme.

SEISMIC PERFORMANCE BENEFIT OF EACH RETROFIT SCHEME

R&C evaluates the seismic performance benefit of each proposed retrofit scheme, in comparison to the un-retrofitted buildings. We do this using a state-of-the-art probabilistic analysis that includes 140,000 earthquake simulations using the Monte Carlo technique. The results provide a detailed picture of the probability of reaching or exceeding the performance levels of concern in a given time frame. The analysis also allows us to evaluate the sources of seismic vulnerability—whether related to structure, utilities, or laboratory contents, and to tailor retrofit measures accordingly.

The probabilistic analysis provides the most complete description of the building seismic performance. The analysis incorporates the full range of possible earthquakes that can affect the Health Sciences buildings—from small frequent earthquakes to a rare large rupture on the San Andreas Fault. All of the potential earthquake scenarios are included according to their probability of occurrence.

The overall probabilistic results are shown in Figure 2. The figure shows the probability of exceeding the given performance levels over the next 30 years. For the buildings in their existing condition (without retrofitting), there is a 57% probability of reaching moderate disruption or worse, a 33% probability of reaching major disruption or worse, and a 23% probability of reaching a life-risk threshold.

Table 2: Scope included in Retrofit Scheme 0 (Cost $0.5M).

Structural Retrofits	Utilities Retrofits	Contents Retrofits
S0: CBC Chapter 34 Minimum = no retrofitting required	U0: CBC Chapter 34 Minimum	C0: CBC Chapter 34 Minimum = no retrofitting required
S1: Column Splice Levels 5.5 & 7.5	U1: Targeted Loss Reduction	C1: Targeted Loss Reduction
S2: Mechanical Tower Dampers		C2: Life Safety + High Value
S3: Laboratory Damper Frames	U3: Enhanced Resilience	C3: Enhanced Resilience

Table 3: Scope included in Retrofit Scheme I (Cost $6.5M).

Structural Retrofits	Utilities Retrofits	Contents Retrofits
S0: CBC Chapter 34 Minimum = no retrofitting required	U0: CBC Chapter 34 Minimum	C0: CBC Chapter 34 Minimum = no retrofitting required
S1: Column Splice Levels 5.5 & 7.5	U1: Targeted Loss Reduction	C1: Targeted Loss Reduction
S2: Mechanical Tower Dampers		C2: Life Safety + High Value
S3: Laboratory Damper Frames	U3: Enhanced Resilience	C3: Enhanced Resilience

Table 4: Scope included in Retrofit Scheme II (Cost $10.0M).

Structural Retrofits	Utilities Retrofits	Contents Retrofits
S0: CBC Chapter 34 Minimum = no retrofitting required	U0: CBC Chapter 34 Minimum	C0: CBC Chapter 34 Minimum = no retrofitting required
S1: Column Splice Levels 5.5 & 7.5	U1: Targeted Loss Reduction	C1: Targeted Loss Reduction
S2: Mechanical Tower Dampers		C2: Life Safety + High Value
S3: Laboratory Damper Frames	U3: Enhanced Resilience	C3: Enhanced Resilience

Table 5: Scope included in Retrofit Scheme III (Cost $22.5M).

Structural Retrofits	Utilities Retrofits	Contents Retrofits
S0: CBC Chapter 34 Minimum = no retrofitting required	U0: CBC Chapter 34 Minimum	C0: CBC Chapter 34 Minimum = no retrofitting required
S1: Column Splice Levels 5.5 & 7.5	U1: Targeted Loss Reduction	C1: Targeted Loss Reduction
S2: Mechanical Tower Dampers		C2: Life Safety + High Value
S3: Laboratory Damper Frames	U3: Enhanced Resilience	C3: Enhanced Resilience

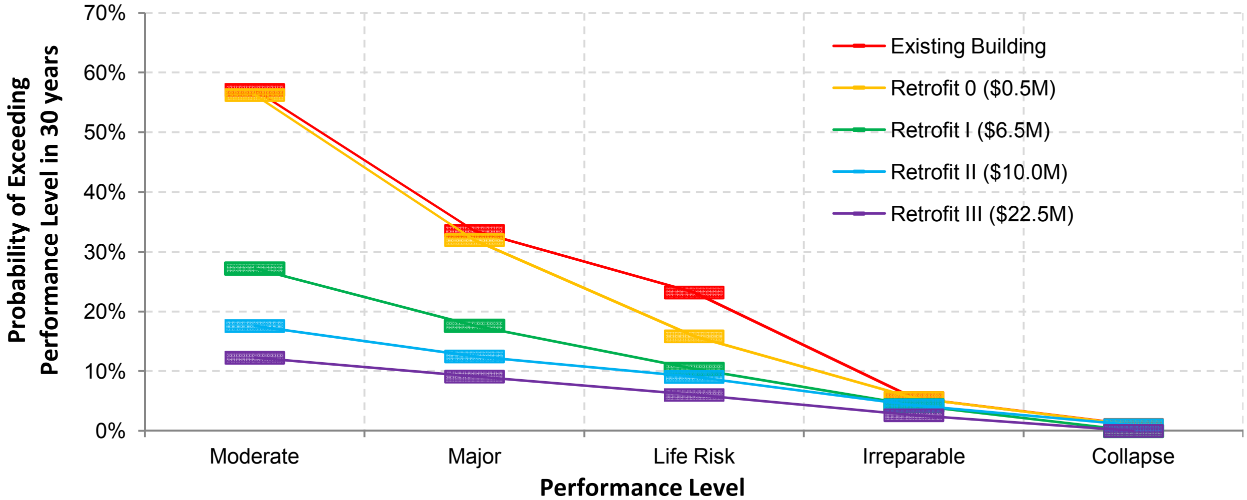


Figure 2: Probability of exceeding project-specific performance levels at least once over the next 30 years. The plot shows the cumulative probability considering structural, laboratory contents, mechanical, plumbing, fire protection, electrical, information technology, and window system damage. The existing building is shown as the base case, along with the results for each proposed retrofit scheme.

Retrofit Scheme 0 is targeted at addressing code issues related to non-structural life risk, and the probabilistic results show that the scheme effectively focuses on this objective, bringing life risk probability down from 23% to 16%, but offering a negligible reduction in the likelihood of disruption to the research functions.

Retrofit Scheme I starts to address resilience of operations, reducing the probability of reaching or exceeding moderate disruption to 27% and the probability of reaching or exceeding major disruption to 18%. Retrofit Scheme II can further reduce these probabilities to 18% and 12% respectively. Retrofit Scheme III has the largest impact in reducing the probability of disruption from earthquake impacts across all performance levels.

UC System Seismic Rating

The UC system Seismic Rating is designed to only consider life safety and protection against collapse. It does not address continued operation, recoverability, or resiliency.

In our opinion, if the code mandated Retrofit Scheme 0 is carried out, the Health Sciences Buildings will have seismic performance consistent with a Level II (Fair) rating according to the UC System. If the rating were based on the structural performance alone, a Level III (Good) rating would be justified—it is the life safety risk of the laboratory contents and building utilities and systems that cause us to judge the rating as Level II (Fair).

To achieve a Level III (Good) rating overall, we believe that Retrofit Scheme II is required. Retrofit Scheme III would also achieve a Level III (Good) rating and would provide substantial additional resilience to the Health Sciences buildings. It might be possible to show that Retrofit Scheme I provides a Level III (Good) rating, but this would require further study.

RECOMMENDATIONS

This report defines four possible Retrofit Schemes, from minimal code compliance to more complete measures, to improve the seismic performance of the Health Sciences buildings. **Based on our understanding of the University's objectives and the value and importance of the research that takes place in the buildings, we recommend implementation of either Scheme II, with an estimated construction cost of $10.1M, or Scheme III, which has an estimated construction cost of $22.4M. The minimum retrofit required to meet Chapter 24 of the California Building Code is Scheme 0, which has an estimated construction cost of $0.5M.**

As the next phases of retrofit planning proceed, refinements to the chosen retrofit scheme can be made to maximize its value to the University and to minimize the cost and disruption of its construction. This includes plans for phasing of construction and development of further details of evaluation and retrofitting. If needed, the probabilistic risk analysis used on this project can readily be used to evaluate refinements in retrofit schemes or to define likely damage scenarios, which can further demonstrate the value of the proposed retrofitting.

Seismic instrumentation in the buildings can help speed the University's recovery and re-occupancy of the laboratories following a damaging earthquake. If funding can be secured for instrumentation, we recommend an instrumentation scheme costing approximately $100,000.

TABLE OF CONTENTS

TABLE OF CONTENTS
(continued)

TABLE OF CONTENTS
(continued)

TABLE OF CONTENTS
(continued)

Volume 2

APPENDICES

Volume 3

APPENDICES

Volume 4

APPENDICES

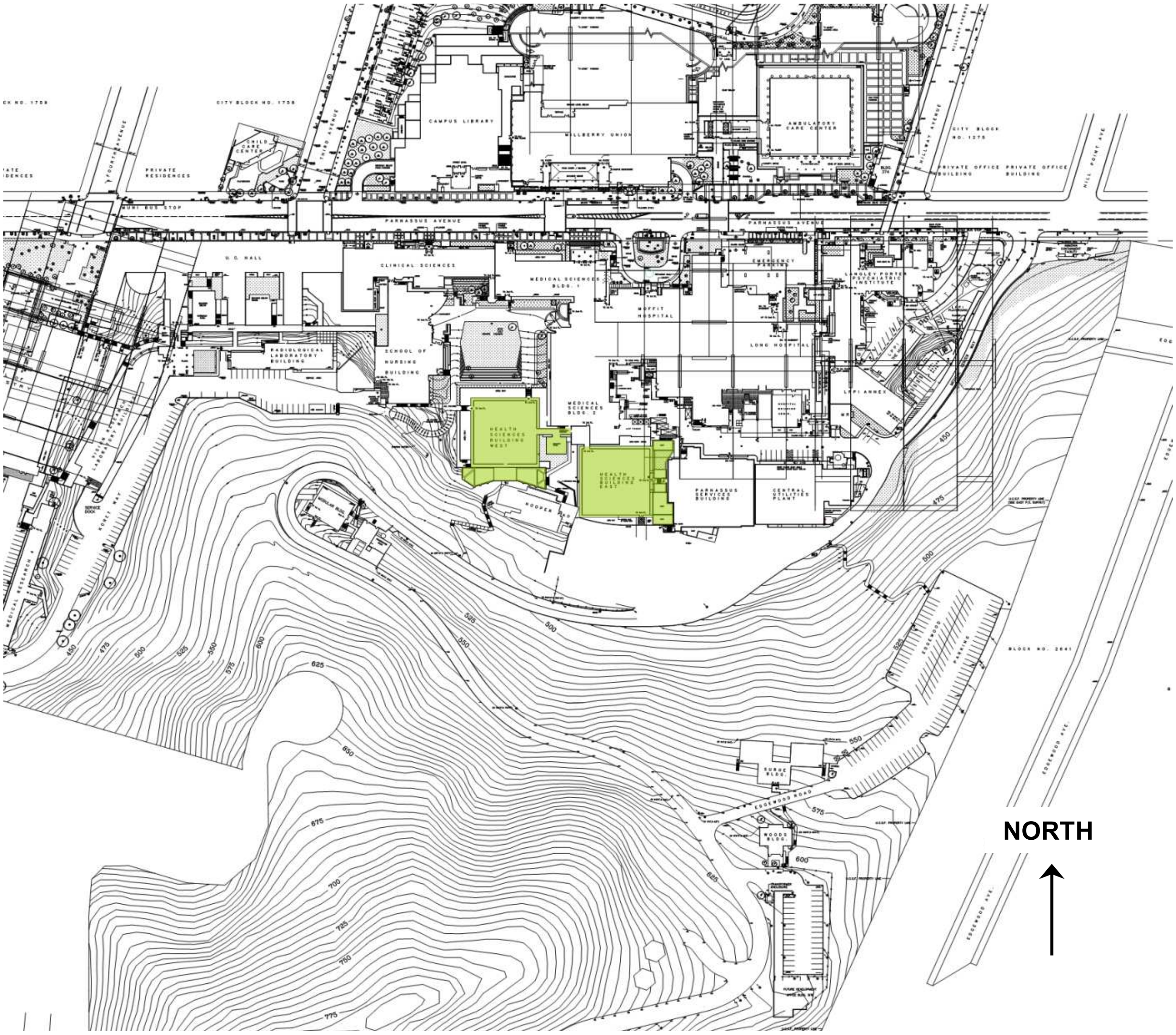


Figure 3: UCSF Parnassus Campus partial site plan showing Health Sciences Instruction and Research complex shaded.

1. INTRODUCTION

The Health Sciences Instruction and Research (HSIR) building complex consists of two 16-story laboratory buildings each with an adjacent stairway and mechanical utility tower supplying heating and ventilation. An elevator corridor tower connects the two laboratory buildings and the nearby Medical Sciences building. The two laboratory buildings are designated as Health Sciences East and Health Sciences West. The complex, constructed in 1963 on Medical Center Way at the Parnassus Campus of the University of California, San Francisco, provides approximately 430,000 ft^2 of space to house laboratories for the University's School of Medicine. In each of the laboratory towers, a steel moment-resisting frame system provides support for both gravity loads and lateral forces. The mechanical and elevator towers use a structural system consisting of steel braced frames embedded in reinforced concrete walls.

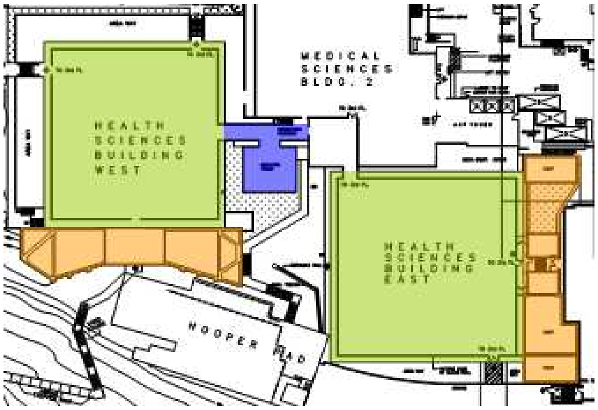


Figure 4: HSIR building complex site plan and project scope. Laboratory buildings are shaded green. Mechanical towers are shaded orange. Elevator tower and connecting corridor are shaded blue.

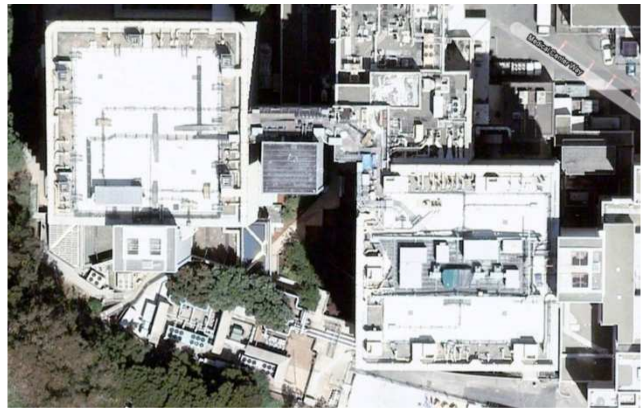


Figure 5: HSIR building satellite view. ©2009 Google, ©2009 GeoEye

1.1. BACKGROUND

Since the initial construction in 1963, numerous interior architectural renovations and mechanical system upgrades and additions have been undertaken, and today the Health Sciences complex is home to $50M in annual research activity. There have been no major *structural* renovations to the buildings. The cumulative value of renovations to the Health Sciences Buildings has recently exceeded 25% of the replacement value of each structure. This threshold, defined by the California Code of Regulations, requires a review of the seismic performance of the building structure and nonstructural components.

In addition to code requirements, the University is also interested in evaluating and improving the earthquake resilience of the buildings to limit potential disruption to research. Recent events at other research institutions have shown that damage and downtime following a natural disaster can lead to a loss of critical equipment, biological samples, or other research data, resulting in possibly irrecoverable set-backs in research progress. After Hurricane Katrina, this damage to research data led to difficulty retaining researchers. In an effort to minimize the possible losses and disruption, the University specified that the study should consider potential retrofit solutions to provide enhanced resilience to the structural and nonstructural components of the building which would limit earthquake damage and resulting impacts on research.

1.2. OBJECTIVES AND SCOPE

To address these issues, this study was commissioned to seismically evaluate the Health Sciences buildings and provide conceptual retrofit schemes along with their associated benefits and costs. The project consists of two objectives:

- An evaluation of the seismic life-safety performance of these buildings with respect to the requirements of the California Building Code and University of California policy. Where deficiencies are identified, we determine the scope and cost of structural and nonstructural retrofit schemes to produce the seismic performance level indicated in the code.
- An evaluation of the scope and cost of structural and nonstructural retrofit measures necessary to provide an enhanced level of resilience for the building and its utilities, systems, and contents, which will limit the potential for irrecoverable disruption to research after a major earthquake.

We evaluated the Health Sciences buildings in two parts, according to the University's two objectives: a code compliance analysis and an enhanced resilience analysis. The California Building Code compliance analysis investigates the adequacy of the building structure and nonstructural components according to the requirements of CBC Chapter 34. The general code standard is to provide Life Safety performance for earthquake motions with a 225-year return period, and to provide collapse-prevention structural performance for earthquake motions with a 975-year return period.

In the enhanced resilience analysis, we carry out a probabilistic and performance-based evaluation of the building structure and nonstructural components to determine the expected seismic performance of building components. Following the analysis of expected performance, we identify several levels of retrofit measures to provide enhanced resilience to the building's research operations.

Each phase of the evaluation includes the coordinated analysis and findings for significant building components including the structural system, building utilities, and seismic anchorage of laboratory contents. Consideration of building utilities and systems includes the following:

- Mechanical systems
- Plumbing systems
- Fire-Protection Systems
- Electrical Systems
- Telecommunications systems
- Information Technology Systems
- Exiting Systems
- Elevator Systems
- Window glazing Systems

The building analysis has been coordinated by Rutherford & Chekene, to include the findings of a team of consultants engaged by the University to provide a comprehensive seismic evaluation. The study is divided into three subject areas for each phase: a structural evaluation, an evaluation of the building systems and utilities, and an evaluation of the adequacy of laboratory contents seismic anchorage. The evaluation team for the project includes the following members:

University of California, San Francisco
Capital Programs and Facilities Management
Michael Bade, Assistant Vice Chancellor of Capital Programs & Campus Architect
Michael Toporkoff, Associate Director Parnassus Campus
Gary Nelson, Project Manager

Project Management, Structural, Geotechnical Engineering:
Rutherford & Chekene
Joseph Maffei, Principal
Lawrence Burkett
Bill Holmes, Principal

Architect:
SmithGroup
Marianne O'Brien

Mechanical, Plumbing, Fire-Protection Engineering:
Gayner Engineers
Shuen Lo

Electrical Engineering:
Cammisa and Wipf Consulting Engineers
George Puffett

Telecommunications, Information Technology Engineering:
Teecom
Kenneth Webb

Cost and Constructability Consultant:
DPR Construction
Gavin Keith
Wayne Revell

Seismic Damping Consultant:
Seismic Isolation Engineering
Ian Aiken

Consultant on Probabilistic Engineering Methods:
AIR Worldwide
Paolo Bazzuro

Our evaluation and design process, described in this report, follows from the scope of work detailed in our proposal to the University of California, San Francisco dated 2 July 2009. The analysis is based on information from the existing documents identified in Table 6, interviews with various UCSF employees, and a series of site visits at the buildings. The objective of this report is to describe the seismic evaluations that have been carried out for the Health Sciences buildings and to present our findings, recommendations, and conceptual retrofit designs along with associated benefits and costs.

Figure 6: View of Health Sciences building complex and adjacent buildings. © 2011 Google

1.3. DESCRIPTION OF HEALTH SCIENCES BUILDINGS

The Health Sciences Instruction and Research complex is located on the north slope of Mt. Sutro at the Parnassus Campus of the University of California, San Francisco. Designed by the Reid Rockwell Banwell and Tarics Architects and Engineers, and first occupied in 1966, the buildings are located on Medical Center Way, adjacent to the Medical Sciences Building, the Parnassus Services Building, and the recently constructed Institute for Regenerative Medicine. The Health Sciences complex consists of five seismically separate buildings including an elevator tower and corridor, and two laboratory buildings, each with its own mechanical utility tower supplying heating and ventilation. See Figure 4 and Figure 5.

The structural systems of the buildings are described in Section 4.1. The two laboratory buildings are steel moment-frame structures made of large built-up sections. Interior girders span 93'-7" to keep the interior of the buildings free of columns. The two mechanical towers and one elevator tower use steel concentric braced frames embedded in concrete walls. At each building, clay and sand overlay steeply sloping sandstone bedrock. The buildings are founded on the bedrock, using drilled piers at deeper bedrock and footings and rock anchors where the bedrock is closer the ground surface.

Building systems—mechanical, plumbing, fire protection, electrical, and information technology—are described in Sections 5 through 7, (along with assessments and retrofit recommendations for these systems). Window glazing, elevator, and exiting systems are described in Section 9. Section 8 describes laboratory equipment and contents.

2. PREVIOUS STUDIES AND AVAILABLE INFORMATION

There is generally very good and complete documentation of the existing structural systems, as described below. There is also good documentation of architectural and mechanical systems, including the numerous remodeling and system upgrade projects that have occurred since the original construction. (Previous involvement by the current project team members on many of these projects has also informed this report.)

There has not previously been a seismic evaluation of the Health Sciences structures. During the time of original construction there were a number of state-of-the-art (for the time) engineering experiments and studies related to the buildings, which are summarized below.

2.1. EXISTING STRUCTURAL DRAWINGS AND DOCUMENTS

UCSF maintains an archive of campus building construction documents in both digital and hard-copy formats. A list of the construction drawings and documents we reviewed during the course of the evaluation is shown in Table 6. None of the many major renovations to the buildings have modified the original structure. Thus, the structural analysis is based on the original construction drawings, including shop drawings provided by the steel fabricator, Bethlehem Steel.

Table 6: List of construction drawings and documents.

Title	Prepared By	Date	Sheets
Health Sciences Instruction and Research Unit 1: Architectural Drawings	Reid Rockwell Banwell & Tarics, Architects and Engineers, San Francisco, CA	August 10, 1962	Structural drawings: Sheets 1-61
Health Sciences Instruction and Research Unit 1: Foundation Drawings	Reid Rockwell Banwell & Tarics, Architects and Engineers, San Francisco, CA	July 25, 1962	Structural drawings: Sheets F1-F19
Health Sciences Instruction and Research Unit 1: Structural Drawings	Reid Rockwell Banwell & Tarics, Architects and Engineers, San Francisco, CA	May 25, 1962	Structural drawings: Sheets S1-S54
Health Sciences: Structural Steel Shop Drawings	Bethlehem Steel Company, Architects and Engineers, Alameda, CA	March 13, 1963	Structural drawings: Sheets 1-161, JS1-JS6A, SW1-SW3, SX2, SX3, TD1-TD4
Health Sciences: Existing Architectural Drawings	UCSF Capital Programs and Facilities Management Digital Archive	-	A1-A20
Health Sciences Instruction and Research Unit 1: Specifications	Reid Rockwell Banwell & Tarics, Architects and Engineers, San Francisco, CA	1962	-
An Investigation for the Earthquake Resistant Design of Large-Size Welded and Bolted Girder to Column Connection	J.G. Bouwkamp, University of California, Berkeley	January 22 - February 1, 1965	-
The Dynamic Behavior of Steel Frame And Truss Buildings	Dixon Rea, J.G. Bouwkamp, R. W. Clough, University of California, Berkeley	September, 1966	-

A number of original structural analyses of the laboratory buildings and mechanical towers are considered in R&C's study of the buildings. At the time of construction, a series of forced vibration tests, component strength tests, and computer analyses were performed by a team of investigators from the University of California, Berkeley led by J.G. Bouwkamp. These analyses are shown in Table 7. The relationship between these original studies and R&C's current analyses are discussed in Section 4.5

Table 7: Summary of structural analysis models and tests results.

Model	Analysis	Software	Description	Comment
1964 Forced Vibration	Experimental	-	Bouwkamp, et al. forced vibration testing of laboratory buildings and mechanical towers to experimentally determine mode shapes and vibrational periods	Results used to verify vibrational period response of current R&C models
1965 Welded Girder Connection Test	Experimental	-	Bouwkamp, test report for welded girder-column connection	Results used to verify connection components in current R&C PERFORM model
1966 Linear	Modal, LRH	FRMDYN	Bouwkamp, et al. 2-D model used to compute translational vibration modes and linear response	Results used to verify vibrational period response of current R&C models
1966 Nonlinear	NLRH	NONLIN	Bouwkamp, et al. 2-D model used to compute nonlinear translational response of a typical plane frame	Results of nonlinear response of the existing laboratory building compared to results from Current R&C PERFORM model

2.2. UCSF Equipment Seismic Anchorage Policy

Anchorage Policy

The current UCSF seismic anchorage policy for laboratory equipment is defined by two documents. The *UCSF Facilities Design Guidelines* dated September 1996, were revised in November 2003 for the Parnassus Campus. This document provides general criteria for anchorage of laboratory contents. According to the document, seismic safety should meet the more stringent of California Title 24 or local regulations, and should include consideration of nonstructural elements and contents. The University of California Industrial Hygiene Program Management Group produced the *Environmental Health and Safety (EH&S) Laboratory Safety Guide, Second Edition*, in September 2007. The report forms the basis of the UCSF anchorage policy, with specific requirements for seismic anchorage of laboratory contents. Taken together, the two documents outline criteria that are more specific and more thorough than the California Building Code criteria for new buildings. A summary of the general requirements is as follows:

- Casework should not be seismic sensitive;
- All shelves shall have a minimum 3/4" lip and the shelves themselves shall be firmly attached to the supports;
- All fixed equipment shall have flexible connections to building utilities;

- Any equipment, 42" or taller which may topple or block egress paths shall be seismically anchored;
- All compressed gas cylinders shall be restrained by racks or brackets with two chains;
- Installation of gas cylinder restraint is subject to review by EH&S;
- Gas cylinders shall be anchored to a permanent building fixture;
- Flammable, toxic, and corrosive liquid storage cabinets shall be seismically anchored;
- Optical benches shall be restrained;
- All biological safety cabinets shall be anchored;
- All refrigerators over five feet tall shall be anchored to the building structure.

There are also some specific anchorage requirements for laboratories with biocontainment precautions. A complete summary of the requirements with citations is provided in Appendix H. These anchorage requirements are not compiled together in either source document.

ANCHORAGE INSTALLATION

Several groups are responsible for the installation of seismic anchorage for laboratory contents. Typically, in new or renovated laboratory spaces, the general contractor or NOR-CAL moving services is responsible for installation of anchorage, except for permanently mounted equipment which is installed by UCSF Facilities Management. In existing laboratory spaces, UCSF Facilities Management is typically engaged by the laboratory users to anchor new equipment or existing equipment which was not previously anchored. In our discussions with Facilities Management, we learned that installation of seismic anchorage is based on previous experience rather than engineered anchorage details.

NOR-CAL has relied primarily on anchorage components and details purchased from WorkSafe Technologies to anchor larger equipment like incubators and sub-80 freezers. These anchorage components typically use double backed adhesive pads to secure equipment to wall mounted restraints. The UCSF Facilities Management group does not have a set of engineered details for seismic anchorage. Anchorage designs are based on experience and feedback from EH&S.

ANCHORAGE ENFORCEMENT

The UCSF Fire Marshall is in responsible charge of verification of seismic anchorage in newly constructed or renovated laboratories. In existing laboratory spaces, the Department Safety Advisor, an EH&S representative, is responsible for verifying that seismic anchorage meets the University criteria. These responsibilities are shown in Table 8.

In the course of our research, we conducted interviews with EH&S Environmental Specialist Frank Billante and the UCSF Campus Fire Marshall, Juan Martin. Mr. Billante is familiar with the *Environmental Health and Safety (EH&S) Laboratory Safety Guide* and its general requirements for seismic anchorage. Additionally, his work is governed by the requirements of California Title 19: Public Safety. The latter document makes limited mention of seismic safety. During our discussions, Mr. Billante mentioned that corridor widths in the HSIR buildings may not meet current code requirements. The allowable corridor widths are based on a legacy agreement with the State Fire Marshall that predates his tenure. As part of his practice, Mr. Billante performs an annual review of the HSIR buildings, including a focus on anchorage of laboratory equipment. In general, he expressed that users of the newer laboratory spaces were generally receptive to the requirements for seismic anchorage. Enforcement in older lab spaces, which are frequently more cluttered, is more difficult.

Juan Martin, the UCSF Campus Fire Marshall, expressed that his enforcement in new construction is based on his experience with the requirements of previous editions of the California Building Code and familiarity with typical anchorage procedures for equipment like freezers or cubicles. Mr. Martin would welcome the distribution of clear criteria documenting a University policy on seismic anchorage of equipment.

Table 8: Equipment seismic anchorage enforcement and installation responsibilities.

Category	Fire Marshall	EH&S Department Safety Advisor	Facilities Management	NOR-CAL	General Contractor
Installation of anchorage for moveable equipment in new construction				✓	✓
Installation of anchorage for permanent equipment in new construction			✓		
Enforcement of anchorage for equipment in new construction	✓				
Installation of anchorage for new equipment in existing laboratory			✓		
Enforcement of anchorage for new equipment in existing laboratory		✓			
Corridor width and storage in new construction	✓	✓			
Enforcement of chemical storage rules		✓			

3. SEISMIC EVALUATION CRITERIA, METHODOLOGY, AND ASSUMPTIONS

3.1. Overall Seismic Criteria

Our seismic evaluation of the HSIR buildings considers two basic criteria:

1. University of California and California Building Code requirements for existing buildings
2. A quantitative evaluation of options for providing an enhanced level of resilience for the building structure, systems, and contents to minimize disruption after an earthquake including estimated benefits and construction costs.

Building Code Criteria

Because the study is triggered by the cumulative value of building renovations to-date, the first criteria is subject to the requirements of CBC Chapter 34. Chapter 34 references the ASCE 41-06 standard, and prescribes minimum seismic performance requirements for structural and nonstructural components at two hazard levels.

At the earthquake demand level associated with a 225-year mean return period, the CBC requires a Life Safety performance level for all structural components and for nonstructural components in areas of public occupancy, and may be interpreted to also require this in areas of egress or public egress. Life Safety structural performance refers to the "post-earthquake damage state in which significant damage to the structure has occurred but some margin against either partial or total structural collapse remains" (ASCE 41-06). In both the structural and nonstructural contexts, Life Safety performance does not mean that all injury risk is eliminated. Rather, it results in a low risk of life-threatening injuries.

For the less frequent earthquake shaking associated with a 975-year return mean period, the CBC requires only Collapse Prevention performance from structural components. At this demand level, there are no code requirements for nonstructural performance. Collapse Prevention refers to the performance level in which the building is severely damaged, but maintains gravity load resistance with no margin against collapse.

A summary of the various requirements is provided in Table 9. The table indicates the sections of the CBC and ASCE 41 that are relevant, and reveals some lack of clarity regarding the Nonstructural Performance Level N-D which governs the code nonstructural requirements for the Health Sciences buildings. Section 1.52 of ASCE 41-06 indicates that nonstructural elements do not need to be retrofitted to preserve egress, while footnote 15 of the referenced requirements implies that retrofitting is required in areas of egress. As discussed in Section 10.3 of the report, we have applied the more conservative interpretation of ASCE 41-06 in determining the scope of Retrofit Scheme 0.

ASCE 41-06 defines the site hazard with a "General Response Spectrum", constructed from codified, mapped spectral acceleration data. Alternately, the standard allows the use of a site-specific response spectrum constructed considering the "geologic, seismologic, and soil characteristics associated with the specific site", provided the resulting spectrum meets certain minimum limits. For the Building Code Criteria, we use site-specific spectra, except where they fail to meet minimum spectral acceleration requirements. These spectra are summarized in Table 10.

Table 9: Building code sections containing seismic criteria applicable to the existing HSIR buildings.

Reference	Requirement
	CRITERIA 1: CALIFORNIA BUILDING CODE CHAPTER 34
CBC Table 3415.5	Performance shall satisfy the more restrictive of S-3 and N-D in BSE-R and S-5 and N-E in BSE-C.
CBC 3419	We use Method B to demonstrate compliance. (Method A is limited to linear evaluation procedures, which we do not recommend for this building).
ASCE 41-06 1.5	Structural Performance Level S-3 = Life Safety. Structural Performance Level S-5 = Collapse Prevention. Level N-E means not considered.
ASCE 41-06 1.5.2.	Nonstructural Performance Level N-D means "high-hazard nonstructural components identified in Table 11-1 are secured to prevent falling into areas of public assembly. Preservation of egress … are not addressed …." Per commentary C1.5.2.4 "it will generally be appropriate to consider … Life Safety Performance for [this] highest-risk subset of the nonstructural components in the building."
ASCE 41-06 Table 11-1	Footnote 15 requires meeting Life Safety for certain items if they "are located in areas of public occupancy or egress." Footnotes 12 and 16 have additional requirements.
CBC 3416.1	BSE-C corresponds to mean return period of 975 years. BSE-R corresponds to mean return period of 225 years.
ASCE 41-06 1.6.2	Site-specific spectra may be provided. Per 1.6.2.1.2, they shall not be less than 70% of the General Response Spectrum "in the period range of greatest significance to the structural response."
ASCE 41-06 1.6.1.3	General Response Spectrum for 975 years is interpolated from 475 years per equation 1-3 ($Ss \geq 1.5g$ per 2003 MCE map). General Response Spectrum for 225 years is also interpolated from 475 years per equation 1-3.
ASCE 41-06 1.6.2.1.6	Where ASCE 41-06 requires S_{xs} or S_{x1}, values may be taken from the site-specific spectrum, with the following minimums. S_{xs} may not be less that 90% of the peak response acceleration at any period. Values derived from S_{x1} shall not be less than 90% of those computed as $S_a = S_{x1}/T$.

RESILIENCE CRITERIA

To evaluate the likelihood of damage and disruption to the Health Sciences buildings under various retrofit schemes, for the range of earthquake shaking intensities, we compute the probability of exceeding key performance levels. We use the following overall procedure in our evaluation:

1. Define project-specific performance states and performance levels. (Sections 3.2 through 3.5)
2. Identify what conditions and types of damage (e.g., to the structure, building systems, or laboratory contents) can affect such performance levels. (Sections 3.2 through 3.5)
3. Identify the engineering demand parameters most closely related to the types of damage that affect the performance levels. (Section 3.6)
4. Quantify seismic hazard probabilistically over all levels of earthquake shaking and develop dispersion-appropriate ground motions for each level. (Section 3.8)
5. Estimate separate fragility functions for each building system that define the probability of reaching or exceeding a certain type or degree of damage, as a function of the associated

engineering demand parameter. Establish the fragility functions for different potential retrofit scopes. (Sections 3.9)

6. Run preliminary analyses and consider scenarios to validate the estimated fragility functions.
7. Perform the nonlinear structural analysis for multiple earthquake ground motions at multiple hazard levels (we use seven earthquake ground motions scaled at seven hazard levels for 49 total analyses). Record the engineering demand parameters of interest for each analysis.
8. Perform a statistical analysis to determine the correlation between engineering demand parameters given the same input motion. Using the correlation data, perform a Monte Carlo simulation to generate 20,000 additional earthquake outcomes with statistical properties matching the nonlinear analyses at each of seven hazard levels (140,000 total outcomes).
9. Using the fragility functions and the results of the Monte Carlo simulation, compute the damage state for each building subsystem for each outcome. Based on the subsystem results, compile the system and overall probability of exceeding a given performance level at each ground motion hazard level.
10. Integrate the performance level results with the site-specific seismic hazard spectra. The results of the integration are the total probability of exceeding a project-specific performance level in a given time frame.
11. Repeat steps 6-9 for each retrofit scheme to compare the seismic risk of the existing structure to the risk of the structure with retrofit measures implemented.

Our calculations of seismic risk are computed using state-of-the-art multi-variant regression and probabilistic analyses, with a computational tool called Monte Carlo simulation. With Monte Carlo simulation, we predict the building response to 140,000 possible earthquake ground motion scenarios and compute distributions of damage. The details of the probabilistic risk assessment are present in Section 3.10 and Appendix L.

For the resilience criteria, we construct site-specific spectra and hazard curves based on the most current USGS data and select input ground motion acceleration records using probabilistic techniques. Table 10 presents a summary of references used in estimating the site uniform hazard spectra. A detailed presentation of the calculation of site hazard and selection of ground motions is included in Section 3.9 and Appendix G.

Table 10: References used for computing earthquake acceleration spectra.

Reference	Remarks
ASCE 7-05 mapped spectral values, based on 2002 USGS hazard data	Used to create “General Response Spectrum” at both 975- and 475-year mean return period hazard levels.
2008 USGS Probabilistic 20%/50 year (225-year mean return period)	Used for site-specific response spectrum at 225-year mean return period.
2008 USGS Probabilistic 5%/50 year (975-year mean return period)	Used for site-specific response spectrum at 975-year mean return period.

3.2. PROJECT-SPECIFIC PERFORMANCE STATES

To identify anticipated disruptions to building function, we define three potential post-earthquake performance states.

FULLY OPERATIONAL PERFORMANCE STATE

A building in the *Fully Operational* performance state following an earthquake would experience only minor disruptions to building function and should receive a green tag inspection allowing the building to be occupied after the earthquake. This performance state is characterized by the following properties:

- Essential research activities continue unabated;
- Full research productivity resumes within days;
- No loss of power or other systems;
- Minor cleanup of some fallen items;
- Building is acceptably safe to occupy in aftershocks (green tag).

Possible visible damage for this performance state includes minor cracking of partitions and some equipment fallen from shelves and counters.

MODERATE DISRUPTION PERFORMANCE STATE

A building in the *Moderate Disruption* performance state would experience some damage to the structure, systems, or contents that affects the ability of the building to perform its research function. This performance state is characterized by the following properties:

- Essential research activities resume in days;
- Full research productivity resumes within weeks;
- Potential interruptions in power for hours;
- Potential interruptions to other building systems repaired in weeks;
- Significant cleanup of fallen items;
- Some experiments need to be re-set or re-run;
- No (or very few) irreplaceable samples lost;
- Building is acceptably safe to allow immediate emergency access to labs for clean-up and restoration of essential research or sample protection (yellow tag).

Possible visible damage includes cracking of partitions, broken exterior window glazing, equipment fallen from shelves and counters, overturning of some large equipment, and damage to concrete at separation joints due to building impact.

MAJOR DISRUPTION PERFORMANCE STATE

The *Major Disruption* performance state describes a building that experiences significant damage in an earthquake, from which some essential research activities are severely disrupted. This could occur from any of the following causes:

- Failure of power or other building systems and their back-up;
- Major disruption and damage to laboratory contents;
- Hazardous chemical release;
- Serious fire or water damage following the earthquake.

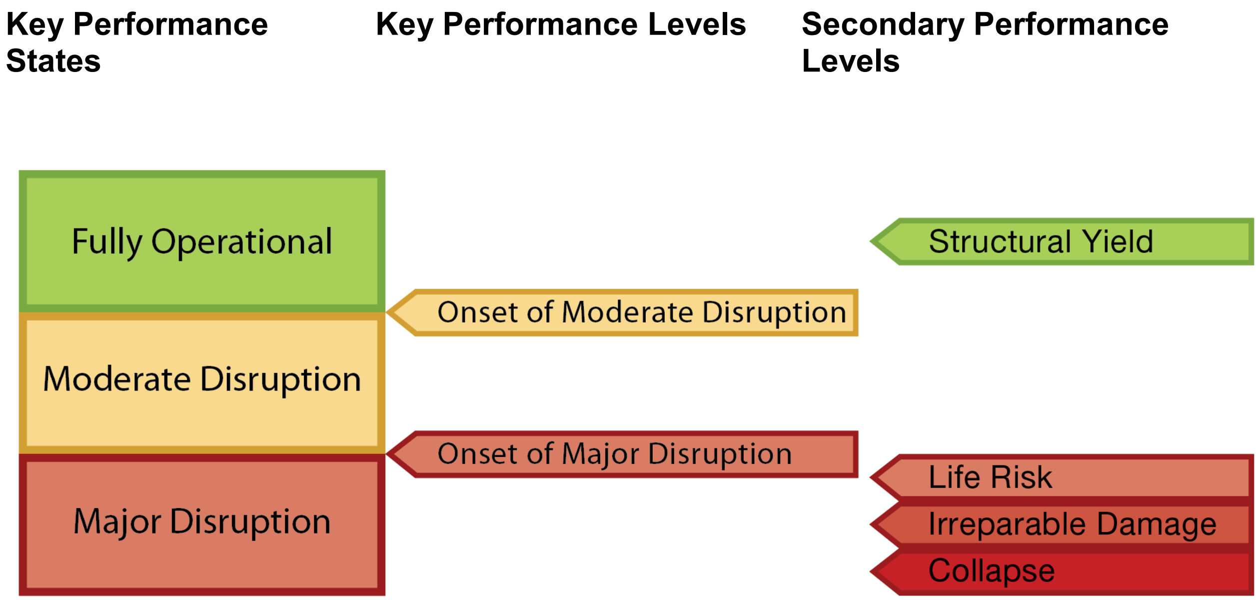


Figure 7: Chart of project-specific performance states and levels used in the HSIR evaluation.

- Loss of irreplaceable samples;
- Serious casualties or deaths;
- Building is not safe to occupy or structural failure prevents entrance to the building (red tag).

Possible visible damage includes steel joint or column splice fracture, significant building residual drift, or full or partial building collapse.

Note that the criteria that define these performance states relate to potential damage in the building structure, utilities and systems, or laboratory contents. In the evaluation to predict which damage state will result from a given level of earthquake motion, we must consider all of these potential sources of damage. The three project-specific performance states are shown in the leftmost column of Figure 7.

3.3. Key Performance Levels

We define two Key performance levels at the boundaries between performance states. Thus, for the three project-specific performance states there are two key performance levels (shown in the middle column of Figure 7):

Onset of Moderate Disruption Performance Level

The *Onset of Moderate Disruption* performance level marks the boundary between *Fully Operational* and *Moderate Disruption* performance states. Building performance below the *Onset of Moderate Disruption* level implies the building is in the *Fully Operational* state. If the building performance measures exceed this level, the building is in the *Moderate Disruption* state.

Table 11: Table showing which component types affect each performance level.

Description	Structure	Utilities and Systems	Contents
Structural Yield	✓		
Onset of Moderate Disruption	✓	✓	✓
Onset of Major Disruption	✓	✓	✓
Onset of Life Safety	✓	✓	✓
Onset of Irreparable Damage	✓	(Only if causing major fire)	(Only if causing major fire)
Collapse	✓		

Onset of Major Disruption Performance Level

The *Onset of Major Disruption* performance level defines the boundary between *Moderate Disruption* and *Major Disruption* performance states. Damage below this measure, but above *Onset of Moderate Disruption,* implies the *Moderate Disruption* state. Damage exceeding the *Onset of Major Disruption* implies the *Major Disruption* performance state.

As shown in Table 11, these two key performance levels can be reached because of damage to the building structure, utilities and systems, or laboratory contents.

3.4. Secondary Performance Levels

In the course of our seismic evaluation, we use secondary performance levels for comparison to seismic guidelines such as FEMA 306 (ATC, 1999) and ASCE 41-06 (ASCE/SEI, 2006). The secondary performance levels can also be indicators of the key performance levels. For example, if the building exceeds the *Onset of Life Risk* or *Onset of Irreparable Damage*, it has automatically exceeded the *Onset of Major Damage* performance level.

The secondary performance levels include the following:

Structural Yield Performance Level

The *Structural Yield* performance level marks the onset of significant yielding in the steel moment frame structure of the building. We set the level at a rotation ductility demand, $\mu = 1.25$, which we evaluate from the results of our nonlinear response-history analyses. The ductility demand is a measure of plastic deformation in the steel structure. As shown in Table 11, this performance level is affected by the building structural system, but not by the utilities, systems, or contents of the building.

Onset of Life Risk Performance Level

The *Life Risk* performance level defines the level of building performance at which either the building structure, the utilities and systems, or the laboratory contents begins to pose a risk to the lives of building occupants. It is comparable to the Life Safety performance level in ASCE 41-06.

ONSET OF IRREPARABLE DAMAGE PERFORMANCE LEVEL

The *Onset of Irreparable Damage* performance level marks the level of damage at which it would not be economically feasible to repair and reoccupy the HSIR buildings. Damage reaching this level would necessitate abandoning (and presumably demolishing and replacing) the buildings. We assume that only structural or fire damage would cause the building to be irreparable and that even severe damage to the building utilities, systems, or laboratory contents would not necessitate complete replacement.

COLLAPSE PERFORMANCE LEVEL

The *Collapse* performance level refers only to the building structural system. It identifies the point at which the building has no additional margin of strength or stiffness to resist collapse.

3.5. RELATIONSHIPS BETWEEN PERFORMANCE LEVELS

Figure 7 shows the project-specific performance state and performance levels with the more disruptive performance at the bottom of the figure, generally corresponding to more intense earthquake shaking. However, since the performance levels depend on the performance of different groups of building components, it is not possible to order them all linearly by increasing damage or earthquake ground motion intensity.

The careful understanding of which performance levels can overlap with others, and which are prerequisites for others is necessary for our explicit evaluation of the probability of the performance levels being reached or exceeded. Previous publications on performance-based design typically have assumed a simple linear progression of non-intersecting performance states, which is not the case for a facility like the Health Sciences complex. Some risk-assessment programs can accept overlapping performance states by collapsing results into a single index such as equivalent dollar loss, but in the process losing the ability to predict damage states. Seismic risk analysis programs and methods (ATC 58, Bazzuro, 2006) of the type previously used cannot be used for this facility for the results we present. Thus, as indicated in Section 3.11, R&C developed our own customized risk-assessment computational platform for this project,

PROGRESSION OF PERFORMANCE LEVELS

Figure 8 illustrates the known relationship of performance levels to each other, as earthquake ground motion intensity increases from left to right in the figure.

The top left of the figure indicates that, by definition, *Onset of Moderate Disruption* must occur prior to (i.e., at a smaller earthquake ground motion intensity than) *Onset of Major Disruption*. To the right of this, the figure shows that, since both *Onset of Life Risk* and *Onset of Irreparable Damage* are defining conditions for *Onset of Major Disruption*, they must occur at or beyond the intensity level for *Onset of Major Disruption*. However, because the two performance states depend on a variety of systems with different capacities, we do not know *a priori* whether *Irreparable Damage* or *Life Risk* would occur first. The nature of structural collapse, at least for the Health Sciences Buildings, is such that it will always pose a risk to human life and result in damage beyond repair. Thus *Onset of Collapse* must occur at or beyond the intensity level of both *Onset of Life Risk* and *Onset of Irreparable Damage*.

The bottom of Figure 8 shows, on a separate track, the hierarchy of performance states that depend solely on structural behavior. For the HSIR structure to become irreparable, we expect that it would have to deform well beyond yield. Thus *Onset of Structural Yield* is shown occurring before *Onset of Irreparable*

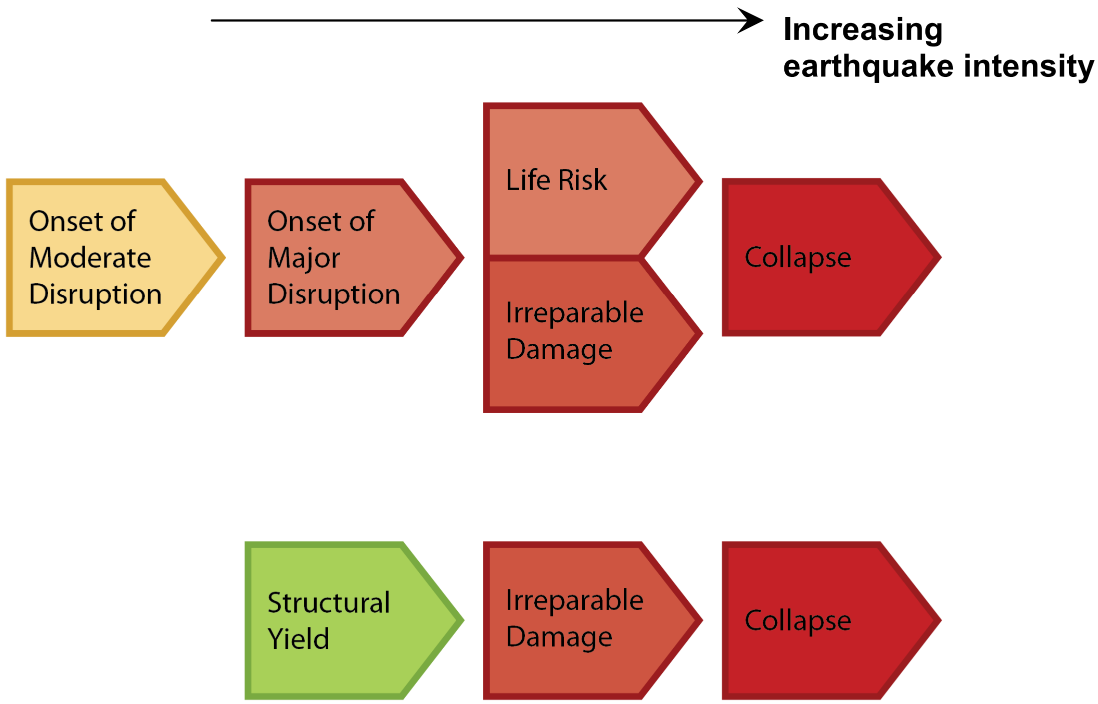


Figure 8: Chart showing the hierarchy of project-specific performance levels used in the HSIR evaluation.

Damage, which, as indicated previously, would occur before *Onset of Collapse*. An exception to this hierarchy of performance levels is the possibility that the structure could become irreparable without yielding because of highly flammable and earthquake-vulnerable contents or systems that cause a major fire, even though the structure did not yield from earthquake shaking. Such a fire occurred in a chemistry laboratory building at the University of Concepción in the 2010 Chile Earthquake.

The *Onset of Structural Yield* performance level cannot be positioned with certainty compared to *Onset of Moderate Disruption*, *Onset of Major Disruption*, or *Onset of Life Risk* performance levels, because each of these three levels can depend on nonstructural damage that could conceivably occur either with or without the structure yielding.

VENN DIAGRAM OF PERFORMANCE STATES

The relationships and overlap between performance states can be shown most completely using a Venn diagram, as shown in Figure 9. Such diagrams explain the relationships between performance states—relationships that must be correctly modeled in the probabilistic seismic risk analysis.

Figure 9 (a) shows the three key performance states, which have no overlap as the facility will be in only one of these states following an earthquake. Figure 9(b) shows details of states that are included within the key state of major disruption. The diagram shows that the presence of irreparable damage, life risk, or collapse triggers the major disruption state, but as indicated in Section 3.2 major disruption can also occur without any of these states occurring, for example through loss of irreplaceable samples, major disruption to contents, etc.

The diagram also shows that irreplaceable damage is not a prerequisite to life risk, nor vice-versa. The structure can be irreparably damaged without necessarily causing life risk. Likewise life risk can be caused by utilities or contents without irreparable structural damage. Collapse on the other hand automatically equates to life risk and irreparable damage.

Figure 9(c) shows the occurrence of structural yielding and its intersection with the other performance states. From the viewpoint of building users, structural yielding is not particularly meaningful because it does not directly indicate the disruption to the function of the building. However it is a useful intermediate index in the engineering evaluation that is readily tracked in the analysis model. As shown in the diagram of Figure 9(c), as well as in Figure 8, structural yield is considered to be a prerequisite to irreparable damage and collapse.

It is possible that life risk can occur without structural yielding, if the life risk comes from damaged utilities or contents rather than from any type of structural instability. Likewise, moderate or major disruption can occur with or without structural yielding. Moderate amount of structural yielding can occur in and earthquake with little or no evidence visible on subsequent inspection. Moderate amounts of yielding may in some cases have no detrimental effect on the subsequent earthquake performance of a building (FEMA 306). Thus it is possible, as shown in Figure 9(c) to have yielding while remaining in the fully operational performance state.

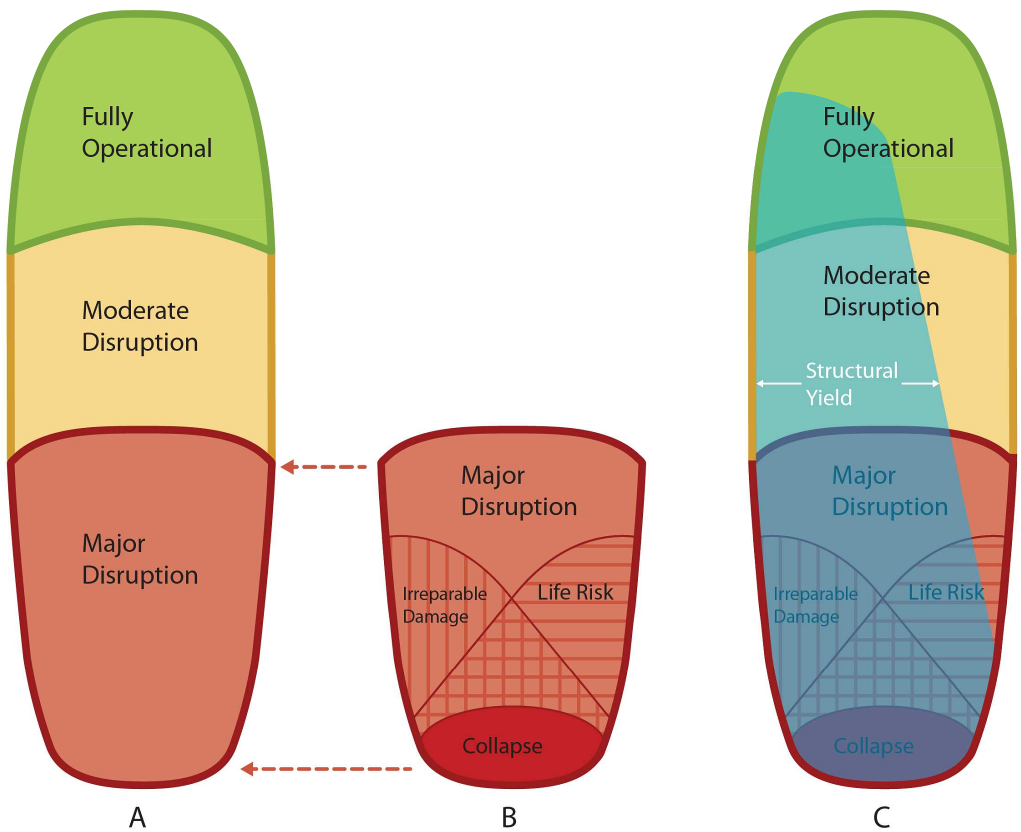


Figure 9: Venn diagram of project-specific earthquake performance states: (a) diagram of the three key performance states, (b) diagram of performance states within the key performance state of major disruption, (c) intersection of the structural yield performance state with other performance states.

3.6. PROJECT-SPECIFIC ENGINEERING DEMAND PARAMETERS

To describe the likelihood of damage and disruption to the Health Sciences buildings under various retrofit options, for the range of earthquake shaking intensities, we compute the probability of exceeding key performance levels in a given time frame. Part of this process is identifying the engineering demand parameters, such as floor acceleration or building interstory drift, that most closely relate to the types of damage that affect the performance levels.

Engineering demand parameters (EDPs) are structural analysis output variables that can be used to measure the performance of building components. Generally, the best demand parameter for any system or type of damage is the one that results in the lowest measurement of variability in the Component's performance, while also being reasonably well correlated with the intensity measure of ground motion that is being used to predict it.

Table 12 summarizes the engineering demand parameters that we use for each of the various systems of the Health Sciences buildings. The EDPs used in our risk assessment are described briefly below, and a complete description is presented in Appendix K.

STRUCTURAL EDPS

We use the peak interstory drift ratio as a parameter to describe the maximum floor deformation in any story during an earthquake. To account on an average sense for the interstory drift distribution in all principal horizontal directions, we use a square-root-sum-of-squares (SRSS) combination for the interstory drift in each plan direction. By considering both the maximum drift and the value at the 4th worst story, we can understand whether the building drifts are concentrated at a single story or distributed over the height of the structure.

Using the two parameters—worst story and 4th worst story—is a way of considering both the severity and the extent of the damage. Separate fragility functions are provided for each parameter. For example, if 2.5% drift in any story is associated with a certain probability of disruption, it is logical that some smaller amount of drift, say 2.0%, would cause a similar level of disruption if it were more widespread (e.g., occurring on at least four of the building's fifteen stories). Although subjective, this type of evaluation that uses a worst and 4th-worst parameter is applied to several of the EDPs.

Residual interstory drift is an important indicator during post-earthquake reconnaissance of building reparability and tagging. We consider the maximum and 4th worst residual drifts to understand the distribution of residual drift over the height of the building. We again use the SRSS combination to account for residual drift in all horizontal directions.

Our analysis of the existing building structure reveals a potential deficiency in the existing steel column splices. To compute the risk associated with the current splices, we monitor maximum and residual splice elongation in the splices at Levels 5.5 and 7.5.

Limiting damage to the mechanical towers is critical for preserving laboratory functionality. We estimate damage to the towers by considering shear force demands along the two tower axes.

Table 12: Building systems affected by each of the engineering demand parameters.

Engineering Demand Parameter (EDP)	Structure	Utilities & Systems	Laboratory Contents	Exterior Windows
	Building System			
Maximum SRSS Interstory drift Ratio (θ_{MAX})	✓	✓		✓
4th Worst SRSS Interstory drift Ratio (θ)	✓	✓		✓
Maximum Peak Horizontal Floor Acceleration (PFA_{MAX})		✓	✓	
4th Worst Peak Horizontal Floor Acceleration (PFA)		✓	✓	
Maximum SRSS Mech. Tower Separation joint Motion		✓		
4th Worst SRSS Mech. Tower Separation joint Motion		✓		
Maximum SRSS Elevator Tower Separation joint Motion		✓		
4th Worst SRSS Elevator Tower Separation joint Motion		✓		
Maximum Splice Elongation at Level 7.5	✓			
Residual Splice Elongation at Level 7.5	✓			
Maximum Splice Elongation at Level 5.5	✓			
Residual Splice Elongation at Level 5.5	✓			
Maximum Residual SRSS Interstory drift Ratio (θ_{R-MAX})	✓			
4th Worst Residual SRSS Interstory drift Ratio (θ_R)	✓			
Maximum Major Axis Mechanical Tower Shear ($V2$)	✓			
Maximum Major Axis Mechanical Tower Shear ($V3$)	✓			

Building Utilities and Systems EDPs

Comprehensive seismic vulnerability surveys for the building utilities are conducted as a part of this study by Gayner Engineers, Cammisa & Wipf Consulting Engineers, and TEECOM Design Group. The components of the building utilities can be classified as displacement-sensitive, acceleration-sensitive, or as sensitive to relative motion across the building separation joints. We track damage to displacement-sensitive components using the EDP maximum SRSS interstory drift. Acceleration-sensitive damage is estimated with peak horizontal floor acceleration. We measure damage to components in the separation joint by tracking the SRSS relative motion at the separation joints. For each category, we track both the maximum story value and the 4th worst story value.

Laboratory contents EDPs

We use the maximum and 4th worst peak horizontal floor acceleration to estimate damage to laboratory equipment. Research has shown that peak acceleration is well correlated to damage in both sliding and rocking components.

Exterior Window EDPs

We were able to obtain test data for similar window wall systems from Stanlock, the original neoprene window gasket provider. Based on the data, we measure maximum and 4th worst interstory drifts to predict window cracking or falling in an earthquake.

3.7. Seismic Hazard and Ground Motions for Code Compliance

Our analysis of the Health Sciences Building complex is divided into two components: code compliance and seismic risk assessment. For the code compliance analysis, we derive acceleration response spectra which satisfy the requirements of Chapter 34 of the California Building Code. Because we perform computer response-history analyses, we develop spectrum compatible ground motion records based on the code compliant response spectra.

To consider the effects of local soil conditions, we undertake a geotechnical investigation to identify the soil classification at the site, which affects seismic hazard and ground motion intensity. The code site class variable describes the degree to which local soil conditions may amplify ground motions of different frequencies. For this study, we elected neither to model soil-structure effects nor to explicitly consider the effects of topography on level of shaking beyond the extent that may be implicitly included in the data used to develop ground motion prediction equations. Therefore, the seismic base for the building is assumed to occur at grade level and to a relatively flat site.

Data for the site class investigation is taken from the original boring logs, which are reproduced on sheet F1 of the HSIR foundation drawings. Based on the soil shear strength, we estimate the soil shear wave velocity over the top thirty meters (v_{S30}) to be approximately 1900 ft/sec. According to ASCE 41-06 standard, this site is defined as Class C. This result was used to define the site general response spectrum and site-specific hazard properties.

Site-specific Response Spectra for Code Compliance

We base the site-specific uniform hazard spectra for the UCSF HSIR seismic study on the most recent USGS probabilistic earthquake ground acceleration data. The data are from 2008, when the USGS revised the gridded hazard values to reflect changes in science related to ground shaking, faulting, and seismicity using the Next Generation Attenuation relations (NGA). The data are provided for a grid of latitude-longitude locations across the United States at 0.05° intervals.

Using the 2008 USGS data, we select the four grid points that bound the HSIR site, as shown in Figure 10, and interpolate hazard data to the site location and construct site spectra for a range of return periods, including those required by Chapter 34 of the CBC, 225 and 975 years. We modify the spectra to account for the HSIR Site Class C soil conditions by calculating the effect of shear wave velocity on the spectral ordinates in the NGA relations using a deterministic spectrum. Additional details of this calculation are provided in Appendix G.

The resulting uniform hazard spectra are plotted in Figure 10. These uniform hazard spectra are defined by the fact that each acceleration value on a given curve has the same mean return period of being reached or exceeded at the site.

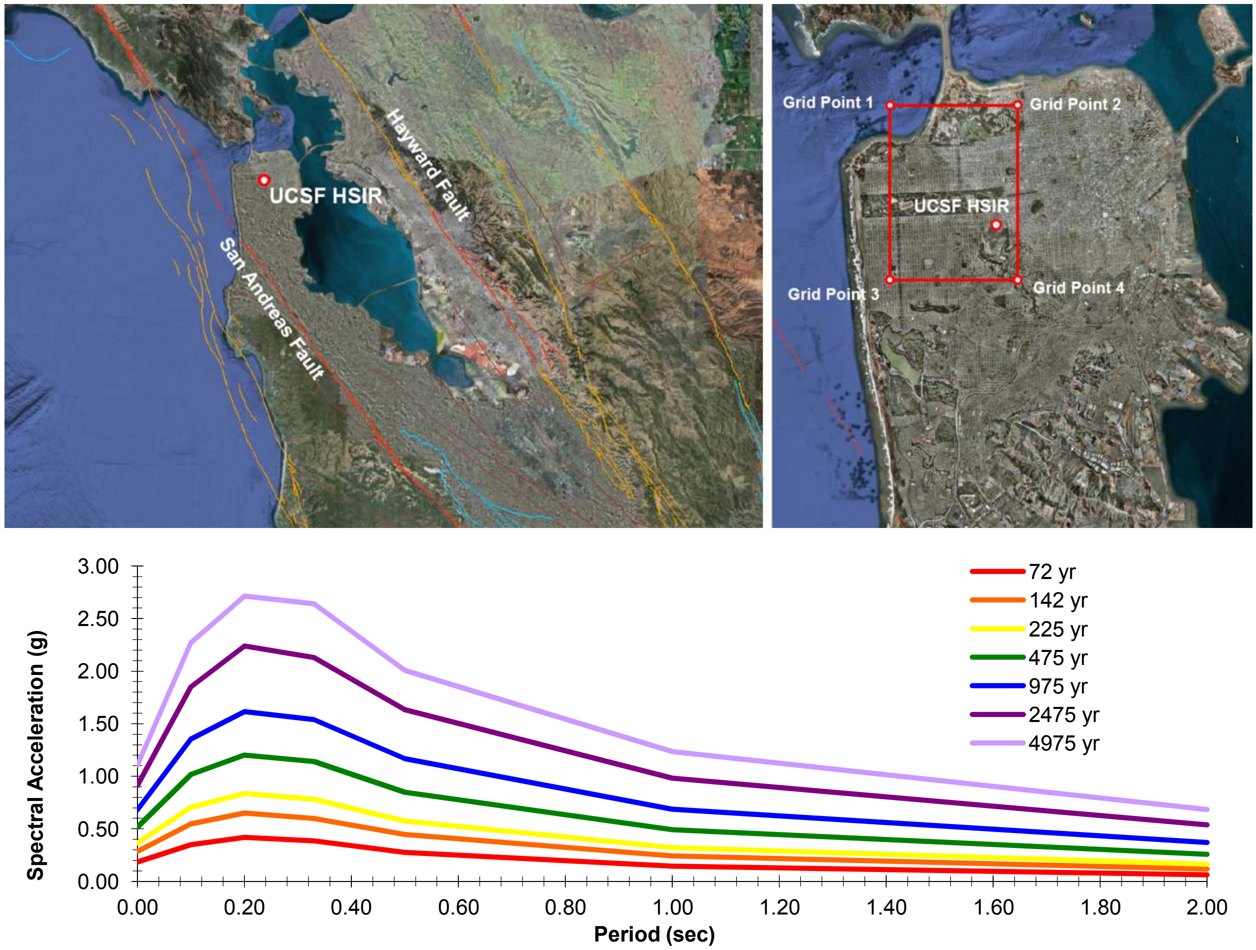


Figure 10: Top row: HSIR site location, © 2009 Google. © 2009 TerraMetrics. Bottom row: HSIR site-specific uniform hazard spectra at several mean return periods

SITE-SPECIFIC RESPONSE SPECTRA FOR CODE COMPLIANCE

We base the site-specific uniform hazard spectra for the UCSF HSIR seismic study on the most recent USGS probabilistic earthquake ground acceleration data. The data are from 2008, when the USGS revised the gridded hazard values to reflect changes in science related to ground shaking, faulting, and seismicity using the Next Generation Attenuation relations (NGA). The data are provided for a grid of latitude-longitude locations across the United States at 0.05° intervals.

Using the 2008 USGS data, we select the four grid points that bound the HSIR site, as shown in Figure 10, and interpolate hazard data to the site location and construct site spectra for a range of return periods, including those required by Chapter 34 of the CBC, 225 and 975 years. We modify the spectra to account for the HSIR Site Class C soil conditions by calculating the effect of shear wave velocity on the spectral ordinates in the NGA relations using a deterministic spectrum. Additional details of this calculation are provided in Appendix G.

The resulting uniform hazard spectra are plotted in Figure 10. These uniform hazard spectra are defined by the fact that each acceleration value on a given curve has the same mean return period of being reached or exceeded at the site.

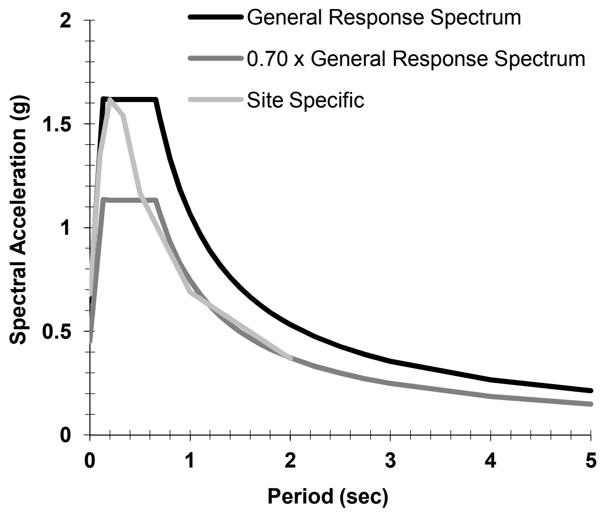


Figure 11: 975 year mean return period General Response Spectrum per ASCE 41-06. Also plotted is the site-specific 975 year response spectrum.

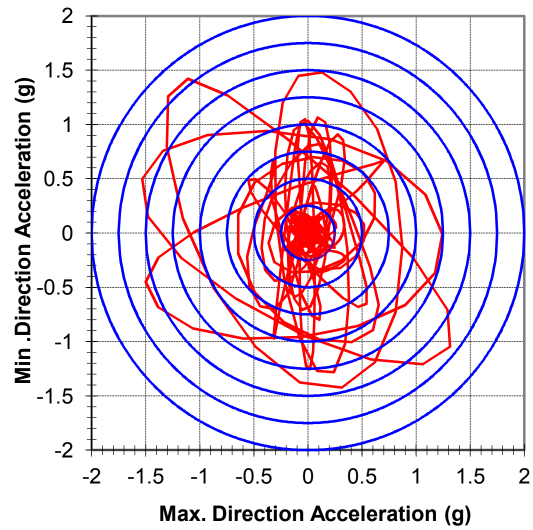


Figure 12: Orbit plot of Friuli, Italy ground motion record, scaled to 975 year mean return period, for period T=0.3 sec.

General Response Spectra for Code Compliance

The general response spectra for the UCSF HSIR study are constructed using the mapped spectral values in the ASCE 7-05 standard, based on the 2002 USGS earthquake hazard data. We construct the general response spectra based on the MCE and 2/3MCE hazard level, which are roughly equivalent to the 975-year return period hazard level and the 475-year return period hazard level, respectively.

A plot showing the general response spectra for the 975-year hazard level is provided in Figure 11. The plot shows the general response spectrum, 70% of the general response spectrum, and the site-specific uniform hazard spectrum. Per the requirement of ASCE 7-06, our criteria 1, code-compliance analyses use the site-specific uniform response spectrum, which is greater than 70% of the general response spectrum at almost all periods.

Earthquake Ground Motions for Code Compliance

Seven two component pairs of time histories are closely matched to the 975-year spectrum using the frequency domain matching procedure included in the program TINKER (Tagasoft, Lafayette CA, 2009). This procedure preserves the phase relationships of the seed records but modifies the Fourier spectra in an iterative fashion until a good match to the target response spectrum is obtained. The general appearance of the acceleration histories is preserved although there is some tendency to add additional cycles in the velocity and displacement histories.

To maintain fault rupture directivity inherent in the original ground motion records, we apply fault normal components of the original ground motion in an orientation normal to the San Andreas Fault. Fault-parallel components of the original ground motion are aligned parallel to the San Andreas Fault. Although rupture directivity is not explicitly considered, some effects are maintained by transferring the original the strike-angle relationship to the San Andreas (meaning the fault normal component of the original record is oriented normal to the San Andreas Fault in the analytical model).

3.8. SEISMIC HAZARD AND GROUND MOTIONS FOR RISK ASSESSMENT

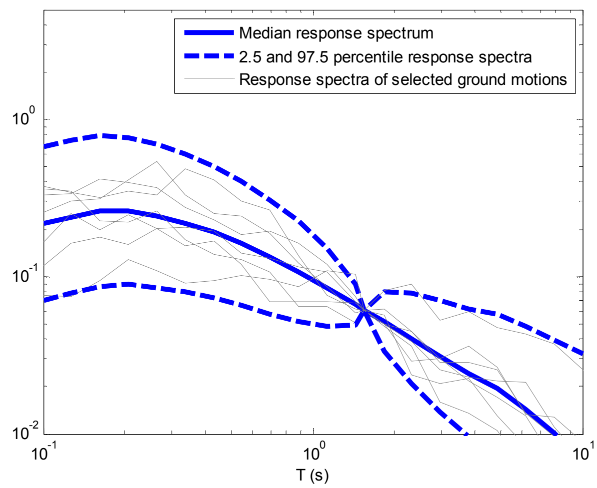


Figure 13: Plot of conditional mean spectrum and response spectra of ground motions selected at the 72 year return period.

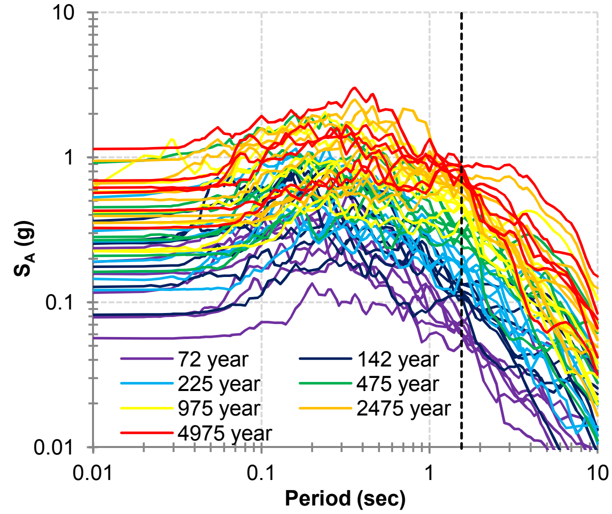


Figure 14: Response spectra of all selected ground motions, colored by mean return period.

The nonlinear analysis for the seismic risk assessment uses unmatched (i.e., amplitude scaled) ground motion records and the Conditional Mean Spectrum, generated at the building first mode period, as our target spectrum. The Conditional Mean Spectrum is an acceleration response spectrum conditioned on the occurrence of a target spectral acceleration at the building first mode period. It provides a more realistic estimate of the expected ground shaking at a site.

While spectrally matched ground motions are well-suited to satisfy the code intent of defining the median structural response, they lack variability in frequency content observed in actual earthquake record response spectra. To be able to predict the occurrence of the full range of possible structural response needed for seismic risk calculations, it is important to include this natural variability. We use the method proposed by Jayaram et al. (2010) to select ground motions considering both the target spectrum mean and variance. The resulting set of records can be considered "dispersion appropriate." We elect to use the horizontal components of the ground motions in our analysis. The building does not contain laboratory contents or equipment that are uniquely vulnerable to vertical excitation and the procedures for accurately modeling vertical earthquake motion are not yet well defined.

The Health Sciences site is located approximately 7 km from the San Andreas fault, and can experience near-fault directivity effects under severe shaking from less frequent ground motions. A commonly accepted approach for estimating the effects of near-fault directivity is based on work by Somerville et al (1997) and Abrahamson (2000). This approach, which is formulated to be included as a part of the seismic hazard analysis procedure includes two terms: the first defining the average increase in spectral acceleration for a site and the second term defining the change in amplitude associated with the fault normal or fault parallel components. It has been shown that when probabilistic spectra are used, the effects of fault directivity computed using this approach are limited, particularly at periods below T=2 sec. (Effects on the deterministic spectrum are typically larger.) Since our analysis of the Health Sciences building uses a probabilistic method and the building period is approximately T=1.55 sec, we do not include the effects of rupture directivity in our analysis.

3.9. BUILDING-SPECIFIC FRAGILITY FUNCTIONS

The seismic risk assessment uses fragility functions (also called fragility curves) to estimate the probability of a specific performance level being reached or exceeded as a function of an engineering demand parameter. A more complete description of the building-specific fragility functions that we develop for this project is presented in Appendix K. Example fragility functions are shown in Figure 15 and Figure 16.

We define fragility functions for six building systems—structure, mechanical and plumbing, electrical, information technology, laboratory contents, and windows—according to the engineering demand parameters that can affect each system, as shown in Table 13. The column of the table labeled existing building shows that we use 38 sets of fragility curves for the risk assessment of the existing building.

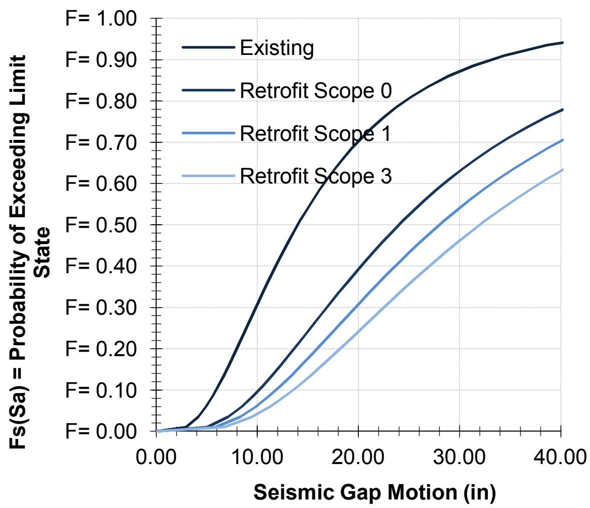


Figure 15: Example mechanical tower joint displacement fragility functions for the *Onset of Life Risk* performance level in the mechanical, plumbing, and fire protection systems.

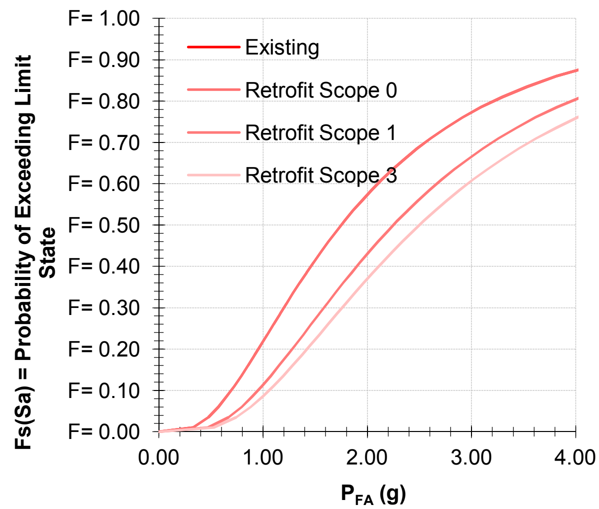


Figure 16: Example floor acceleration fragility functions for the *Onset of Life Risk* performance level in the mechanical, plumbing, and fire protection systems.

As shown in the next column of the table, for Retrofit Scheme 0, we provide some limited retrofit measures for mechanical systems and we therefore create eight revised sets of fragility curves related to the performance of mechanical systems. (The retrofit schemes are described in Section 10)

Likewise for Retrofit Scheme I, we implement anchorage and other retrofit measures for contents, mechanical, electrical, and IT systems, creating 26 revised sets of fragility functions. Retrofit Schemes II and III implement enhanced measures for contents, mechanical, and electrical systems, leading us to create 18 additional sets of fragility curves.

In these cases, the increasing benefit of more extensive retrofitting is reflected in fragility functions that show less damage for the given demand parameter. Increasing darkness of shading in the table indicates providing fragility functions for more extensive retrofit measures, equating to lower likelihood of damage.

By contrast, the benefit of the particular structural retrofit measures recommended for HSIR does not require modifying the structural fragility functions, because the effect of the structural measures is accounted for by revising the nonlinear analysis model. If dampers or column splice retrofitting is added to the analysis model, such measures act to reduce the demand parameters—acceleration, interstory drift, etc.

PERFORMANCE LEVELS AFFECTED

Not all building systems affect all of the six performance levels that we evaluate. Table 14 shows that structural performance affects all six performance levels, therefore requiring a set of six fragility functions for each structural demand parameter. Contents, mechanical, and window performance affect three of the six performance levels—moderate and major disruption, and life-safety. Each of these systems is associated with fragility functions in sets of three.

Electrical system performance affects the two key performance levels of moderate and major disruption. The performance of information technology systems is assumed to affect only the moderate disruption level.

DEVELOPMENT OF FRAGILITY FUNCTIONS

We use an iterative process to define and verify fragility functions. A number of constraints and judgments are logically applied to develop each function, as outlined in Appendix K. Generally we define an index value for the function—often the median value, and then define the variability term β. There are methods and consensus for defining β for certain situations, including that presented by Cornell et al (2006) and Maffei et al (2006). For aleatory variability β_R, we can also gain insight from the dispersion of our analysis results.

All of the systems except the exterior windows are comprised of a number of components or subsystems with unique seismic vulnerabilities. The fragility functions that we develop are based on classes of components and subsystems with similar seismic vulnerabilities. For each subsystem, we create composite fragility functions based on our professional judgment, research findings, detailed comparisons of relative fragility, and verification studies.

Table 13: Required sets of fragility functions. Each shaded cell indicates a distinct set of project-specific fragility functions developed by R&C for the risk assessment. Increasing darkness of shading indicates providing fragility functions for more extensive retrofit measures, equating to less damage for the given EDP. Un-shaded cells indicate fragility functions that are not changed from those applicable to the existing building.

Building System	Engineering Demand Parameter (EDP)	Existing Building	Retrofit Scheme 0	Retrofit Scheme I	Retrofit Scheme II and III
Structure	Max Interstory drift				
	4th Worst Interstory drift				
	Max Residual Drift				
	4th Worst Residual Drift				
	Splice Elongation 7.5				
	Splice Elongation 5.5				
	Residual Splice Elongation 7.5				
	Residual Splice Elongation 5.5				
	Mechanical Tower V2				
	Mechanical Tower V3				
Contents	Max PFA				
	4th Worst PFA				
Mechanical	Max Mech Gap Closing				
	4th Worst Mech Gap Closing				
	Max Elev Gap Closing				
	4th Worst Elev Gap Closing				
	Max PFA				
	4th Worst PFA				
	Max Interstory drift				
	4th Worst Interstory drift				
Electrical	Max Mech Gap Closing				
	4th Worst Mech Gap Closing				
	Max Elev Gap Closing				
	4th Worst Elev Gap Closing				
	Max PFA				
	5th Worst PFA				
	Max Interstory drift				
	5th Worst Interstory drift				
Information Technology	Max Mech Gap Closing				
	4th Worst Mech Gap Closing				
	Max Elev Gap Closing				
	4th Worst Elev Gap Closing				
	Max PFA				
	6th Worst PFA				
	Max Interstory drift				
	6th Worst Interstory drift				
Windows	Max Interstory drift				
	4th Worst Interstory drift				

Table 14: Fragility functions in each set. Each shaded cell indicates a performance level for which R&C develops a fragility function. Only structural performance affects yielding, irreparable damage, and collapse performance levels. Only structural, mechanical, or window performance affects life-safety. Information technology performance can cause, at worst, moderate disruption.

Building System	Engineering Demand Parameter (EDP)	Struct. Yield	Moderate Disruption	Major Disruption	Life Risk	Irreparable Damage	Collapse
Structure	Max Interstory drift	■	■	■	■	■	■
	4th Worst Interstory drift	■	■	■	■	■	■
	Max Residual Drift		■	■		■	
	4th Worst Residual Drift		■	■		■	
	Splice Elongation 7.5	■	■	■	■	■	■
	Splice Elongation 5.5	■	■	■	■	■	■
	Residual Splice Elongation 7.5		■	■		■	
	Residual Splice Elongation 5.5		■	■		■	
	Mechanical Tower V2	■	■	■	■	■	■
	Mechanical Tower V3	■	■	■	■	■	■
Contents	Max PFA		■	■	■		
	4th Worst PFA		■	■	■		
Mechanical	Max Mech Gap Closing		■	■	■		
	4th Worst Mech Gap Closing		■	■	■		
	Max Elev Gap Closing		■	■	■		
	4th Worst Elev Gap Closing		■	■	■		
	Max PFA		■	■	■		
	4th Worst PFA		■	■	■		
	Max Interstory drift		■	■	■		
	4th Worst Interstory drift		■	■	■		
Electrical	Max Mech Gap Closing		■	■			
	4th Worst Mech Gap Closing		■	■			
	Max Elev Gap Closing		■	■			
	4th Worst Elev Gap Closing		■	■			
	Max PFA		■	■			
	5th Worst PFA		■	■			
	Max Interstory drift		■	■			
	5th Worst Interstory drift		■	■			
Information Technology	Max Mech Gap Closing		■				
	4th Worst Mech Gap Closing		■				
	Max Elev Gap Closing		■				
	4th Worst Elev Gap Closing		■				
	Max PFA		■				
	6th Worst PFA		■				
	Max Interstory drift		■				
	6th Worst Interstory drift		■				
Windows	Max Interstory drift		■	■	■		
	4th Worst Interstory drift		■	■	■		

3.10. Loma Prieta Scenario Verification of Existing Building Fragility

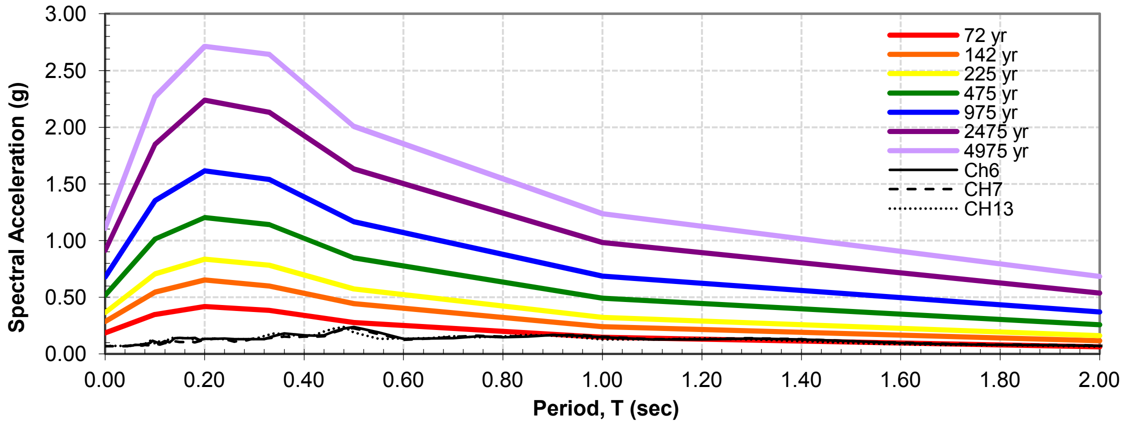


Figure 17: Comparison of recorded Loma Prieta spectra and site-specific uniform hazard spectra at various mean return periods. Channel 13 of the Loma Prieta accelerogram is oriented North/South. Channels 6 and 7 are oriented East/West.

To test the adequacy of the fragility functions that we developed for the existing building, we compare the damage predicted by our fragility functions and risk analysis to actual observations of damage at the Health Sciences complex following the 1989 Loma Prieta earthquake. Our test results predict minimal damage to the building structure, laboratory contents, or building utilities, which matches well with the reported level of damage.

Using the Center for Engineering Strong Motion Data (CESMD) database, we located ground motion accelerograms recorded during the Loma Prieta earthquake from a station near the Health Sciences complex. The station is located at the UCSF School of Nursing, in a reinforced concrete building. The recordings were taken at the level of the slab-on-grade. (There are no free-field records in the vicinity for the Loma Prieta earthquake.)

Figure 17 shows a comparison of the recorded Loma Prieta spectra to the site-specific uniform hazard spectra. At and above the fundamental period of the laboratory buildings (1.55 sec), the spectra are approximately equal to the 72-year mean return period uniform hazard spectra. At periods shorter than 0.8 seconds, the amplitudes of the Loma Prieta spectra are lower than the 72-year spectral acceleration levels. The reduction in high frequency content is consistent with the base slab averaging effect described in the FEMA 440 report (FEMA 440: Improvement of Nonlinear Static Seismic Analysis Procedures, 2005). The implication of the results, that the first mode spectral accelerations caused by the Loma Prieta earthquake can be expected to be reached or exceeded at the site about every 72 years, is consistent with our understanding of the consensus judgment of seismologists.

We take the Loma Prieta ground motion record and use it as input to our nonlinear analysis model of the structure, recreating (in a virtual sense) the effects of the Loma Prieta earthquake on the buildings. From the output, we extract results for each of the EDPs used in our probabilistic risk assessment.

Figure 18 and Figure 19 compare the resulting interstory drift and peak horizontal floor acceleration response. The response values for interstory drift are roughly equivalent to the median response from the set of ground motions selected to match the 72-year mean return period spectrum. However, peak horizontal floor acceleration values, which are strongly influenced by higher mode effects, are reduced.

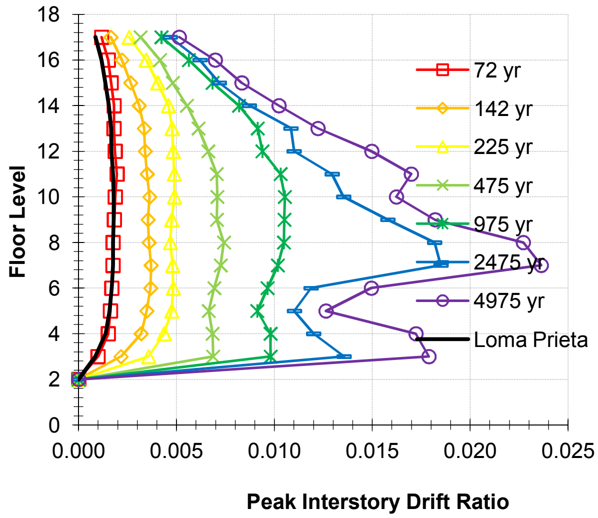


Figure 18: Comparison of peak interstory drift ratios caused by the ground motions used in the probabilistic analysis and the Loma Prieta record.

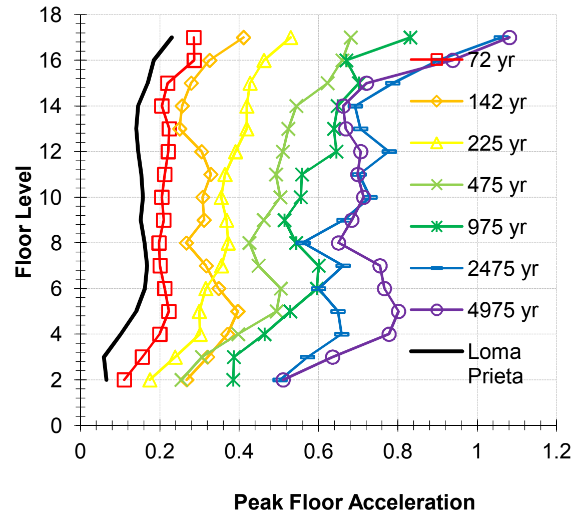


Figure 19: Comparison of peak horizontal floor acceleration response caused by the ground motions used in the probabilistic analysis and the Loma Prieta record.

Using the EDP output and the existing building fragility functions, we are able to compute the probability that the building would exceed specific performance states in the Loma Prieta scenario. The results show that the building was unlikely to experience disruption from damage to the building structure, mechanical systems, laboratory contents, or windows. Detailed results are presented in Appendix K. Of the EDPs that we use, our models predict that the greatest risk of disruption in the Loma Prieta ground shaking would come from seismic gap closing. Accounts of damage to the buildings following Loma Prieta confirm that no significant damage occurred.

According to our team's consultants who have a long experience with the buildings, the Loma Prieta earthquake did not result in a lasting disruption to the laboratories. Electrical service was not disrupted and major mechanical systems functioned in an uninterrupted fashion. It was reported that some of the exterior fume exhaust risers suffered minor damage as a result of the earthquake. Jeff Roloff, the ThyssenKrup elevator representative for the campus reports that the Health Sciences elevators were undamaged. These observations validate the assumptions used in our fragility functions at lower levels of shaking.

3.11. Probabilistic Risk Assessment Methodology

To determine the probability that the buildings enter or exceed the performance states of interest in a given period of time, we carry out an explicit seismic risk assessment considering the full range of possible earthquake ground motion intensity. For this task, we use a customized risk-assessment computational platform developed specifically for this project. The methodology is presented in detail in Appendix L.

As a part of this risk assessment, we use the results of our nonlinear structural analysis to predict damage to each building system. From our set of 49 earthquake record runs—seven levels of shaking measured in terms of spectral acceleration at the fundamental period of vibration of the buildings with seven ground-motion pairs[1] at each level—we record the result for each of the 16 engineering demand parameters (EDPs) that we use. Each EDP at each level of shaking is considered to be lognormally distributed random variable[2] with mean and standard deviation estimated from the results of the runs. Hence, the set of random variables comprising the natural logarithm of each one of the 16 EDPs is jointly normally distributed. Given this assumption, the joint distribution of the 16 EDPs is fully known once the correlation structure among all the EDPs is known. The correlation structure of all the EDPs is empirically estimated based on the approach outlined in the following section.

Statistical Correlation of Demand Parameters

To establish the likelihood that various potential types of damage may occur in a given period of time, we need to estimate the correlation (or the covariance) matrix of the 16 EDPs. Different types of damage are better correlated with different EDPs. For example, the HSIR laboratory contents are vulnerable to floor acceleration, while certain mechanical systems are vulnerable to separation joint closing. The two different types of damage are better correlated to different response measures, but these measures are not completely independent from each other. Both floor acceleration and separation joint closing will increase with the intensity of shaking, however some particular ground motions will affect one parameter more than the other.

Figure 20 shows a graph with correlation coefficients of the 16 EDPs. The areas of the graph shown in red and yellow indicate the parameter combinations that are most highly correlated. In general similar types of EDPs tend to be well correlated—for example, the separation joint closing (mechanical gap) EDPs are well correlated to interstory drift. The areas of the graph shown in purple indicate the lowest amount of correlation. Many of these occur for measures of column splice elongation that remain zero until the column yields in tension, which only happens under the action of the most severe ground motions.

Although 49 nonlinear analysis runs is more that is normally carried out in a seismic evaluation, it is not fully sufficient to provide all of the details of the response measure correlation structure that might be desired. To use more ground motions, run more analyses, and determine different sets of correlation coefficients at different intensity levels of shaking. For example, our current results show that maximum interstory drift is 99% to 100% correlated to 4th worst interstory drift. One might expect that if only larger shaking intensities are considered, interstory drift could concentrate at a level, and the two parameters would become less correlated. However, this approach would require carrying out a number of additional dynamic analyses way beyond the resources available and, therefore, for our analysis (per the

[1] Note that only the two horizontal components of the ground motion records were used as input to the structural response analyses. The vertical component, which does not affect considerably the performance of the structure, was not utilized.
[2] This assumption is, in general, tenable for most of the EDPs.

recommendation of our expert sub-consultant) we computed one set of correlation coefficients using all 49 analyses. This solution provides good statistical significance while keeping the number on analysis runs reasonable.

SIMULATION OF PROJECT-SPECIFIC ENGINEERING DEMAND PARAMETERS

The inherent random nature of earthquake occurrence and the variability in the specific characteristics of the ground motions generated by each earthquake at the site mean that it is impossible to model with certainty the response of the Health Sciences buildings to future seismic loads. Hence, to probabilistically estimate the building response we adopted the following approach:

From our set of 49 nonlinear analyses, we use the mean, variance, and covariance of the resulting EDPs to simulate the outcomes of many possible future earthquakes. Given that a full integration of the joint normal distribution necessary for the close-form computation of the probability of different damage states is computationally unmanageable, we use the Monte Carlo simulation technique to generate these additional outcomes.

Figure 21 shows how the simulated outcomes, shown with red points in the figure, compare with the outcomes from each of our 49 response-history analysis runs, shown with blue points. The set of simulated outcomes is produced to have the same statistical properties as the set of analysis outcomes—thus the two sets of points show a similar pattern of distribution of on a plot like the one shown. Plots like this also provide a check of correlations, and can indicate if the correlation tends to depend on intensity. Figure 22 shows the joint probability distribution of two EDPs based on the correlation data.

Given the simulated response outcomes and fragility functions, we identify the expected damage in each of the simulated cases. Computationally, for each ground motion level defined in terms of spectral acceleration at the fundamental period of the building of the ground motion input records, we calculate the number of realizations which result in each performance state, both as a result of damage to individual systems and as an overall result. The long-term probability of the building exceeding any one of such damage states can be obtained by convolving the cumulative results with the ground motion hazard curve.

More precisely, we present the results of our analysis as the probability of reaching or exceeding the project-specific seismic performance states defined in Section 3.2. Since the probabilities of exceeding the states are low in a single year, we express data over a 30 year period, which equates to a typical long-term planning horizon.

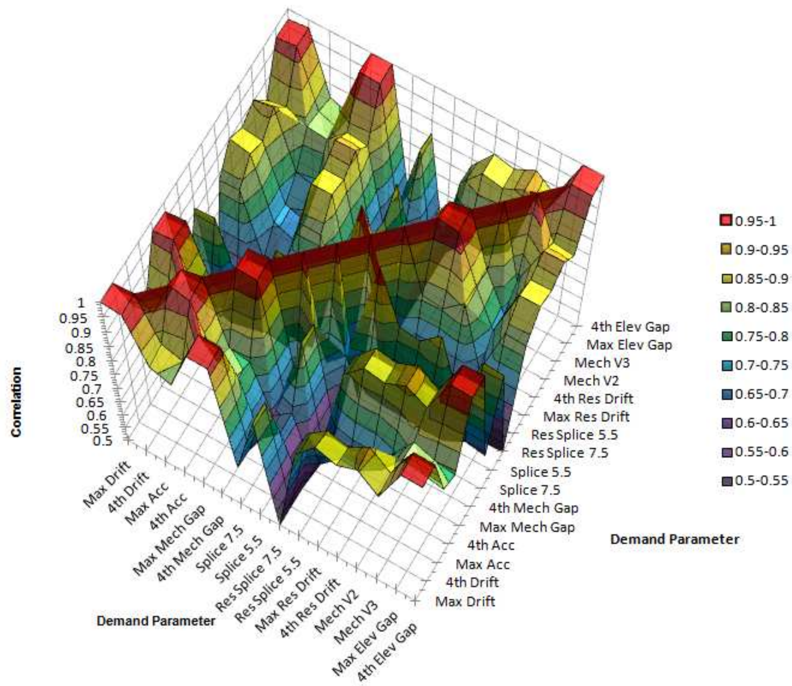


Figure 20: Surface plot of correlation coefficients for all EDPs in the existing building extracted from the empirical data obtained from response-history analyses using 49 ground motion records.

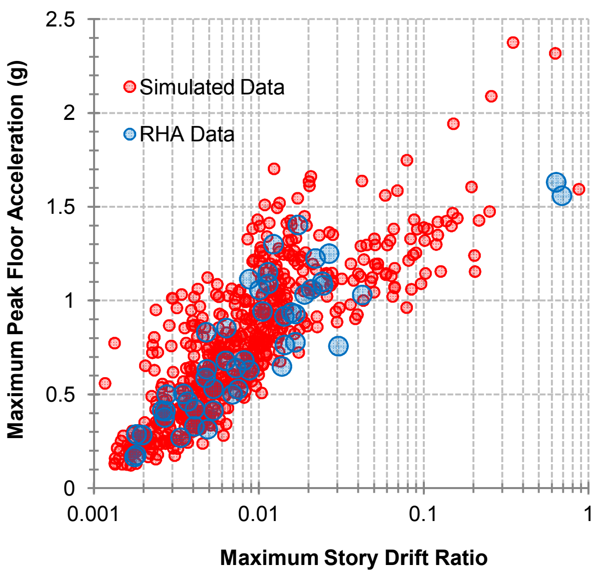


Figure 21: Plot of maximum interstory drift ratio and maximum peak floor acceleration data from response-history analysis and Monte Carlo simulation. The points at the far right represent structural collapse caused by large earthquake ground motions.

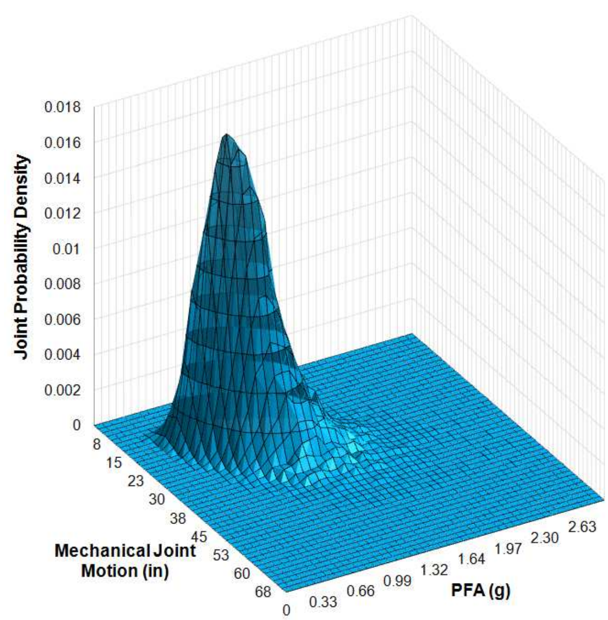


Figure 22: Joint lognormal probability density function of maximum mechanical tower separation joint closing and maximum peak horizontal floor acceleration (correlation coefficient, r=0.82).

3.12. Structural Material Properties

Properties of structural materials used in the building analysis are shown in Table 15. To determine the materials, we reviewed the original project specifications, retrieved from the UCSF archive. Expected properties are computed based on information determined in previous evaluations of buildings from the same era or from code-referenced standards, such as ASCE 41-06.

Table 15: Material properties.

Material	Description	Expected Strength	Basis
Reinforced Concrete Materials			
Reinforcing Steel	Intermediate grade, deformed	$f_y = 47$ ksi	ASCE 41-06, other buildings from same era
Normalweight Concrete	3000 psi nominal	$f'_c = 4500$ psi	ASCE 41-06, other buildings from same era (Weight = 150b/ft^3)
Lightweight Concrete	3000 psi nominal	$f'_c = 4500$ psi	ASCE 41-06, other buildings from same era (Weight = 115/ft^3)
Structural Steel Materials			
All structural steel, except as noted below	A7	$F_y = 33$ ksi $F_u = 61$ ksi	Project specifications, ASCE 41-06, AISC MSC 6th ed.
Column Plates	A373	$F_y = 32$ ksi $F_u = 58$ ksi	Project specifications, AISC MSC 6th ed.
Plates > 1/2" thickness to be welded	A373	$F_y = 32$ ksi $F_u = 58$ ksi	Project specifications, AISC MSC 6th ed.
Rivets	A195	$F_{y\text{-}T} = 78$ ksi $F_{u\text{-}T} = 39$ ksi $F_{y\text{-}V} = 24$ ksi $F_{u\text{-}V} = 59$ ksi	Project specifications, ASTM Standard, *Guide to Design Criteria for Bolted and Riveted Joints*
Welding Electrode	A233	$F_u = 60$ ksi	Project specifications, ASCE 41-06, AISC MSC 6th ed.
High Strength Bolts	A325 HSB	$F_{N\text{-}T} = 90$ ksi $F_{N\text{-}V} = 48$ ksi	Project specifications, AISC MSC 13th ed.

4. BUILDING STRUCTURE

4.1. DESCRIPTION OF EXISTING STRUCTURES

Laboratory Buildings

The two laboratory towers, designated Health Sciences East (HSE) and Health Sciences West (HSW), are identical in plan except that they are rotated 90° relative to one another. The typical floor plate of each building covers approximately 12,900 ft^2, with 10,800 ft^2 enclosed by window glazing at typical floors. Due to the sloped nature of the site, the two buildings are founded at different elevations. The HSE building slab-on-grade, slightly downhill from its counterpart, is located at an elevation of 397'-6", at the 1st Floor Level. Just uphill, the HSW building slab-on-grade occurs at 410'-2", the 2nd Floor Level. In each building, the lowest level is largely below grade with concrete walls extending up one floor to the steel frame system. Typical floor-to-floor heights in each building are 13'-0" over the steel-framed levels. Since both towers share a common Main Roof elevation of 606'-0", the HSE building, at 208'-6", is one story taller than the HSW.

The two laboratory buildings share similar foundation systems. At each building, clay and sand overlay steeply sloping sandstone bedrock. At locations where the bedrock is just below the slab-on-grade, columns are supported on shallow spread-footings with rock-anchors extending into the sandstone to resist overturning forces. More commonly, piers occurring on the building grids, are drilled through the sand and clay and belled in the sandstone. Because the bedrock slopes steeply, pier lengths vary from approximately 25 ft to 65 ft. Reinforced concrete walls, approximately 2'-0" thick span between the tops of the piers beneath the slab-on-grade.

The grid system for the HSE lab building is shown in Figure 25. Although the framing for the HSW building is rotated 90° in the clockwise direction, it is otherwise identical. The grid system is doubly symmetric, with three bays along each axis. The middle bay column spacing is 33'-4", with exterior bay spans at 30'-1½". Columns are located only at the perimeter grids leaving a large, uninterrupted floor plate. Interior columns are oriented so that their strong-axis is supporting a long-span girder, with the weak-axis connecting a shorter girder to an adjacent perimeter column. The strong-axis of the four corner columns in the HSE building is oriented for bending in the North/South direction.

The seismic lateral force-resisting system for the laboratory buildings is a steel moment frame. The column sections are built-up, shop-riveted I-sections consisting of a web plate (from 3/4" to 1½" thick), two flange plates (1" to 3¾" thick), and four angles (L5x5x1/2" to L6x6x3/4") tying the web and flange together. At interior columns, the gross section dimensions vary based on the flange plate thickness but are approximately 3'-0" x 3'-0". Corner column dimensions vary similarly, at approximately 2'-4" x 2'-4". Column splices consist of bolted web and flange plates.

At the building perimeter, girders are 3'-6" deep I-sections at all levels, built-up of continuously fillet welded web and flange plates. The girder strength is varied over the height of the frame by varying the flange thickness. Interior girders, spanning 93'-7", are identical at all stories, consisting of welded, built-up I-sections 3'-6" deep. At the interior grid intersections, girders running East/West in the HSE building are continuous. Interrupted North/South girders have bolted flange plate moment connections. Since the building story height is limited to 13'-0" and the long interior spans necessitate deep girders there is insufficient overhead space to locate the utilities required to service the laboratories. To fit the utilities, the girders have numerous large web penetrations to permit ductwork and piping in the space above the ceiling. Continuous diagonal web stiffeners are used to transfer shear along the length of beams.

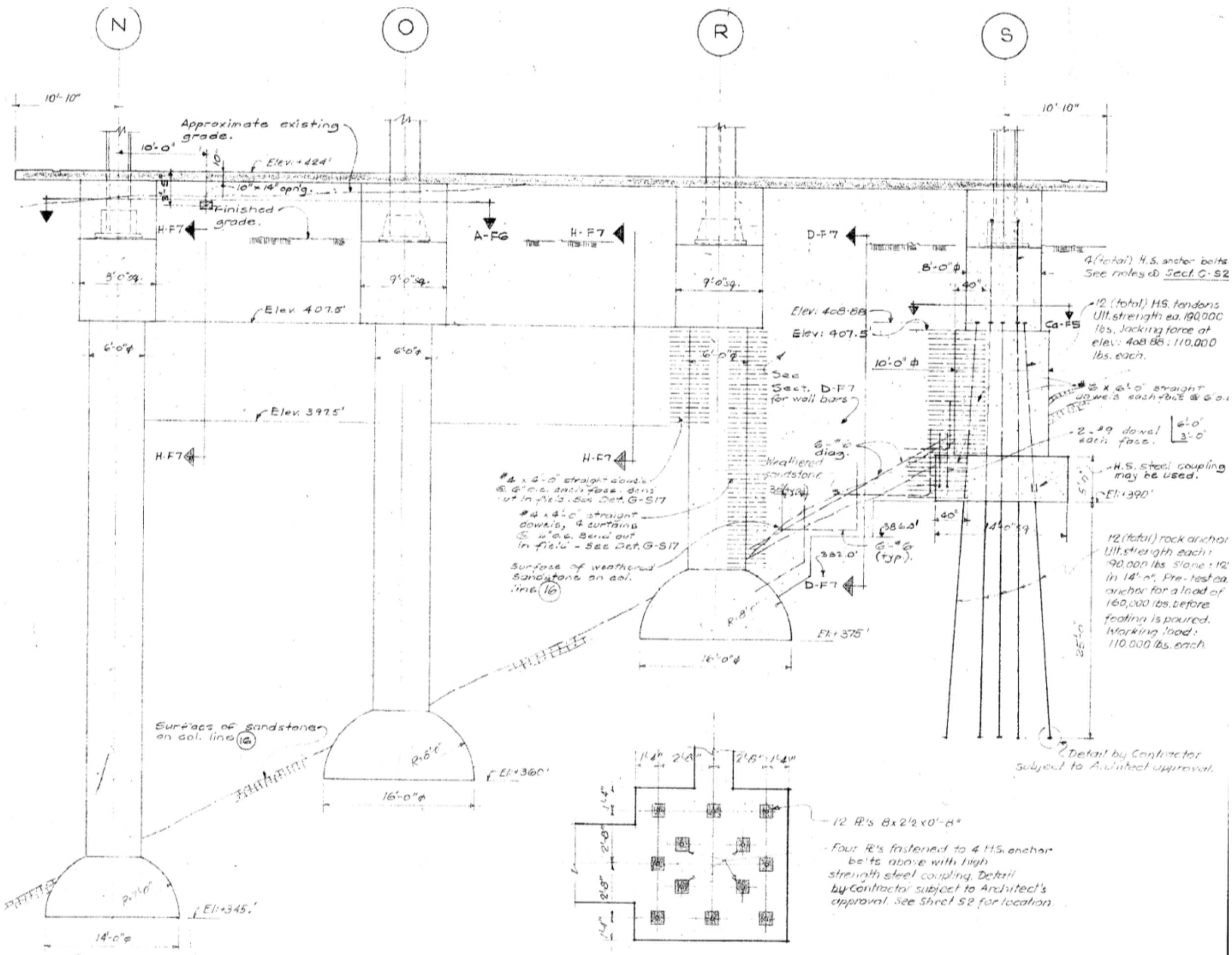


Figure 23: Typical foundation elevation showing belled piers and shallow footing with rock anchors into sandstone bedrock.

The laboratory buildings use two distinct types of beam-column moment connections. In the column strong direction, the connection consists of a pair of WT-sections (cut from a W36x300), flange bolted to the column. The WT web is bolted to the beam flanges. A smaller WT shear tab is bolted to the beam web. In the column weak direction, continuity plates are complete penetration welded to the column flanges and web. These continuity plates extend beyond the column and are bolted to the beam flanges. A shear tab is bolted to the beam web and to a shear plate welded to the column web and continuity plates.

The remainder of the gravity resisting system includes W24 (x100, x110) beams which span between interior girders. There is one W24 beam per bay. W10 purlins span between W24 beams, spaced at approximately 6’-6” on center. The slab cantilevers 10’ from the perimeter frame on each face to create a perimeter walkway to achieve the total 116’-4” floor plate dimension. The perimeter spandrel is a W18x50.

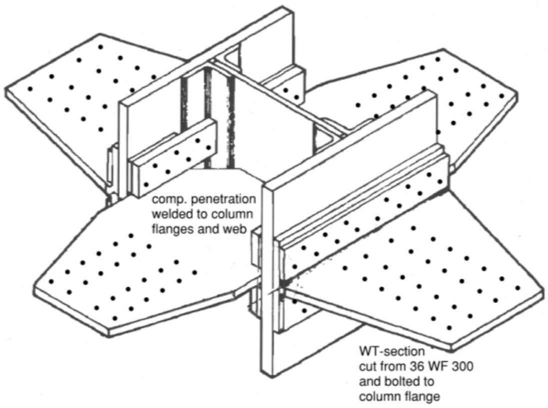


Figure 24: Typical beam-column connection. From Rea et al. (1966)

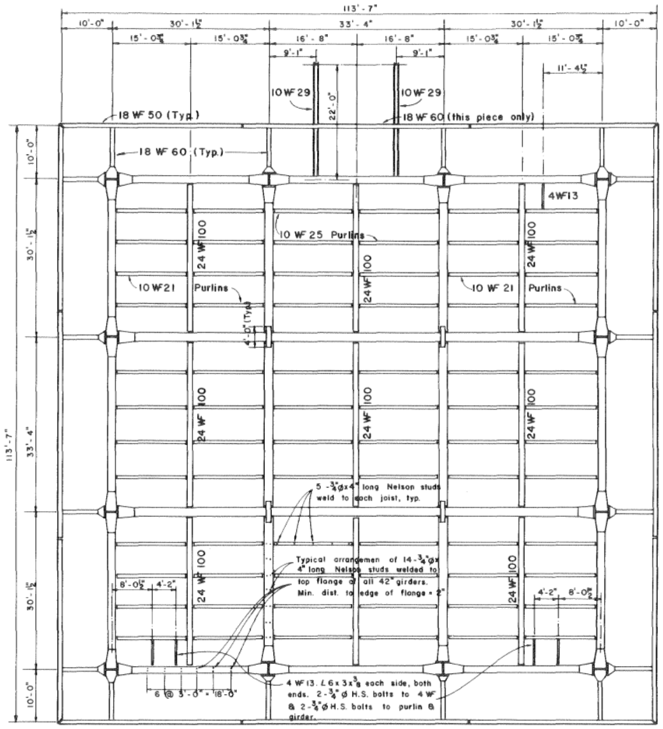


Figure 25: Typical laboratory floor framing. From Rea et al. (1966)

At typical steel framed levels, headed steel studs are present on beams and girders in sufficient quantity to develop partial composite action in the floor slab. Slabs are 5½" thick, of lightweight concrete, without metal decking. Although there are a few minor slab penetrations are the perimeter of the slab, there are no interior shafts for ductwork or plumbing.

Mechanical and Elevator Towers

Designers were able to forgo slab openings because utility services are provided at each level by the mechanical towers. Each laboratory building has its own mechanical tower, with which it shares a common structural base elevation. At the HSE building, the mechanical tower is situated to the east of the laboratory building, with a slab-on-grade occurring at 397'-6". The mechanical towers extend three stories above the main roof level to 636'-0". The lowest two levels of each tower function as an air intake. Reinforced concrete retaining walls and roof slab form a horizontal tube structure extending north and south from the vertical portion of the tower. Air intake occurs at each end of the horizontal tube, and the air is forced into ducts running up the tower to supply each laboratory floor. Like the laboratory buildings, the mechanical towers are founded on belled piers embedded into the sandstone bedrock.

Each mechanical tower has three bays. At the HSE tower, the North/South grid spacing is 12'-4" – 10'-8" – 12'-4". East/West grids are spaced at 20'-0". At typical levels, the two exterior bays have no floor slab and function as duct and utility riser space. The center bay contains a stairwell and partial slab. The structural system for the mechanical towers is made up of concentric steel braced frames embedded in 8" thick reinforced concrete walls. In the short direction of the tower plan, there are four walls and four corresponding frame bays. In the long direction of the tower plan, each tower has two walls and two chevron frame bays. The steel frames are made up of W14 columns, with double channel beams and double channel braces.

An elevator tower and connecting corridor links the HSE building with the HSW building via a small addition to the southwest corner of the Medical Sciences Building. The elevator tower seismic force resisting system is similar to the one used in the mechanical towers. Steel concentric braced frames, consisting of W14 columns with double channel beams and braces are embedded in reinforced concrete walls. In the case of the elevator tower, there are two walls in each direction. Two additional interior reinforced walls running North/South lack braced frames, but have internal steel gravity framing as well as several punched openings to provide elevator access at each floor. The elevator tower base occurs at 400’-6”, Level 1, with the roof level at 626’-0”. Beneath steel columns embedded in interior concrete walls, drilled piers extend to the sandstone bedrock. Exterior steel braced frame columns are supported on drilled piers that are belled 16’-0” into bedrock. Columns baseplates are tied to the foundation using post-tensioned rock-anchors embedded into the sandstone.

The north part of the elevator tower is a connecting corridor, which is joined to the HSW building and the Medical Sciences Building addition.

HEALTH SCIENCES EAST ROOFTOP EQUIPMENT BRIDGE

A unique feature of the Health Sciences East Building is the rooftop electrical system. The HSE roof houses several pieces electrical switchgear, a SCADA distributed processing unit, an emergency generator, and a motor control center (MCC), with a total weight of 150,000 lb. The electrical components are supported on an equipment bridge which spans 93'-7" between the two interior columns on Line C and the two interior columns on Line F. In the long direction, the equipment bridge consists of two parallel double angle trusses. Framing in the short direction is W18 beams. W6 filler beams support the equipment on a 1½" pultruded fiberglass grating. In-plane stiffness is provided by horizontal WT braces in the plane of the deck system.

The bridge was originally constructed in 1992, and underwent a structural retrofit in 2008 as part of a project to modernize the high voltage switchgear. In that retrofit, the equipment bridge was repainted and plain high strength bolts were replaced with galvanized high strength bolts in an effort to minimize corrosion.

Figure 26: HSE roof equipment bridge and equipment.

Figure 27: HSE roof equipment bridge column support.

4.2. LABORATORY BUILDINGS

COMPONENT CAPACITY AND GOVERNING BEHAVIOR MODES

To determine the analytical properties of the structural members of the perimeter moment frame system, for the subsequent dynamic analyses, R&C calculates strength and deformation capacities for the critical components and the connections between these components. Expected material properties (as shown in Table 15) are used for these calculations. The critical elements of the structural force resisting system include:

a) Corner and intermediate built-up columns.
b) Exterior and interior girders.
c) Corner column strong-axis connections.
d) Corner column weak-axis connections.
e) Intermediate column strong-axis connections.
f) Intermediate column weak-axis connections.
g) Panel zones in corner column strong-axis direction.

a) Corner and intermediate built-up columns:

For the columns we calculate axial force capacities in compression and tension for both the built-up columns and the column splices that are located at mid-story height every other story. The axial strength and axial loads for three different expected demand conditions (gravity loads only, gravity + MCE seismic overturning forces, gravity - MCE seismic overturning forces) are depicted in Figure 28 and Figure 29. The calculations confirm that both the corner columns and the intermediate columns have sufficient strength to resist axial compression loads for the three expected demand conditions. However, splice tension capacities for the corner columns are insufficient to resist elastically the tension loads that develop under combined gravity and seismic loading conditions. Hence, for large earthquake loads the corner splices at level 5.5 (and potentially at level 7.5) are expected to yield in tension. This deficiency is further evaluated in our overall analysis and assessment. The flexural strength of the steel columns is dependent on the level of axial force. Because of this and the fact that the steel column cross sections only change every two stories, the column available flexural strength given the level of axial force under MCE seismic demands is smaller in even stories than in odd stories (see blue line in Figure 30). The flange plates of the built-up columns are shop-riveted to the web plate with continuous double angles. However, the 1-inch rivets spaced nominally in the column clear spans between floor levels, do not have enough strength to develop the plastic capacity of the flange plates and double angles. Based on research findings on riveted connections, we conclude that this is an acceptably ductile mechanism, and we account for the limited shear flow capacity of the rivets by adjusting the flexural strength of the corner and intermediate columns accordingly. Figure 30 and Figure 31 show the column strong-axis strength and Figure 32 and Figure 33 depict the column weak-axis strength. Flexural strengths for the three demand levels are also shown.

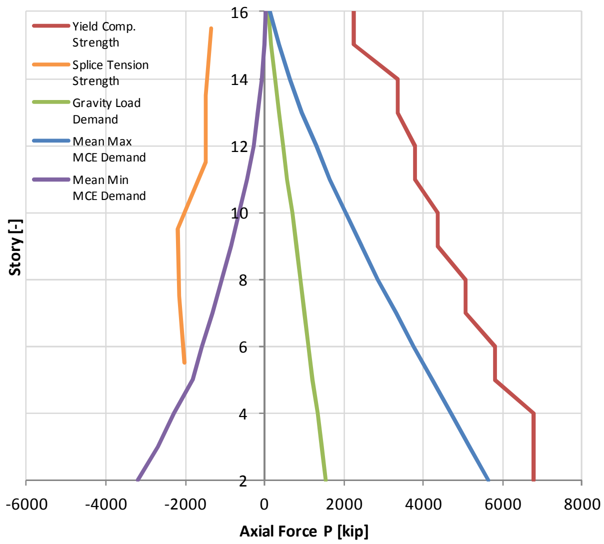


Figure 28: Corner column and corner column splice axial strength compared to MCE demands.

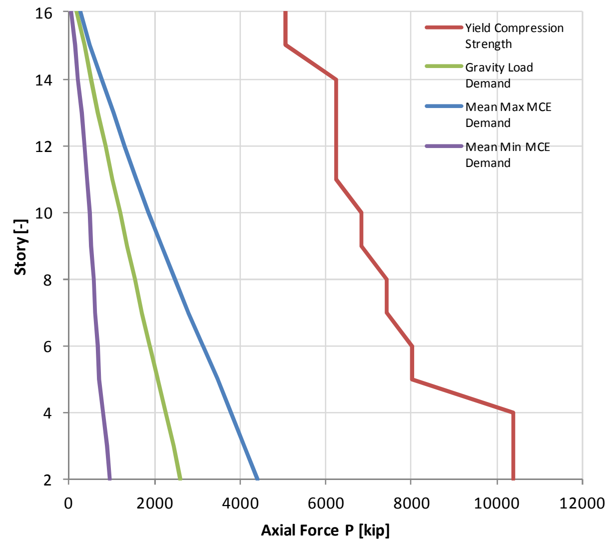


Figure 29: Intermediate column axial strength compared to MCE demands.

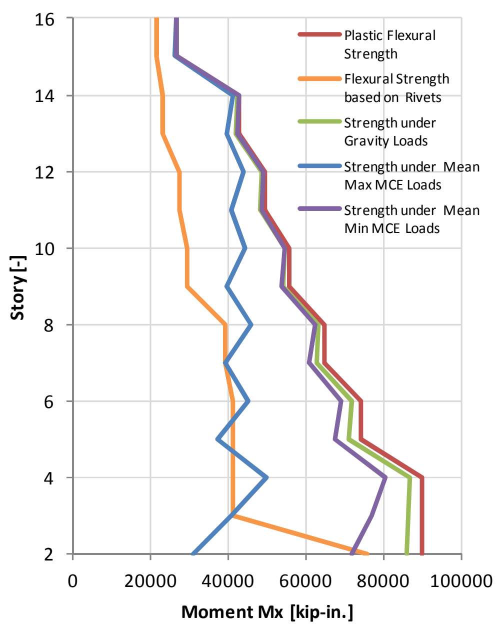


Figure 30: Corner column available strong-axis flexural strength under given axial forces.

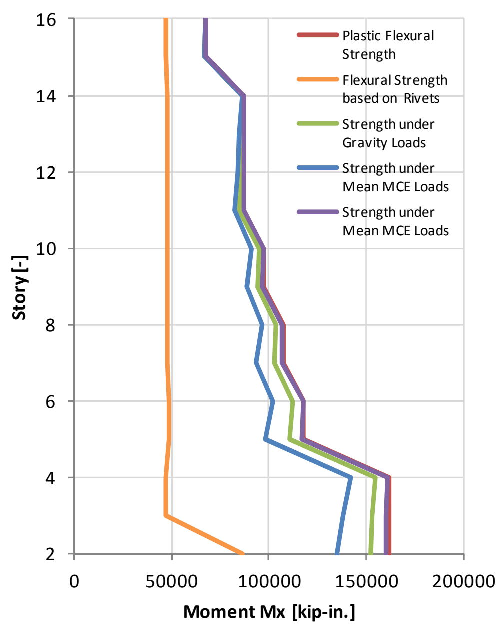


Figure 31: Intermediate column available strong-axis flexural strength under given axial forces.

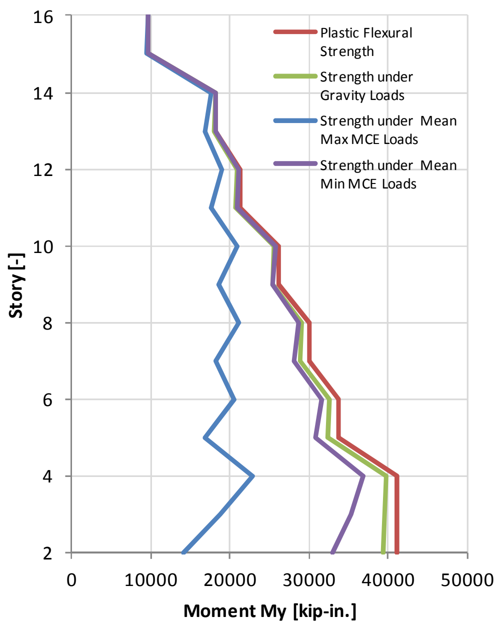


Figure 32: Corner column available weak-axis flexural strength under given axial forces.

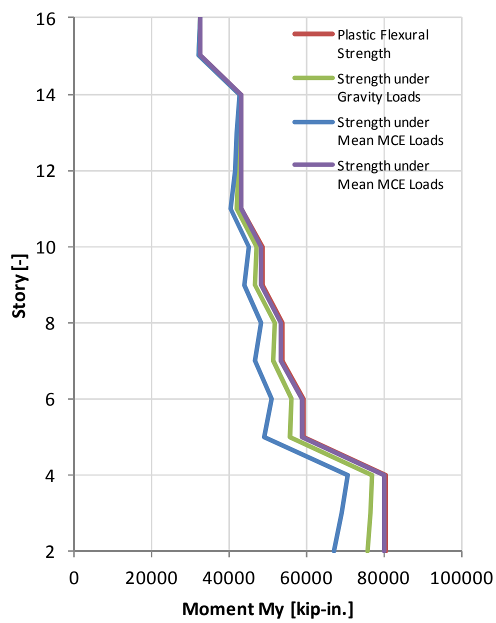


Figure 33: Intermediate column available weak-axis flexural strength under given axial forces.

b) Exterior and interior girders:

For the girders we compute the flexural strength for the steel members using hand/spreadsheet calculations. We determine the full composite negative and positive moment strengths and moment-curvature relationships from fiber models with an effective concrete slab width of 80 inches. After calculating the capacity of the shear studs, we determine that the girders are acting as partially composite limiting the positive (slab in compression) moment strength. Figure 34 shows a comparison of the full composite and partial composite moment strength of the girders. The negative moment strength is not affected by the limited shear flow of the studs. The positive and negative strengths of the girder WT-connections, which are discussed below are also depicted in Figure 34. From this evaluation of composite action, Figure 35 and Figure 36 illustrate the governing girder-end strengths along the north-south and the east-west moment frames.

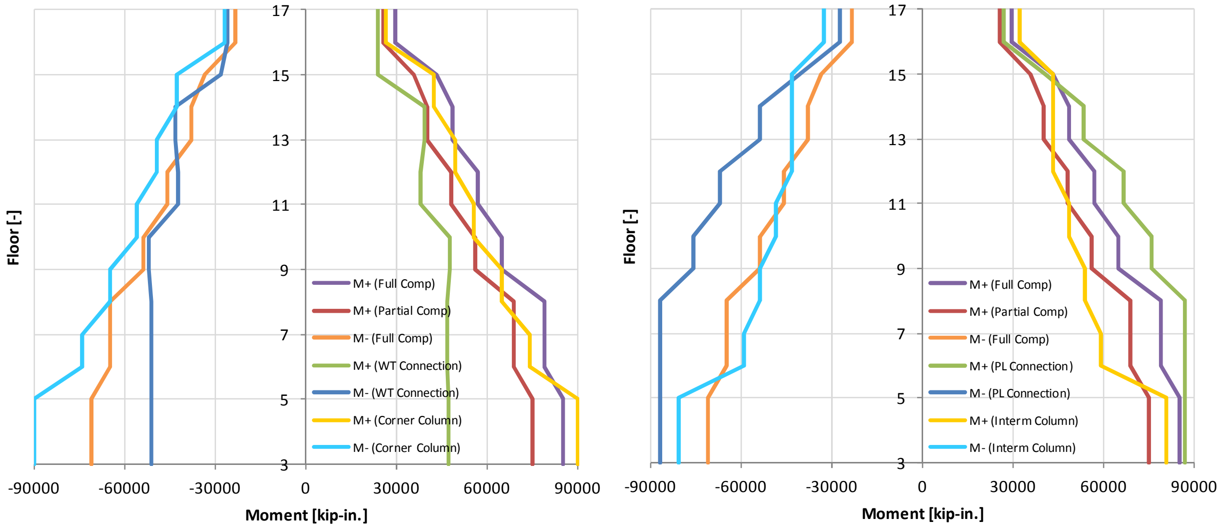


Figure 34: Girder flexural strengths for strong axis (left) and weak axis (right) connections.

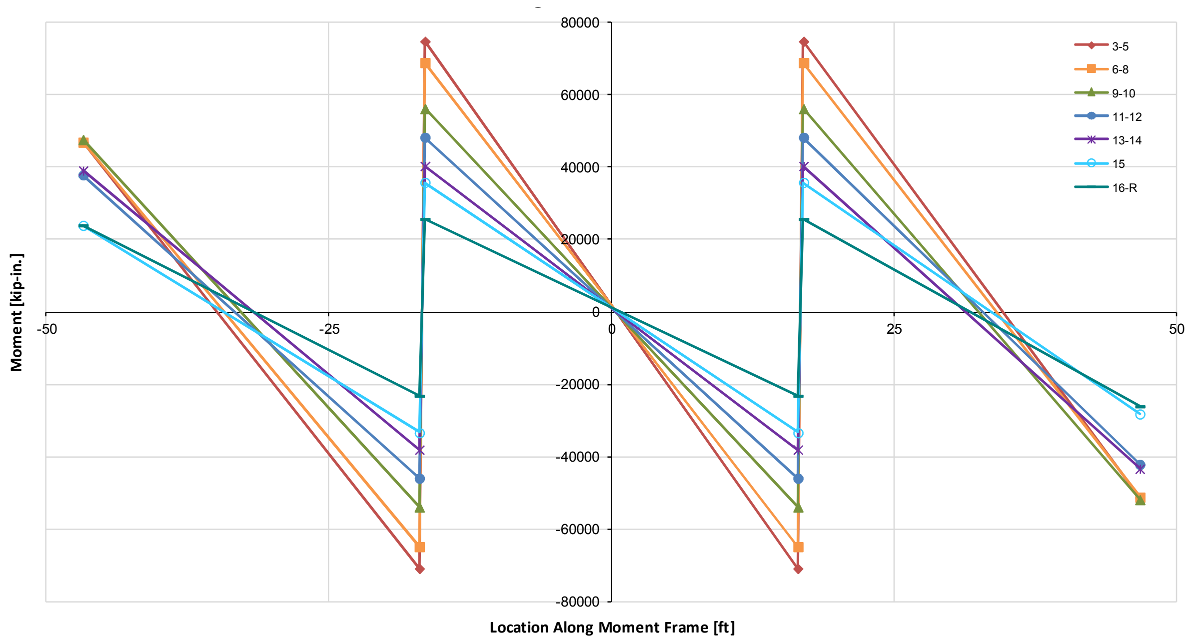


Figure 35: Girder-end moment strength along north-south perimeter frame. The colored lines illustrate the girder-end capacities at different floor levels (girder groups) over the building height.

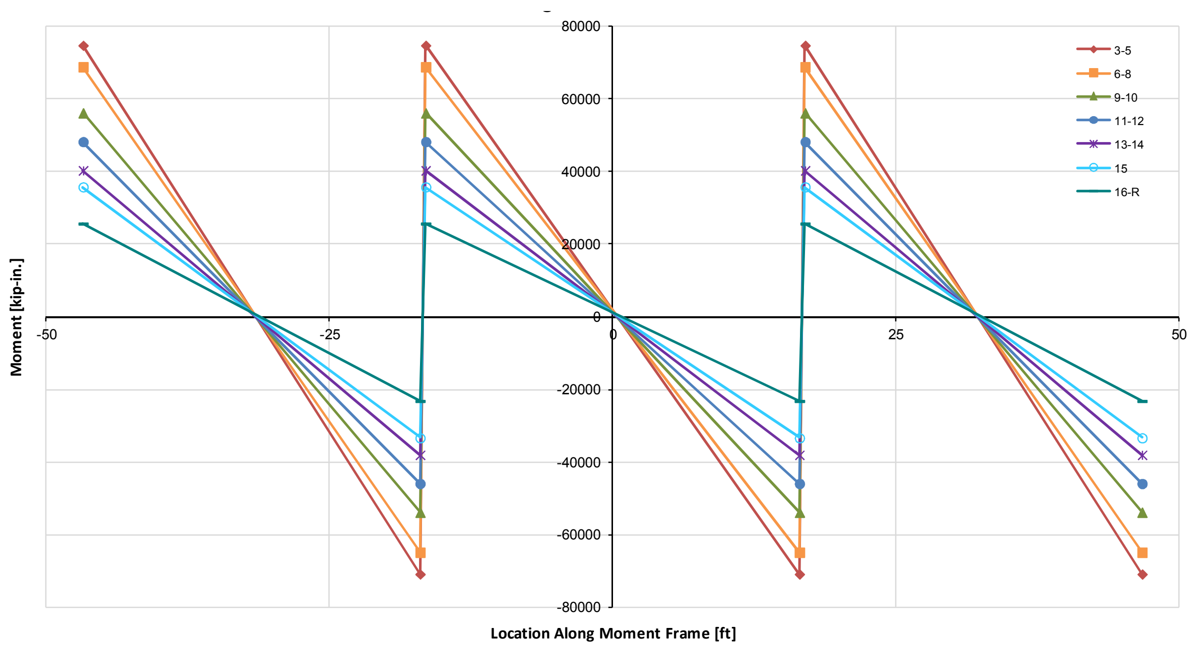


Figure 36: Girder-end moment strength along east-west perimeter frame. The colored lines illustrate the girder-end capacities at different floor levels (girder groups) over the building height.

c) *Girder connection to strong axis of corner column:*

The girder flange connection to the corner column strong-axis consists of short horizontally oriented WT-sections (cut from 36WF300 to 36WF170) that are bolted to the column and girder flanges with 1.25-inch diameter high strength bolts. The WT-section, the number of WT-flange bolts, and the number of WT-stem bolts reduce moving up the building. We investigate several potential mechanisms, which are summarized in Table 16.

Table 16: Summary of potential mechanisms for girder flange connection to strong axis of column.

Component	Yield or Failure Mechanism
WT-Stem	- gross section yield of the stem - net section fracture of the stem - block shear failure of the stem - buckling of the stem
WT-Stem Bolts	- bolt slip - bolt shear failure - bearing failure at stem bolt holes
WT-Flange	- plastic mechanism in flange - shear mechanism in flange
WT-Flange Bolts	- bolt tension failure

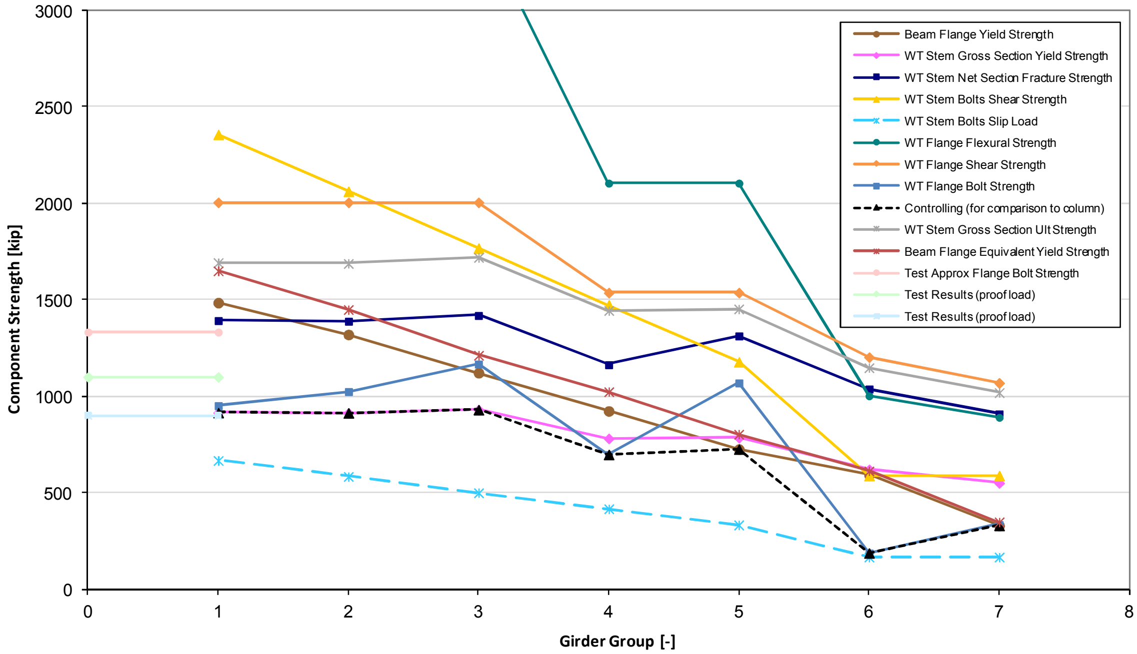


Figure 37: Component strength of girder tension flange connection to strong axis of corner column. The girder groups 1 through 7 along the X-axis of the graph represent the girders at floor levels 3-5, 6-8, 9-10, 11-12, 13-14, 15 and 16-R.

For increasing flange axial forces on this connection, R&C determines the following sequence of limit states: First, the bolts on the WT-stem will slip into bearing. For floor levels 3-10 the connection undergoes gross section yielding thereafter. For small plastic rotations this limit state is a ductile mode, but for large plastic rotations the short gage length of the WT-stem and the moderate ductility of the A373 steel render the gross section yield limit state unreliable. It is more likely that once the WT-stem strain-hardens the WT-flange bolts serve as the critical component. For floor levels 11 and 12 the WT-flange bolt tension limit state occurs before the gross section yielding limit state of the WT-stem. For the remaining floor levels the connection strength is controlled by girder flange yielding. The comparison of the strength limit states of the different components of the strong-axis girder flange connection is illustrated in Figure 37. We conclude that the strong-axis girder flange connection provides a relatively ductile behavior for the expected plastic rotation demands through a combination of WT-stem gross section yielding and plastic deformation of the WT-flange. However, a semi-ductile behavior of this connection could occur through WT-flange bolt tension failure or WT-stem net section fracture.

The girder web connection to the corner column strong-axis consists of vertically aligned double angle sections riveted to the column with closely spaced, staggered 1-inch rivets and bolted to the girder web with 1.25-inch high strength bolts. From strength calculations we determine that this web connection provides sufficient strength to transfer shear demands.

d) *Girder connection to weak axis of corner column:*

The girder flange connection into the weak axis of the corner column consists of ASTM A373 steel plates that are full penetration welded to the web and the inside of the two flanges of the column. Girder flanges are bolted to theses continuity plates with 1.25-inch high strength bolts.

The plate thickness and the number of bolts reduce moving up the building. We investigate potential mechanisms, which are summarized in Table 17.

Table 17: Summary of potential mechanisms for girder flange connection to weak axis of column.

Component	Yield or Failure Mechanism
Continuity Plate	- gross section yield of the plate - net section fracture of the plate - block shear failure of the plate bolt pattern - block shear failure of the plate welds
Flange Bolts	- bolt slip - bolt shear failure - bearing failure at bolt holes
Column Welds	- weld failure

For increasing flange axial forces on this connection we determine the following sequence of limit states: First, the bolts connecting the girder flange to the continuity plate will slip into bearing. Because the yield strength of the girder flanges is lower than the yield strength of the continuity plates the corner column weak-axis connections are controlled by gross section yielding of the girder flanges for all floor levels. This is a ductile failure mechanism. The comparison of the strength limit states of the different components of the weak-axis girder flange connection is shown in Figure 38. It should be noted that the hinge formation due to the yielding of the girder flanges will be away from the column by a distance equal to the length of the continuity plate. Hence, the effective over-strength of the continuity plates compared to the girder flanges is smaller than what is shown in Figure 38 because of the moment gradient. We conclude that the weak-axis girder flange connection provides a ductile behavior which is controlled by the yielding of the girder flanges or possibly by the combined yielding of the girder flanges and the continuity plates. For our computer structural analyses we locate the girder plastic hinges at the ends of the continuity plates.

The web connection of the girder to the corner column weak-axis consists of a single, vertically aligned steel web plate that is fillet welded (both sides) to the column web and the continuity plates at the top and bottom. A pair of web splice plates (one on each side) are bolted to the web plate and the girder web to connect the two together. From strength calculations we compute that for most floor levels this web connection provides sufficient strength to transfer shear demands by itself. For a small number of floors the contribution of the flange continuity plates is required to transfer shear demands.

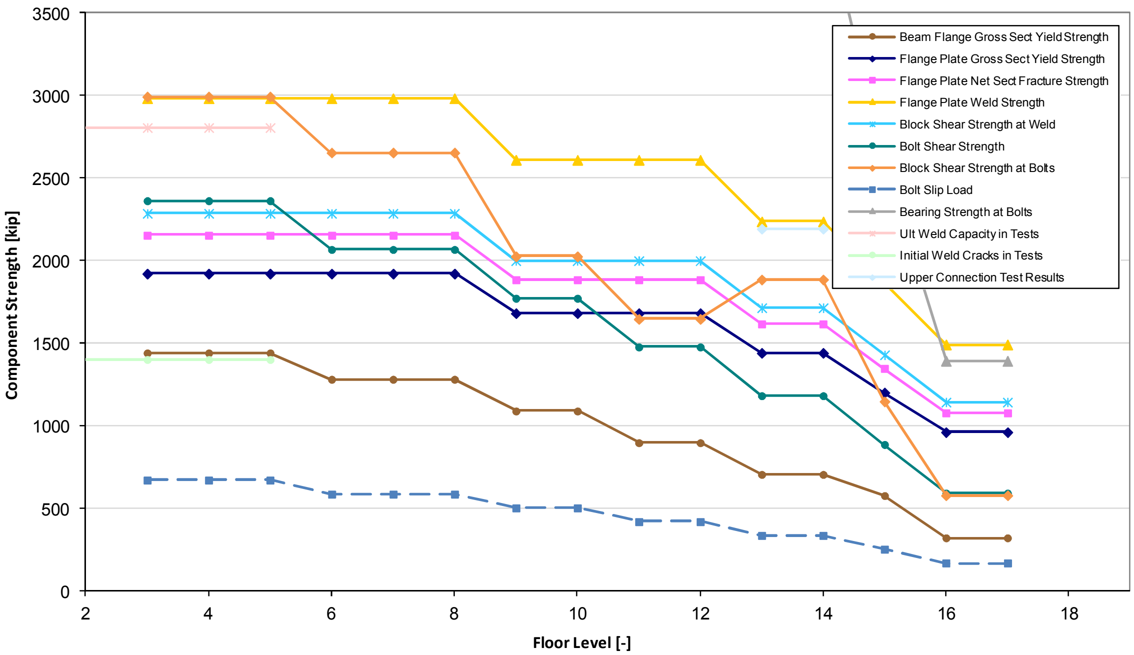


Figure 38: Component strength of girder tension flange connection to weak axis of corner column.

e) Girder connection to strong axis of intermediate column:

Similar to the corner column strong-axis connections, the intermediate column strong-axis girder flange connections consists of WT-sections (cut from 36WF300) that are bolted to the column and girder flanges with 1.25-inch high strength bolts. For the intermediate columns, the WT-section, the number of WT-flange bolts and the number of WT-stem bolts are the same for all floor levels. The same potential mechanisms as for the corner column connection, which are summarized in Table 16, are evaluated for the intermediate column connection. We find that after the WT-stem bolts slip into bearing, the next limit state which is controlling the connection strength is the yielding of the girder flanges. Other potential mechanisms, such as WT-stem gross section yielding, WT-stem bolt shear failure and WT-flange bolt tension failure have significantly higher strength. Hence, we can conclude that the intermediate column strong-axis connection will remain elastic and the interior girders will form plastic hinges. This means that for our computer structural analyses we locate the girder hinges away from the column face at the inside end of the WT-section.

f) Girder connection to weak axis of intermediate column:

The intermediate column weak-axis connections are identical to the corner column weak-axis connections described with results as described earlier.

g) Panel zones of corner columns in strong-axis direction:

For the corner columns, the 36-inch by 42-inch panel zones consist of single built-up steel plates. The thickness of the panel zones (equal to the web plate thickness of the column section) reduces from 1.5 inches to 0.75 inches moving up the building. Due to the orientation of the intermediate columns only the corner column panel zones contribute to the overall strength of the perimeter moment frame in the corner column strong-axis direction. We model corner column panel zones as nonlinear and intermediate column zones as linear to account for panel zone deformations. Our calculations show that the nominal strength of the panel zone webs are the weakest components in the girder-column joints for all floor levels. We determine that the built-up column sections have sufficient capacity to mobilize the column flanges and increase the panel zone strength. As can be seen from Figure 39, the panel zone strength with flange contribution is still the weakest component in the girder-column joint. However, if we consider the reduction in panel zone shear demand from the counteracting effects of the column shears above and below the joint, the panel zones have sufficient strength to transfer the actual shear demand.

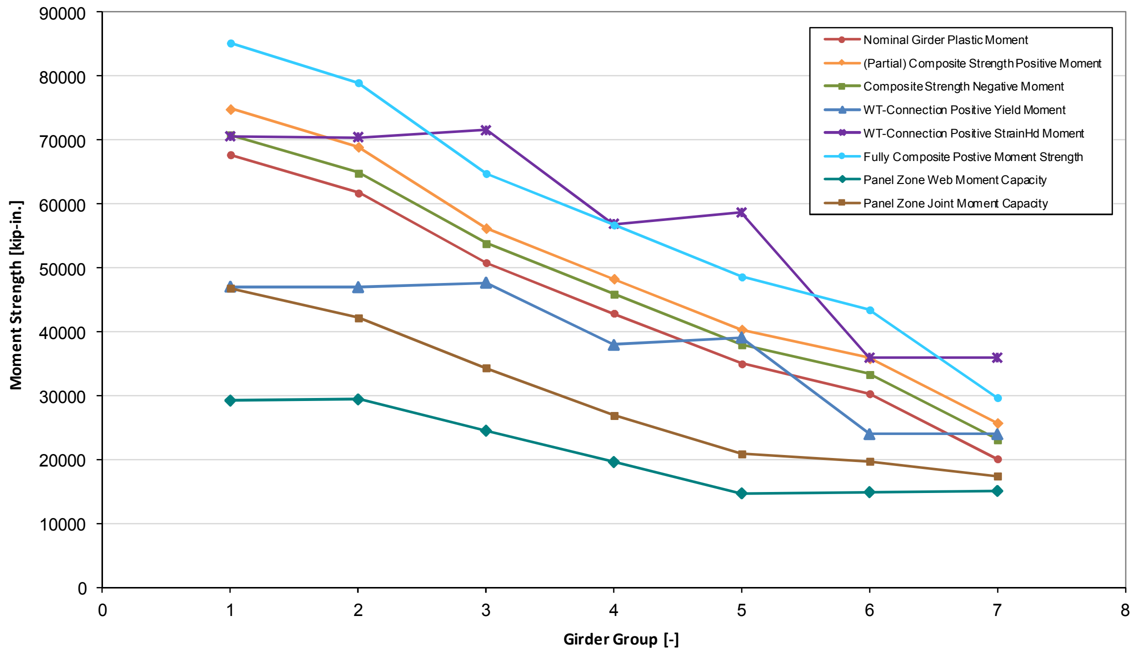


Figure 39: Girder moment strength compared to panel zone strength. The girder groups 1 through 7 along the X-axis of the graph represent the girders at floor levels 3-5, 6-8, 9-10, 11-12, 13-14, 15 and 16-R.

4.3. MECHANICAL TOWERS

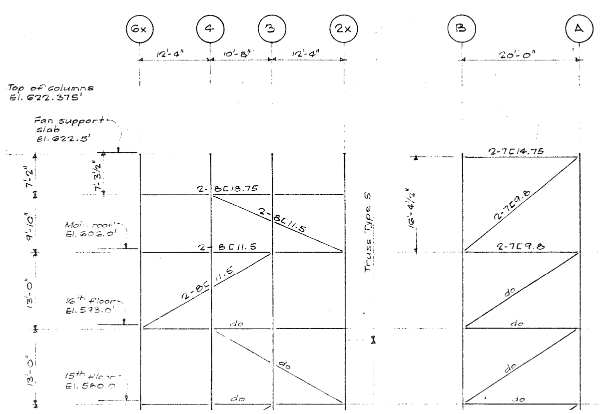


Figure 40: Typical mechanical tower bracing configurations. The top floors of the HSE tower are shown. The HSW tower is similar.

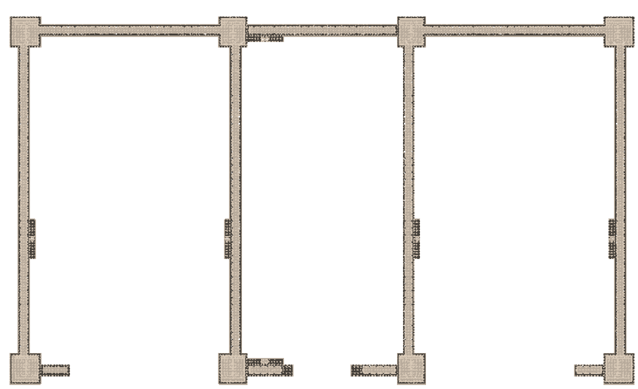


Figure 41: Fiber model of mechanical tower concrete walls including reinforcing and vertical structural steel, shown as a plan section.

COMPONENT CAPACITY AND GOVERNING BEHAVIOR MODES

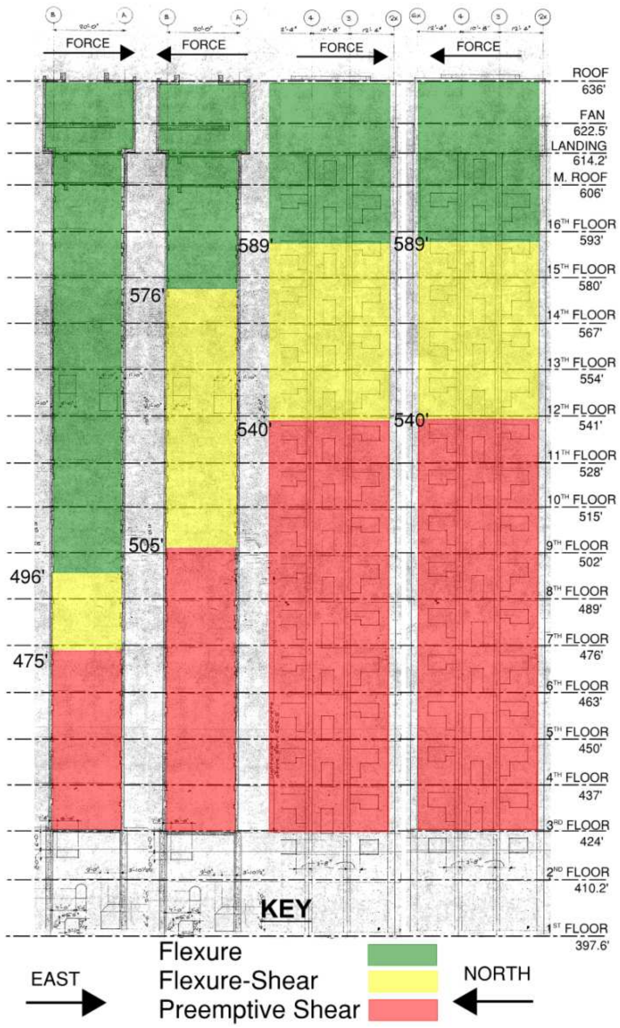


Figure 42: Behavior modes of the HSE mechanical tower walls expressed in terms of height of the resultant seismic force. Results for the HSW tower are similar.

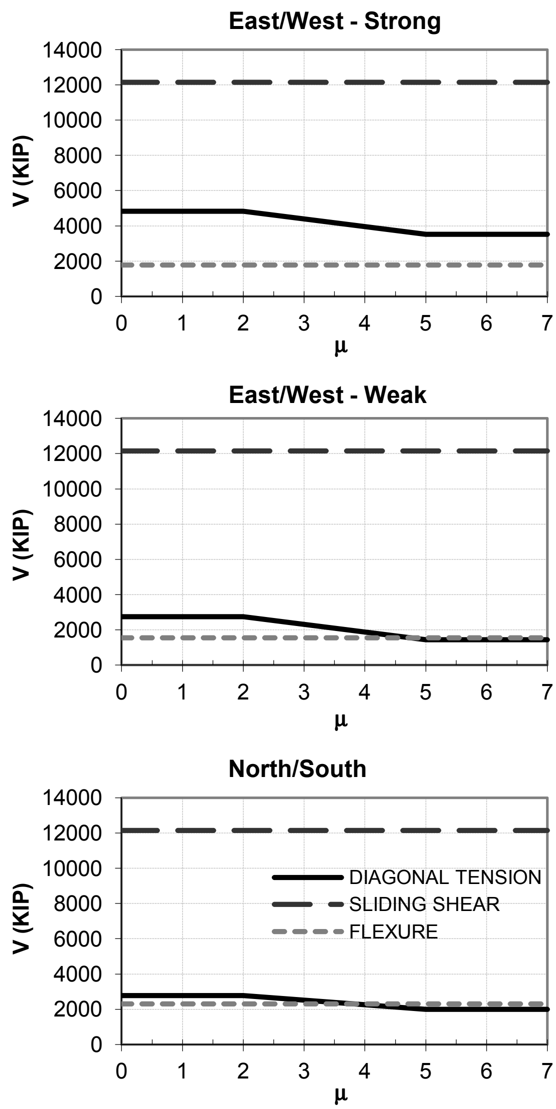


Figure 43: Behavior modes of the HSE mechanical tower walls expressed in terms ductility and assuming an inverted triangular load pattern. Results for the HSW tower are similar.

To establish of the expected structural behavior the mechanical towers, when subjected to lateral earthquake forces and deformations, we calculate the strength of each tower in each direction in flexure, shear, and sliding shear. We assume composite behavior between the steel frame system and reinforced concrete walls. Columns and the vertical component of brace area act as flexural reinforcement. Our calculations confirm that the splices of the W14 columns have the capacity to develop the tension yielding of the columns. The model uses expected material properties. We calculate the flexural strength of the tower in each direction at each story level using a moment curvature analysis on a discretized cross section. Because the wall reinforcement is spaced 10 inches O.C., we chose a spalling concrete model without a confined core region. An image of a typical cross-section model for the mechanical tower is shown in Figure 41. Output from the analysis includes moment-curvature diagrams and axial force-moment interaction diagrams, which are used to define the tower nonlinear behavior in the dynamic model.

For each tower in each direction we also determine the strength for several shear failure modes. Comparing the flexural and shear capacities, we determine the expected wall behavior. The behavior modes considered include flexural yielding, flexural yielding followed by shear failure at larger ductility, preemptive shear failure, or sliding shear failure. For shear strength in diagonal tension, we use an equivalent truss model and treat the steel beams and the horizontal component of the steel brace area as shear reinforcement in cases where potential diagonal cracks must cross such members.

Figure 42 indicates that the behavior of the walls is dependent on the height of the lateral force resultant, equal to the ratio between the base shear and overturning moment. The figure illustrates the expected behavior modes for the mechanical tower in terms of the height above base of the lateral force resultant for which the structure will be govern by capacity in flexure rather than shear. Flexure-governed behavior is preferable to shear-governed behavior, which is associated with strength degradation and potential concentrations of lateral deformation [SEAOC 1999].

In simplified seismic design the lateral force resultant is assumed to be at about two-thirds of the height of a building. Figure 42 shows that for a resultant force at this height the towers are flexure or flexure-shear governed in all directions. In actual earthquake shaking the resultant height is likely to lower as a result of higher mode effects. For a resultant below 45% to 60% of the building height the figure shows that the walls tend to become shear governed. This issue is further evaluated in our overall analysis and assessment.

That all of the steel diagonal braces in the tower short direction slope in a common direction means that the shear capacity of the building is asymmetric. The tower is considerably stronger in the loading direction that generates a shear crack that must cross the steel brace. The result is that the HSE mechanical tower is stronger in shear in the direction away from the lab building. The HSW tower is stronger in shear in the direction towards the lab building. Figure 42 illustrates this point for the HSE tower. The green, flexure controlled region extends lower when the force is acting to the east, away from the lab building. The ratio of HSE tower mass to expected base shear capacity is 0.52 g in the weak direction and 0.60 g in the strong direction.

In the strong direction of the tower, the braces in the two long walls alternate slope at adjacent floor levels. However, the braces in each wall slope in the same direction at a given level. Therefore, the shear capacity of the long direction walls is controlled by the capacity of a shear crack that does not cross the steel brace. Figure 42 slows that the expected failure mode in the north/south direction is independent of the direction of loading. In this case, the ratio of mass to base shear capacity is 0.78 g for both directions.

4.4. Elevator Tower

Description of Existing Structure

The structure of the Health Sciences elevator tower is similar to the system employed in the mechanical towers. Again, the core has three bays with steel braced frames consisting of double channel beams and braces with W14 columns embedded in 8-inch thick reinforced concrete walls. The two frames in the east/west direction have a typical chevron configuration of braces. In the north/south direction, four braces form a chevron brace pattern spanning three bays in each of the perimeter walls. Two interior walls on Line K and L have punched openings for elevator access and no braced frames.

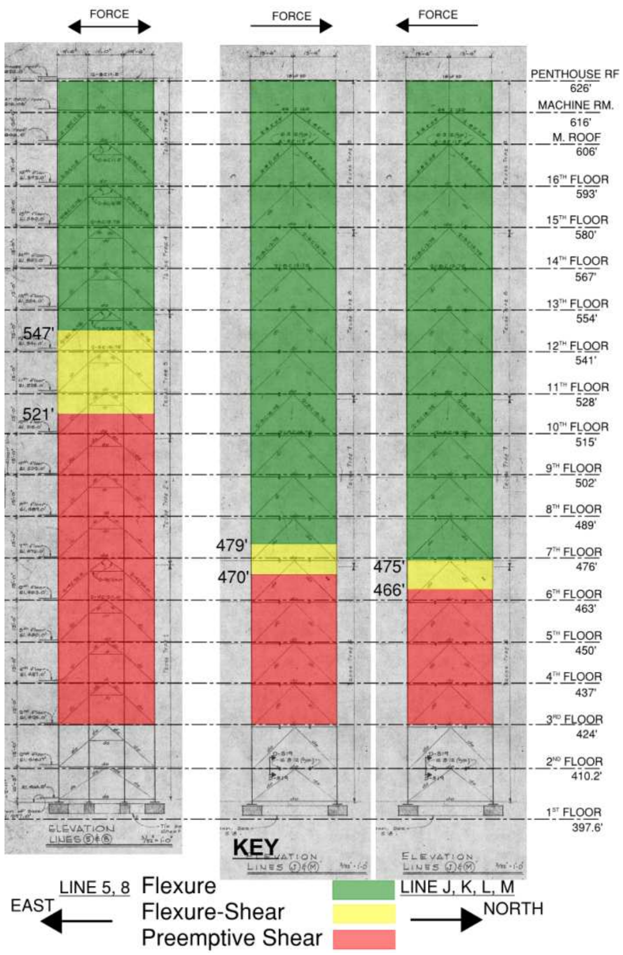


Figure 44: Behavior modes of the Elevator tower walls expressed in terms of height of the resultant seismic force.

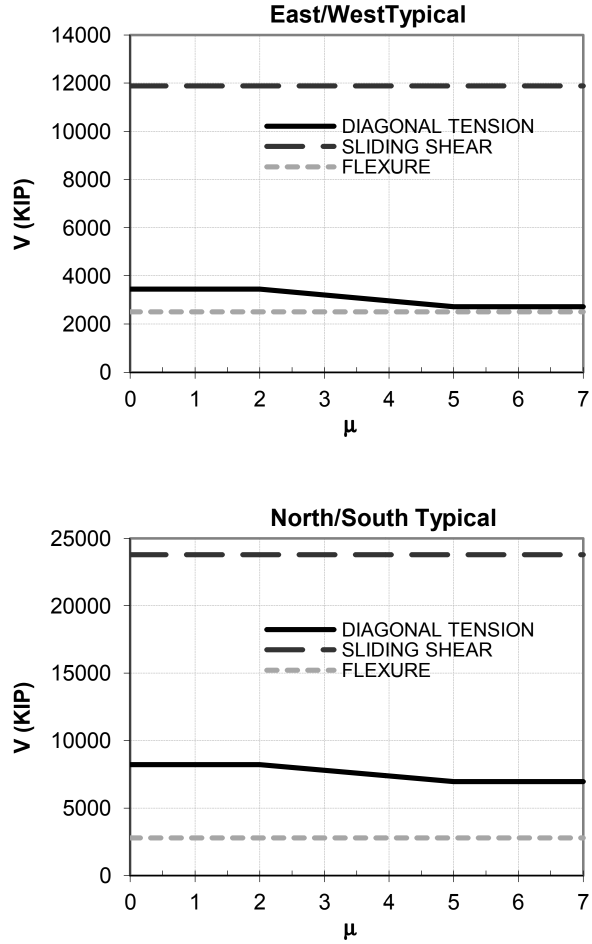


Figure 45: Behavior modes of the elevator tower walls expressed in terms ductility and assuming an inverted triangular load pattern.

The elevator tower differs from the mechanical towers by the presence of an attached corridor. The corridor links the Health Sciences West Building with the Increment 2 addition to the Medical Sciences Building. The corridor structure consists of a one-way formed reinforced concrete floor slab. Four W14 steel columns form moment frames with W18 beams spanning East/West. No beams are present in the North/South direction. Because the stiffness of the steel moment frames is small compared to the elevator tower walls, the corridor can induce plan torsion in the elevator tower by shifting the center of mass north, away from the tower's center of rigidity and center of lateral strength.

Component Capacity and Governing Behavior Modes

Our analysis of the elevator tower is similar to that used with the mechanical towers. We determine the flexural strength, stiffness, and ductility properties of a tower cross section assuming composite behavior between the steel frame and reinforced concrete walls. We determine the strength of the tower for the behavior modes of flexural yielding, flexural yielding followed by shear failure at a larger ductility, preemptive shear failure, and sliding shear failure.

In this case the wall behavior is nearly symmetric about both its north/south and east/west principal axes. Results are shown in Figure 44 and Figure 45. The tower is very likely to be flexure governed in the north/south direction. The ratio of tower seismic mass to expected base shear capacity is between 0.94 and 1 g, which indicates high strength. In the east/west direction, the behavior mode is more sensitive to the resultant height (i.e., the ratio of overturning moment to base shear). The ratio of mass to expected base shear capacity is 0.90 g. Such a high strength ratio implies that the tower is unlikely to experience sufficient force or ductility demand to initiate a shear failure.

4.5. Nonlinear Response-history Modeling

To evaluate the expected response of the existing laboratory structures, R&C builds a three dimensional, analytical model of the Health Sciences East (HSE) building. We only model the HSE building with its mechanical tower because it is the more critical of the two laboratory buildings considering that it is one story taller than the HSW building and supports heavy mechanical equipment at the roof level. We examine the structural behavior of the building for seven ground motions at seven different hazard levels. The selection of this suite of forty-nine ground acceleration records is described in Section 3.8 of this report. The primary analysis technique that R&C uses to study the response of the HSE building under the dynamic earthquake loading is the nonlinear response-history (NLRH) analysis method. The structural software package that we employ to carry out these NLRH analyses is Perform-3D, a nonlinear finite element program that is specialized for the displacement-based and capacity design of building structures.

The objectives of the analyses include:

- To determine floor displacements, interstory drifts, seismic gap displacements, and total floor accelerations to evaluate nonstructural components and laboratory contents.
- To determine the demand levels and likelihood of inelastic deformation demands exceeding estimated capacities for all elements and connections of the seismic-force-resisting system.
- To determine the possibility for a localization of nonlinear seismic demands leading to the potential for the building to form a story mechanism.
- To evaluate the seismic force path through connections relative to the capacities of the individual connection components.
- To determine if seismic forces will be carried adequately by each connection component providing its intended response.
- To evaluate the details of the force path acting on the built-up members to determine if these structural components perform integrally.
- To determine the interaction effects of the laboratory building with the adjacent mechanical and elevator towers.

Table 18 summarizes basic modeling assumptions and approaches for the analysis of the laboratory building.

Table 18: Summary of structural modeling assumptions.

Category	Structural Modeling Assumption
Geometry	- The base of the model is located at elevation 409'-0" (2 ft below the 2nd floor level) and is assumed to be fixed, reflecting the embedment in concrete of the built-up steel columns. - The floor levels are modeled at the center lines of the 42-inch deep girders with floor-to-floor heights of typically 156 inches. - The floor diaphragms are modeled as flexible diaphragms using membrane elements between the girders. The membranes are defined as shell elements with a negligible out-of-plane bending thickness.
Weight	- The self weight of the structure is assembled from the self weight of all the frame and shell elements and additional point loads for the roof equipment bridge. - An expected live load of 10 psf is included, and modeled by increasing the self weight of the shell elements accordingly.
Mass	- Building masses are concentrated at the floor levels and are defined as translational (H1, H2, V) masses lumped at 16 nodes per floor level. - To account for the roof equipment bridge, the bridge mass is assigned to the 4 nodes corresponding to the top of the intermediate columns that support the bridge. - An expected live load of 10 psf is included in the 16 lumped masses per floor level.
Nonlinearities	- Concentrated (zero-length) rigid-plastic hinges are used to model element plasticity in columns and girders. - The plastic hinges have fixed locations that R&C determines before performing the NLRH analyses. - Elastic-perfectly-plastic M-hinges (rotation type) are used for the exterior and interior girders. - Tri-linear M-hinges (rotation type) are used for the corner girders. - Elastic-perfectly-plastic PMM-hinges (steel rotation type) are used for the columns and column splices. - The panel zones of the corner columns are modeled as inelastic tri-linear panel zones based on the Krawinkler model. - The panel zones of the interior columns are modeled as linear-elastic panel zones based on the Krawinkler model. - The base hinge of the mechanical tower is modeled as an elastic-perfectly plastic PMM-hinge (concrete rotation type) with a hinge length of 125 inches.
Damping	- Modal viscous damping equal to 1% of critical is assigned to modes 1 trough 50, which corresponds to periods between 1.554 and 0.157 seconds. - To also provide energy dissipation for the higher modes (larger than mode 50), a small amount of Rayleigh damping is assigned to the model.

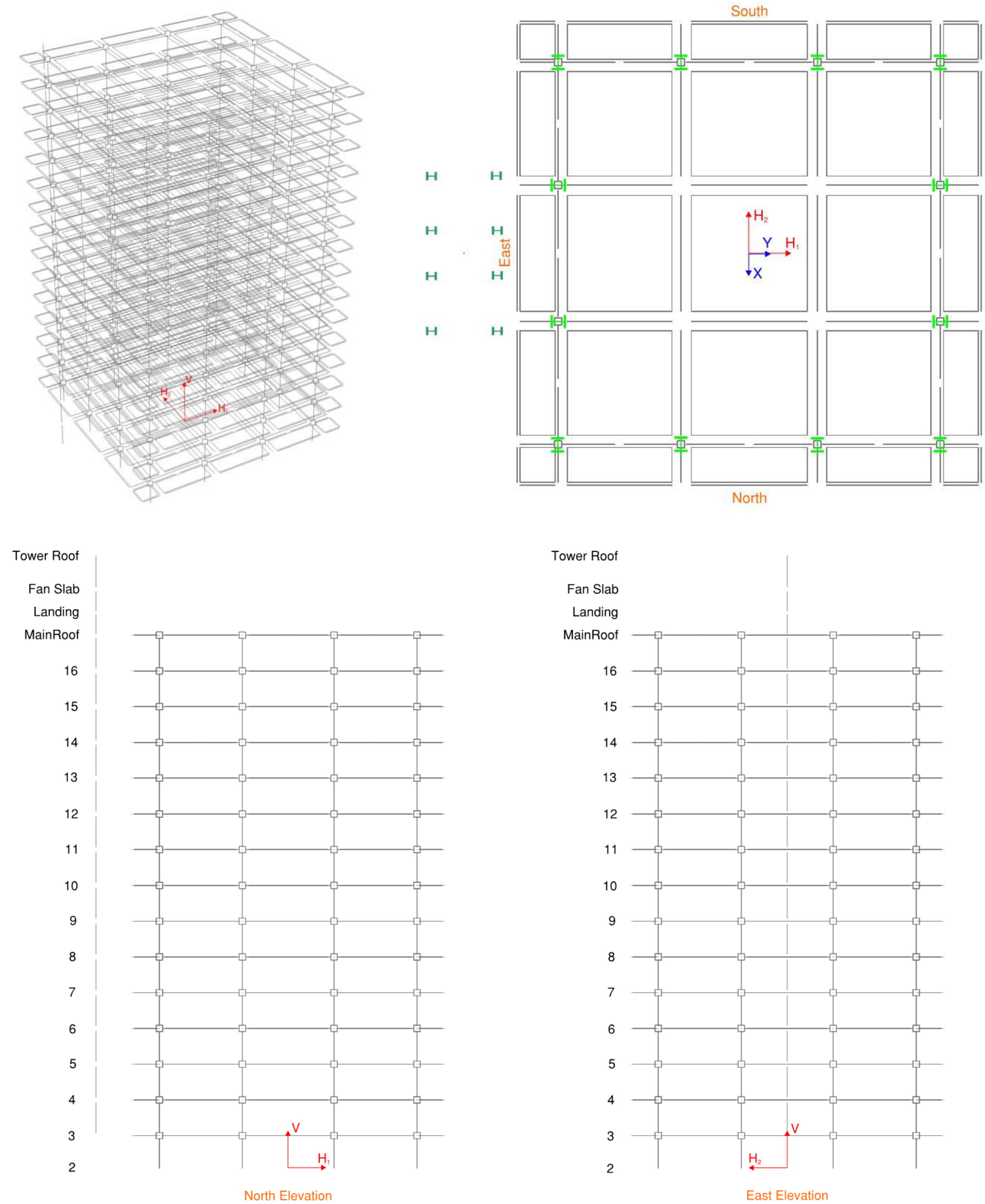


Figure 46: Perspective, plan and elevation views of the Perform-3D model.

The geometry of the Perform-3D model of the HSE laboratory building and mechanical tower is shown in Figure 46. For clarity, the global axes (H1, H2, V) and the orientations of the steel columns have been superimposed on the drawings. The mechanical tower is modeled as a 2,544-inch tall stick with a concrete PMM hinge at the base. Story heights are typically 156 inches except for the second story which is 153.25 inches. Hence, the overall height of the 15-story laboratory building model is 2,337.25 inches.

Along the perimeter moment frames of the laboratory building the column spacing in the exterior bays is 361.5 inches and the interior bay spans 400 inches. The shell elements which are used to model the flexible floor diaphragm can be seen in the plan view of the structural model.

The gravity loads and the seismic weights are summarized in Table 19. For the seismic weight, which corresponds to the masses that we assign to the Perform-3D model, the weights of partitions, cabinets, ceilings, lighting and miscellaneous items are reduced, because we assume that during an earthquake they do not participate with their full mass to the inertia forces. From the table it can be seen that the total distributed roof weight is smaller than the total distributed weight of a typical floor level. However, this difference is less pronounced for the seismic weight than for the gravity load. From the gravity load and seismic weight takeoff we calculate corresponding floor weights and floor masses which are summarized in Table 20. The reported floor weights and masses include the contribution of the 10 psf live load and the reported roof weight and mass include the contribution of the equipment bridge and other miscellaneous roof installations. The total gravity load in the Perform-3D model amounts to 26,800 kips and the total reactive seismic mass is equal to 24,400 kips. To capture second order effects from axial column loads, gravity loads are applied to the Perform-3D model before running the earthquake load cases and P-Delta effects are turned on in all the frame elements.

Table 19: Gravity load and seismic weight takeoff.

	Gravity Load		Seismic Weight	
Smeared Loads [-]	Typ. Floor [psf]	Roof [psf]	Typ. Floor [psf]	Roof [psf]
5.5 in LWT concrete slab	55.0	55.0	55.0	55.0
Steel Framing	23.3	15.5	23.3	15.5
Partitions/Cabinets	20.0	-	10.0	-
Flooring	2.0	-	2.0	-
Roofing	-	3.0	-	3.0
Insulation	-	3.0	-	2.0
Deck deflection added fill	3.0	3.0	3.0	3.0
Fireproofing	1.0	1.0	1.0	1.0
MEP-Sprinklers	5.0	5.0	5.0	5.0
Ceiling & Lighting	5.0	-	3.0	-
Miscellaneous	5.0	5.0	3.0	3.0
Total	**119.3**	**90.5**	**105.3**	**87.5**
Concrete	55.0	55.0	55.0	55.0
Steel	23.3	15.5	23.3	15.5
Other	41.0	20.0	27.0	17.0

Table 20: Floor weights and floor masses.

		Hand Calculation		Perform-3D Model		Perform-3D Model	
Level [-]	Elevation [ft]	Weight [kip]	Cum. Weight [kip]	Weight [kip]	Cum. Weight [kip]	Mass [kip]	Cum. Mass [kip]
Roof	606.0	1769	1769	1573	1573	1731	1731
16	593.0	1698	3467	1714	3287	1517	3248
15	580.0	1736	5203	1743	5030	1556	4803
14	567.0	1746	6949	1758	6788	1565	6369
13	554.0	1748	8698	1759	8547	1568	7936
12	541.0	1764	10461	1777	10324	1583	9520
11	528.0	1773	12235	1782	12106	1593	11112
10	515.0	1789	14024	1803	13909	1608	12721
9	502.0	1801	15824	1809	15718	1620	14341
8	489.0	1828	17653	1843	17561	1648	15988
7	476.0	1839	19491	1847	19408	1658	17646
6	463.0	1839	21330	1858	21266	1658	19304
5	450.0	1883	23213	1886	23152	1703	21007
4	437.0	1883	25096	1895	25047	1703	22710
3	424.0	1883	**26980**	1799	**26846**	1703	**24412**

Note: The roof weight and roof mass include the additional weight/mass due to the roof equipment bridge and other miscellaneous roof installations.

To capture the nonlinear plastic behavior of the girders, columns, column splices and corner column panel zones, a range of compound components are setup in the Perform-3D model. The frame element components consist of rigid end zones, concentrated M-hinges or concentrated PMM-hinges and linear-elastic zones. The eight different types are illustrated in Figure 47. Interior column panel zones are assigned a linear-elastic behavior and corner column panel zones are modeled as inelastic tri-linear based on the Krawinkler model. As can be seen from Figure 47 rigid end zones in all the girders are assigned equal to half the depth of the columns and similarly rigid end zones in all the columns are assigned corresponding to the depth of the girders.

Based on the relative capacities of the composite girder and the girder connection to the strong-axis of the corner columns, R&C determines the following modeling approach for the corner girders. We place the girder plastic hinges at the face of the column because we assume that the WT-connection is controlling the plastic hinge location. A tri-linear M-hinge is defined with the yield moment strength equal to the capacity of the WT-connection based on gross section yielding of the WT-stem. We include the contribution of the slab (limited in either direction by the shear strength of the shear connectors) in the capacity of the connection. A strain hardening factor based on a linear interpolation from the yield point to the ultimate stress (assumed to occur at a strain of 18%) is used to define the tri-linear behavior of the Perform-3D moment hinge. We transform strains to hinge rotation based on a constant curvature assumption over the free span length of the WT-stem. For the ultimate hinge rotation we also incorporate the contribution of a partially yielded inner bolt row on the WT-flange and a combination of slip and bearing deformation.

For the exterior girders R&C determines the following frame element compound components. Because the gross section yielding of the girder flanges controls this connection, the girder plastic hinges are assigned a fixed distance away from the face of the columns. In between the column face and the plastic hinge an elastic element component is defined that corresponds to a portion of the length of the flange connection plate. For the nonlinear hinge we use an elastic-perfectly-plastic moment rotation relationship with the yield moment strength equal to the partial composite strength of the girder with slab for positive moments and the full composite strength of the girder with slab for negative moments.

The interior girders connecting to the strong-axis of the interior columns are also controlled by the gross section yielding of the girder flanges. We therefore model them similarly to the exterior girders with the plastic hinge located a fixed distance away from the interior column face. The length of the elastic component that is assigned between the column face and the plastic hinge corresponds to the length of the WT-connection. The nonlinear hinge is again modeled with an elastic-perfectly-plastic behavior corresponding to the partial and full composite strength of the interior girder with slab.

For the columns R&C defines compound frame elements with elastic-perfectly-plastic PMM hinges at the ends of the rigid end zones. Because the elements are defined at the center lines of the 42-inch deep girders the column hinges are located 21 inches above and below the girders. For the PMM-interaction surface we use the axial compression yield force of the corresponding column cross section, the moment strength based on the limited rivet shear flow in the strong-axis direction and the plastic moment in the weak-axis direction. The remaining interaction surface parameters were left at their default values.

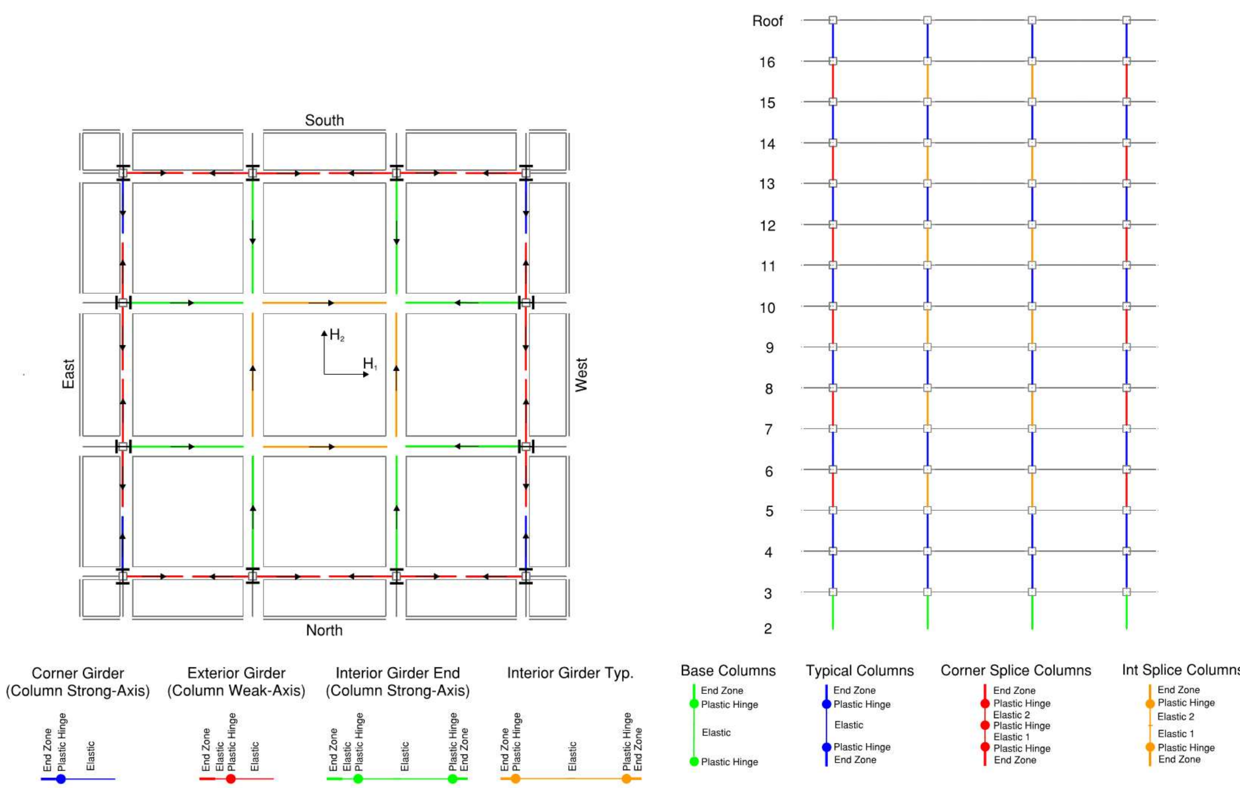


Figure 47: Compound components in the Perform-3D model for the girder and column elements.

Because we find that the corner column splices at levels 5.5 and potentially 7.5 will yield in tension in the larger ground motions, we model the splices of all the corner columns as concentrated PMM hinges at column mid-height. This allows us to track the amount of plastic elongation of the splices in case they yield under seismic tension loads. To define the interaction surfaces of the PMM hinges, R&C determines tension and compression yield forces (they are asymmetric), and plastic moments for the strong- and weak-axis directions. Because gross section yielding of the splice plates is the governing behavior mode, we base our calculations on the gross section yielding of the splice plates alone for the tension side and on the gross section yielding of the splice plates plus the column flanges for the compression side. The remaining interaction surface parameters that define the transition between these key points are based on the Perform-3D default values.

The girder plastic hinge properties for the Perform-3D model are summarized in Table 21 and the column plastic hinge properties are summarized in Table 22.

Table 21: Girder plastic hinge properties in Perform-3D model.

Girder Type	Corner Girders Strong-Axis Connection			Exterior Girders Weak-Axis Connection		Interior Girders Strong-Axis Connection	
Description	WT-Connection Yield Moment	WT-Connection Yield Moment	WT-Connection Ultimate Moment	Partial Composite Positive Moment	Full Composite Negative Moment	Partial Composite Positive Moment	Full Composite Negative Moment
Section [-]	My + [kip-in.]	My - [kip-in.]	Mu + / Mu - [kip-in.]	My + [kip-in.]	My - [kip-in.]	My + [kip-in.]	My - [kip-in.]
16-R	23965	-26013	27382	25754	-23130	45395	-42900
15	23965	-28135	31022	35836	-33330	45395	-42900
13-14	39105	-43275	49460	40382	-37990	45395	-42900
11-12	37925	-42095	54999	48302	-45900	45395	-42900
9-10	47675	-51845	65582	56222	-53820	45395	-42900
6-8	46972	-51142	75812	68938	-64840	45395	-42900
3-5	47053	-51223	75962	74852	-70840	45395	-42900

Table 22: Column plastic hinge properties in Perform-3D model.

Column Type	Corner Columns				Interior Columns		
Description	Compression Yield Force	Splice Tension Yield Force	Moment based on Rivet Shear Flow	Plastic Moment	Compression Yield Force	Moment based on Rivet Shear Flow	Plastic Moment
Section [-]	Py + [kip]	Py - [kip]	Mpx [kip-in.]	Mpy [kip-in.]	Py + [kip]	Mpx [kip-in.]	Mpy [kip-in.]
15.5-R	2252	-1343	21316	9671	5049	46848	32447
13.5-15.5	3341	-1491	23033	18210	6237	47415	43139
11.5-13.5	3787	-1485	27260	21217	6237	47415	43139
9.5-11.5	4356	-2170	29518	26210	6831	47698	48485
7.5-9.5	5074	-2162	39356	29978	7425	47982	53831
5.5-7.5	5792	-2030	41257	33747	8019	48527	59177
2-5.5	6782	-	41219	41172	10395	47283	80561

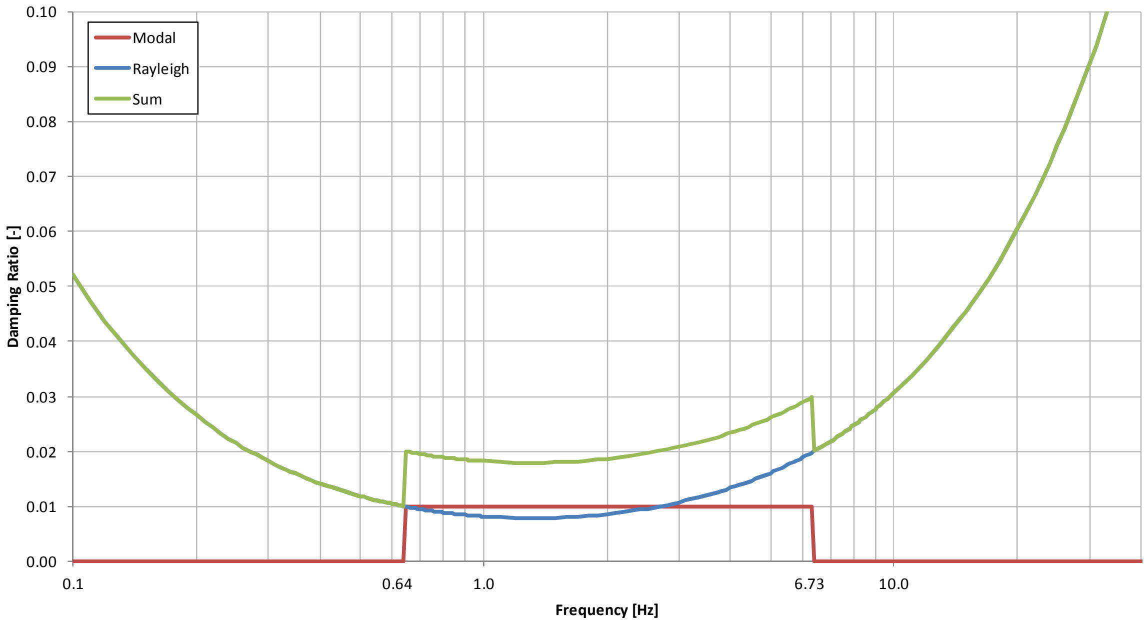


Figure 48: Modal and Rayleigh viscous damping ratios in the Perform-3D model.

The energy dissipation from nonstructural components and from other mechanisms that are not directly captured by the explicitly defined nonlinear hinges is modeled using viscous damping. We are using a combination of modal and Rayleigh viscous damping as illustrated in Figure 48 to obtain approximately 2-3% damping over a large range of building periods. As can be seen, the modal viscous damping portion is equal to 1% of critical assigned to modes 1 through 50. This corresponds to frequencies between 0.64 Hz and 6.73 Hz. To also provide energy dissipation for the higher modes (above 6.73 Hz), we assign the Rayleigh damping portion equal to 1% of critical at 0.64 Hz and 2% of critical at 6.73 Hz.

SUMMARY OF STRUCTURAL ANALYSIS MODELS

The results of several original structural analyses of the Health Science buildings are considered in this investigation. At the time of construction, a series of forced vibration tests, component strength tests, and computer analyses were performed by a team of investigators from the University of California, Berkeley led by J.G. Bouwkamp.

Table 23 lists the original analyses and also summarizes the current analyses, which include component-level fiber models in XTRACT, and three dimensional PERFORM models of the HSE building used to perform nonlinear response-history analysis. The comment column in the table indicates how original analyses are compared to current analyses.

Table 23: Summary of structural analysis models and tests results.

Model	Analysis	Software	Description	Comment
1964 Forced Vibration	Experimental	-	Bouwkamp, et al. forced vibration testing of laboratory buildings and mechanical towers to experimentally determine mode shapes and vibrational periods	Results used to verify vibrational period response of current R&C models
1965 Welded Girder Connection Test	Experimental	-	Bouwkamp, test report for welded girder-column connection	Results used to verify connection components in current R&C PERFORM model
1966 Linear	Modal, LRH	FRMDYN	Bouwkamp, et al. 2-D model used to compute translational vibration modes and linear response	Results used to verify vibrational period response of current R&C models
1966 Nonlinear	NLRH	NONLIN	Bouwkamp, et al. 2-D model used to compute nonlinear translational response of a typical plane frame	Results of nonlinear response of the existing laboratory building compared to results from current R&C PERFORM model
Current Component Fiber	Component NL	XTRACT	R&C fiber models of mechanical and elevator tower sections and composite floor slab and girders	Component results used to define nonlinear hinge characteristic in PERFORM and SAP2000 models
Nonlinear - Existing and Retrofitted Structure Model	NLRH	SAP2000	R&C 3-D model of HSE with concentrated moment-rotation nonlinear components at beam and column hinge zones including versions with and without damping retrofits	Results were not used. FNA solver results were unreliable and lack P-delta analysis capability. Direct integration solution of model was not possible
Current Nonlinear - Existing Structure Model	NLRH	PERFORM	R&C 3-D model of HSE with concentrated moment-rotation nonlinear components at beam hinge zones and panel zone and concentrated moment-rotation PMM components at columns. Explicitly includes potential yielding behavior at column splices. Three versions used: existing structure, retrofit with dampers at the mechanical tower, and retrofit with dampers at the mechanical tower and laboratory frames	Results used to describe the seismic performance of the existing and retrofitted laboratory buildings

4.6. Analysis Findings for Existing Structure

To study the behavior and evaluate the performance of the HSE laboratory building we record a range of measurement indices during the nonlinear response-history analyses of the three-dimensional Perform-3D model. We use the Perform-3D Application Programming Interface (API), developed by Prof. T.Y. Yang, to simplify the extraction and the post-processing of the extensive amount of NLRH analysis output data.

MEASUREMENT INDICES

The directly recorded and/or calculated measurement indices include:

- Building periods and mode shapes to compare and calibrate against experimental forced vibration tests that were performed during construction in 1964 on the bare steel moment resisting frame with concrete slabs. Periods and mode shapes are also used to determine the global behavior of the HSE laboratory building and mechanical tower in the linear-elastic range.
- Plastic hinge rotation maps to determine yield sequences and compare component demands against capacities.
- Distribution of floor displacements, velocities and total accelerations to determine floor-by-floor demand intensities and evaluate acceleration sensitive building components and contents.
- Distribution of interstory drift ratios and interstory velocities to determine a global measure of demand intensity and identify the potential for the building to form a soft story mechanism.
- Distribution of residual interstory drift ratios to evaluate post-earthquake reparability.
- Story shear and overturning moment demands of the laboratory building and the mechanical tower to check shear capacities.
- Corner column and intermediate column axial tension and compression demands, strong- and weak-axis moment and shear demands to check column capacities of the laboratory building.
- Relative displacements at building separation joints to determine joint opening and closing and transverse displacements across joints.
- Floor response spectra to evaluate building and mechanical tower contents. Response spectra are generated for two orthogonal directions as well as for the vector norm of floor accelerations.

PERIODS AND MODE SHAPES

The periods and mode shape directions that R&C determines from the analysis model are summarized in Table 24 below. The table also lists the effective modal masses and the cumulative sums of the effective modal masses. The effective modal masses provide a means to determine the significance of a vibration mode. The higher the effective modal mass, the stronger that particular mode will be excited when the structure is subjected to ground shaking. North-South vibration modes are highlighted in red, East-West vibration modes in blue and torsional modes in green.

As mentioned earlier we use the experimentally determined periods and mode shapes from forced vibration tests to calibrate the elastic properties of the analytical model before running the NLRH analyses. The forced vibration tests were performed in 1964 by Bouwkamp, et al. on the bare steel perimeter moment resting frame with concrete slabs. With our building model representing the mass and weight of the steel frame and concrete slab only, we are able to match the experimentally determined periods to within 5%. We then increase the masses and the weights to their full in-service values and calculate the periods summarized in Table 24. With the additional mass and weight the first mode period of the laboratory building increases from the experimentally determined 1.18 seconds of the bare steel building to the analytically determined 1.55 seconds of the fully operational building.

Table 24: Periods of the laboratory building and mechanical tower for in-service masses and weights. Effective modal masses and cumulative sums of the effective modal masses are also shown.

Mode [-]	Period [sec]	Direction [-]	H1 [%]	H2 [%]	V [%]	H1 Cum [%]	H2 Cum [%]	V Cum [%]
1	1.554	N-S (Lab)	0%	67%	0%	0%	67%	0%
2	1.528	E-W (Lab)	67%	0%	0%	67%	67%	0%
3	1.451	E-W (Tower)	7%	0%	0%	74%	67%	0%
4	1.140	Torsional (Lab)	0%	0%	0%	74%	67%	0%
5	1.029	N-S (Tower)	0%	7%	0%	74%	74%	0%
6	0.548	N-S (Lab)	0%	11%	0%	74%	85%	0%
7	0.542	E-W (Lab)	11%	0%	0%	85%	85%	0%
8	0.419	Torsional (Lab)	0%	0%	0%	85%	85%	0%
9	0.315	N-S (Lab)	0%	4%	0%	85%	89%	0%
10	0.314	E-W (Lab)	4%	0%	0%	89%	89%	0%
11	0.285	E-W (Tower)	2%	0%	0%	91%	89%	0%
12	0.284	Floor Vert. (Lab)	0%	0%	4%	91%	89%	4%
13	0.281	Floor Vert. (Lab)	0%	0%	25%	91%	89%	29%
14	0.278	Floor Vert. (Lab)	0%	0%	0%	91%	89%	29%
15	0.276	Floor Vert. (Lab)	0%	0%	0%	91%	89%	29%
16	0.274	Floor Vert. (Lab)	0%	0%	0%	91%	89%	29%
17	0.272	Floor Vert. (Lab)	0%	0%	0%	91%	89%	29%
18	0.270	Floor Vert. (Lab)	0%	0%	0%	91%	89%	29%
19	0.269	Floor Vert. (Lab)	0%	0%	0%	91%	89%	29%
20	0.267	Floor Vert. (Lab)	0%	0%	0%	91%	89%	29%
21	0.266	Floor Vert. (Lab)	0%	0%	0%	91%	89%	30%
22	0.265	Floor Vert. (Lab)	0%	0%	0%	91%	89%	30%
23	0.265	Floor Vert. (Lab)	0%	0%	1%	91%	89%	31%
24	0.264	Floor Vert. (Lab)	0%	0%	1%	91%	89%	32%
25	0.262	Floor Vert. (Lab)	0%	0%	0%	91%	89%	32%
26	0.260	Floor Vert. (Lab)	0%	0%	4%*	91%	89%	36%
27	0.248	Torsional (Lab)	0%	0%	0%	91%	89%	36%
28	0.247	N-S (Tower)	0%	2%	0%	91%	91%	36%
29	0.217	E-W (Lab)	2%	0%	0%	93%	91%	36%
30	0.216	N-S (Lab)	0%	2%	0%	93%	93%	36%
31	0.173	E-W (Lab)	0%	0%	0%	93%	93%	36%
32	0.172	N-S (Lab)	0%	0%	0%	93%	94%	36%
33	0.172	Torsional (Lab)	0%	0%	0%	93%	94%	36%

The first three mode shapes of the laboratory building are shown in Figure 49. The first mode vibration occurs along the North-South direction of the building and the second mode vibration occurs along the East-West direction of the building. The torsional vibration around the vertical Z-axis corresponds to the third mode of the laboratory building. The additional flexibility added by modeling the panel zones in the column strong-axis directions reduces the overall stiffness of the building in the North-South direction enough for the first mode to occur along that direction.

Note that the most prominent vertical floor vibration mode has a period of T = 0.281 sec. The period is roughly equivalent to floor systems we have designed for hospital patient care with vibration sensitivity criteria. As a result, we do not consider that the laboratory floors are unusually sensitive to vertical accelerations and we chose not to consider the effect of vertical ground motions in our analysis.

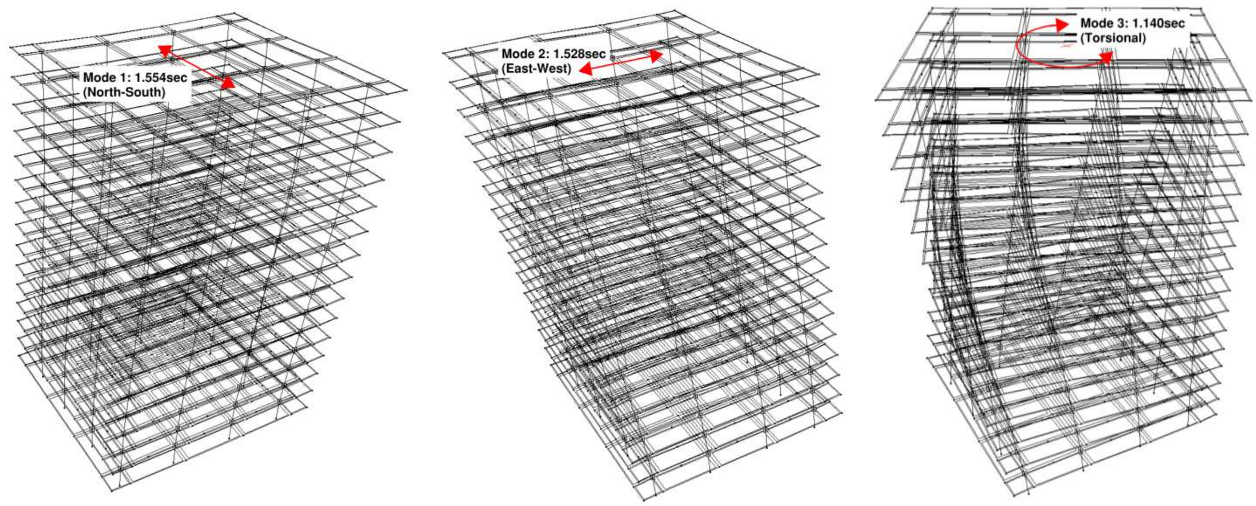


Figure 49: First three mode shapes of the HSE laboratory building.

ELASTIC BEHAVIOR MODES

Table 25: Period ratios for the laboratory building.

	N-S		E-W		Torsional	
Mode	Period	Ratio	Period	Ratio	Period	Ratio
-	sec	-	sec	-	sec	-
1	1.554	1.0	1.528	1.0	1.140	1.0
2	0.548	2.8	0.542	2.8	0.419	2.7
3	0.315	4.9	0.314	4.9	0.248	4.6
4	0.216	7.2	0.217	7.0	0.172	6.6

Note: Pure shear beam period ratios: 1 : 3 : 5 : 7
Pure flexural beam period ratios: 1 : 6 : 18 : 34

Table 26: Period ratios for the mechanical tower.

	N-S		E-W	
Mode	Period	Ratio	Period	Ratio
-	sec	-	sec	-
1	1.029	1.0	1.451	1.0
2	0.247	4.2	0.285	5.1

Note: Pure shear beam period ratios: 1 : 3 : 5 : 7
Pure flexural beam period ratios: 1 : 6 : 18 : 34

One fairly quick method to gain some insight into the global, linear-elastic behavior of a tall structure that is similar to a cantilever beam with distributed mass and elasticity is to determine the modal period ratios. Generally the dynamic properties of tall buildings tend to be the result of a combination of shear and bending deformations. If the tall building exhibits a pure flexural cantilever beam behavior the period ratios are close to 1 : 6 : 18 : 34 and if the tall building behaves like a pure shear beam the period ratios are close to 1 : 3 : 5 : 7.

To evaluate the global behavior of the HSE laboratory building and tower R&C determines the period ratios as described. Table 25 summarizes the period ratios for the laboratory building and Table 26 for the mechanical tower.

We can conclude that the behavior of the laboratory building is governed by pure shear deformations because the period ratios are almost identical with the ones for a pure shear beam. In contrast, the behavior of the mechanical tower is the result of a combination of both shear and flexural deformations. Because the period ratios for the mechanical tower are closer to the ones for a pure flexural beam we can conclude that the mechanical tower is more flexure governed than shear governed in its linear-elastic range.

Inelastic Behavior Modes

To investigate the inelastic behavior of the HSE laboratory building R&C determines plastic hinge rotation demands for different hazard levels and compares them against rotation capacities. We consider the following three hazard levels to determine the sequence in which inelastic deformations occur in the structural building components: 1) The 225-year return period hazard level that corresponds to the serviceability earthquakes, 2) the 475-year return period hazard level that corresponds to the design basis earthquakes (DBE), and 3) the 975-year return period hazard level that corresponds to the maximum considered earthquakes (MCE). For each of the three hazard levels we use Perform-3D to determine the plastic hinge rotation demands and to compute the mean plastic hinge rotations over the seven ground motions.

Figure 50 through Figure 52 depict demand capacity ratios (D/C ratios) for mean plastic hinge rotations of 0.005 radians for the three hazard levels. D/C ratios of an entire element are controlled by the hinge component with the largest D/C ratio inside that element. We find that for the 225-year hazard level the base columns (more specifically the PMM hinges at the base of those columns) are the first components in the overall building structure to undergo inelastic behavior. As can be seen from Figure 50, plastic rotations are on the order of 0.001 radians for all 12 base columns, which we can consider the onset of yielding at the base of the structure. If we increase the hazard level to the 475-year return period, plastic rotations in the base columns increase by a factor of four to 0.005 radians. The corner columns between floors three and four are the next components to undergo inelastic deformations with plastic rotations of the order of 0.0025 radians. In addition, the corner columns up to the seventh floor and the intermediate columns in the east-west frames between floors six and nine are at the onset of yielding for the 475-year hazard level.

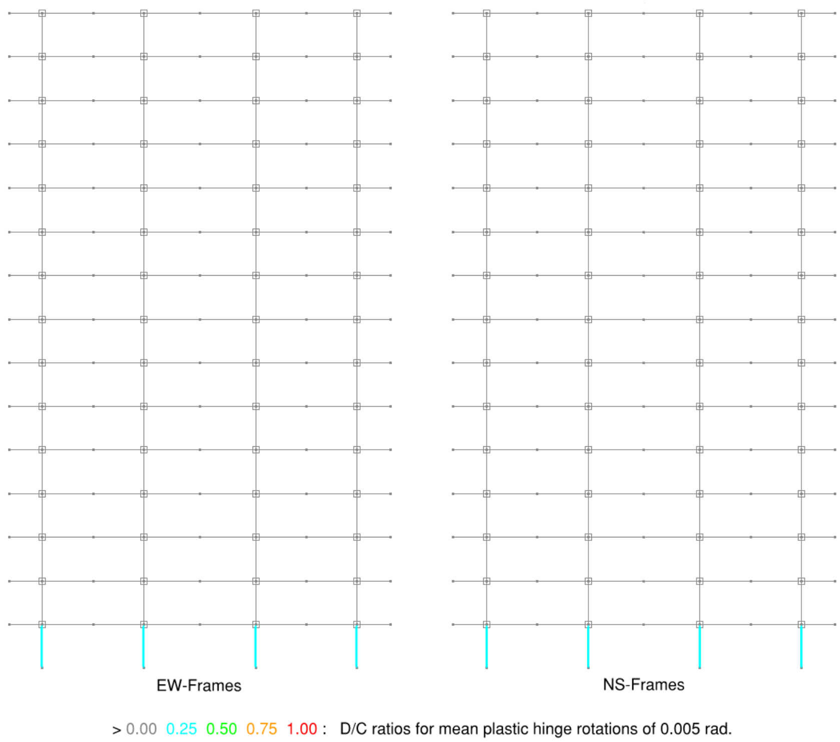


Figure 50: D/C ratios for the mean of the 225 year return period motions.

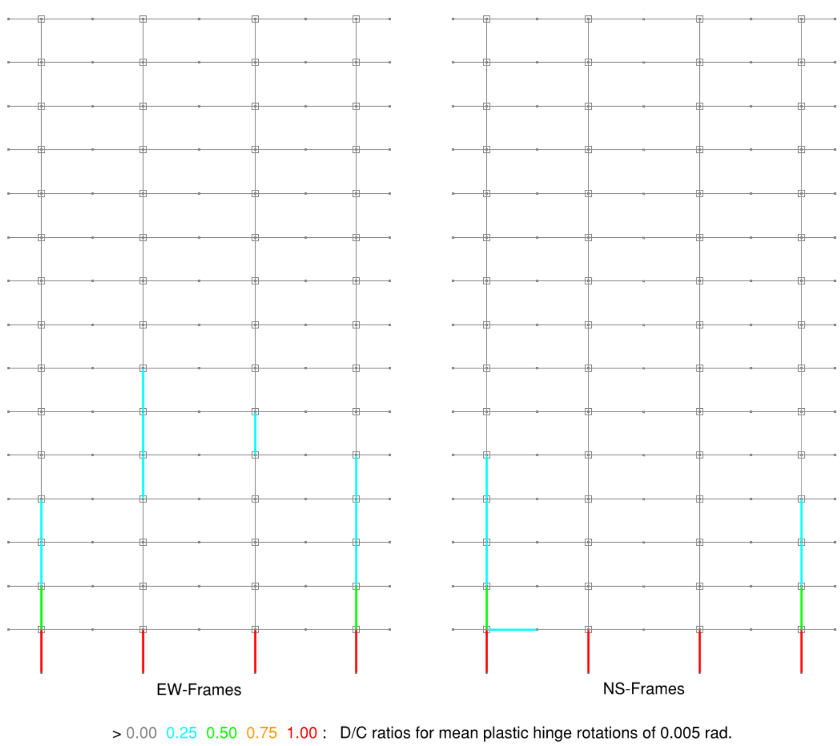


Figure 51: D/C ratios for the mean of the 475 year return period motions.

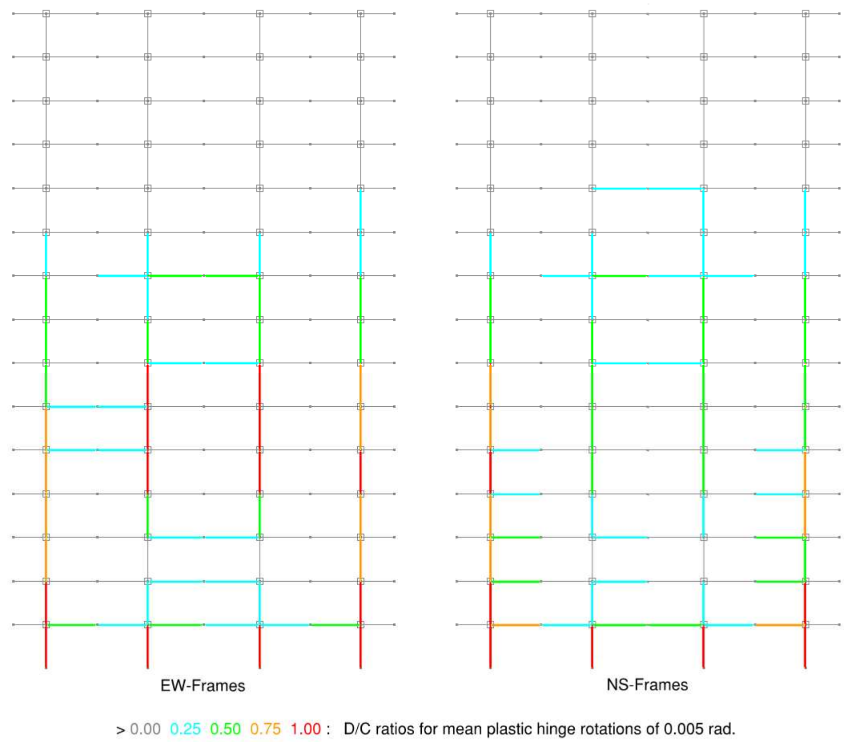


Figure 52: D/C ratios for the mean of the 975 year return period motions.

The reason that the intermediate columns in the east-west frames start yielding before the ones in the north-south frames is due to the girder connections to the weak-axis of the corner columns being stronger than the girder connections to the strong-axis of the corner columns. This means that the intermediate columns in the east-west frames participate more and sooner in the overall frame action because all the girder to column connections are the same and moments are therefore transferred more uniformly to all the columns in the east-west frames.

Figure 52 shows the D/C ratios for mean plastic hinge rotations of 0.005 radians for the MCE hazard level. We find that the corner columns up to floor level nine exhibit moderate inelastic behavior with the second and third story columns exceeding plastic rotation demands of 0.005 radians. The corner columns between floors nine and thirteen and the intermediate columns in the north-south frames up to floor level thirteen experience small plastic hinge rotations or are at the onset of yielding. Similar to the DBE hazard level, the intermediate columns in the east-frame undergo larger plastic hinge rotations than the ones in the north-south frames. From Figure 52 we can see that the intermediate columns between floor levels six and nine exhibit inelastic deformations exceeding 0.005 radians. In addition, several girders form plastic hinges for the 975-year hazard level, especially the girder connections to the strong-axis of the columns up to floor level seven.

In general R&C concludes that the laboratory buildings exhibit inelastic behavior modes where yielding occurs well distributed throughout the building with no soft-story mechanisms forming. The columns in the lower stories (especially the base story) form plastic hinges before the girders, but for the larger hazard levels the girders also form plastic hinges, distributing inelastic deformations well throughout the building. From the MCE hazard level we also conclude that the base story as well as stories six through eight are slightly softer than the remaining stories, meaning that they undergo somewhat larger inelastic deformations than the other stories.

Critical Response Quantities

Several of the measurement indices that are described earlier and that R&C is using to evaluate the behavior of the existing building as well as the proposed retrofitting scopes are discussed next. Figure 53 through Figure 58 summarize response quantities for the existing laboratory building and mechanical tower for the 975-year return period hazard level which corresponds to the maximum considered earthquakes. Typically, response quantities represent SRSS values of the global X and Y direction results except for the separation joint displacements, which are reported in the two perpendicular directions separately.

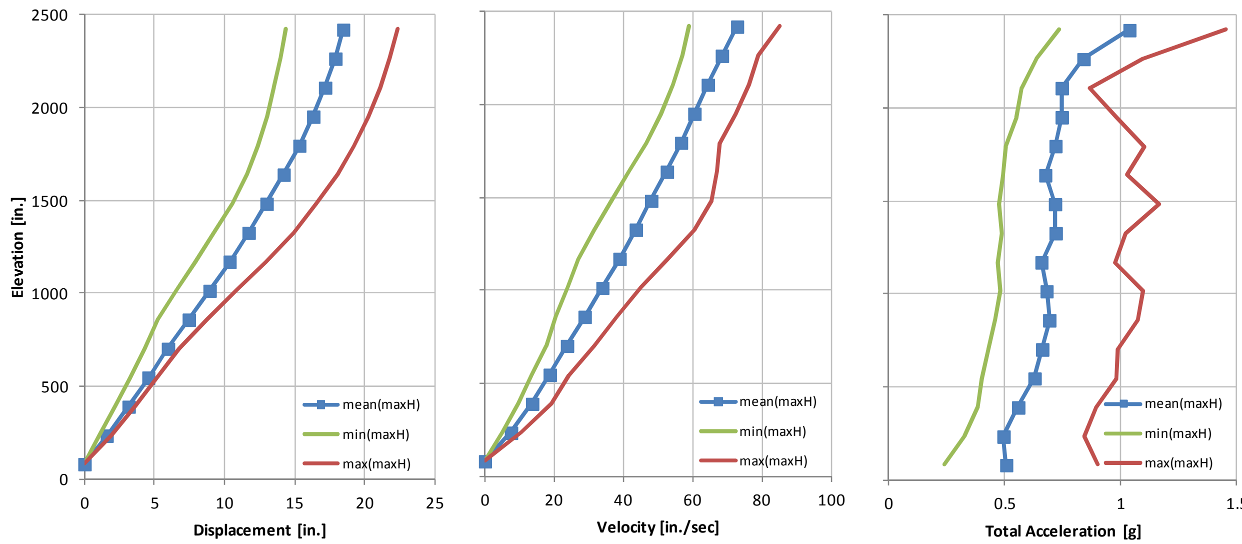


Figure 53: Floor displacements, floor velocities and floor total accelerations.

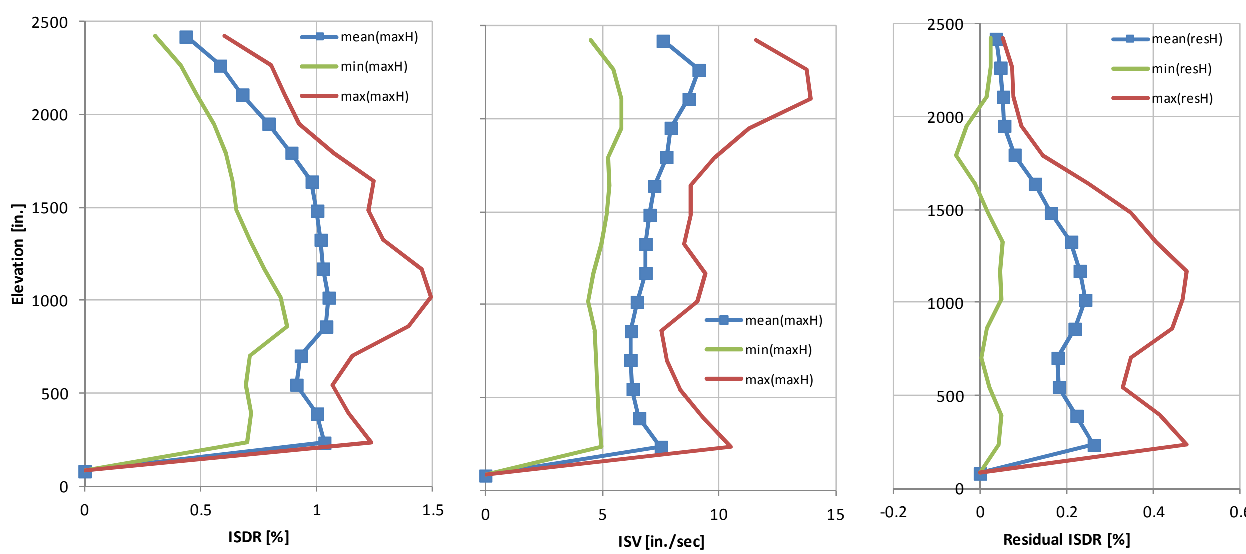


Figure 54: Interstory drift ratios, interstory velocities and residual interstory drift ratios.

We can see from the graphs that the mean SRSS roof displacement is equal to 18.4 inches and the mean SRSS roof velocity is equal to 72.8 in./sec. Mean total SRSS floor accelerations slowly increase from 0.63 g at the fifth floor to 0.75 g at the fifteenth floor. However, the mean total SRSS floor accelerations increase rapidly over the last two floors reaching 1.04 g at the roof level. This means that the laboratory building experiences seismic whipping behavior in the top two floors due to higher mode effects. From Figure 54 we can see that mean SRSS interstory drift ratios for the MCE hazard level are around 1% for most stories up to the eleventh. Above the eleventh story they start decreasing until they reach approximately 0.5% in the sixteenth story. R&C again concludes that the laboratory building does not form any soft stories when subjected to the MCE ground motions. The mean SRSS residual interstory drift ratios are on the order of 0.2 %.

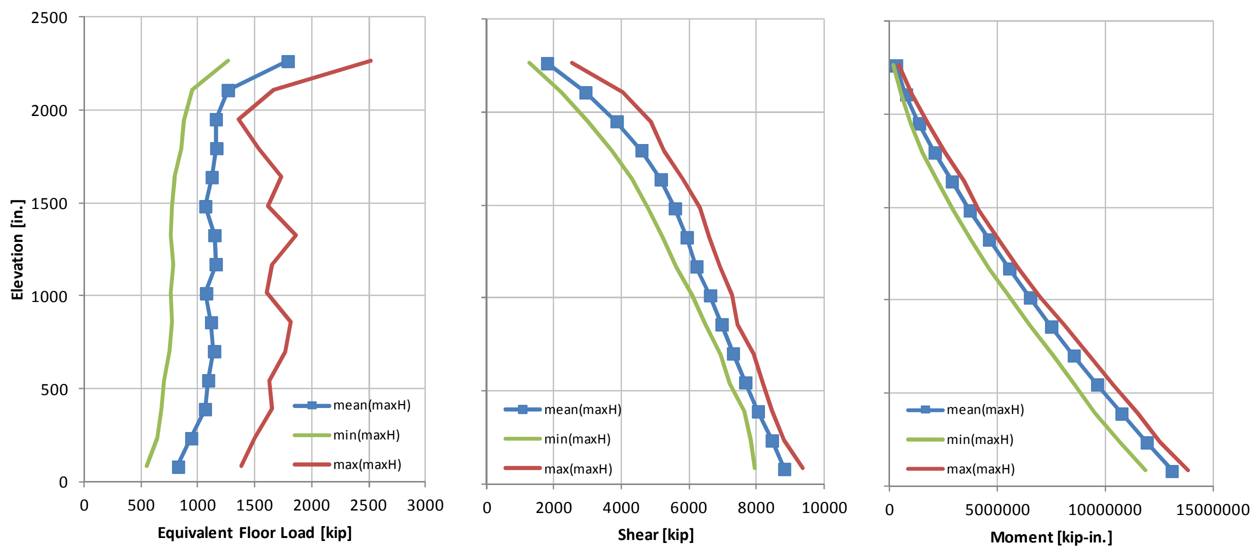


Figure 55: Equivalent seismic floor loads, story shear forces and story overturning moments.

Figure 55 depicts equivalent seismic floor loads, story shear forces and story overturning moments. We can observe that the mean SRSS equivalent seismic floor load distribution closely resembles the mean total SRSS floor acceleration distribution, meaning that story shears are mainly generated by the inertia forces of the floor masses. The mean SRSS base shear for the laboratory structure is equal to 8,815 kips which corresponds to 36% of the seismic weight of the building.

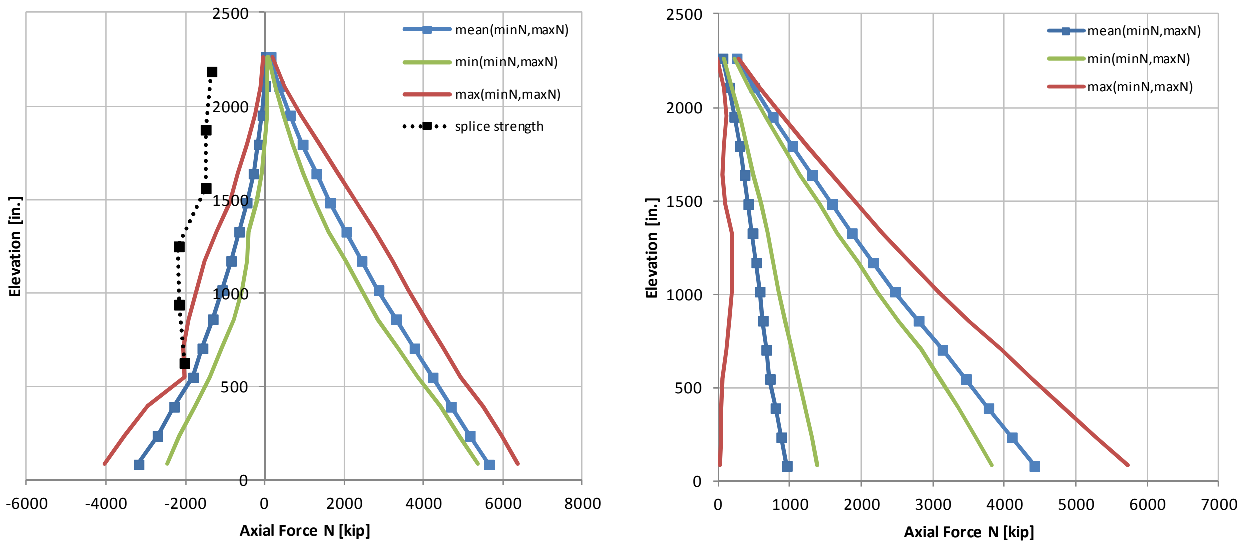


Figure 56: Axial forces for corner columns (left) and intermediate columns (right).

The mean SRSS base overturning moment is equal to 13,077,944 kip-in., which corresponds to the seismic weight of the building applied 536 inches above the base of the building or approximately at the fifth floor.

Figure 56 illustrates the minimum and maximum axial forces in the corner columns and the intermediate columns. On the corner column graph we also show the splice tension strength for the six column splices. R&C finds that for several of the MCE ground motions the corner column splices at Level 5.5 yield in tension and do not close again in compression. For the corner columns mean tension forces at the base are equal to -3182 kips and mean compression forces at the base are equal to 5645 kips. As we can see from the second graph in Figure 56, the intermediate columns experience no tension forces. However, for the most severe of the MCE ground motions gravity compression loads are nearly compensated for by the seismic tension forces.

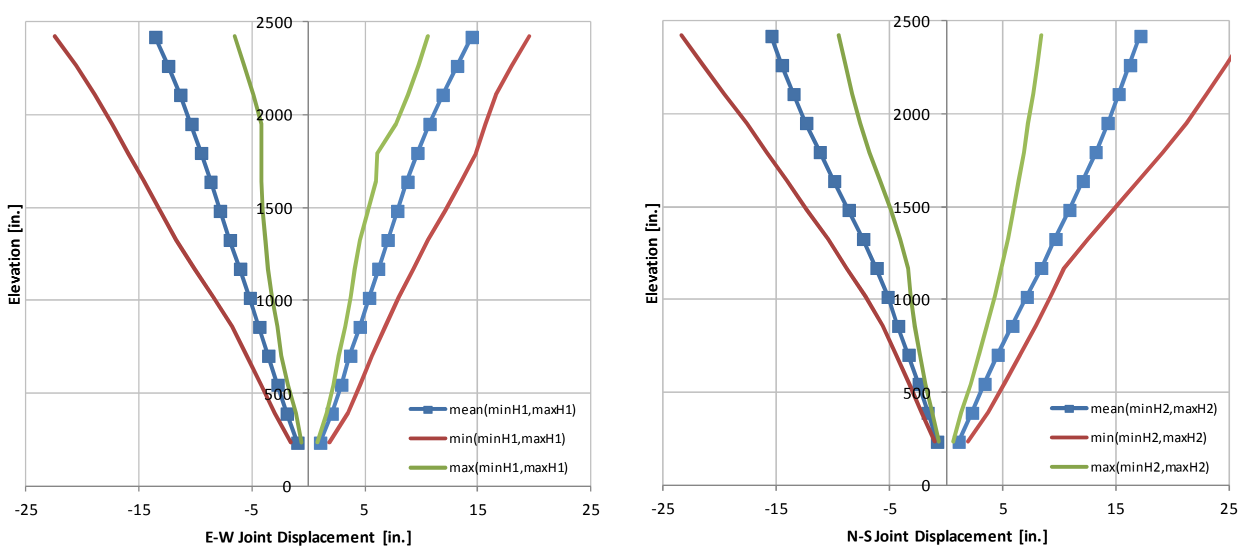


Figure 57: Separation joint displacements in E-W and N-S directions.

To evaluate the relative movement at building separation joints and asses if utilities contained in or crossing such joints would be damaged at the MCE hazard level, R&C computes the relative displacements between the laboratory building and the mechanical tower from the NLRH analyses. Joint opening and closing are reported in the first graph in Figure 57 and transverse displacements are reported in the second graph. We can see that at the roof level of the laboratory building the seismic separation joint closes or opens 14.5 inches on average and shears 17.1 inches on average. For the most severe of the MCE ground motions, joint displacements are on the order of 25 inches. R&C concludes that seismic separation joints close entirely and are also subjected to very large shearing deformations at the MCE hazard level. This would clearly damage utilities contained in or running across these separation joints during an MCE event.

To evaluate laboratory contents we plot mean SRSS pseudo acceleration floor spectra for floor levels 2, 7, 12 and 17 in Figure 58. Spectra are plotted for damping values of 1%, 2%, 5% and 10%. However, most laboratory contents will have very little inherent damping and R&C suggests to use the 1% and 2% curves. We can see that pseudo accelerations peak at frequencies of 0.62 Hz, 1.78 Hz and 3.21 Hz, which closely correspond to the first three modes of the laboratory building. Pseudo accelerations at these peaks are the largest at the roof level and reach 8.5 g at 1.78 Hz for the 1% damped case.

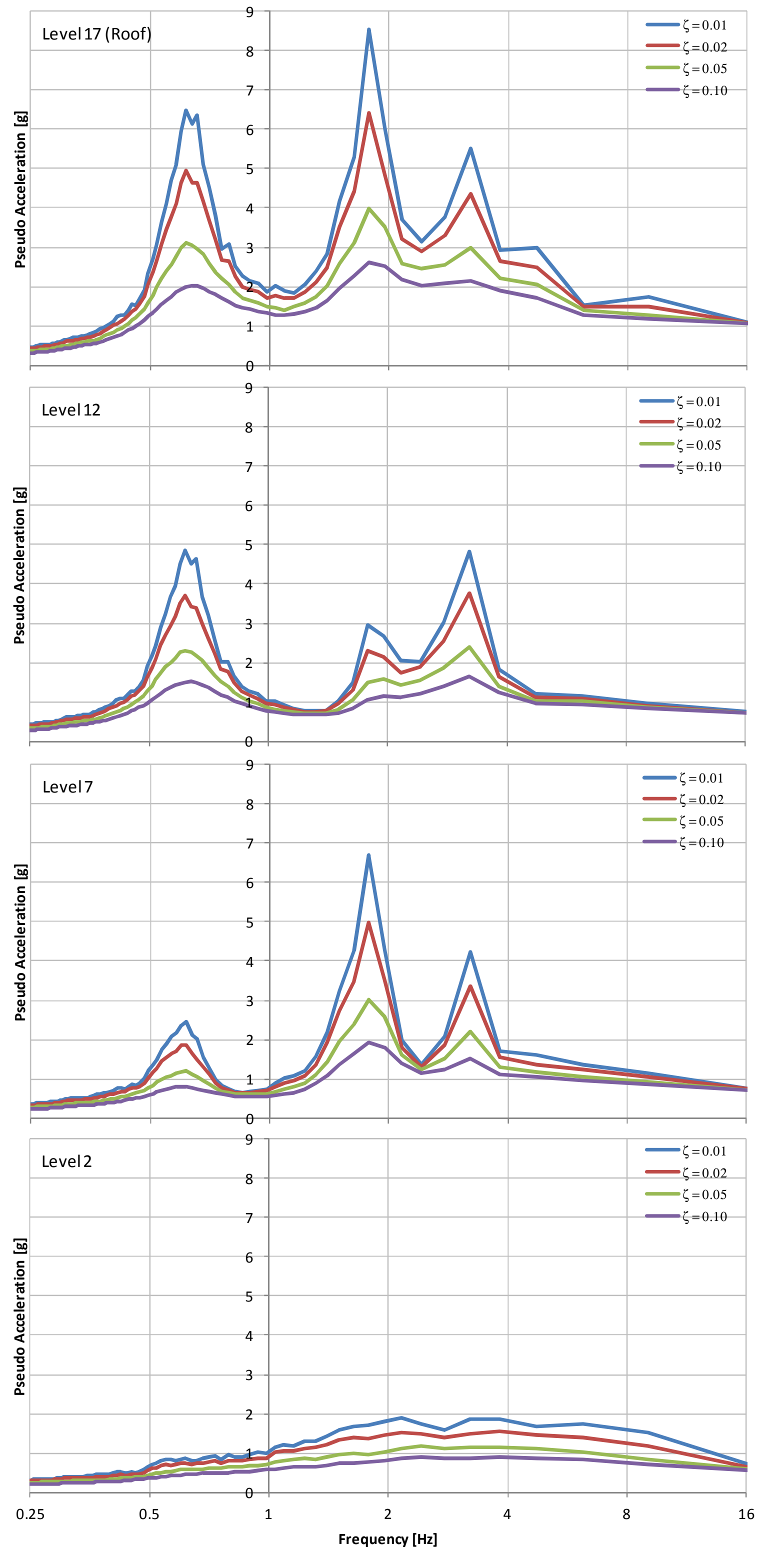


Figure 58: Pseudo acceleration floor spectra for levels 17, 12, 7 and 2.

4.7. SUMMARY OF POTENTIAL SEISMIC-STRUCTURAL DEFICIENCIES

Table 27: Potential seismic-structural deficiencies and findings.

Potential Seismic Deficiency	How Evaluated	Finding
Relative movement at building separation joints (damaging utilities contained in or crossing the joint).	NLRH analysis to determine amount of joint opening, closing, and transverse relative displacement for all levels of ground motion input.	Joints will entirely close in the largest ground motions. Retrofitting at joints is recommended. This can included retrofitting the utilities and/or cutting away some structure. At the mechanical towers, dampers across joints could greatly mitigate this deficiency.
Column splices	NLRH analysis with explicit nonlinear modeling of splice yielding and residual deformation.	In the largest ground motions, splices in the four corner columns will yield in tension and may not close again in compression. We expect the splice yielding to be ductile, but a brittle fracture could possibly occur. Could lead to yellow or red tag but unlikely to lead to collapse, since 8 of the 12 columns are interior and do not have high splice demands. Issue applicable at levels 5.5 and 7.5 only.
Mechanical tower structure, potential shear critical wall behavior	Hand calculations and explicit NLRH analyses. Capacities per FEMA 306 approach. Assume composite action between steel and concrete.	Concrete walls in some directions of motion are likely to be shear governed or flexure-shear governed. Retrofit by tower damping can make walls more likely to be flexure governed. Concrete shear failures should be less consequential because of the complete steel frame. Possible retrofit using shotcrete or horizontal fiber wrap.
Elevator tower structure, potential shear critical wall behavior	Hand and spreadsheet calculations to establish strength and behavior modes. Capacities per FEMA 306 approach. Assume composite action between steel and concrete.	No retrofit needed. Concrete walls in the north-south direction are flexure governed. Those in the east-west direction are shear governed. In both directions lateral strength is such that behavior will be linear in all but the strongest earthquakes.
Elevator tower structure, plan torsion and performance of steel moment frames.	Hand and spreadsheet calculations Considering comparative strength and stiffness between moment frames and concrete walls, and considerations of centers of mass, stiffness, and shear.	No retrofit needed. Stiffness and strength of concrete walls moderate the demands on the steel moment frames and limit the amount of deformation in plan torsion.
Potential story concentration or mechanism in laboratory buildings	NLRH of laboratory structure	The rare ground motions that collapse the structure do involve a story concentration, with different motions tending to affect different stories. Without retrofit, the collapse probability is low—comparable to a UC Level III (Good) rating. Retrofit with damper frames improves collapse probability.
Riveting of column flanges cannot transfer full column moment capacity	Modification of column capacity in the NLRH analysis	This effect is accounted for in the analysis and is assumed to be reasonably ductile; no specific retrofit is needed for this issue.
Panel zone and beam connection capacity	Comparative strength evaluation of all possible yielding or failure modes for the panel zones and beam connections	This effect is accounted for in the analysis and is assumed to be reasonably ductile; no specific retrofit is needed for this issue.

Table 27 summarizes the principal structural characteristics and details of the buildings that represent potential seismic deficiencies to be evaluated. The table summarizes how the deficiencies are evaluated and what our finding is for each potential deficiency. As shown in the last column of the table, retrofitting is recommended to address the relative movement at building separation joints, and to address column splices. Retrofitting against the shear governed behavior of the concrete walls in the mechanical tower, might also be considered.

None of the potential deficiencies of the structure require retrofitting to meet California Building Code requirements, based on our evaluation summarized below.

Ability to Meet California Building Code, Chapter 34 without Structural Retrofit

Chapter 34 of the building code requires evaluating structural performance at two separate levels of seismic hazard. At the *BSE-R* hazard level, associated with a 225-year return period ground motion, the Code requires *Life Safety (S-3)* structural performance. At this performance level, it is assumed that the structure may have experienced significant damage, but that it maintains some residual strength and stiffness capacity to resist collapse. Although some injuries may occur, it is assumed that the risk of life threatening injury is low. Our analyses of the existing structure at this ground motion level indicate that it meets the required performance.

The second requirement of Chapter 34 is that, at the *BSE-C* hazard level, which is associated with a 975-year return period ground motion, the structure is required to satisfy *Collapse Prevention (S-5)* requirements. This performance level is used to describe a structure which has undergone significant damage, but retains its vertical-load carrying capacity but with no extra margin of strength or stiffness to resist collapse. Our evaluation finds that the existing structure without retrofitting meets this standard.

For 975-year ground motions there may be some yielding and residual deformation of the column splices. We expect the splice yielding to be ductile, but a brittle fracture could possibly occur. Fracture or excessive residual deformation at the splice could lead to a yellow tag or red tag posting of the building after a large earthquake, but we conclude that such damage is unlikely to lead to collapse. This partly because high splice demands can only occur at the four corner columns, and the other eight of the 12 columns are interior and do not have high splice demands.

4.8. Potential Structural Retrofit Measures

While the structure does not need retrofitting to meet the minimum requirements of the California Building Code for existing buildings, we have defined retrofit measures that can substantially improve the buildings seismic performance related to UCSF's desire for resilience and continued operation of research following an earthquake. These measures address column splices, dampers at the mechanical tower separation joints, and damper frames throughout the laboratory buildings.

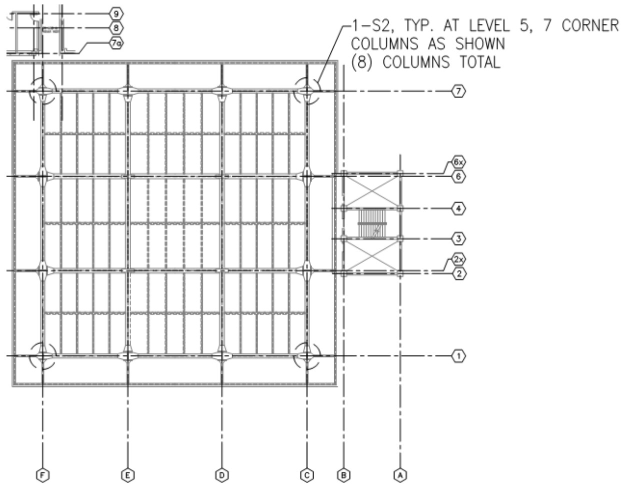


Figure 59: Typical corner column splice retrofit location at Health Sciences East. Health Sciences West is similar.

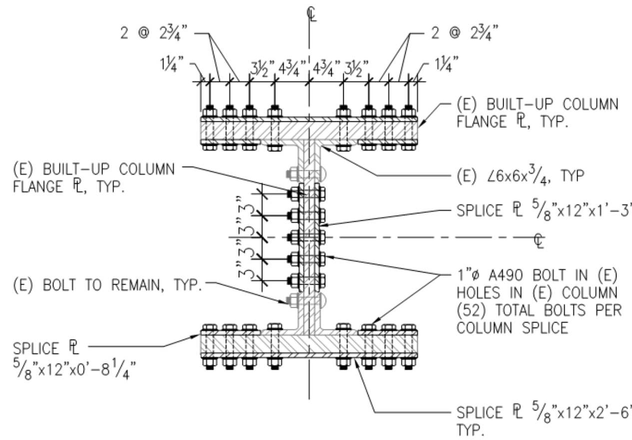


Figure 60: S1: *Column Splice* retrofit section.

COLUMN SPLICE RETROFIT (RETROFIT SCOPE S1)

The existing splices consist of A373 plates bolted to each side of the column flange plates and web plate. Our analysis shows that the splices are limited by yielding of the gross section, a relatively ductile mechanism. However, the splices will experience multiple cycles of tension during larger earthquakes. Elongation or fracture of the splice plate under low-cycle fatigue may lead to concentrations of damage in frames or increase the likelihood of disruption due to yellow or red tag following an earthquake.

The *S1: Column Splice* retrofit scope strengthens the splices of the four corner columns of each laboratory building at Levels 5.5 and 7.5. The scope includes removing the existing bolted splice plates and replacing them with new 5/8" A572 Grade 50 plate, connected with 1" diameter A490 bolts in the existing bolt holes. The revised connection is sized such that the plates will experience gross section yield rather than an undesirable brittle failure mechanism. The retrofit occurs at eight total locations around the perimeter of each building, and can be done with the building occupied.

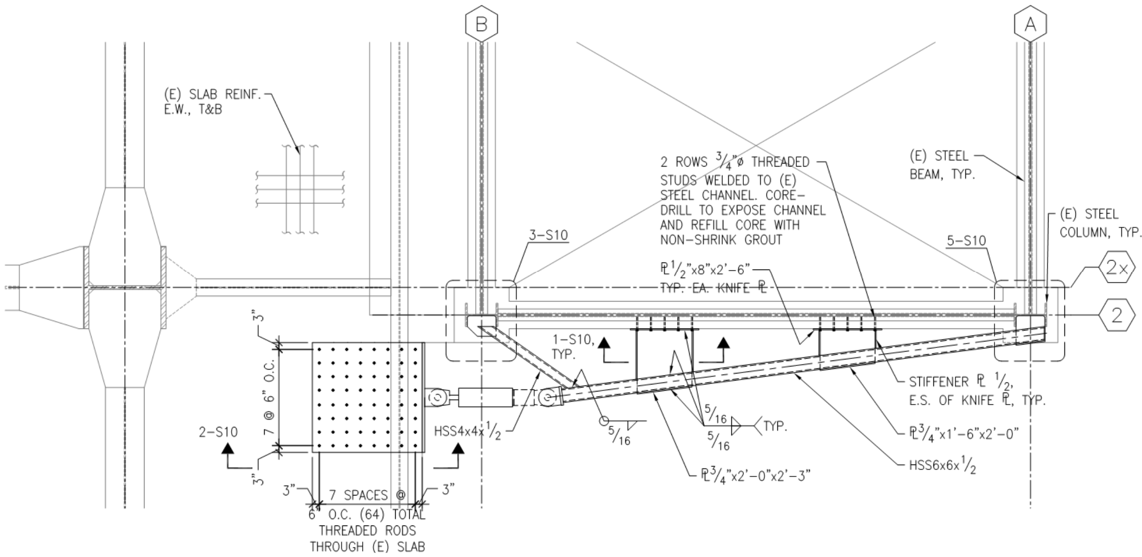


Figure 61: ***S2: Mechanical Tower Dampers*** **retrofit partial plan. This detail is oriented identical to Figure 59, with the half of the mechanical tower shown at right and the laboratory building steel framing shown at left.**

MECHANICAL TOWER DAMPERS (RETROFIT SCOPE S2)

The *S2: Mechanical Tower Damper* retrofit scope places fluid viscous dampers between the mechanical tower and the adjacent laboratory building in order to protect critical building systems located in the separation joint between the two structures. Our design provides dampers at four levels somewhat concentrated toward the top of the buildings: Roof Level and Levels 15, 12, and 8. The retrofit occurs at each mechanical tower with a damper on each side of each tower located and supported based on two criteria: (a) that the dampers, working together, provide a force that initially acts concentrically with the centers of stiffness of the buildings on either side, and (b) that the dampers accommodate without interference the maximum relative joint movements in opening closing, and shearing deformation.

This leads to a design in which each damper is attached on one end to a new external horizontal steel frame attached to the side of the mechanical tower as shown in Figure 61. The new frame is connected to the existing steel beams and columns of the mechanical tower using horizontal gusset plates. The other end of the damper is connected to the perimeter of the adjacent laboratory building, using steel plates through-bolted above and below the reinforced concrete "eyebrow" slab that is outside the window wall. The fluid viscous dampers are pinned at each end.

Another benefit of this retrofit measure is that the structural retrofit work occurs almost entirely outside of the building envelope and therefore minimizes disruption to building occupants.

The fluid viscous damper acts to dissipate seismic energy present in the differential motion of the laboratory building and the mechanical tower. Our analyses indicate that the laboratory building deflects primarily in a shear mode, while the mechanical tower deflects largely in a flexural mode. During the course of an earthquake simulation, the two buildings move out-of-phase, which causes differential motion at the separation joint. The fluid viscous dampers act to limit this differential motion, helping the buildings move in-phase.

The force in fluid viscous dampers is dependent on the relative velocity between the two ends of the device. As a result, the addition of dampers to the building does not change the stiffness properties of the structure. The dampers decrease drifts and accelerations, generally without increasing forces on the structure, because the damping-force response is out of phase with the building deflections. Our design is based on using viscous dampers with a nonlinear velocity relationship (coefficient $\alpha = 0.4$). Such dampers have the property of limiting the maximum force that can be transmitted through the device, which reduces the possibility of overloading the damper or support structure in larger earthquakes.

By helping the buildings move in-phase, the dampers greatly limit damage to building utilities that are located in the separation joint between the mechanical tower and the laboratory building. Our analyses show that the dampers (in the direction that they act) also slightly decrease the overall laboratory building deflections and accelerations. This improvement helps to protect utilities and contents throughout the laboratory building. Our analyses have indicated that the vertical location of the dampers over the height of the mechanical tower affects the overall system performance. Placing dampers lower would provide additional benefit by further reducing the laboratory building response, but it would increase shear demands at the base of the mechanical tower. Our design, with dampers at the Roof Level and Levels 15, 12, and 8 balances the benefit to laboratory building performance with modestly increasing the shear demand on the mechanical tower.

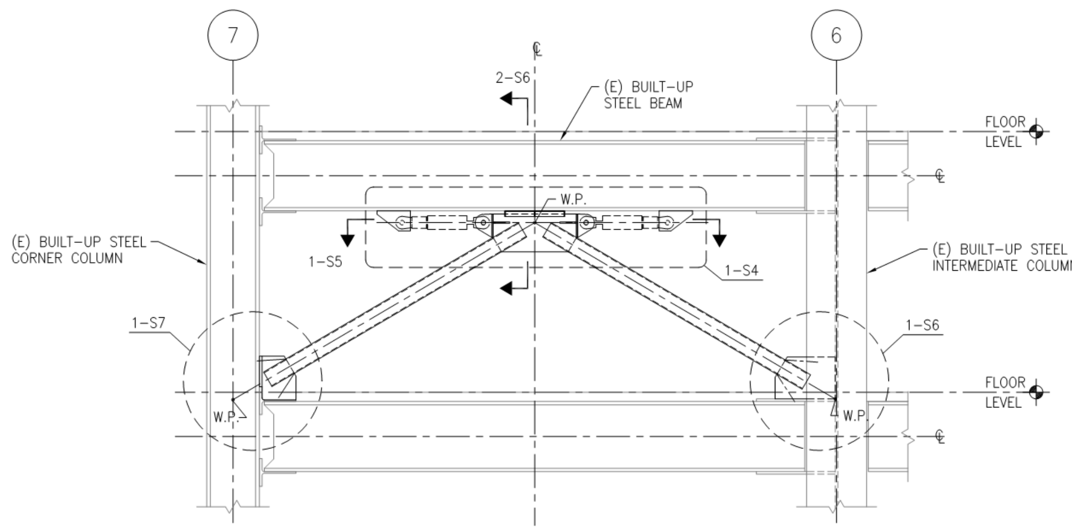


Figure 62: Typical frame elevation of S3: *Laboratory Damper Frame* retrofit. Four total similar frame bays occur at each level of each laboratory building.

Laboratory Damper Frames (Retrofit Scope S3)

The retrofit scope *S3: Laboratory Damper Frames* places four damping frames around the perimeter of each level of the laboratory buildings. The frames substantially reduce seismic drifts and accelerations, which reduces the likelihood of damage to the building frame in an earthquake. Because the damper frames work to decrease building response, without adding strength or stiffness, the retrofit also helps to protect building utility and laboratory components sensitive to drift or floor accelerations.

The damper frames consist of a chevron HSS tube frame, connected at its base with gusset plates to the beam-column intersection. As opposed to a traditional braced frame, the damper frame does not connect directly to the beam above. Rather, the gusset plate at the top of the chevron links the frame to a pair of fluid viscous dampers which are in turn connected to the beam above. As the building drifts in an earthquake, the upper floor moves relative to the damper frame, which is rigidly attached to the level below. This relative motion between floor levels causes the dampers to extend and compress, dissipating energy and damping the buildings motion.

The retrofit scope adds four damping frames to each steel-framed level of each laboratory building, with one damper frame at each side of the perimeter. Each side of the building has three bays, which are potential locations for the damper frame. Considering the architectural layout, SmithGroup has identified preliminary locations for the damper frames in the Health Sciences East Building in order to mitigate functional conflicts.

For the detailing of the damper frames we have worked with Cast Connex Corporation and confirmed the feasibility of a prefabricated cast connection for the top of the chevron, as an alternative to a shop and field fabricated assembly of gusset plates. The cost estimate for this retrofit scope considers options for both the cast and shop-fabricated connections. Complete details of this retrofit scope are presented in Appendix A.

Similar to the *S2: Mechanical Tower Damper* retrofit scope, we propose to use fluid viscous dampers with a nonlinear force versus velocity relationship ($\alpha = 0.4$) which protects the building frame by limiting the maximum force which can be transmitted by damper components. Because the damper force is dependent on interstory velocity, which is out-of-phase with building drifts, the demands on most of the structural elements in the building are reduced with this retrofit scope.

The damper frames can be installed individually while the building is occupied. The work to install the frames includes the local removal of finishes and fireproofing, the delivery to each installation location of the dampers and steel pieces (possibly in sections via elevator), the welded and bolted assembly, and the restoration of finishes. A relatively small work area and lay-down area is required around each installation.

4.9. Analysis Findings for Structural Retrofit Scopes

In this section R&C summarizes several measurement indices that we extract and process from Perform-3D NLRH analysis models of the three proposed structural retrofit scopes. To simplify comparisons among the existing structure and the retrofit scopes we present the same set of measurement indices that we discussed in Section 4.6 for the existing structure.

Column Splice Retrofit (Retrofit Scope S1)

For the column splice retrofit scope we find that by strengthening the corner column splices at Levels 5.5 and 7.5, the splices no longer yield in tension when subjected to the MCE hazard level. As we can see from Figure 63, the new tension splice strength curve (black dotted line) is now outside the most severe tension demand curve (solid red line), meaning that for the 975-year return period hazard level all of the column splices have sufficient strength to remain elastic.

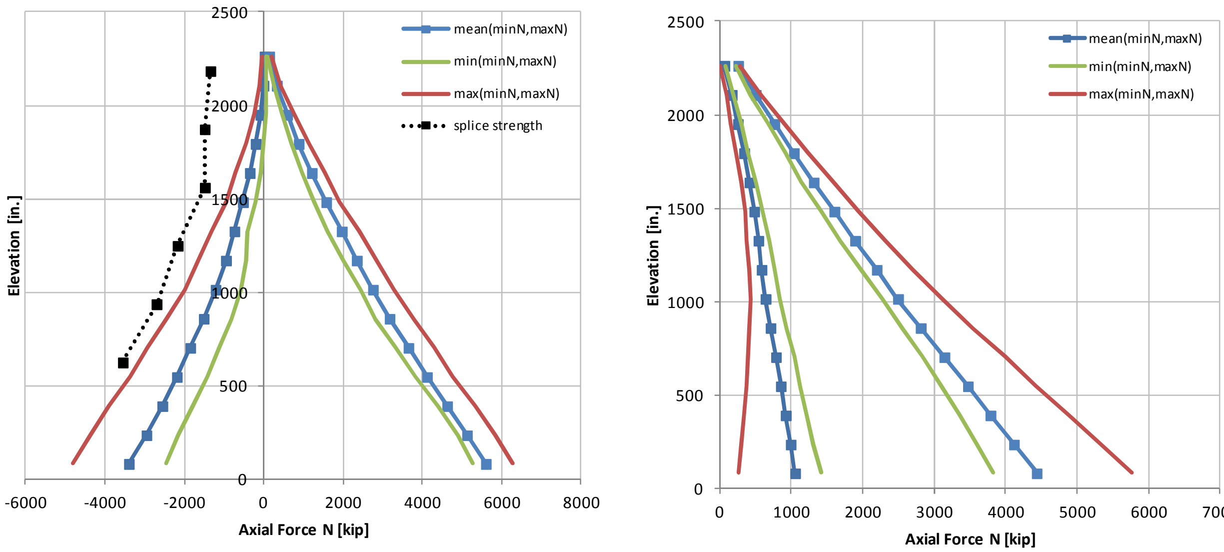


Figure 63: Axial forces for corner columns (left) and intermediate columns (right).

We find that all other measurement indices remain the same as for the existing structure. Hence, we do not repeat them here, but refer the reader to look at Section 4.6 instead.

MECHANICAL TOWER DAMPERS (RETROFIT SCOPE S2)

Figure 64 through Figure 70 summarize response quantities for the mechanical tower dampers retrofit scope. As before, we report measurement indices for the 975-year return period hazard level which corresponds to the maximum considered earthquakes. Typically, response quantities represent SRSS values of the global X and Y direction results except for the separation joint displacements, which are reported in the two perpendicular directions separately.

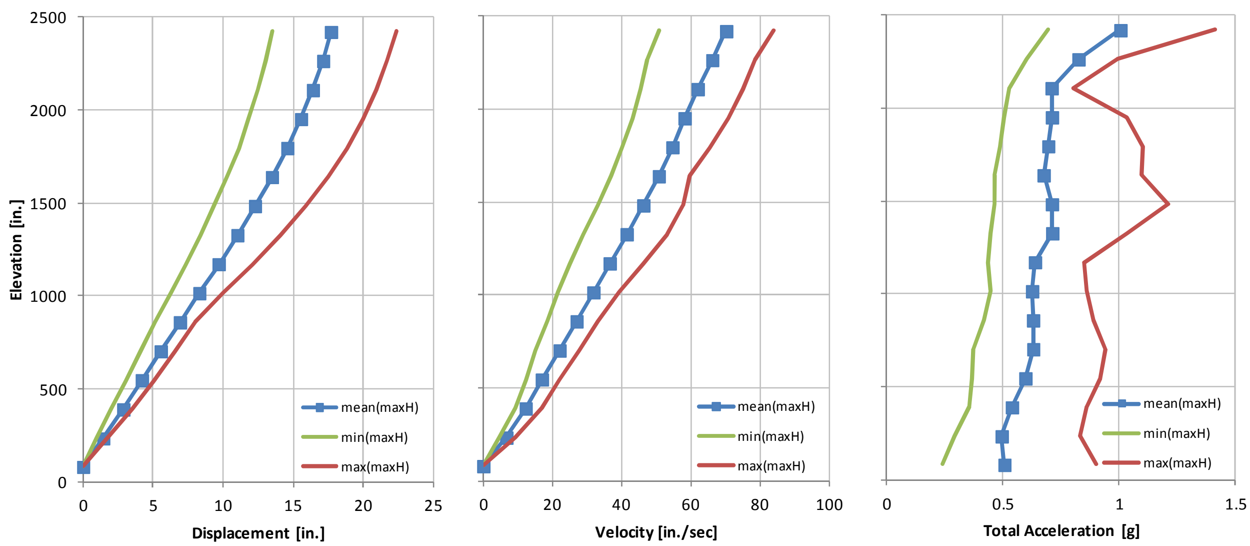


Figure 64: Floor displacements, floor velocities and floor total accelerations.

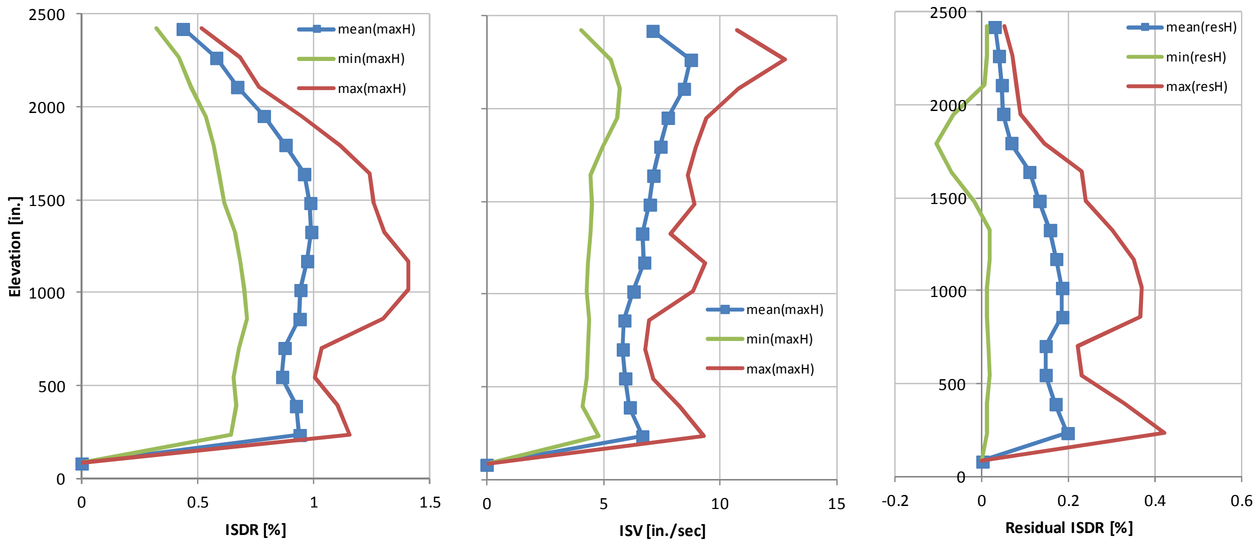


Figure 65: Interstory drift ratios, interstory velocities and residual interstory drift ratios.

We can see from Figure 64 that the mean SRSS roof displacement is equal to 17.7 inches which is a slight reduction from the 18.4 inches in the un-retrofitted structure. The mean SRSS roof velocity reduces from 72.8 in./sec to 70.2 in./sec. Mean total SRSS floor accelerations slowly increase from 0.60 g at the fifth floor to 0.71 g at the fifteenth floor. Mean total SRSS accelerations for the top two floors are again much higher and reach 1.00 g at the roof level. We can conclude that accelerations are overall slightly reduced as compared to the existing structure but higher mode whipping effects are not alleviated by this retrofit scope. From Figure 65 we can see that mean SRSS interstory drift ratios for the MCE hazard level are still around 1% for most stories up to the eleventh. Above the eleventh story they start decreasing till they reach approximately 0.5% in the sixteenth story. R&C again concludes that the laboratory building does not form any soft stories when subjected to the MCE ground motions. The mean SRSS residual interstory drift ratios are on the order of 0.2 %.

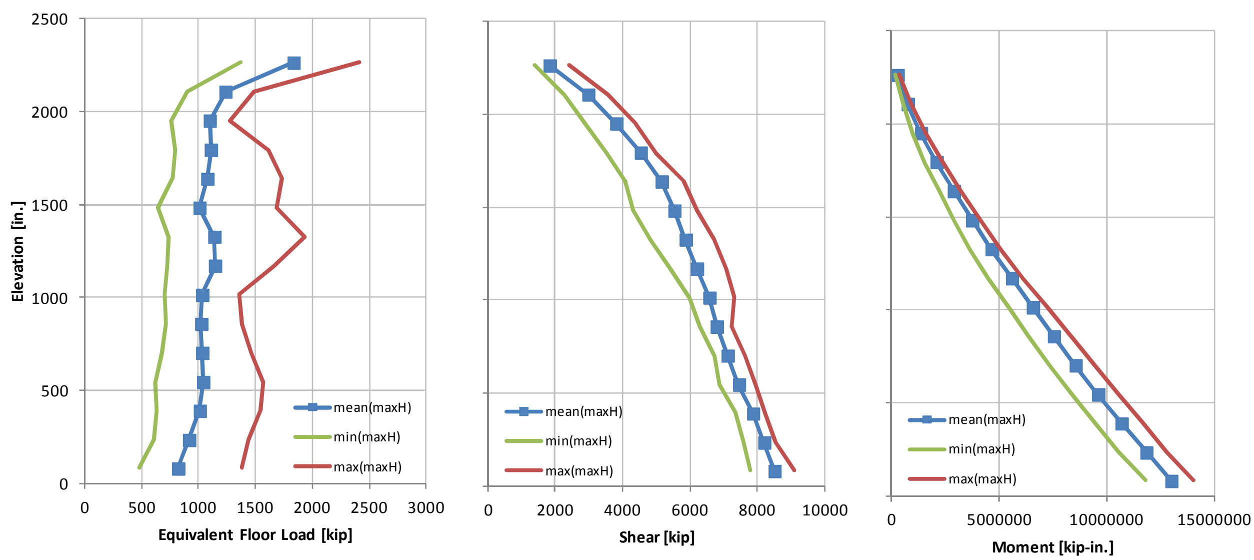
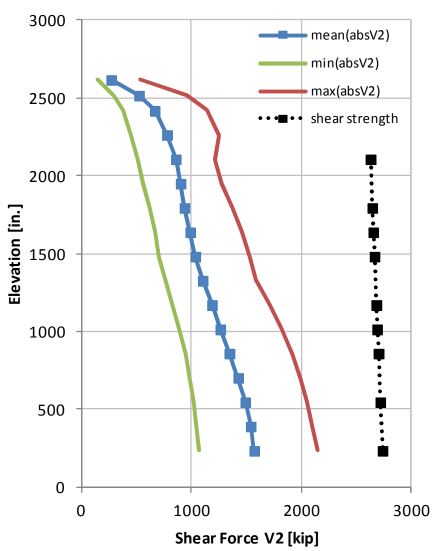


Figure 66: Equivalent seismic floor loads, story shear forces and story overturning moments.

Figure 66 depicts equivalent seismic floor loads, story shear forces and story overturning moments. As with the existing structure we can observe that the mean SRSS equivalent seismic floor load distribution closely resembles the mean total SRSS floor acceleration distribution, meaning that story shears are mainly caused by the inertia forces of the floor masses. The mean SRSS base shear for the laboratory structure is equal to 8,506 kips which corresponds to 35% of the seismic weight of the building. The mean SRSS base overturning moment is equal to 12,993,081 kip-in., which corresponds to the seismic weight of the building applied 532 inches above the base of the building or approximately at the fifth floor. R&C concludes that story shears and overturning moments in the laboratory building are slightly reduced by this retrofit scope that places fluid viscous dampers between the mechanical tower and the laboratory building.

However, we find that mean shear demands in the mechanical tower increase in the direction of the dampers because of the force transfer that is occurring across the dampers from the laboratory building into the mechanical tower. Figure 67 shows the comparison of the shear distributions in the mechanical tower for the existing building (on the left) and retrofit scope S2 (on the right). Mean base shears in the weak-axis direction of the mechanical tower increase from 1569 kips for the existing structure to 1906 kips for the retrofit scope S2. The increase in shear is equal to 337 kips which is almost identical to the 309 kip reduction of the mean base shear in the laboratory building.

Figure 67 also shows the shear strength distribution over the height of the mechanical tower as a dotted black line. We can see that the shear strength at the base of the mechanical tower exceeds the largest shear demand caused by the most severe of the MCE ground motions. R&C concludes that the shear strength of the mechanical tower is sufficient to make the mechanical tower damper retrofit scope feasible.



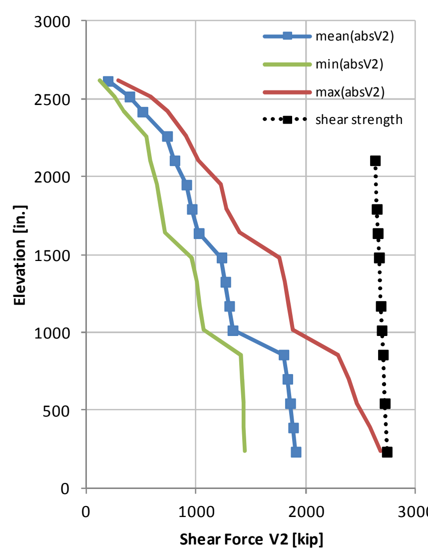


Figure 67: Mechanical tower story shear forces for the existing configuration (left) and mechanical tower damper retrofit (right).

Figure 68 illustrates the minimum and maximum axial forces in the corner columns and the intermediate columns. On the corner column graph we also show the splice tension strength for the six column splices. If retrofit scope S2 is selected R&C recommends that it is applied in combination with retrofit scope S1. The figures in this section reflect that recommendation by including retrofit scope S1 in the results presented here. As for retrofit scope S1, we can therefore conclude that the tension splice strength curve of the retrofitted corner column splices (black dotted line) is outside the most severe tension demand curve (solid red line), meaning that for the 975-year return period hazard level all of the column splices have sufficient strength to remain elastic.

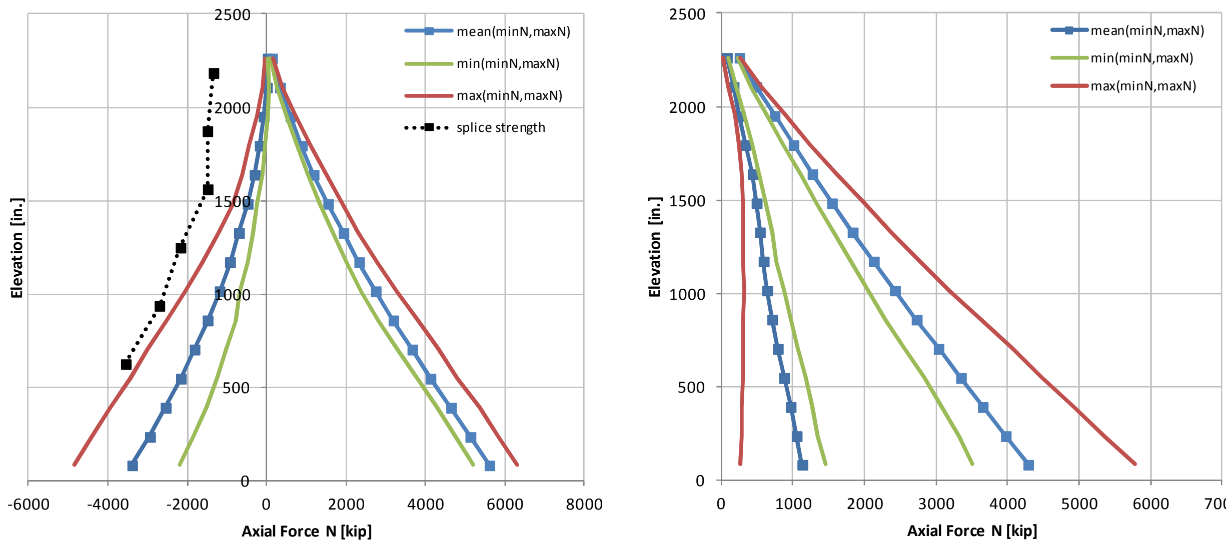


Figure 68: Axial forces for corner columns (left) and intermediate columns (right).

To evaluate the relative movement at building separation joints and asses if the mechanical tower damper retrofit scope is effective in reducing joint displacements in the direction of the dampers (E-W direction), R&C again computes the relative displacements between the laboratory building and the mechanical tower from the NLRH analyses. Joint opening and closing are reported in the first graph in Figure 69 and transverse displacements are reported in the second graph. We can see that at the roof level of the laboratory building the seismic separation joint closes or opens 2.2 inches on average and shears 17.1 inches on average. The largest mean joint opening or closing is equal to 3.3 inches and occurs at the ninth floor level. For the most severe of the MCE ground motions, the maximum joint closing or opening is equal to 6.5 inches and occurs at the tenth floor level. Transverse shearing deformations are nearly on the same order as for the un-retrofitted structure. R&C concludes that the proposed retrofit scope S2 is effective in reducing joint closing and opening deformations to levels where utilities contained in the seismic separation joints are protected from damage and utilities running across the seismic separation joints can be designed to accommodate such deformations.

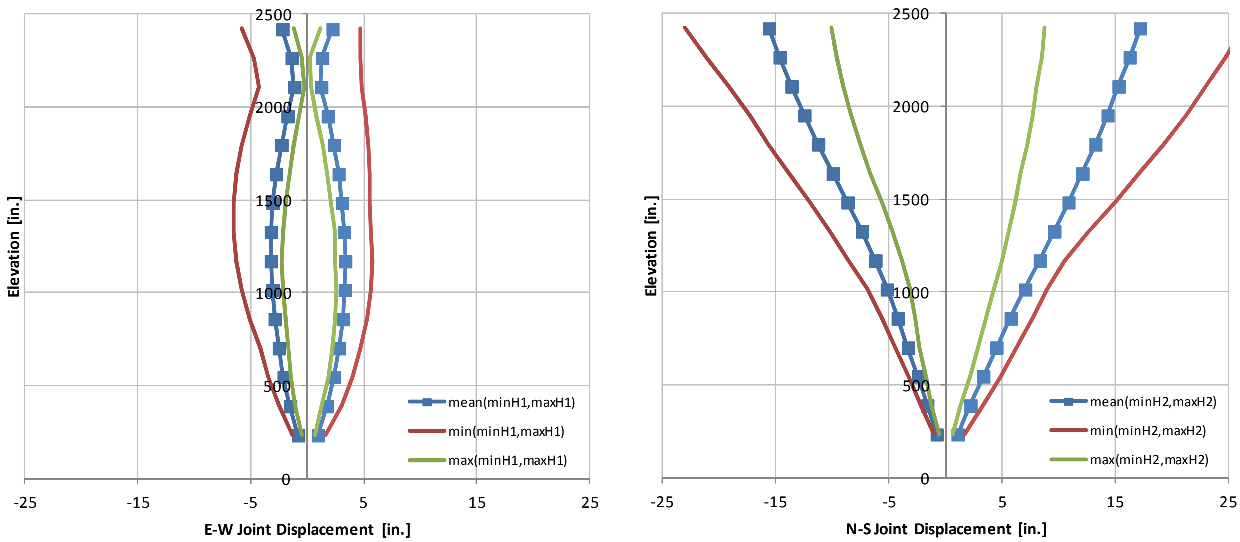


Figure 69: Separation joint displacements in E-W and N-S directions.

To evaluate laboratory contents we again plot mean SRSS pseudo acceleration floor spectra for floor levels 2, 7, 12 and 17 in Figure 70. Spectra are plotted for damping values of 1%, 2%, 5% and 10%. However, most laboratory contents will have very little inherent damping and R&C recommends to use the 1% and 2% curves. We can see that pseudo accelerations peak at frequencies of 0.62 Hz, 1.78 Hz and 3.21 Hz, which closely correspond to the first three modes of the laboratory building. Pseudo accelerations at these peaks are the largest at the roof level and reach 7.8 g at 1.78 Hz for the 1% damped case. This is nearly a 10% reduction from the largest mean SRSS pseudo acceleration recorded in the un-retrofitted laboratory building. R&C concludes that most pseudo acceleration demands for the 1% damped case are reduces by approximately 10% by the mechanical tower damper retrofit scope.

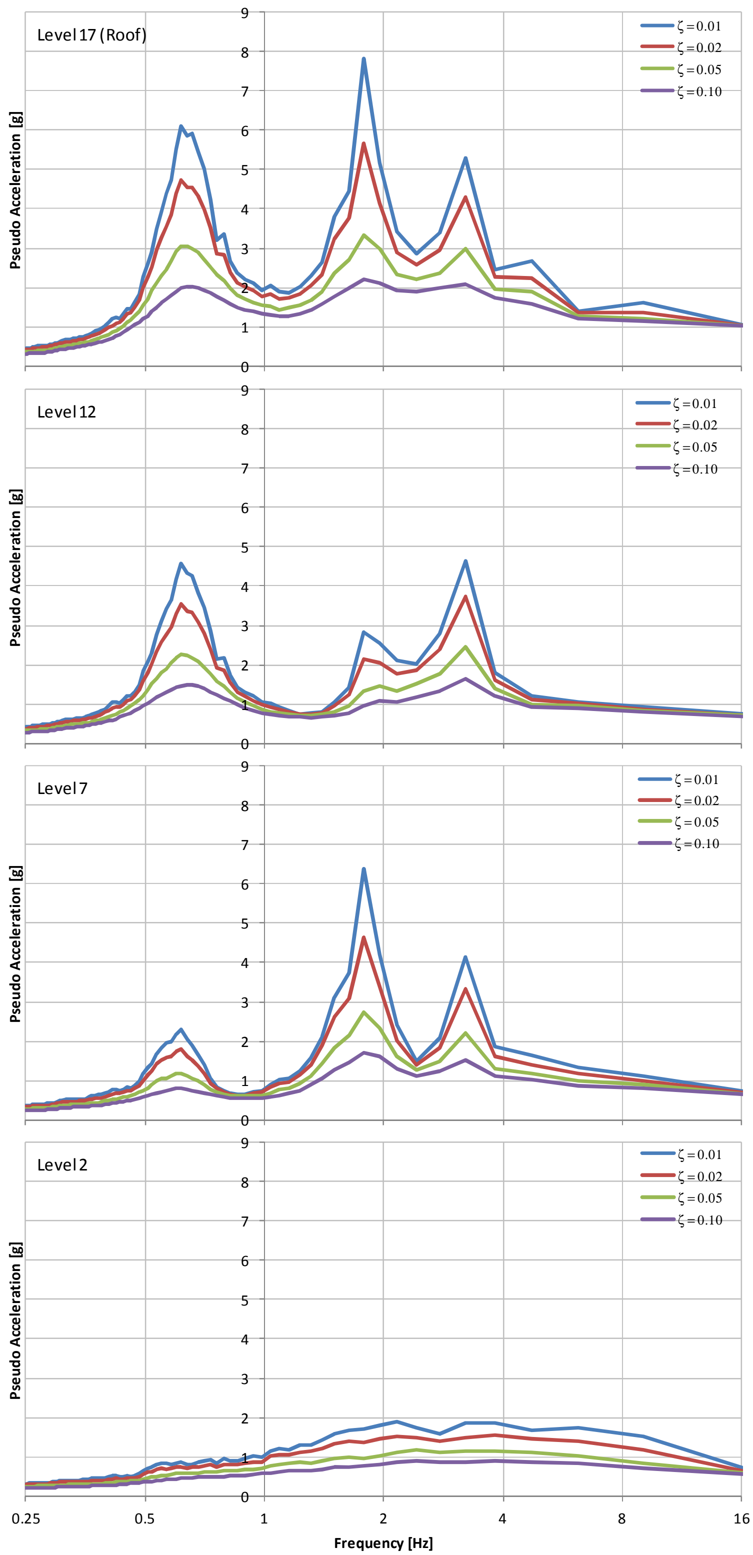


Figure 70: Pseudo acceleration floor spectra for levels 17, 12, 7 and 2.

LABORATORY DAMPER FRAMES (RETROFIT SCOPE S3)

Figure 71 through Figure 76 summarize response quantities for the laboratory damper frame retrofit scope. If retrofit scope S3 is selected, R&C recommends that it is applied in combination with retrofit scopes S1 and S2. The figures and conclusions we present in this section reflect that recommendation by including retrofit scopes S1 and S2 together with S3. As before, we report measurement indices for the 975-year return period hazard level which corresponds to the maximum considered earthquakes. Typically, response quantities represent SRSS values of the global X and Y direction results except for the separation joint displacements, which are reported in the two perpendicular directions separately.

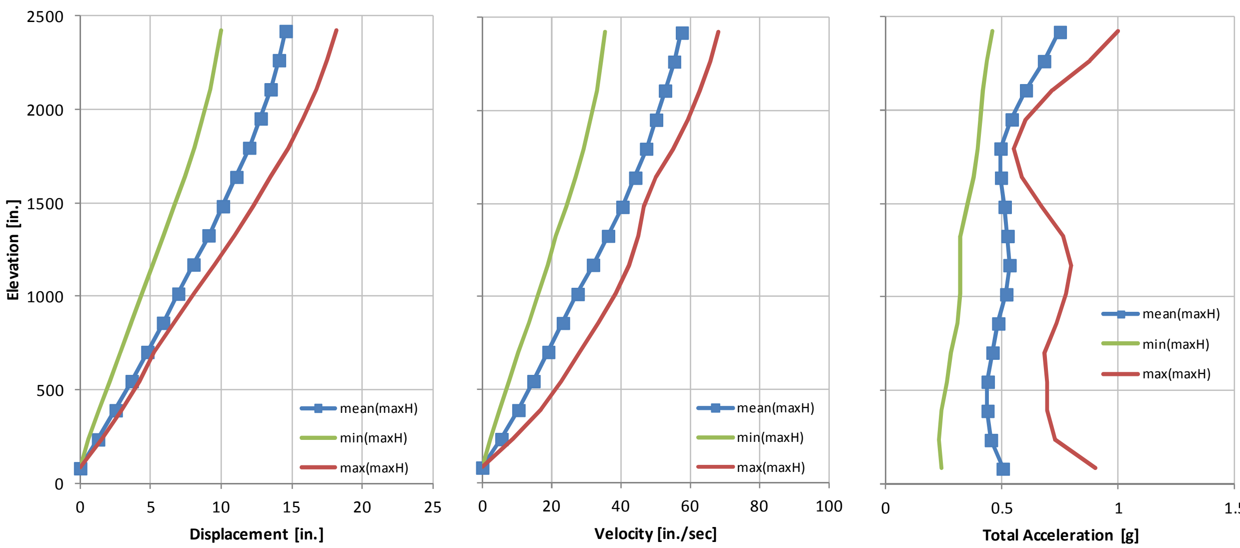


Figure 71: Floor displacements, floor velocities and floor total accelerations.

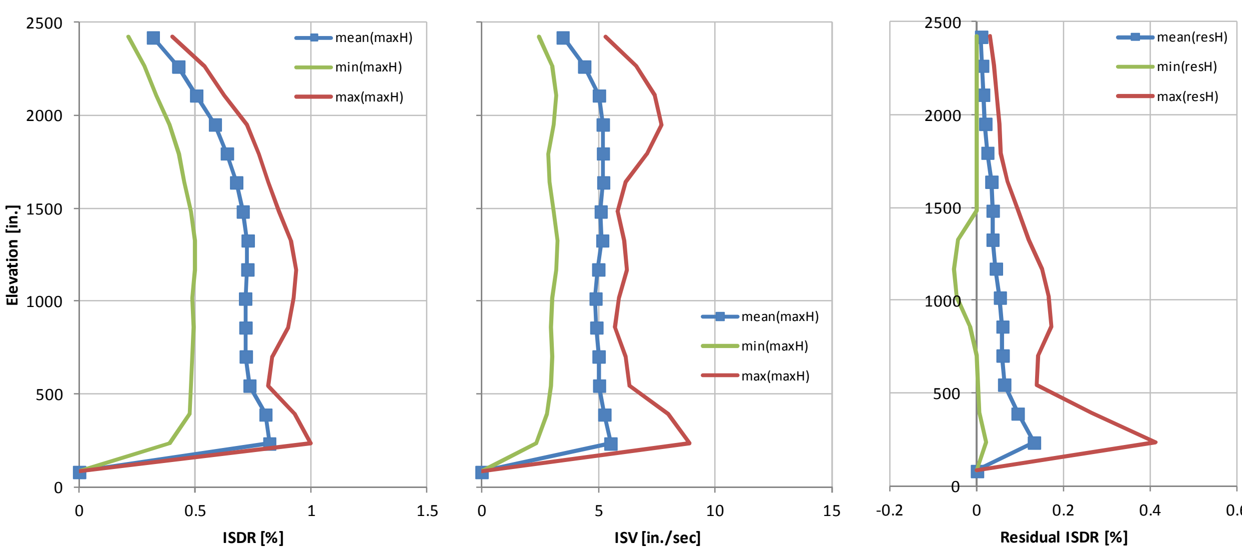


Figure 72: Interstory drift ratios, interstory velocities and residual interstory drift ratios.

We can see from Figure 71 that the mean SRSS roof displacement is equal to 14.5 inches, which is a 21% reduction from the 18.4 inches in the un-retrofitted structure. The mean SRSS roof velocity reduces from 72.8 in./sec to 57.6 in./sec which is also a 21% improvement over the existing laboratory building . Mean total SRSS floor accelerations improve the most and are approximately constant at 0.5 g for floor levels two through thirteen instead of increasing with height. At floor level thirteen that is a reduction of almost 30%. Above the thirteenth floor, mean total SRSS accelerations increase linearly and reach 0.75 g at the roof level. This corresponds to a 28% reduction over the roof accelerations reported for the existing structure. We can conclude that floor displacements, velocities and accelerations are significantly reduced as compared to the existing structure but higher mode whipping effects are not alleviated by this retrofit scope.

From Figure 72 we can see that mean SRSS interstory drift ratios for the MCE hazard level are reduced to around 0.73% for most stories up to the eleventh. Above the eleventh story they start decreasing till they reach 0.32% in the sixteenth story. R&C concludes that the laboratory building does not form any soft stories when subjected to the MCE ground motions and mean interstory drifts are reduces by 25% as compared to the existing structure. The mean SRSS residual interstory drift ratios are 0.1% or smaller.

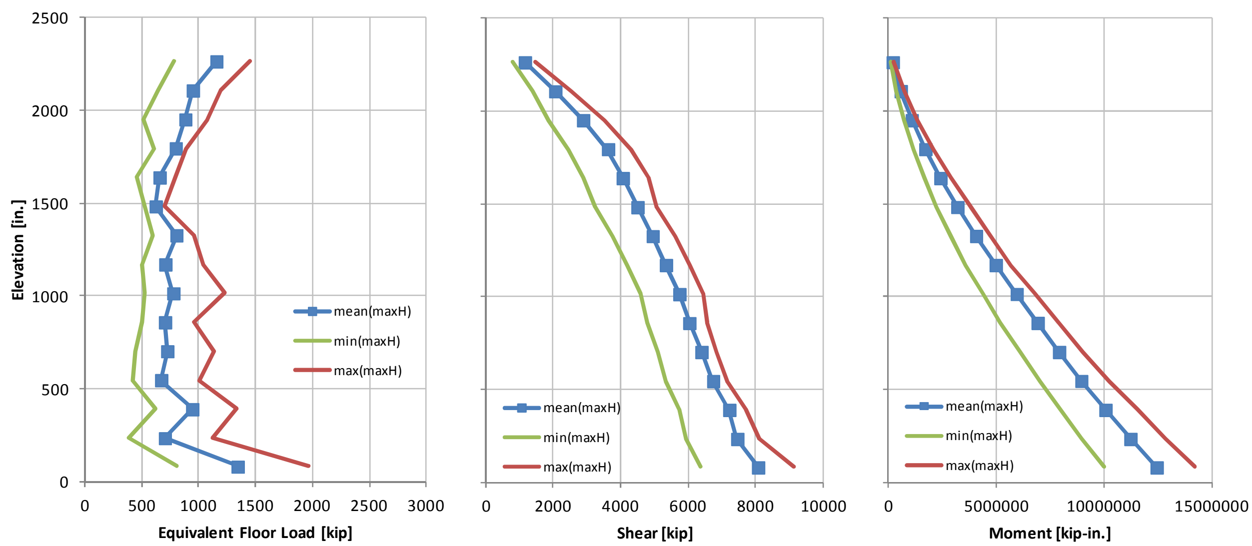


Figure 73: Equivalent seismic floor loads, story shear forces and story overturning moments.

Figure 73 depicts equivalent seismic floor loads, story shear forces and story overturning moments. We can observe that the mean SRSS equivalent seismic floor load distribution no longer closely resembles the mean total SRSS floor acceleration distribution because story shears are not only caused by the inertia forces of the floor masses but also by the damping forces of the laboratory damper frames. The mean SRSS base shear for the laboratory structure is equal to 8,070 kips which corresponds to 33% of the seismic weight of the building. The mean SRSS base overturning moment is equal to 12,436,700 kip-in., which corresponds to the seismic weight of the building applied 509 inches above the base of the building or right below the fifth floor. R&C concludes that story shears and overturning moments in the laboratory building are reduced by approximately 10% by this retrofit scope.

Because the results we present here for retrofit scope S3 include retrofit scopes S1 and S2, mean shear demands in the mechanical tower are again larger in the direction of the dampers as compared to the existing structure. Figure 74 shows the comparison of the shear distributions for the existing building (on the left) and retrofit scope S3 (on the right). However, base shears in the weak-axis direction of the mechanical tower do not increase as much as for retrofit scope S2. The increase in shear is equal to 90 kips as opposed to the 337 kips for retrofit scope S2. R&C concludes that the shear strength of the mechanical tower is sufficient to make the laboratory damper frame retrofit scope in combination with scopes S1 and S2 feasible.

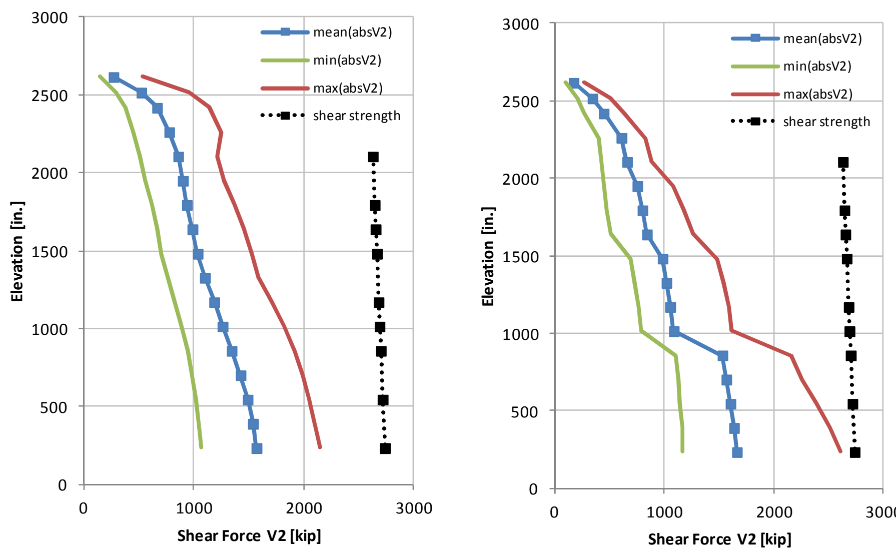


Figure 74: Axial forces for corner columns (left) and intermediate columns (right).

Figure 75 illustrates the minimum and maximum axial forces in the corner columns and the intermediate columns. On the corner column graph we also show the splice tension strength for the six column splices. As for retrofit scope S1 and S2, we observe that the tension splice strength curve of the retrofitted corner column splices (black dotted line) is outside the most severe tension demand curve (solid red line), and we conclude that for the 975-year return period hazard level all of the column splices have sufficient strength to remain elastic. For the corner columns mean tension forces at the base are equal to -3039 kips and mean compression forces at the base are equal to 5504 kips. These base forces are slightly smaller than the ones reported for the un-retrofitted laboratory building.

As for the other retrofit scopes we report opening and closing deformations for the seismic separation joints in the first graph in Figure 76 and transverse displacements in the second graph. We can see that at the roof level of the laboratory building the seismic separation joint closes or opens 1.6 inches on average and shears 14.5 inches on average. The largest mean joint opening or closing is equal to 2.5 inches and occurs at the ninth floor level. For the most severe of the MCE ground motions, the maximum joint closing or opening is equal to 5.0 inches and occurs at the tenth floor level. Compared to retrofit scope S2 we achieve an additional 20% reduction of the joint deformations in both, the east-west and north-south directions. R&C concludes that the proposed retrofit scope S3 is effective in reducing joint closing and opening deformations to levels where utilities contained in the seismic separation joints are protected from damage and utilities running across the seismic separation joints can be designed to accommodate such deformations.

To evaluate laboratory contents we again plot mean SRSS pseudo acceleration floor spectra for floor levels 2, 7, 12 and 17 in Figure 77. Spectra are plotted for damping values of 1%, 2%, 5% and 10%. However, most laboratory contents will have very little inherent damping and R&C recommends to use the 1% and 2% curves. We can see that pseudo acceleration peaks are significantly reduced for this

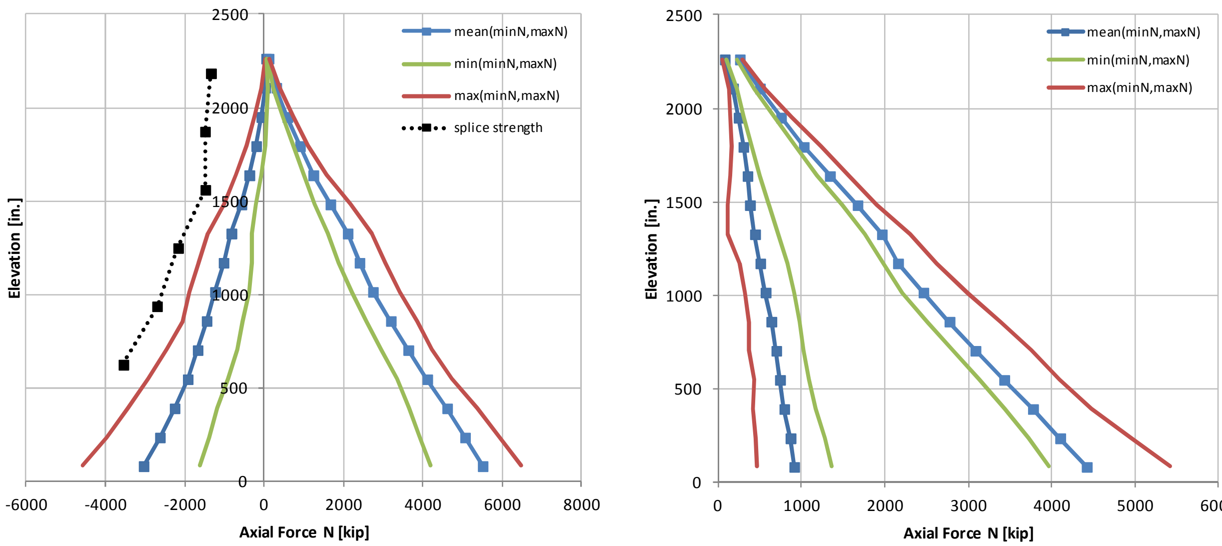


Figure 75: Axial forces for corner columns (left) and intermediate columns (right).

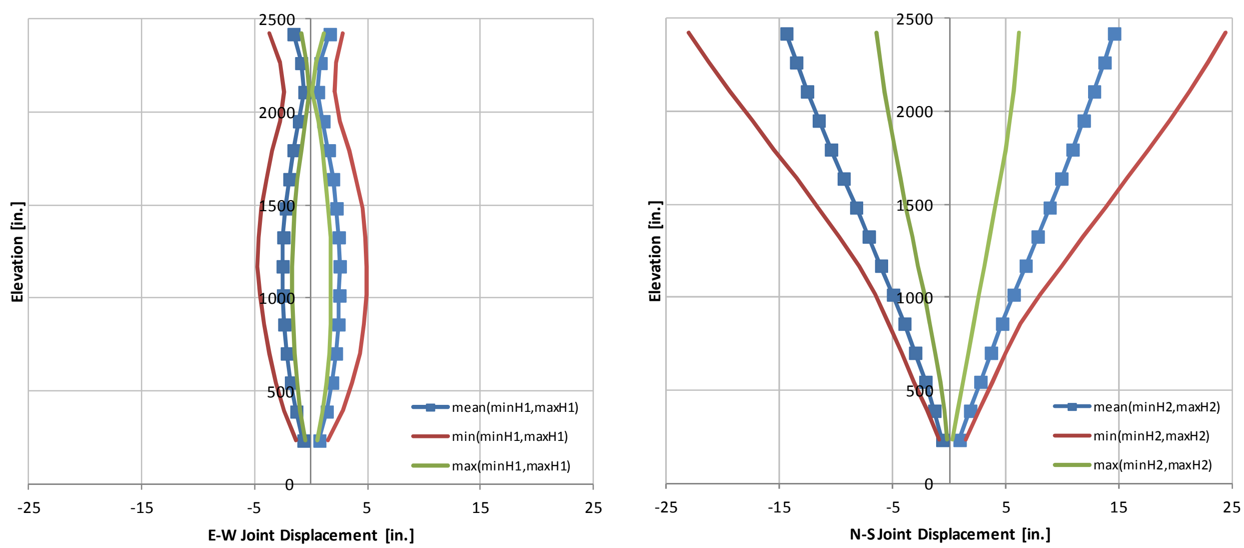


Figure 76: Separation joint displacements in E-W and N-S directions.

retrofit scope as compared to the existing structure. The largest pseudo acceleration occurs at the roof level and is equal to 4.2 g at 0.70 Hz for the 1% damped case. This is an exceptional 50% reduction from the largest mean SRSS pseudo acceleration recorded in the un-retrofitted laboratory building. We can observe that the frequencies of the peaks occur at 0.70 Hz, 1.95 Hz and 3.82 Hz. We can explain this shift towards higher frequencies and away from the first three modal frequencies of the laboratory building by the supplemental damping of approximately 10% of critical that we introduce through the laboratory damper frames. R&C concludes that the laboratory damper frame retrofit scope is extremely effective in reducing demands throughout the building. With this retrofit scope we achieve reductions of the pseudo acceleration demands of 50% for the entire frequency range, meaning that acceleration sensitive building contents will only see half of the demands as compared to the existing laboratory structure.

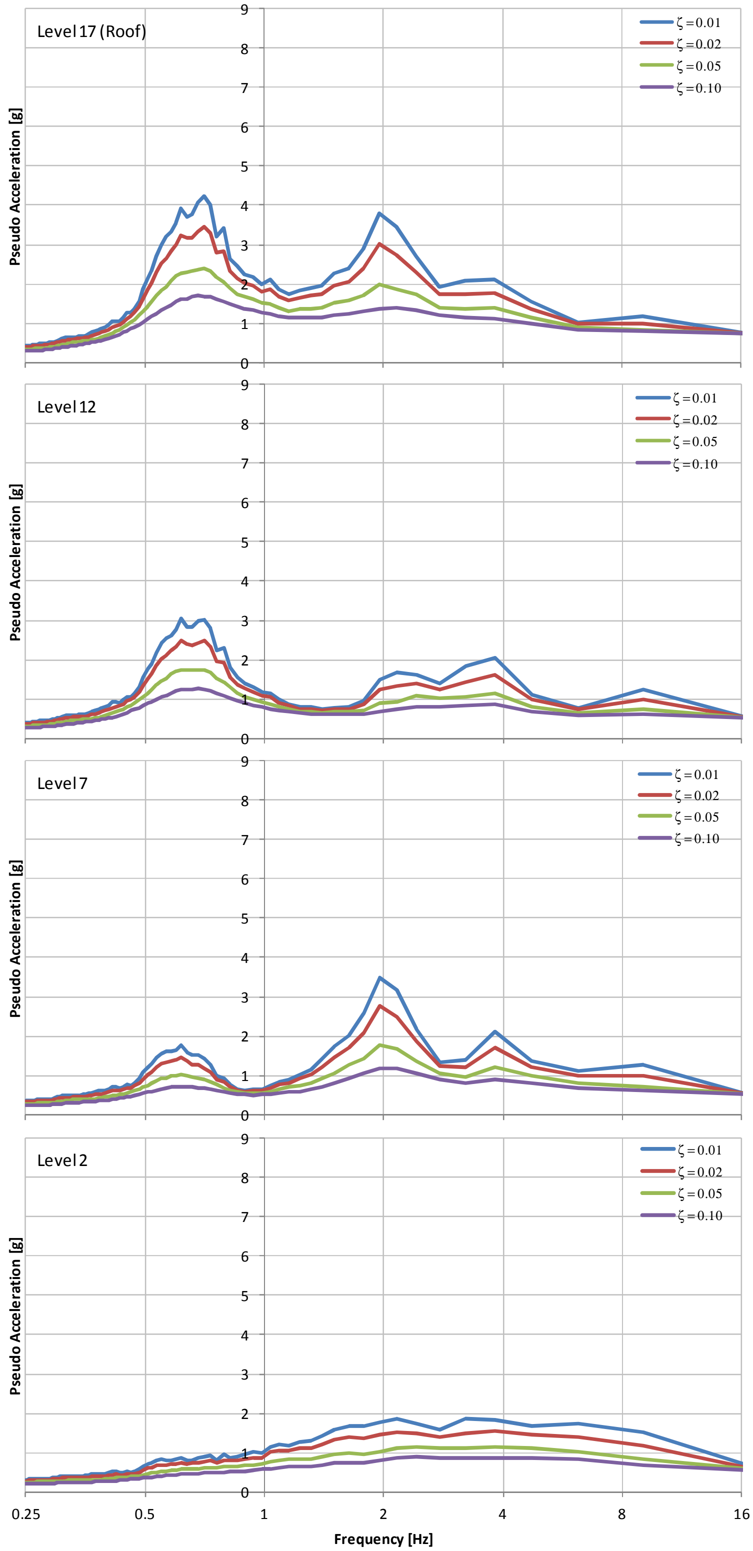


Figure 77: Pseudo acceleration floor spectra for levels 17, 12, 7 and 2.

Table 28: Performance levels affected by structural EDPs in the seismic risk analysis.

Engineering Demand Parameter (EDP)	Performance Level: Yield	Moderate Disrup.	Major Disrup.	Life Risk	Irrep.	Collapse
Maximum SRSS Story Drift Ratio (θ_{MAX})	✓	✓	✓	✓	✓	✓
4th Worst SRSS Story Drift Ratio (θ)		✓	✓	✓	✓	✓
Maximum Residual SRSS Story Drift Ratio ($\theta_{R\text{-}MAX}$)		✓	✓		✓	
4th Worst Residual SRSS Story Drift Ratio (θ_R)		✓	✓		✓	
Maximum Splice Elongation at Level 7.5	✓	✓	✓	✓	✓	✓
Residual Splice Elongation at Level 7.5		✓	✓		✓	
Maximum Splice Elongation at Level 5.5	✓	✓	✓	✓	✓	✓
Residual Splice Elongation at Level 5.5		✓	✓		✓	
Maximum Major Axis Mechanical Tower Shear (*V2*)	✓	✓	✓	✓	✓	✓
Maximum Major Axis Mechanical Tower Shear (*V3*)	✓	✓	✓	✓	✓	✓

4.10. STRUCTURAL FRAGILITY FUNCTIONS

The structural component of our seismic risk assessment for the Health Sciences buildings uses ten EDPs to estimate damage and disruption occurring in the structural system. The parameters we track from the nonlinear response-history analyses are shown in Table 28 with the performance levels affected by each EDP. We observe that residual displacements cannot cause yielding of the structural system, since the building frame must have already yielded to create a residual displacement. Similar logical conditions preclude some EDPs from affecting the *Life Risk* and *Collapse* performance levels.

The fragility functions for each EDP and performance level are based on judgment and expert opinion informed by research findings and our experience in the analysis of similar structures. We propose structural retrofit measures in Section 4.8 that modify the demands on the structural system, but not the capacities. As a result, we use the fragility functions developed for the existing building configuration throughout our analysis. Further discussion of the structural EDPs and fragility functions is included in Section 3 and Appendix K.

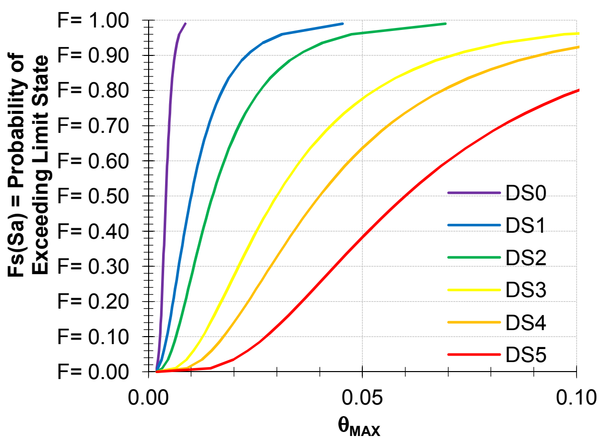


Figure 78: Example fragility functions for maximum interstory drift ratio.

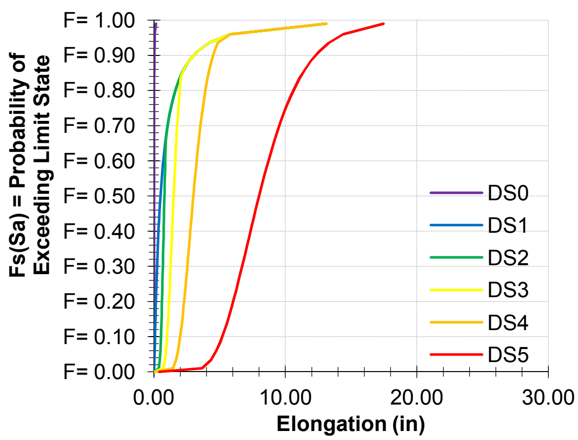


Figure 79: Example fragility functions for maximum Level 5.5 splice elongation.

5. MECHANICAL, PLUMBING, AND FIRE PROTECTION SYSTEMS

5.1. INTRODUCTION

The purpose of this study is to evaluate the mechanical, plumbing and fire protection system support and anchorage integrity for the entire building, as part of the seismic evaluation of the building structure conducted by Rutherford & Chekene. The evaluation is based on field observation, review of available existing drawings, and interviews with UCSF maintenance personnel. Field investigation is limited to areas that are readily accessible and observable. Due to the existing hazardous material in the older floors, investigation above the ceiling space is limited to the newer remodeled floors. The criteria recommended were to evaluate the system based on the continuing operations of these systems after an earthquake

Three building system retrofit categories will be used to evaluate the mechanical, plumbing, and fire protection systems. The equipment included in each retrofit scope is determined based on the expected increase in building resilience. Cheaper items and those components which contribute most to building resilience are considered the highest retrofit priority.

The retrofit scopes are:

M0: CBC CHAPTER 34 MINIMUM REQUIRED

This retrofit is a minimum level retrofit that protects areas of public occupancy from Life Safety falling and chemical hazards in public egress paths.

M1: TARGETED LOSS REDUCTION

This retrofit provides a limited cost-effective seismic retrofit which addresses the performance of building system components which have the greatest impact on building functionality, direct economic loss, or life safety. This retrofit scope may be the most appropriate approach when limited funding hinders the implementation of a more comprehensive seismic retrofit scheme.

M2: ENHANCED RESILIENCE

This retrofit provides a more complete seismic retrofit of building systems. This retrofit scope adds further protection for systems and equipment intended to reduce laboratory down time, preserve valuable research, and improve functionality following an earthquake.

The building systems that fall within each retrofit category are:

A. Retrofit Category M0:

1. All ductwork and piping above the elevator lobby.

2. All ductwork and piping above HSW 3rd Floor classrooms and lobby.

3. Hazardous material plumbing and piping at the separation joint between the mechanical towers and the laboratory buildings.

B. Retrofit Category M1:

1. All supply, general exhaust, and fume exhaust systems that serve the laboratory areas. In this case, it will be levels 2 to 16 in HSE and levels 4 to 16 in HSW. Primarily, this is to maintain the proper building pressure to allow occupants to exit the area in the event of emergency and prevent possible hazardous fumes from migrating outside the containment areas.

2. Temperature control system. This is required to maintain the operations of the building supply and exhaust air system.

3. Natural gas piping distribution.

4. Fire protection system.

5. Fuel oil system to the emergency generator on HSE roof.

C. Retrofit Category M2:

1. Heating hot water system.

2. Chilled water system.

3. Steam and condensate return system.

4. Cold water system.

5. Hot water system.

6. Waste and vent system.

7. De-ionized water.

8. Compressed air.

9. Vacuum.

10. Storm drain.

The HSIR building is a 16-story building, built around 1963, which is separated into 3 main buildings: Health Science East (HSE); Health Science West (HSW); and the Elevator Tower, located between HSE and HSW. Each building is separated with a separation joint. HSE starts from Level 1, while HSW and the elevator tower start at Level 2, except for the tunnel and equipment level in the elevator tower, which starts at Level 1. In addition, HSE and HSW have an exterior stair tower which is separated by a separation joint.

The system evaluation is based on the vulnerability of components to the relative building movements (separation joint motion), floor plate differential movement (relative floor displacement), and floor acceleration. The table in Appendix C summarizes the retrofit categories, evaluation, and repair/upgrade work recommendation for the mechanical, plumbing, and fire protection systems.

The lowest level of each building is semi-underground with the floor slab above this level fixed (no building gaps). Therefore, the evaluation for separation joint motion starts on the floor slab above the lowest floor; in this case Level 3 on HSE and Level 4 on HSW and elevator tower. The lowest level of HSE and HSW contains the building HVAC, plumbing utilities, and fire protection infrastructure, including the supply fan systems for the building towers. The equipment at this level is evaluated primarily on the integrity of the support and anchorage of the equipment with respect to the floor acceleration. There is an interconnecting tunnel between HSE and HSW which runs on the south side of the elevator tower and HSW. This tunnel contains HVAC, plumbing and fire protection water pipes.

The following sections describe the mechanical, plumbing, and fire protection systems.

5.2. Mechanical

1. Supply Air System (SF)

   The existing supply air systems for HSE and HSW are essentially identical. The systems consist of two built-up air handling systems located on the lowest floor of each building. Two supply air ducts rise in the stair tower and supply air from Levels 4 to 16 of HSE and HSW tower. Levels 1 to 3 of HSE and the Level 3 classrooms of HSW are served by separate supply fan systems. The tower air handling systems were replaced in 2 phases: 2001 for HSE, and 2005 for HSW. The entire existing air handling system was replaced with brand new built-up air handling systems consisting of new vane axial fans, heating and cooling coils, multi-stage air filter banks, and sound attenuators. New equipment anchorage has been upgraded based on the applicable code at the time of the project approval. In this case, the HSE tower system upgrade follows 1997 CBC and HSW tower system upgrade follows 2001 CBC requirements. Part of this project included repair and upgrade of the supply duct risers related to the performance of the system. It does not include upgrade of the duct riser supports and anchorages. We recommend visually inspecting the support and anchorage of the duct risers. Levels 1 to 3 of the HSE and Level 3 of the HSW supply air handling systems are original and consist of heating and cooling coils and a single stage air filter. Base frame, spring vibration isolators and seismic snubbers had been added some time in the early 1980's. Since these fans are located in levels 1 and 2, respectively, which is slab on grade, the anchorage of this equipment should be checked against the current code requirements. The following summarizes the supply air handling systems for each tower.

Table 29: HSIR supply fans.

Tower	Fan Tag	Year Built	Fan Type	Location	Floor Served	Notes
HSW	SF-1A, 1B	2005	Vane Axial	2nd Floor	Levels 4 to 16 East	1
	SF-2A, 2B	2005	Vane Axial	2nd Floor	Levels 4 to 16 West	1
	SF-3	1963	SWSI	2nd Floor	Levels 2 & 3	2
HSE	SF-4	1963	SWSI	1st Floor	Levels 1, 2, & 3	2
	SF-5A, 5B	2001	Vane Axial	1st Floor	Levels 4 to 16 North	1
	SF-6A, 6B	2001	Vane Axial	1st Floor	Levels 4 to 16 South	1
Elevator Tower	SF-7	1963	DWDI	1st Floor	Levels 1 & 2, Tunnel, Mechanical & Electrical Transformer Room	

Notes:
1. Work includes completely new supports, vibration isolators, and seismic restraints.
2. Fans are mounted on base frame with vibration isolators and seismic snubbers, which were installed in the mid 1980's.

Figure 80: Typical floor main ductwork crossing the separation joint between the stair tower and HSE or HSW.

Figure 81: Typical duct penetrations through stair tower shaft.

Each floor of the Tower building is supplied by two supply air duct risers with 6 fixed branch ducts. Figures 1 and 2 in Appendix C show a typical floor main supply and exhaust air duct distribution for old and newer (remodeled) floors. Due to the depth of the steel structure, the 6 main floor supply ducts are routed through pre-cut openings in the steel girders. The main supply duct branch exits the stair tower supply air shaft and crosses the separation joint between the building and the stair tower. The main duct penetration through the shaft wall is equipped with fire/smoke dampers which were installed as part of the air handling system upgrade project, and a flexible duct connection (with about 1-inch gap between the duct through the stair tower shaft and the building duct). See Figure 80 and pictures PT-9 and 10 of Appendix C. The gap for the flexible duct is not large enough to accommodate the building movement in an earthquake. Due to the duct configuration and available space in the building separation joint, it is not practical to install a flexible duct joint in the main supply duct exiting the riser shaft, to overcome the seismic movement of the building. Because of this, we recommend that UCSF prepare a procedure to repair or replace the duct in minimal time in the event of an earthquake.

The original supply air system is a constant volume tempered air system. In subsequent remodel projects, particularly post-1990, the supply air system has been converted into single duct variable air volume with hot water reheat coils. In addition, duct-mounted cooling coils were added to provide supplementary cooling to the respective floors (see Figure 81 and picture PT-11 in Appendix C). Most of the supply ductwork is not very large, and they are supported based on SMACNA Duct Construction Standard recommendations. Most of the post-1990 remodeled main ductwork has been replaced with much better duct joints. The originally installed ductwork has mostly S and drive transverse joints. The integrity of these joints should be evaluated, repaired or replaced in future remodel projects. The pre-2001 code duct-mounted cooling coil support and anchorage should be re-evaluated based on the current code.

2. Exhaust Air System (EF)

Similar to the supply air systems, the general exhaust air systems for HSE and HSW are essentially identical in the tower portion. There are miscellaneous exhaust fans that are serving specific areas in Levels 1 and 2.

The tower general exhaust system consists of two exhaust fans located in the 18th Floor fan room of the stair tower structure of HSE and HSW. The exhaust fans in HSE tower are the original fans (1963); the fans in HSW were replaced in 2005. Two exhaust air ducts rise in the stair tower and exhaust the room air from Levels 4 to 15 of HSE and HSW.

Levels 1, 2, and 3 of HSE are served by a separate exhaust fan located in the 1st Floor exhaust fan room. This fan discharges air through the 2nd Floor slab to the exterior with about a 25-foot tall stack. The stack is supported from the Medical Sciences Building (MSB) exterior wall. The support should be separated from the MSB. However, this duct survived the Loma Prieta earthquake due to the natural flexibility of the ductwork. In addition, it is the discharge side of the exhaust fan. For these reasons, is it not a priority to fix this.

Level 3 of HSW is exhausted by a separate exhaust fan which is located in Level 2. There are other exhaust fans which serve other miscellaneous areas.

Except for the HSW tower general exhaust fans, all the existing floor-mounted fans are mounted on base frames with spring vibration isolators and seismic snubbers which were installed in the early 1980’s. The anchorage of this equipment should be checked against the current code. The following table summarizes all the existing building exhaust fans in the HSIR tower.

Table 30: HSIR general exhaust fans.

Tower	Fan Tag	Year Built	Fan Type	Floor Served	Notes
HSW	EF-1	2005	DWDI	Levels 4 to 16 East	1
	EF-2	2005	DWDI	Levels 4 to 16 West	1
	EF-3	1963	DWDI	Levels 2 rooms	3
	EF-10	1963	SWSI	Levels 2 shops	3, 4
	EF-15	1963	SWSI	Level 2	3, 4
HSE	EF-4	1963	DWDI	Levels 1 & 2, 3	2
	EF-5	1963	DWDI	Levels 4 to 16 North	2
	EF-6	1963	DWDI	Levels 4 to 16 South	2
	EF-9	1963	DWDI	Level 1 Electrical Transformer Room	2
Elevator Tower	EF-8	1963	Roof Exhauster	Toilets in Elevator Tower	5
	EF-13	1963	Cabinet Fan	Level 1 & 2 tunnel and Mechanical Room	

Notes:
1. Work includes completely new supports, vibration isolation, and seismic restraints.
2. Fans are mounted on base frame with vibration isolators and seismic snubbers which was installed in the mid-1980’s.
3. Ceiling-hung on vibration isolators.
4. Due to inaccessibility, it is not known whether the fans still exist or if they are still operating. UCSF maintenance suspected that they are no longer used.
5. Mounted on roof curb.

The exhaust duct distribution is similar to the supply air duct distribution. See Figures 1 and 2 in Appendix C for typical floor exhaust duct distribution. Similar to the supply duct, the main

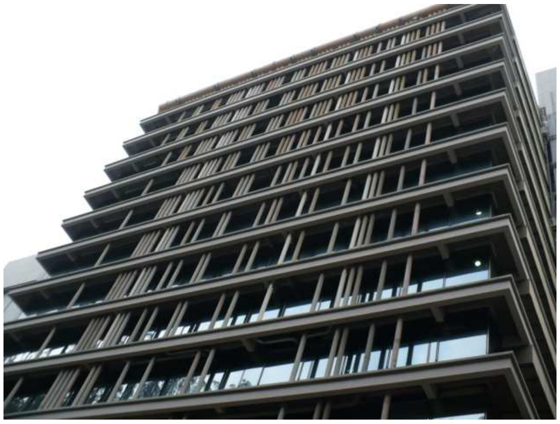


Figure 82: Typical exterior fume exhaust duct risers - HSE south face.

Figure 83: Typical exterior fume exhaust duct risers – HSW north face.

duct risers were repaired and upgraded in the 2001 and 2005 infrastructure upgrade work of HSE and HSW, respectively. The same deficiencies and limitations for the supply duct also exist in the general exhaust duct distribution.

3. Fume Exhaust System (FH)

Each fume hood from a typical floor is ducted to the exterior of the building, penetrates through the building window glazing, and rises up through a pre-punched hole on the building ledges to the roof. Figure 82 and Figure 83 and Pictures PT 1 and 2 in Appendix C show the typical fume exhaust duct risers in HSE and HSW. The portion of the fume exhaust duct indoors is constructed of type 316 stainless steel. The fume exhaust duct outdoors is constructed of fiberglass-reinforced plastic. The transition of material occurs after the duct penetrations through the building window glazing. The pre-punched holes on the ledges were sized to accommodate a 9-inch duct with about 1/2 inch annular space, which is filled with caulking material. The existing duct penetration openings through the floor ledges were notched, such that the duct risers on the upper floor rest on ledge and the duct from the lower floor riser penetrates the ledge and clip in to the upper floor duct. Newer duct penetrations were cut as one size holes. Short pipe clamps were used to support the duct risers at each level. Figure 84 and Figure 85 and Pictures PT-3 through 8 show the typical fume exhaust duct riser construction and arrangements.

In the past seismic event, some of these duct riser joints came apart. These duct risers need to be evaluated against the differential movement of each respective floor plate. One possible solution is to install a flexible duct such as Mercer Rubber Flexible duct connector at each level to compensate for the floor movement, or to enlarge the duct penetration openings through the ledges, and support the duct risers with isolators, such as Mason SLF at each level and allow the fiberglass duct to flex naturally. Fume exhaust duct from the general chemical fume hoods are collected in two fume exhaust duct manifolds on the roof. In HSE, one manifold was sized to serve about 2/3 of the building fume exhaust air, with four fume exhaust fans (one as backup). The other was sized to serve about 1/3 of the building fume exhaust air with two fume exhaust fans (one as backup). In HSW, the two manifolds were connected to become one. The HSE tower fume exhaust system was upgraded around 2001 and the HSW tower system was upgraded around 2005. The HSE manifold fume exhaust fans are mounted on base rail with

Figure 84: Typical multiple fume exhaust duct risers through the ledge.

Figure 85: Typical multiple fume exhaust duct risers through the ledge above.

housed spring vibration isolators. HSW manifold fume exhaust fans are mounted on a base frame with unhoused spring isolators and seismic snubbers.

Some special fume hoods or biosafety cabinets are exhausted by independent individual small fume exhaust fans located on the roof. These fans are mounted on base rails and vibration isolators. The fans on HSE roof are supported by two kinds of vibration isolators. The older fans (pre-1998 building code) are mounted on rubber in shear (Mason ND), while the new fans are mounted on base rails with open spring vibration isolators and seismic snubbers or housed spring vibration isolators. The fans on HSW are mostly supported by restrained, double sphere, neoprene rubber (M.W. Sausse Model FUD) isolators. A few fans are supported by housed spring vibration isolators or open spring with seismic restraints. Pictures PT 26, 27, 29, and 30 in Appendix C show the typical small fume exhaust fan support. The small fans on HSE that are mounted on rubber, in-shear isolators do not have seismic restraint capability. These should be replaced with new seismic restraint isolators.

Figure 86: Typical fume exhaust fan HSE with un-restrained rubber isolators.

Figure 87: Typical manifolded fume exhaust fan base frame with spring vibration isolators and snubbers.

4. Miscellaneous Fan Coil Units (FC or SF)

   Due to high equipment heat rejection load and lack of air conditioning in the building, subsequent remodel projects have installed a number of ceiling mounted fan coil units in rooms with high equipment heat rejection, such as some tissue culture labs, equipment rooms, freezer rooms, etc. We recommend evaluating the supports/anchorage for those units installed under pre-1998 building codes.

5. Heating Hot Water System (HHWS/R)

   Heating hot water is generated in each building (HSE & HSW) via two steam-to-hot water heat exchangers, and three hot water circulating pumps in HSE and two pumps in HSW. The equipment is located in the lowest level of each building tower. The heat exchangers are existing original units, installed in 1963. The heat exchangers are mounted on a base frame with a small diameter pipe leg at each corner of the frame. See pictures P1I and P1J in Appendix C. The support for the heat exchangers should be upgraded to meet the current code. The hot water circulating pumps and the piping infrastructure were replaced and upgraded in 2000 and 2005 in HSE and HSW, respectively. The heating hot water pumps are anchored directly to the concrete housekeeping pad. The heating hot water system is currently serving the main supply fan preheat coils, zone reheat coils, and the existing perimeter baseboard heaters. Heating hot water supply and return piping are distributed throughout the building with the pipe risers located in the stair towers and two corners of the floor. Figures 3 and 4 show a typical floor arrangement of the pipe risers and distribution in the older and newer (remodeled) floors, respectively. Heating hot water pipes from the stair towers exit the pipe riser shaft, cross the building separation joint, and enter the building. There is currently no pipe separation joint expansion compensation. The years 2000 and 2005 hot water system upgrade included the installation of new additional heating hot water supply and return risers mounted on the exterior face of each HSE and HSW stair tower respectively with branch stub-out at each floor (starting from Level 4 and up) to provide supplemental heating hot water to each floor. Figure 88 and Pictures PT 31 and 32 in Appendix C show these risers. In addition, new heating hot water mains were also routed through building seismic gaps. In an earthquake, these pipes may be crushed by the two buildings, or break by the excessive building movements. This condition needs to be corrected by locating the pipe risers outside the building seismic gap and flexible piping.

Figure 88: New HSE HHSW/R risers (2000) – south face of stair tower.

Figure 89: Typical floor chilled water risers through seismic gap with valve stub-outs and heating hot water sub-outs in building gap between stair tower and HSE or HSW.

6. Chilled Water System (CHWS/R)

The primary source of chilled water for the HSIR buildings comes from the central utility plant. There was a backup air-cooled chiller farm located on Hooper pad which could supply chilled water to HSW tower or back-feed the central chilled water loop. The back-up air-cooled chillers are no longer in use and were abandoned in place. The chiller farm equipment will not be evaluated, because it will be removed in the near future. Twenty-four-inch diameter chilled water supply and return pipes exit the central utility plant and distribute chilled water to the PSB, MSB and HSIR buildings. Sixteen-inch chilled water supply and return piping enter the east side of the HSE tower at Level 1 and run along the Level 2 south breezeway, rising up to Hopper pad (Level 4) to the HSW tower. Each tower building has its own VFD-driven tertiary pumps (3 in HSE and 4 in HSW) which distribute chilled water to the main air handling system cooling coils, duct-mounted cooling coils, and fan coil units in the tower floors. Chilled water supply and return piping rise next to the stair towers of HSE and HSW inside the building separation joint, and branch out at each floor. Figure 89 and Picture PT24 in Appendix C shows a typical chilled water pipe riser in the building seismic gap. These risers need to be relocated to outside the seismic gap, possibly on the same stair tower face as the heating hot water risers. On the 8th floor, there are 6" CHWS/R pipes that originate from HSW chilled water piper risers and run around the east corridor of HSW, and cross the elevator tower into HSE and MSB buildings. See Drawing M8 in Appendix C. These pipes were installed between 1992 and 1993. This set of chilled water pipes crosses a number of building separation joints. This should be minimized or corrected. For correction measures, we recommend separating the chilled water distribution from HSE and HSW, and supplying the HSE part of the building from HSE chilled water risers. New pairs of chilled water risers are required, or larger pipe risers should be installed as part of the relocation work of the existing pipe riser work.

The cooling coil in SF-3, which serves the 3rd Floor HSW classroom is served by an air-cooled chiller and a circulating pump located on the Hopper pad. Piping from this chiller is independent from the central utility chilled water piping distribution. The chilled water pipes from this chiller drop down from Hooper pad at Level 5 to Level 2 of HSW in the separation joints between the stair tower and HSW. The chilled water system was installed in the mid 1980's. Chilled water pipe risers through the building seismic gap in this case should be acceptable, due to the small amount of seismic movements near the base of the building.

7. Steam and Condensate Return System (LPS, MPS, MPR)

Low pressure (15 psig) and high pressure steam (200 psig) is distributed from the central utility plant to HSIR. The 18" main low pressure steam enters HSE in Level 1, to serve the heating hot water heat exchanger in HSE and HSW. The main low pressure piping continues up to HSE 2nd floor south breezeway, up to Hooper pad where it is capped for future connection to the west campus. High pressure steam is reduced to medium pressure (60 psig) and low pressure (15 psig) to be used as backup and supplement the low pressure steam system. Medium pressure steam serves the autoclaves, washers, and other steam-using equipment in HSE and HSW towers. The medium pressure pipe rises up the building in the stair tower, exits the floor with 1-1/4" pipe, and crosses the separation joint with a Z-shape loop and no other seismic expansion compensation. These joints should work for small seismic movements at the lower levels, but can be damaged in large seismic movements on the upper levels.

Condensate return from each floor of the HSE and HSW towers is collected into a condensate return riser in one corner of each floor (SW corner in HSE and NE corner in HSW) from Levels 2 to 15. Figure 3 in Appendix C shows a typical floor medium pressure steam and condensate return piping distribution. The entire distribution system was built in 1963. Subsequent floor remodel projects on selected floors provide new steam and condensate return branch lines, and may have re-routed the condensate return riser to accommodate the new architectural partition layout.

8. Temperature Control System

The temperature control system is a mix of pneumatic and direct digital control. The older floors/systems are pneumatic controlled. The newer floors/systems (post-mid-1980's) are a combination of pneumatic and direct digital control (DDC). The post-mid-1990's floors/systems are mostly direct digital controls. Pneumatic control air is obtained from the compressed air loop (see plumbing narrative) located in Level 1 of the HSIR tower. Control wires in the conduit riser goes up to the roof and are used to control and monitor the fume exhaust fans on the roof, the fire/smoke damper modulations, in each level, and communicate with the temperature control panels in each floor (the ones that have DDC control). The pneumatic piping distribution and the DDC control distribution consist of small pneumatic tubings and conduits with small localized control panels located in the ceiling space. Due to the size and flexibility of the pneumatic tubing and electrical conduits, the integrity of these systems would not be significantly impacted by any seismic event category, with the possible exception of some infrastructure control conduit risers located in the separation joint between the stair and the building towers.

5.3. PLUMBING

Plumbing systems in the HSIR Buildings consist of cold water (industrial – ICW and domestic – DCW), natural gas (G), de-ionized water (DI), hot water (industrial – IHW and domestic – DHW), compressed air (CA), vacuum (VAC), sanitary waste and vent (W&V), acid waste and vent (AW & AV), and rain water leader (RWL). All main plumbing utilities originate from the lower floors of the HSIR Buildings and rise up in the stair tower and ledge of HSE and HSW, except for the acid vent risers, which rise in 4 corners of the floors. Plumbing piping and acid waste exit the stair tower, and cross the building seismic gap to HSE and HSW, with Z-shaped loop. There is no other seismic expansion compensation. Similar to the steam and condensate return piping distribution, the Z-shaped loop may be adequate for smaller seismic movements at the lower levels, but may be inadequate for larger movements at the upper levels. Some seismic movement compensation is currently provided by the pipe offset between the building gap. No other seismic expansion compensation is provided. See Figures 5, 6, 7, and 8 in Appendix C for typical pipe risers, distributions and elevations in the HSIR tower. The waste and vent pipe risers in the 4 corners of the floors are rigidly supported at each level. The integrity of these pipe risers need to be evaluated on floor plate differential movements and floor acceleration.

There is a fuel oil (FO) system in HSE that serves the emergency generator located on the roof of HSE.

1. Cold Water (ICW & DCW)

   High and low pressure potable cold water mains enter Level 1 of HSE tower and are distributed throughout the HSIR buildings. Cold water is split into potable and industrial cold water. The industrial cold water main is protected with reduced pressure backflow preventer in each building to prevent contamination of the potable water system. The industrial cold water is distributed throughout the building to the lab fixtures and equipment, and the industrial water heaters. Potable water is distributed to serve the toilets, janitor closets, drinking fountains, and the domestic hot water heater.

2. Hot Water (IHW & DHW)

   Industrial hot water in HSE is generated by two 350-gallon storage type steam-to-hot water generators located in the Level 1 mechanical room. This equipment is still original, installed in 1963. See pictures P1F and P1G in Appendix C. Industrial water heater in HSW is generated by two semi-instantaneous steam-to-hot water heaters located in the Level 2 mechanical room. See Figure 91 and Picture P2N in Appendix C. This equipment was installed in 2009. Industrial hot water is distributed throughout the building to serve the lab fixtures and equipment. The distribution system has a return loop and a circulating pump.

Figure 90: HSE 1st Floor Mechanical Room – domestic hot water generator.

Figure 91: HSW 2nd Floor plumbing room – industrial hot water generator.

3. Natural Gas (G)

   1-1/4" high pressure gas enters HSE and HSW at Levels 1 and 2, respectively. High pressure gas is reduced to 7" water column (w.c.) and distributed throughout the building laboratory spaces. Gas main in each building is metered.

4. De-ionized Water (DI)

   De-ionized water is generated in the central utility plant, and is piped to Level 1 of HSE into 4 main DI water storage tanks (two 2800-gallon tanks in SF-5 outside air plenum and two 3200-gallon tanks in SF-6 outside air plenum). From these tanks, DI water is pumped to Medical Sciences buildings and the HSIR 600-gallon base DI tanks (typical of 2) located in the Level 1 tunnel of the elevator tower and pumped up to Level 17 of HSE and HSW stair tower DI water storage tanks. The storage in each tower consists of two horizontally mounted 400-gallon aluminum tanks. The base and Level 17 tanks are all original equipment built in 1963. The support framing appears to be adequate. However, we have no information on the anchors. This should be evaluated further. The plastic storage tanks were added in later years. We have no record on the time they were installed.

   From the Level 17 tanks, DI water is distributed down in the stair tower mechanical shaft throughout the building. The primary material for the DI water piping is PVC. Occasionally, stainless steel pipe is used for floor distribution.

5. Compressed Air (CA)

   80 psig compressed air is generated by two air compressors with one air receiver located in Tunnel 1 of the elevator tower. The compressed air receiver is quite tall (about 7 to 8 feet) and is anchored to the concrete floor via four angle clips with 1/4" concrete expansion anchors. The support and anchors appear to be inadequate. This needs further evaluation. Compressed air from the central utility plant is used to supplement the compressed air need for HSIR. Compressed air is piped throughout the building for shop and lab use and temperature control in the entire HSIR Building. The compressed air equipment appears to be the original equipment installed in 1963. The equipment anchorage appears to be in good condition.

6. Vacuum (VAC)

   About 20" mercury (H.G.) vacuum is generated by two vacuum pumps with 2 accumulators located in Level 1 of the elevator tower. Vacuum is piped through the building to the lab vacuum outlets. The vacuum pumps and one smaller accumulator tank appear to be new (we have no records on when they were installed). The large vacuum accumulator tank appears to be original (1963). Similar to the compressed air tank, the vacuum accumulator support and anchorage need further evaluation. The existing vacuum pumps will be replaced by the end of summer, 2010.

7. Waste and Vent (W, V)

   Waste and vent piping serves the drinking fountains in HSE and HSW floors. Waste from the toilets and janitor's closet in the elevator tower is collected to a riser in the plumbing shaft. Waste lines from these areas are combined with acid waste and storm drain lines in the sanitary sewer manholes outside the building.

8. Acid Waste and Vent (AW, AV)

   Lab waste from each floor is collected into a 6-inch acid waste main running in the center of the floor, routed to the stair tower, and down in the stair tower mechanical shaft to the lower level. The main acid waste is combined with the sanitary sewer waste and storm drain lines in the sanitary sewer manholes outside the building.

   Acid vent from the lab fixtures is piped and collected into one of the acid vent risers located in the corner of the floor and up to the roof.

   The original acid waste pipe material is PVC. Subsequent remodel projects have replaced some of the branch waste lines with Duriron. The original pipe material on the acid vent risers is glass. Subsequent remodel projects have replaced some of the glass pipe risers with Duriron. Vent piping above the ceiling is mostly fire retardant polypropylene.

9. Storm Drain (RWL)

   Rainwater is collected through roof drains into a rainwater leader riser located in the ledge of HSE and HSW across the stair tower. The rainwater leader in the elevator tower is located in the plumbing riser shaft. Storm drain piping is combined with the sanitary sewer and acid waste lines in the sanitary sewer manholes outside the building.

10. Fuel Oil (FO)

   There is a fuel oil system in HSE that serves the emergency power generator located on the HSE roof. The generator currently provides emergency power to the fume hood exhaust fans located on the HSIR and MSB Increment II roof. The fuel oil system consists of a 1,000 gallon, above ground tank and transfer pump, which are located on Level 3, above the HSE supply air intake. 1-1/2” fuel oil supply and 1-1/4” fuel oil return pipe inside a 9” containment pipe cross a seismic gap at Level 3.5 and rise up the building through one of the existing fume exhaust duct riser knockouts at the northeast corner of HSE, to the roof. In a severe seismic event, the pipe risers may be damaged, due to floor differential displacement and acceleration. However, the size of the pipes and the space inside the containment pipe will allow some flexibility in the fuel oil pipes to compensate the floor differential displacement. The best scenario is for UCSF to provide emergency power to those fume hood exhaust fans served by the emergency generator from the building emergency power circuits, so that the emergency generator and the fuel oil system can be eliminated and removed from the site. See electrical narrative regarding this issue.

5.4. Fire Sprinkler (FSP)

Fire water to the HSIR Building is fed from the campus fire water loop which originates from the fire water reservoir in the hills south of the HSIR Building. The 12” fire water main enters HSE Level 1 and is distributed to three wet stand pipes; a 6-inch wet stand pipe in the ledge of HSE and HSW across the stair tower and two 3-inch wet stand pipes on 2 floor corners opposite the stair towers. The 6-inch wet stand pipe is also used to supply water for the fire sprinkler system at each floor. The 3-inch wet stand pipe serves the floor fire hose cabinets. These risers are supported rigidly through each level. The integrity of these pipe risers needs to be evaluated on floor plate differential movement and floor acceleration.

The 12" diameter fire water pipe main in HSE Level 1 splits northward to supply fire water to the Medical Science (MSB), Clinical Science (CSB), and UC Hospital (UCH) buildings, and westward to supply fire water to the elevator tower, HSW, Nursing Building, and the west part of the Parnassus Campus. The campus fire water distribution was built in 1995.

5.5. SUMMARY

Appendix C shows typical mechanical system distribution within the building. Appendix C shows the results of our equipment and system distribution surveys which show existing conditions and deficiencies that will need to be evaluated and corrected.

In general, the largest concern for the mechanical, plumbing, and fire protection are related to the locations where piping crosses the separation joints without any expansion compensation, and piping is installed in the seismic gap. The figures and pictures in Appendix C illustrate typical and specific installation of the mechanical systems in the HSIR tower. Previous sections have described the various conditions of the M/P/FP system with respect to the HSIR separation joints, with some sample of correction work recommendations. These recommendations can be applied to other systems not discussed in detail in this report. In addition, there is another separation joint on the north side of HSE which adjoins the Medical Science Building (MSB); see Figure 9. This separation joint contains numerous exhaust duct and chilled water pipe risers that serve the MSB spaces. Half of the duct risers and chilled water pipe risers serve the 7th Floor Human Prion Lab which is a biosafety Level 2 + laboratory.

The other ducts serve the 2nd Floor LARC washroom, 3rd Floor offices, and 13th Floor Lab. The main concern on the HSE Building in this separation joint is the north side fume exhaust duct risers. Some of the seismic gaps are covered by sliding metal plates which have a potential to slice the duct risers if the building movements are large (more than 9 inches of movement).

The floor-to-floor drift is not a major concern for the pipe risers, since with the exception of the 6-inch wet stand pipe, most of them are located in pre-cut slots which have excess openings to compensate for the movements. The glass acid vent risers have transverse joints have rubber gaskets with stainless steel bands which provide a little flexibility. The rain water leader has ball and spigot joint fittings which allow some movement on the pipe. The integrity of the 6-inch wet standpipe needs further evaluation. However, the survival of this pipe is questionable in a major seismic event. According to the structural analysis, the largest floor drift occurs between Levels 7 to 10. Fortunately, Levels 7 to 10 in both HSE and HSW have been remodeled. Most of the pipe risers have been re-routed/offset to fit the new partition layout, which provides further flexibility to the piping system. The perimeter fume exhaust duct risers may not survive a major seismic event. Unlike the interior pipe risers, there are no horizontal duct offsets in the risers to provide additional compensation for movement.

The floor acceleration is more difficult to predict or analyze, since this depends on the methods and type of structural anchorage used in the design of the equipment support, and there is no means to check this easily without performing a destructive analysis or test. Visual observation only will not provide sufficient clues or information on the condition of the supports and anchorage. To comply with the current code, the attachment to concrete structure on equipment installed pre-1998 CBC should be analyzed, particularly large and heavy equipment, such as ceiling-mounted fan coil units, main pipe risers in the stair breezeway, and main supply and general exhaust risers above the ceiling. This will require further study and investigation. Building system retrofit matrix in Appendix C shows the retrofit category and seismic component that may cause the major damage to the particular mechanical system.

6. ELECTRICAL SYSTEMS

6.1. INTRODUCTION

The Health Science Instruction and Research (HSIR) buildings were originally built in 1962 and have undergone extensive renovations. These renovations include several major electrical infrastructure projects. The 120/208 volt substation HSIR-2 was added in 1983. In 1992 a major electrical expansion was undertaken. A structural steel bridge and equipment platform was constructed at the roof of HSE to accommodate the new electrical equipment. This expansion included 12kV switchgear, single-ended 277/480 volt substation HSIR-3 and double-ended 120/208 volt substation HSIR-4A/4B. The Library Release Space Improvements project added an emergency generator and a walk-in enclosure with motor control centers at the bridge/equipment platform at HSE roof. The Outside Plant SCADA project added a pre-fabricated equipment building at HSE roof. HSE Improvements Phase 1 added a non-automatic transfer switch and distribution panel at HSE roof. HSW Improvements Phase 1 added a stainless steel walk-in equipment enclosure to house motor control centers, transfer switches, and distribution panels. Numerous other renovations have taken place. These typically involve remodeling a single floor in either HSE or HSW into a modern laboratory suite. The remodel projects would typically involve new panelboards and feeders, lights and lighting control panels, as well as branch wiring for lighting, convenience receptacles, fire alarm, etc.

The renovation projects were typically designed per the building codes in effect at the time of design, including structural and seismic anchoring codes. Most equipment installed in the renovation projects included some type of anchorage. Building codes are updated every three years and the requirements usually become more stringent. Although electrical equipment provided with renovation projects may have had code-compliant seismic anchorage at the time of their installation, the anchorage may not comply with current codes relative to seismic anchorage.

The HSIR buildings have an emergency power system. A portion of the system is served by a local generator rated 200kW located at the roof of HSE. Another portion of the system is served by remote generators located at the Central Utilities Plant (CUP) through an emergency power substation rated 1500 kVA. The emergency power system serves fume hood fans, ventilation fans, emergency egress lighting, and other emergency loads. The local generator has on-site fuel sufficient for 24 hours of operation. The CUP generators are supplied by on-site fuel storage tanks with sufficient capacity to provide 24 hours to several days of operation, depending on loading conditions and other campus needs.

This seismic retrofit study investigates various electrical systems, the main distribution elements of these systems, and the distribution on two representative floors. This report presents apparent seismic vulnerabilities in the electrical systems related to interstory drift, acceleration, and seismic gap closing and possible retrofit measures.

This report identifies two levels of seismic retrofit scope for the electrical systems. These two levels are *Targeted Loss Reduction* and *Enhanced Resilience*. Retrofit scope for *Targeted Loss Reduction* addresses systems related to life safety and systems having major impact on building functionality or economic loss. These systems are primarily the emergency power system, emergency/egress lighting, and the fire alarm/life safety system. Retrofit scope for *Enhanced Resilience* addresses electrical systems required for minimizing laboratory downtime, preserve valuable research, and improve functionality following an earthquake. These systems are the normal and standby power systems, general building lighting, the SCADA and power monitoring systems, and the security system.

6.2. ELECTRICAL SYSTEMS INCLUDED IN SCOPE

The electrical systems that are included in the scope of this study are the following:

- Medium Voltage (12kV) Power
- Low Voltage (208V and 480V) Power
- Emergency Power Distribution System
- Emergency Generator System
- Lighting System
- Fire Alarm and Life Safety System
- Campus Supervisory Control And Data Acquisition SCADA and Power Monitoring Systems

6.3. MAIN DISTRIBUTION ELEMENTS

The main distribution elements for each of the electrical systems are described below for each of the systems:

1. Medium Voltage (12kV) Power

   - Primary Switching Station PSS-HSI-SS1 located outdoors, on grade, at the 2nd Floor courtyard near the elevator tower. This equipment is in good condition, was installed in 2002, and is seismically anchored to a concrete slab at grade level. Vulnerability is minimal and no retrofit work is recommended.

   - Primary Switching Station PSS-HSI-SS2 located on the equipment bridge/platform at the roof of the HSE tower.

     Primary Switching Station PSS-HSI-SS2 is anchored to the equipment bridge/platform at the roof of the HSE tower. However, this anchorage was designed per code requirements in effect in 1992 and the anchorage may not comply with current codes that consider the building height at the anchorage location and the effects on seismic forces. This equipment anchored to the equipment bridge/platform and vulnerability is minimal. Retrofit work recommended would be detailed evaluation of the existing anchorage and possible addition of anchors. This equipment is part of the normal/standby power system and retrofit would be provided for *Enhanced Resilience* category.

   - 12kV feeders running vertically from grade level to the roof of HSE.

     The 12kV feeders running vertically from grade level to the roof of HSE are anchored to the Core building where they come out of the ground. The feeders then cross the separation joint between the Core building and HSE and are anchored to the "eyebrows" at each floor of HSE. The feeders cross the building separation joint at the 2nd floor level without any expansion fittings in the conduits. Because these feeders are located in the seismic gap between two structures they are vulnerable to damage resulting from interstory drift at the upper floors. The vulnerability is minimal where the feeders come out of the ground and cross the seismic gap because relative building motion and interstory drift is minimal at grade level. The 12kV feeders are part of the normal and standby power systems and retrofit would be required for *Enhanced Resilience*. The recommended retrofit is to relocate the feeders from the eyebrow on the north side of HSE to the west side of HSE, out of the seismic gap. Expansion fittings at the 2nd floor level would also be added as part of this relocation.

- 480 volt feeders, ground system conductor, and SCADA fiber optics running vertically from grade level to the roof of HSE.

  These electrical systems run vertically from grade level to the roof of HSE and are run parallel to the 12kV feeders. Because systems are located in the seismic gap between two structures they are vulnerable to damage resulting from interstory drift at the upper floors, similar to the 12kV feeders. These systems are part of the normal and standby power systems and SCADA system, and retrofit would be required for *Enhanced Resilience*. The recommended retrofit is to relocate the conduits and wiring from the eyebrow on the north side of HSE to the west side of HSE, out of the seismic gap.

2. Low Voltage (208V and 480V) Power

- Substation HSIR-1A/1B, 277/480 volt double-ended substation located at the 1st Floor main electrical room. The substation consists of two 1500 kVA dry type substation transformers and 2000 amp switchgear. This equipment is in good condition, was installed in 2002, and is seismically anchored to a concrete slab at grade level. Vulnerability is minimal and no retrofit work is recommended.
- Substation HSIR-2, 120/208 volt double-ended substation located at the 1st Floor main electrical room. The substation consists of one 1000 kVA dry type substation transformer and 4000 amp switchgear. This equipment is in good condition, was installed in 2002, and is seismically anchored to a concrete slab at grade level. Vulnerability is minimal and no retrofit work is recommended.
- Substation HSIR-3, 277/480 volt substation located on the equipment bridge/platform at the roof of the HSE tower. The substation consists of one 1500 kVA liquid-filled substation transformer and 3200 amp switchgear.

  Substation HSIR-3 transformer and switchgear are anchored to the equipment bridge/platform at the roof of the HSE tower. However, this anchorage was designed per code requirements in effect in 1992 and the anchorage may not comply with current codes that consider the building height at the anchorage location and the effects on seismic forces. This equipment anchored to the equipment bridge/platform and vulnerability is minimal. Retrofit work recommended would be detailed evaluation of the existing anchorage and possible addition of anchors. This equipment is part of the normal/standby power system and retrofit would be provided for *Enhanced Resilience* category.
- Substation HSIR-4, 120/208 volt double-ended substation located on the equipment bridge/platform at the roof of the HSE tower. The substation consists of two 1000 kVA liquid-filled substation transformers and 3200 amp switchgear. The 120/208 volt switchgear was recently replaced due to severe corrosion and deterioration of the original switchgear. The new switchgear is located in a stainless steel, conditioned, electrical equipment enclosure (electrocenter). The electrocenter is anchored to the equipment bridge/platform. The new switchgear and electrocenter is heavier than the original switchgear. The adequacy of the equipment bridge/platform to accept the additional weight was evaluated in accordance with the 2007 California Building Code and the anchorage of the new equipment was designed to comply with the 2007 CBC. The vulnerability of this equipment is minimal and no retrofit work is recommended.

  Substation transformers T4A and T4B of Substation HSIR-4A/4B are anchored to the equipment bridge/platform at the roof of the HSE tower. However, this anchorage was designed per code requirements in effect in 1992 and the anchorage may not comply with current codes that consider the building height at the anchorage location and the effects on seismic forces. This equipment anchored to the equipment bridge/platform and vulnerability is minimal. Retrofit work recommended would be detailed evaluation of the existing anchorage and possible addition of anchors. This equipment is part of the normal/standby power system and retrofit would be provided for *Enhanced Resilience* category.

- Emergency power distribution system consists of 480 volt feeders, distribution panels, automatic transfer switch, and motor control centers in HSW. This system primarily serves ventilation fans and fume hood fans as required by the California Building Code for H8 occupancies. The emergency power equipment was installed in 2005, is in good condition and is anchored to the HSW building structure at the 2nd floor or at the roof. The anchorage was designed and installed in accordance with the 2004 California Building Code. The vulnerability of this equipment is minimal and no retrofit work is recommended.

- The emergency power distribution system also includes a 480 volt feeder, 480-208Y/120 volt step down transformer, distribution panel, and a 120/208 volt riser with emergency power panelboards on each floor of the HSIR complex. The riser and panelboards serve emergency egress lighting, fire alarm system, and other selected loads. This emergency power equipment is in good condition and is anchored to concrete floor slab at the main electrical room at the 1st floor and panelboards are mounted on the concrete walls of the stacked electrical closets at floors 2 through 16. The vulnerability of this equipment appears to be minimal but since this equipment is part of the emergency power system retrofit work recommended would be detailed evaluation of the existing anchorage and possible addition of anchorage. This retrofit would be included in the *Targeted Loss Reduction* category.

- The emergency power distribution system is supplied by emergency diesel generators (EDGs) located at the Central Utilities Plant (CUP). The EDGs generator power at 12kV and the power is distributed to emergency power substation EHSIR on a 12kV emergency power feeder. Substation EHSIR consists of a high voltage fused switch, 12,000-480Y/277 volt substation transformer, and 480/277 volt switchboard. The EDGs, 12kV emergency feeder, and Substation EHSIR are located outside of the HSIR buildings and are therefore outside the scope of the HSIR seismic study.

- Bus Risers EL and WL, rated 1000 amp 277/480 volts. These bus risers serve primarily lighting, and run from the 1st floor to the 16th floor vertically through the stacked electrical closets located in the core, or connecting building between HSE and HSW. The stacked electrical closets have concrete floors and concrete walls. The bus risers are anchored to the concrete floors and walls and vulnerability is minimal.

- Bus Risers EP1SW, WP2NE, EP3SW, WP4NE, rated 1000 amp 120/208 volts. These bus risers serve primarily receptacle power load, and run from the 2nd floor to the 16th floor in HSE and from the 3rd Floor to the 16th Floor in HSW. These busses are located in the southwest and northeast corners of the HSE and HSW towers. These bus risers are anchored to the concrete floor but in most cases do not have attachment to walls. These busses are more vulnerable than the lighting busses and they typically serve laboratory loads on each floor. Failure of these busses would have greater impact on building functionality and economic loss.

  Power Bus Risers EP1SW, WP2NE, EP3SW, WP4NE and Lighting Bus Risers EL and WL are rigid assemblies consisting of solid aluminum bus bars enclosed within a steel housing.

The busses are supported/anchored at each floor but they do not have any expansion fittings. The bus sections at each floor would tend to move with the floor during an earthquake. Differences in relative motion at different floors would cause the rigid busway assemblies to deflect, possibly damaging the busses or causing the bolted joint of the busway sections to fail. Retrofit work recommended would be detailed evaluation of the existing anchorage and possible addition of anchors. The lighting risers EL and WL are part of the normal/standby power system and retrofit would be provided for *Enhanced Resilience* category. Since the power bus risers have higher vulnerability and also serve critical research loads the retrofit work for these busses is recommended to be included in the *Targeted Loss Reduction* category.

- Motor Control Centers MCC-A, MCC-C, and MCC-E. These motor control centers are located in the mechanical spaces at the 1st Floor in HSE and 2nd Floor in HSW.

  MCC-A and MCC-C are old original motor control centers dating back to 1962. The anchorage of these motor control centers is questionable. Retrofit work recommended would be detailed evaluation of the existing anchorage and likely addition of anchors. This equipment is part of the normal/standby power system and retrofit would be provided for the *Targeted Loss Reduction* and *Enhanced Resilience* category

- Stainless steel walk-in equipment enclosure at HSW roof housing motor control centers, transfer switches, and distribution panels.

  Stainless steel walk-in equipment enclosure at HSW roof housing motor control centers, transfer switches, and distribution panels are anchored to the roof of the HSW tower. However, this anchorage was designed per code requirements in effect in 2004 and the anchorage may not comply with current codes that consider the building height at the anchorage location and the effects on seismic forces This equipment was installed in 2005, is in good condition and is anchored to the HSW building structure at the roof. The anchorage was designed and installed in accordance with the 2004 California Building Code. The vulnerability of this equipment is minimal and no retrofit work is recommended

- Distribution feeders and branch circuit wiring at the main electrical room, the roof, and the representative floors (HSE 3 and HSE 16).

  Some conduits of various electrical systems cross the building separation joints without expansion fittings or flexible conduit at the crossing point. These conduits are vulnerable to damage due to the lack of expansion fittings or flexible conduit. Retrofit work recommended would be addition of expansion fittings of flexible conduit to the conduit systems at the building separation joints and re-pulling of the related wiring in the conduit(s). This work would be provided for the *Targeted Loss Reduction* category for conduits and wiring associated with the emergency power system and the fire alarm and life safety system. For conduits for all other systems this work would be provided for the *Enhanced Resilience* category.

  120/208 volt and 480 volt feeder conduits cross the building separation joints at the roof. These feeders typically consist of liquid tight flexible conduit at the building separation joint crossings to allow for relative motion. However, the liquid tight conduit is showing signs of deterioration due to exposure to the elements. The length of flexible conduits at the separation joints should be compared with the expected relative movement of the buildings at the separation joints to verify sufficient slack exists to accommodate the expected movement. Retrofit work recommended would be addition of expansion fittings and replacement of the deteriorated flexible conduit at the building separation joints at the roof and re-pulling of the related wiring in the conduits. Since these conduits are associated with the normal/standby power system this work would be provided for the *Enhanced Resilience* category.

3. Emergency Generator System

- Emergency Generator, rated 200kW 277/480 volts, in walk-in type sound attenuated weather housing located on the equipment bridge/platform at the roof of the HSE tower. This generator serves individual fume hoods at HSW, HSE and the adjacent Medical Sciences Building. The housing also includes a fuel oil day tank, electrical distribution panel, and other accessories. Fuel oil to serve the generator is pumped from a fuel oil storage tank at the 5th floor. The fuel oil tank provides for a minimum of 24 hours of operation. The generator is anchored to the equipment bridge/platform and vulnerability is minimal.

  The Emergency Generator and walk-in enclosure and day tank are anchored to the equipment bridge/platform at the roof of the HSE tower. However, this anchorage was designed per code requirements in effect in 1992 and the anchorage may not comply with current codes that consider the building height at the anchorage location and the effects on seismic forces. Retrofit work recommended would be detailed evaluation of the existing anchorage and possible addition of anchors. This equipment is part of the emergency power system and retrofit would be provided for the *Targeted Loss Reduction* category

  A master plan for emergency and standby power has been developed for the campus. This master plan calls for a new 2000kW generator to serve the HSIR complex. Upon implementation of this element of the master plan, the existing 200kW generator at the roof of HSE will no longer be required and should be removed. Depending on the timing of the new emergency generator relative to the timing of the retrofit work, the recommended retrofit work would be changed to include the removal of the existing generator and enclosure and the related accessories, wiring, and fuel oil system

- Fuel oil day tank located within the generator enclosure and double-wall fuel oil piping rising from the fuel oil storage tank at the 5th floor level up to the HSE roof.

4. Lighting System

- Lights are located throughout all floors of HSE and HSW. The main elements are the luminaries (fixtures), branch circuit panelboards, and lighting control panels or other control devices. This study focuses on HSE 3 and HSE 16 as representative floors.

  The light fixtures in floors that have been renovated in the last 15-20 typically have support wires independent of the ceiling support system. They also typically have swing restraints on suspended and pendant type fixtures to prevent light fixtures from colliding with other fixtures or systems as the light fixtures swing during seismic events. The representative floors, HSE 3 and HSE 16 have been renovated in the last 20 years and the lights at these floors do not pose vulnerabilities. However, other floors in HSE and HSW have not been renovated and light fixtures may not have the required independent support wires and swing restraints.

  Recommended retrofit measure to limit the seismic vulnerability of this component is addition of independent support wires and swing restraints to existing lighting fixtures. This work is recommended for the *Enhanced Resilience* category.

5. Fire Alarm and Life Safety System

- Fire Alarm and Life Safety System components are located throughout all floors of HSE and HSW. The main Building Fire Control System (BFCS) panels are located at the 3rd Floor in the Core building connecting HSE and HSW.

  The Building Fire Control System (BFCS) cabinets and Fire Alarm Distribution Panels (FDPs) were installed as part of the campus-wide Fire and Life Safety System Upgrade project done in the early 1990's. However, this anchorage was designed per code requirements in effect in at the time and the anchorage may not comply with current codes that consider the building height at the anchorage location and the effects on seismic forces. Retrofit work recommended would be detailed evaluation of the existing anchorage and possible addition of anchors. This equipment is part of the fire alarm and life safety system and retrofit would be provided for the *Targeted Loss Reduction* category

6. Campus SCADA and Power Monitoring Systems

- The SCADA system Distributed Processing Unit DPU-16 is located at the 1st Floor Main Electrical room. Modbus Gateway MBG-16 is mounted inside the DPU-16 cabinet.
- The SCADA system DPU-11 is located in a pre-fabricated equipment enclosure/building on the equipment bridge/platform at the roof of the HSE tower. Modbus Gateway MBG-11 is mounted inside an enclosure that is located inside the SCADA building.

  The SCADA pre-fabricated building is anchored to the equipment bridge/platform at the roof of the HSE tower. However, this anchorage was designed per code requirements in effect in 1992 and the anchorage may not comply with current codes that consider the building height at the anchorage location and the effects on seismic forces. This equipment anchored to the equipment bridge/platform and vulnerability is minimal. Retrofit work recommended would be detailed evaluation of the existing anchorage and possible addition of anchors. This equipment is part of the SCADA system and retrofit would be provided for *Enhanced Resilience* category.

7. INFORMATION TECHNOLOGY SYSTEMS

7.1. INTRODUCTION

The purpose of this study is to evaluate the technology information support systems for both the HSE and HSW buildings, as part of the seismic evaluation of the building structure conducted by Rutherford & Chekene. This evaluation is based on our 10 years of experience working in these buildings on multiple TI upgrade projects, field observations, and interviews with UCSF OASIS ENS staff.

The information support systems evaluation included the impact of the relative building movements (separation joint motion), and floor acceleration rating at both low and high vulnerability. Two system retrofits are presented based on by criteria created by Rutherford and Chekene (CBC Chapter 34, *Targeted Loss Reduction*, and *Enhanced Resilience*). See the Building System Retrofit Matrix – Information Systems for more detail.

Originally HSE and HSW buildings were built with multi-pair copper risers extending from the basement levels to the top floors in each tower to provide phone service from AT&T. In the 1980's UCSF installed its own PBX (phone system) in the basement of Long Hospital and extended services to the HSE and HSW towers via copper tie cables to the basement levels and used the existing building risers to deliver voice services.

In the late 1980's with the proliferation of voice services additional risers were added to dedicated floors. These new risers were distributed from the MSB Telecom Rooms that are vertically stacked and located in the Medical Science Building near the HSE and HSW elevator tower. Copper cable was then run from MSB Telecom Rooms via cable tray located in the hallway corridors to new telecom rooms or wall mounted termination fields.

With the development of IP connectivity via fiber optic cable in the 1990's, fiber optic cable was installed to most floors in the HSE and HSW towers directly from the MSB Telecom Room via the existing corridor cable trays.

Since the early 1990's there have been a substantial amount of tenant space improvements (related to technology) that have lead to the installation of additional copper and fiber optic cables from the MSB Telecom Rooms to both towers.

Along with the installation of copper and fiber optic backbone has been the installation of network equipment racks which hold most, if not all, network equipment in the towers. Equipment racks are installed in two formats: wall mounted racks or floor mounted racks. While these racks are robust enough to hold and support most equipment they typically fail do to the manner in which they were installed. In the HSE and HSW buildings most wall mounted racks are toggle bolted to drywall and floor mounted racks are installed to manufactures instructions which are not compliant with the requirements of seismic design category D.

This seismic retrofit study investigates various information support systems, the main distribution and racking elements of these systems. This report presents apparent seismic vulnerabilities relative to floor acceleration, and seismic gap closing as it applies to the information support systems and possible retrofit measures.-

7.2. Information Support Systems Included In Scope

The information support systems that are included in the scope of this study are the following:

- Interbuilding Cable Tray System
- Network Equipment Racks

7.3. Information System Elements

The main distribution elements for each of the information systems are described below for each of the systems:

1. Interbuilding Cable Tray System

- As discussed above in the Information Technology Support Systems Introduction there is an existing cable tray (12"W x 4"D) that connects the MSB, HSE and HSW building and is installed in the ceiling of the elevator tower and crosses all three separation joints (see attached sketch).
- The cable tray system houses almost all fiber optic cables that provide data connectivity to both towers and about 40% of the multipair copper cables that supply voice connectivity. Also collocated in the cable tray are copper and fiber optic backbone cables that tie directly between labs located in the MSB, HSE and HSW towers.
- The cable tray system is continuous and does not include separation joints at key intersections and has limited flexibility to handle stresses during any type of separation joint motion. Cable tray retrofit should be considered during a *Target Loss Reduction* retrofit and has a low vulnerability during floor acceleration.

  A simple and cost effective retrofit measure to limit the seismic vulnerability of this component during a separation joint motion moment would be to modifying the existing cable tray at the separation joints to provide slack in the components to accommodate seismic drift.

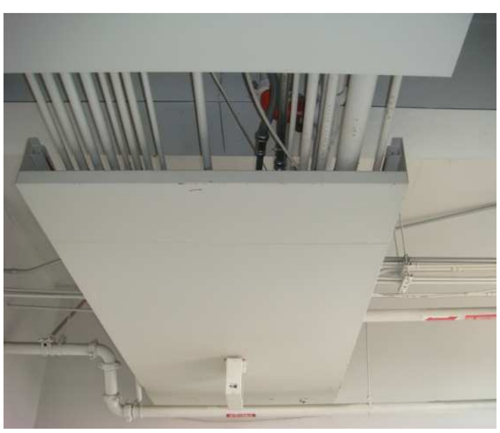


Figure 92: Typical conduit routing between connecting corridor and laboratory building.

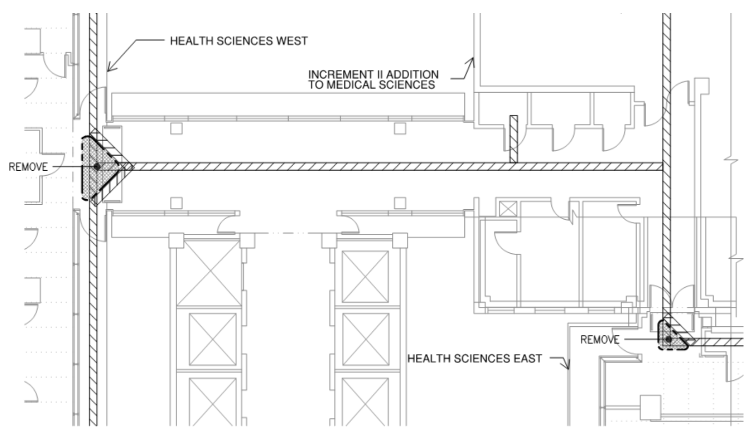


Figure 93: Schematic illustration of cable tray retrofit to increase slack in information technology cables.

2. Network Equipment Racks

- Network Equipment Racks have only been installed in the HSE and HSW towers since the late 1980's. The racks consist of two types: wall mounted and floor mounted.
- Wall mounted racks are typically mounted directly to walls via toggle bolts and can range in size from 19”W x 12”-36”H. Wall mounted racks are sporadic in both towers and quantities may be 15-25 racks total between both HSE and HSW towers. These racks typically house voice and data station cables (copper 4-pair cables) and network equipment switches. The majority of these racks have been installed in a method that is not compliant with the requirements of seismic design category D. Most racks have less than 50 lbs of equipment but need to be addressed in this report due to loss of network connectivity (usually local to the lab) and possible bracing failure due to floor acceleration. This type of equipment is similar in size and content and is discussed in further detail in the Lab Equipment Section below.
- Floor mounted equipment racks (19”W x 7’H) are similar to wall mounted racks when it comes to the content of the racks but can exceed 100 lbs if local UPS equipment is also installed. Like wall mounted racks, most floor mounted racks have been installed since the mid 1980's do not meet the current seismic design category D requirements. The only racks that meet this requirement are those installed after 2000 as this is when structural designs for equipment racks became mandatory with most designs and TI updates at UCSF. The importance of most floor mount racks is that they serve much larger areas (multiple labs if not multiple floors) and the replacement time for some of this equipment would be much longer (days to weeks depending on the event) than they wall mounted racks.
- Racks while robust are susceptible to seismic events and should be reviewed on an individual basis for structural integrity. The best plan would be to place the review and update of anchoring in the *Targeted Loss Reduction* model at a minimum with the understanding that floor acceleration has the greatest effect on their sustainability.
- A simple and cost effective retrofit measure to limit the seismic vulnerability of this component during a high floor acceleration moment would be to modify the mounting of wall racks by adding a plywood backboard between the wall & rack and replace the concrete anchors in floor mounted racks. All of this could be done without impacting equipment or affecting the users in most cases.

8. LABORATORY CONTENTS AND EQUIPMENT

8.1. LABORATORY CONTENTS AND EQUIPMENT RETROFIT SCOPES

INTRODUCTION

The goal of the HSIR contents analysis is to define the benefits and costs associated with the provision of seismic anchorage for laboratory contents and equipment, leading to recommended retrofit options for UCSF. The California Building Code, Chapter 34 seismic evaluation defines a legal minimum standard of protection. To minimize direct economic losses and downtime following an earthquake, it may be beneficial to provide additional seismic restraint beyond the minimum levels prescribed by the CBC. However, it is generally not feasible to provide anchorage for all laboratory equipment. Such an anchorage policy would be prohibitively expensive and would negatively affect productivity for laboratory occupants.

To quantify the benefits and costs of securing laboratory equipment over a range of options, we have identified four possible levels of seismic anchorage scope. The lowest level provides a minimum cost approach associated with anchoring the minimum amount of equipment. This scope, *CØ*, is defined by the requirements of the California Building Code, Chapter 34. At each subsequent anchorage scope, additional anchorage requirements are added as more pieces of equipment are restrained, representing an increased investment in seismic anchorage and additional benefits. The additional, increasing anchorage scopes we have defined are: *C1: Targeted Loss Reduction, C2: High Value, and C3: Enhanced Resilience.* Each scope is associated with a set of specific anchorage criteria, which identify the equipment targeted for seismic retrofit. These criteria are outlined in Table 31. Additionally, each scope is inclusive of all the equipment anchored in a lower scope. Thus, all equipment that is anchored in scope *C1* will, by definition, be anchored in scope *C2*.

RETROFIT SCOPE TO MEET CALIFORNIA BUILDING CODE, CHAPTER 34 (SCOPE C0)

Laboratory equipment anchorage scope *CØ* is defined to be equal to the minimum requirements of a seismic evaluation triggered by Chapter 34 of the California Building Code. Chapter 34 covers existing structures and defines performance criteria associated with a seismic hazard level. At the *BSE-R* hazard level, representing a ground motion with a 225-year return period, nonstructural components must meet the *Hazards Reduced (N-D)* performance level defined in the ASCE 41-06 Standard. Hazards reduced performance is intended provide Life Safety protection to areas of public occupancy by securing high hazard components. Specifically, ASCE 41-06 1.5.2.4 requires that contents "are secured to prevent falling into areas of public assembly. Preservation of egress, protection of fire suppression systems, and similar life-safety issues are not addressed." Footnote 15, Table 11-1, requires Life Safety performance "Where heavy nonstructural components are located in areas of public occupancy or egress." We interpret this footnote to refer to areas of public occupancy or public egress, not to general exit paths, an interpretation consistent with ASCE 41-06 1.5.2.4.

At the Health Sciences Building complex, the majority of the floor area is dedicated laboratory and research space which does not provide public occupancy. As a result, the only applicable requirement of the CBC refers to the containment of hazardous chemicals. We interpret that the *Hazards Reduced* performance objective is satisfied when strong acids, bases, and toxic or radioactive chemicals are stored in self-closing, self-latching chemical cabinets which are seismically anchored or have a minimum horizontal dimension greater than 2/3 the cabinet height in order to prevent tipping. In these situations, chemicals which may combine to form hazardous compounds should be stored separately.

Table 31: Retrofit options for laboratory contents.

Category	CØ : CBC Chapter 34	C1: Targeted Loss Reduction	C2: High Value	C3: Added Resilience
Shape Criteria	No Criteria	Anchor: items > 50 lb stored > 5 ft from floor; items > 500 lb; items > 200 lb and min horizontal dimension < 1/2 height	Anchor: items > 20 lb stored > 5 ft from floor; items > 5 ft tall; items > 200 lb; items with min horizontal dimension < 2/3 height; items that could block exit path	Anchor: items > 20 lb stored > 5 ft from floor; items > 5 ft tall; items > 200 lb; items with min horizontal dimension < 2/3 height; items that could block exit path
Wheels	No Criteria	Anchor wheeled equipment with weight > 200 lb	Anchor all wheeled equipment	Anchor all wheeled equipment
Gases	No Criteria	Anchor all tall compressed gases and LN2	Anchor all compressed gases and LN2	Anchor all compressed gases and LN2
Fridges/ Freezers	No Criteria	Refer to Shape criteria	Refer to Shape criteria	Anchor all tall fridges, freezers. Provide interior protection for valuable, sensitive contents
Chemicals	Acids, toxics, radioactive	Acids, toxics, radioactive	Acids, toxics, radioactive	Acids, toxics, radioactive
Cabinets	No Criteria	Refer to Shape criteria	Anchor tall, moveable. Provide positive latches.	Anchor all. Provide interior restraint for valuable, sensitive contents
Shelving	No Criteria	No Criteria	Provide 1 1/2" lip, anchor shelves	Provide 4" lip/restraint, anchor shelves
Item Value	No Criteria	>$50,000	>$20,000	>$10,000
Research Samples	No Criteria	Protect irreplaceable samples	Protect irreplaceable samples	Protect irreplaceable and high-value samples
Importance	No Criteria	No Criteria	No Criteria	Important equipment or equipment required to provide function for important equipment
Partitions	No Criteria	No Criteria	No Criteria[3]	Check partition adequacy in existing labs[2]
Existing Anchorage	No Criteria	Existing anchorage is assumed to be sufficient. Remove slack from existing chains or straps	Existing anchorage is assumed to be sufficient. Remove slack from existing chains or straps	Check existing seismic anchorage in existing labs. Remove slack from existing chains or straps
Restraint Track	No Criteria	Add track to all newly constructed lab benches	Add track to all lab benches	Add track to all lab benches

<u>Application of *CØ* Scope to HSE16 and HSE3</u>

In the HSE16 laboratory we visited, we did not observe any dedicated hazardous chemical storage. Laboratory chemicals are present in numerous spaces, but we are not aware of the nature of the

[3] If the criteria are applied to new or fully remodeled laboratories, partitions and backing should be designed for seismic anchorage of attached contents.

chemicals. At the HSE3 laboratory, we observed that hazardous chemical storage satisfied the code-specified performance objective. Strong acids and bases were stored self-closing, self-latching chemical cabinets with a low aspect ratio which would resist tipping. Flammable chemicals were stored in a fire cabinet, seismically anchored to a partition wall. Additional chemicals were stored in cold rooms and on lab benches, but we are not aware of the nature of the chemicals. We were informed by Frank Billante of UCSF EH&S that a University hazardous chemical storage policy, which meets CBC Chapter 34 requirements, is in place. Beyond this action, no further work is required for the labs to meet contents retrofit scope *CØ* .

C1: TARGETED LOSS REDUCTION

We define contents anchorage scope *C1: Targeted Loss Reduction* by the criteria identified in Table 31. This anchorage scope provides restraint for most compressed gas cylinders, sub-80 freezers, bio-safety cabinets, and stacked incubators. Generally, the *C1* scope includes the following items: heavy equipment stored on shelves, very heavy equipment stored on the floor, or heavy equipment with a high aspect ratio, which is likely to overturn. Additionally, this anchorage scope protects the most valuable laboratory components by requiring seismic anchorage for contents valued at more than $50,000. For the UCSF HSIR laboratory inventory, 20% of laboratory contents fall into this category.

For this retrofit scope, are not investigated for their ability to provide seismic restraint. Existing seismic anchorages are also assumed to be sufficient, unless they are clearly deficient upon inspection. Loose chains or straps in existing seismic anchorages are to have the slack removed as part of this scope.

Contents anchorage scope *C1* is not defined by the CBC. We created this scope in order to provide targeted seismic anchorage of the components which pose the greatest danger to human life or the most severe direct economic loss. The *C1: Targeted Loss Reduction* anchorage scope is not intended to prevent the risk of all serious injury, loss of laboratory equipment functionality, or destruction of laboratory contents. The *C1* retrofit scope may be the most appropriate approach when limited funding does not permit the implementation of a more comprehensive seismic anchorage scheme.

Application of *C1* Scope to HSE16

At the newly renovated HSE16 laboratory, we documented the location of 120 pieces of major laboratory contents. The pieces can be categorized into 58 contents categories. Typically, chemical bottles and glassware, books, small bench-top equipment, small microscopes, and typical computers and computer monitors were omitted from our inventory. In this laboratory, 59 of the 120 pieces of contents had some form of seismic anchorage at the time of our visit. Applying the *C1: Targeted Loss Reduction* anchorage scope to the HSE16 laboratory spaces we visited would result in requiring seismic anchorage for 63 total pieces of equipment or 53% of the contents we inventoried. Of these 63 items, 23 did not already have seismic anchorage. Typical contents anchored by implementation of the *C1: Targeted Loss Reduction* anchorage scope would include the following:

- 18 compressed N2 and CO2 gas cylinders
- 14 incubators
- 2 nitrogen cryo-storage containers
- 7 freezers
- 6 bio-safety cabinets
- 2 microscope air tables (valued at $60,000 each)
- 14 other items

63 total items

Application of *C1* Scope to HSE3

The HSE3 laboratory space is slightly older than HSE16. At the HSE3 laboratory, our inventory includes 90 items in 66 categories. Again, small equipment is intentionally omitted from our documentation. In this laboratory, 26 total items were seismically anchored. When the *C1: Targeted Loss Reduction* anchorage scope is applied to this laboratory, 39 items, or 43% of the contents in our inventory requires seismic anchorage. Of these 39 items, 23 are currently unanchored. Typical equipment affected by implementation of the *C1: Targeted Loss Reduction* anchorage scope would include the following:

- 7 liquid nitrogen dewars
- 6 incubators
- 9 bio-safety cabinets/fume hoods
- 3 nitrogen cryo-storage devices
- 1 DNA isolation system (valued at $75,000)
- 13 other items

39 total items

C2: High Value

We define contents anchorage scope *C2* according to the criteria identified in Table 31. The seismic anchorage provided in this scope includes everything anchored under the scope of *C1*, as well as a number of additional items like most full height refrigerators and freezers, heavy bench equipment like some centrifuges, and cabinets and shelving. In this case, tall cabinets should be anchored, with positive latches. Shelves should have a minimum 1½" lip.

We identified this scope with the goal of providing protection for the most valuable research contents and equipment and to provide additional protection against overturning and falling of dangerous items. This means that moderately heavy items stored on shelves should be moved or restrained, and heavy, floor mounted contents should be anchored. Additionally, *C2* requires anchorage for the high replacement values items defined by scope *C1*. This scope does not explicitly address the functionality of laboratories or laboratory equipment, the protection of contents within refrigerators, freezers, or cabinets, or the prevention of all injuries due to an earthquake.

Application of *C2* Scope to HSE16

When we apply the *C2: High Value* scope to the HSE16 laboratory inventory, a total of 92 of the 120 major contents items, or 77%, require seismic anchorage. Of these 92 items, 33 currently lack seismic anchorage. Typical work associated with this retrofit scope would include the following items:

- 63 items anchored per retrofit scope C1
- 10 cabinets
- 11 sets of shelves
- 4 refrigerators/freezers
- 4 lighter nitrogen cryo-storage containers

92 total items

Application of *C2* Scope to HSE3

At the HSE3 laboratory spaces, results are similar. For the *C2: High Value* scope, a total of 68 items of the 90 items, or 76%, require seismic anchorage. In this older laboratory, 39 of these items did not have seismic anchorage at the time of our visit. (We have observed the trend that in recently renovated laboratories, a greater proportion of the contents are anchored.) Under the *C2* scope, the following items would require seismic anchorage:

- 39 items anchored per retrofit scope C1
- 9 refrigerators/freezers
- 7 sets of shelves
- 3 cabinets
- 3 liquid nitrogen dewars
- 2 centrifuges
- 5 other items

  68 total items

C3: ENHANCED RESILIENCE

Contents anchorage retrofit scope *C3* is defined according to the requirements of Table 31. In addition to the criteria identified for scope *C2*, this retrofit scope adds further protection for samples and equipment intended to reduce laboratory downtime, preserve valuable research, and improve functionality following an earthquake. Protection is provided for the storage of valuable samples. At shelves, larger lips or additional restraint are required to protect contents from falling. Under these criteria, the adequacy of existing partitions and existing equipment anchorage to resist the demands imposed by earthquake forces is reviewed. Contents that are critical to laboratory productivity and equipment with a moderately high replacement cost are anchored.

In the *C3: Enhanced Resilience* scope, we recognize that providing anchorage to primary storage containers, like refrigerators, freezers, or cabinets, can protect the container from serious damage. However, in the research laboratory setting, the samples themselves can be of greater value than the primary container. In some cases, critical samples may be irreplaceable. In this instance, anchoring only the primary container may not afford protection for sample specimens. Retrofit scope *C3* addresses this issue by requiring secondary protection for the most valuable samples stored in refrigerators, freezers, or cabinets. Potential secondary protection measures include having copies of specimens stored offsite or requiring secondary packaging in freezers to limit the movement of contents.

We were informed that the University is in the process of creating an offsite freezer storage facility at Oyster Point in San Francisco. It is our understanding that the seismic adequacy of this facility has not been thoroughly investigated. Since it is likely that a large earthquake would cause significant ground shaking at both the Parnassus campus and at the Oyster Point Facility, one or both locations should consider additional secondary packaging and freezer anchorage requirements. Such requirements would ensure that samples would not be destroyed due to shaking. An alternative would be to provide offsite storage at a location further from the Parnassus campus to minimize the danger that a single earthquake would affect both sites simultaneously.

At this retrofit level, additional contents items are anchored due to their importance to laboratory functionality or replacement cost. During our laboratory inventory, managers completed a questionnaire in which they scored laboratory equipment based on its importance to maintaining laboratory productivity and research. These scores are used to identify important equipment which must be operational following an earthquake. Additionally, at this level, equipment that is necessary to support the function of important equipment should be protected. For example, computers that are necessary for data extraction from important bench-top equipment should be anchored. Further, contents with a replacement cost greater than $10,000 require anchorage under this scope. Approximately 40% of the laboratory equipment included in the UCSF HSIR inventory has a value greater than $10,000.

In addition to providing protection for valuable contents, the *C3: Enhanced Resilience* scope requires a comprehensive review of existing laboratory conditions. The adequacy of existing partitions and existing seismic anchorage will be reviewed. Where partitions are found to be deficient, secondary supports, like strong-backs or supplementary frames will be provided, or contents will be anchored to the slab. In cases where the existing seismic anchorage is found to be deficient, new anchorage will be provided.

The target of the *C3: Enhanced Resilience* retrofit scope is to minimize the effects of downtime, direct economic loss, and loss of productivity following an earthquake. This scope is not intended to provide complete protection of all contents or eliminate all injury risk, which would not be economically feasible or practically attainable.

<u>Application of *C3* Scope to HSE16</u>

When we apply the *C3: Enhanced Resilience* retrofit scope to the HSE16 laboratory, 107 of the 120 contents items, or 89%, require some form of seismic anchoring. 48 of these pieces are currently unanchored. However, because this scope requires review of existing anchorage adequacy, all 107 items would require some level of review or anchorage. Typical work for this anchorage scope would include the following items:

- 92 items anchored per retrofit scope C2
- 2 centrifuges
- 2 refrigerators
- 2 microscopes
- shelves- taller lip or additional restraint
- freezers and cabinets – secondary protection scheme

<u>Application of *C3* Scope to HSE3</u>

At the HSE3 laboratory spaces, the *C3: Enhanced Resilience* retrofit scope affects 83 of 90 major contents items we inventoried or 92% of the items. 57 of the items we have identified as requiring anchorage under this scope are currently unanchored. This retrofit scope would include items such as the following:

- 83 items anchored per retrofit scope C2
- 1 light nitrogen cryo-storage container
- various bench-top equipment including a centrifuge, incubator, gene sequencer, spectrophotometer
- shelves- taller lip or additional restraint
- freezers and cabinets – secondary protection scheme

Table 32: Summary of laboratory contents retrofit scopes.

Contents Items Anchored	C0 : CBC Chapter 34	C1: Targeted Loss Reduction	C2: High Value	C3: Added Resilience
HSE16				
Items requiring new anchorage	0	23	33	48
Items already anchored assumed acceptable	0	40	59	0
Items already anchored to be investigated for adequacy	0	0	0	59
Total items anchored in this scope	0	63	92	107
Percentage of inventoried items	0%	53%	77%	89%
Investigate partition adequacy	No	No	No	Yes
Tighten slack chain anchorage	No	Yes	Yes	Yes
HSE3				
Items requiring new anchorage	0	23	42	57
Items already anchored assumed acceptable	0	16	26	0
Items already anchored to be investigated for adequacy	0	0	0	26
Total items anchored in this scope	0	39	68	83
Percentage of inventoried items	0%	47%	69%	86%
Investigate partition adequacy	No	No	No	Yes
Tighten slack chain anchorage	No	Yes	Yes	Yes

8.2. LABORATORY CONTENTS FRAGILITY FUNCTIONS

For the study of the seismic performance of the HSIR laboratory contents, we define the critical operational unit to be individual laboratory-floors. Our contents fragility functions describe the level of disruption to a characteristic laboratory-floor.

We use the maximum and 4th worst SRSS peak horizontal floor accelerations (*PFA)* as the EDPs for the investigation of laboratory contents performance. Peak floor acceleration describes the maximum acceleration experienced by an infinitely stiff oscillator attached to the building structure during an earthquake.

The performance levels for the laboratory contents risk assessment are defined in Section 3: a *Fully Operational* state, a *Moderate Disruption* state, a *Major Disruption* state, and a *Life Risk* state. We develop fragility functions for the *Onset of Moderate Disruption*, *Onset of Major Disruption,* and *Life Risk* performance levels for the existing building and each of the laboratory contents retrofit scopes.

The fragility functions for the existing building configuration reflect the actual equipment condition observed during our inventories of the HSE16 and HSE3 laboratories. In each lab, some equipment had existing seismic anchorage. The fragility functions represent the composite capacity of all laboratory equipment at a floor, including both the components with seismic anchorage and the unrestrained components. We use the results of research conducted by Chaudhari and Hutchison (2006) and Lopez Garcia and Soong (2002) as the basis for our functions. The retrofit scopes anchor progressively more equipment, increasing the overall resistance of the laboratories to damage and disruption.

Typical laboratory contents fragility functions for the existing building are shown in Figure 94. A comparison of fragility functions for various contents retrofit scopes is shown Figure 95. A complete presentation of all fragility functions used in the risk assessment of laboratory contents is presented in Appendix K.

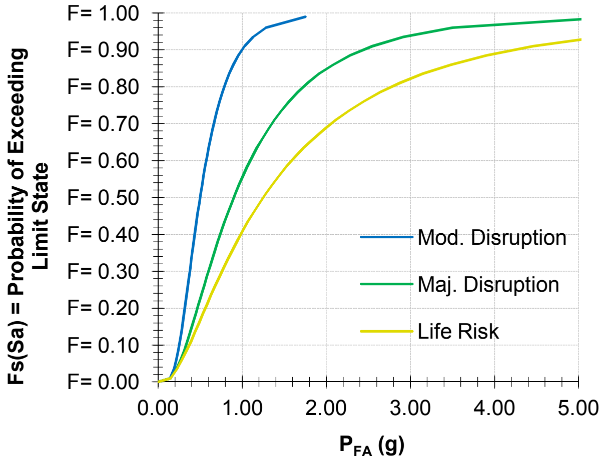


Figure 94: Laboratory contents fragility functions for existing building condition and maximum peak horizontal floor acceleration EDP.

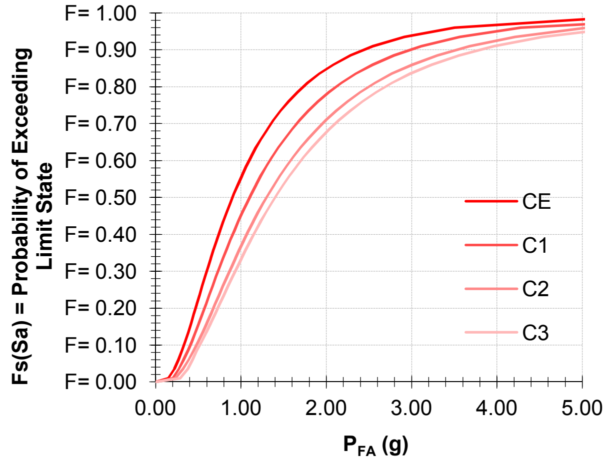


Figure 95: Comparison of laboratory contents fragility functions for existing building condition and three contents retrofit scopes. The fragility functions are shown for the maximum peak horizontal floor acceleration EDP

9. OTHER SYSTEMS

9.1. WINDOW GLAZING SYSTEMS

EXISTING LABORATORY WINDOW GLAZING SYSTEM

The existing window glazing at the Health Sciences laboratory buildings consists of a curtain wall system with open-web mullions. The windows are sensitive to interstory drift, the relative motion that occurs between adjacent floors in an earthquake. Because damage to the curtain wall system can affect the functionality of laboratories, we include fragility functions based on laboratory testing as part of our probabilistic assessment of overall seismic risk. The analysis indicates the windows contribute minimally to the overall probability of disruption to the Health Sciences buildings and thus we do not recommend any seismic retrofit of the curtain wall system.

The window glazing spans between floor slabs covering the exterior of each building. Mullions are typically spaced at 6'-6" horizontally, and the glass at each story is divided vertically into three separate panels. The largest panel is located in the middle, with a vertical span of 9'-4". The smaller top panel spans 1'-6" vertically, while the bottom panel spans 1'-1". According to the original design drawings, the window glazing is 1/4" plate glass secured with a continuous neoprene perimeter gasket. The vertical mullions of the curtain wall system have a unique steel truss design, formed by a bent round bar as shown in Figure 99.

Our review of the window glazing is system is based on the following information:

- Reid, Rockwell, Banwell, and Tarics architectural curtain wall drawings, sheets AA33-AA38 dated May 25, 1962;
- Original *Health Sciences Instruction & Research, Unit 1* specifications Section 16: Window Wall, Section 17: Aluminum Windows, and Section 22: Glass and Glazing;
- *Stanlock Glazing Gasket Test Report* (Schoeneck, 1971);
- Site visits conducted on October 27, 2009 and July 7, 2010.

Figure 96: Exterior view of laboratory building window glazing.

Figure 97: View of window glazing a laboratory building corner

Figure 98: Typical interior view of laboratory building window glazing.

The original project specifications prescribe performance criteria for the completed curtain wall system. Among the requirements outlined in *Section 16: Window Wall* are the following:

- Window glass set in neoprene shall be cut to within 1/32 inch of the specified dimensions;
- The curtain wall system shall accommodate an ambient temperature range of 120°F without experiencing detrimental effects;
- The curtain wall system shall withstand wind loads of 20 lb/ft^2 normal to the wall in both the positive (inward) and negative (outward) directions;
- Mullion deflections shall not exceed L/175 under the specified wind loads;
- Neoprene gaskets shall meet tensile strength, elongation, hardness, brittleness, and weather resistance criteria.

Figure 99: Curtain wall open web mullion and neoprene gasket.

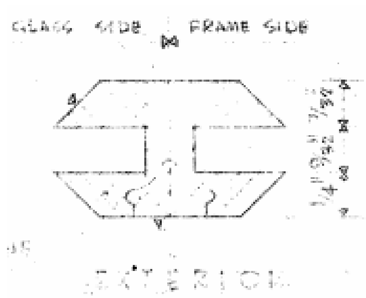


Figure 100: Original architectural drawing of neoprene gasket, sheet AA36.

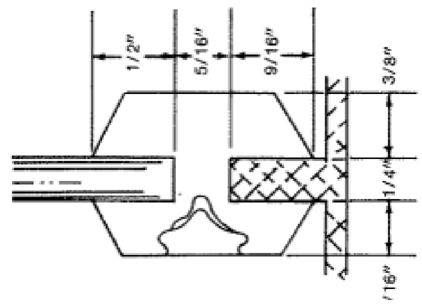


Figure 101: Dimensions of tested Stanlock neoprene gasket.

EVALUATION METHOD, CRITERIA, AND ASSUMPTIONS

In the course of our research, we contacted Stanlock, the manufacturer of the neoprene gaskets used in the construction of the curtain wall system. The manufacturer was able to provide a 1971 earthquake racking test report for the neoprene gasket system. Our analysis is on the findings of that study and experience with similar curtain wall systems.

The 1971 test report investigated the load-deformation behavior of windows subjected to racking forces. Since the system performance is dependent on the glass dimensions, tests were conducted on nominally sized windows as well as 1/16 inch oversized windows and 1/16 undersized windows. Each window frame was racked in-plane with a reversing cyclic static load up to a maximum load of 450 lb. The test data is presented in the form of asymmetrical backbone load - deformation plots. The test configuration and results are shown in Figure 102 and Figure 103.

To apply the general findings of the Stanlock study to the Health Sciences buildings, we compute the drift at which each of the tested window configurations comes into contact with the surrounding frame and compare the result to the maximum tested drift. Based on the results of our structural analysis, we know that the majority of drift in the laboratory buildings occurs over the column free-height, between panel zones. Using the drift from the tested configurations, we estimate a mean interstory drift capacity for the Health Sciences windows as described below. We use this capacity to develop fragility functions for the windows.

ANALYSIS RESULTS, DEFICIENCIES, AND FINDINGS

Based on the window geometry, we estimate that the glass in the Stanlock test report begins to bear on the surrounding frame at an interstory drift between 0.8% and 1.0%, depending on the glass dimensions. The load-deformation curve becomes nearly vertical, implying imminent failure at drifts between 1.1% and

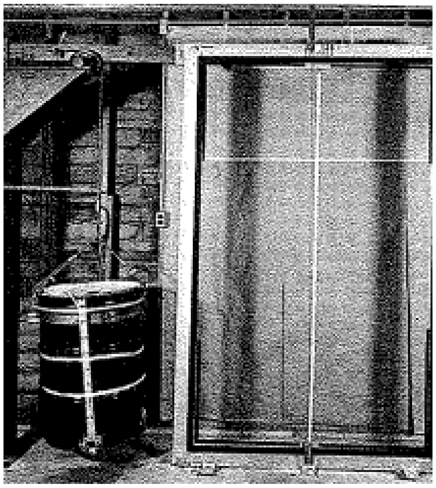


Figure 102: Window frame racking test-setup (from Stanlock 1971).

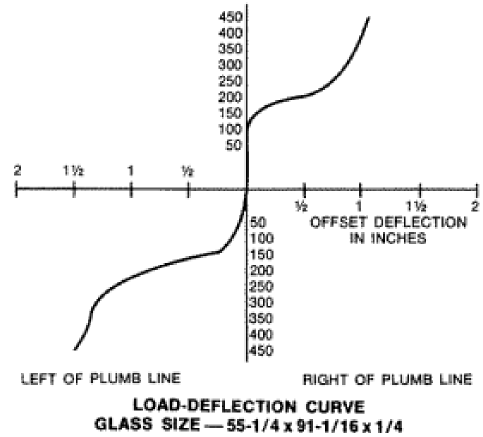


Figure 103: Window frame racking load - deformation backbone curve for nominally sized window (from Stanlock, 1971).

1.9%. When we apply these failure drift capacities to the Health Sciences window configuration, we estimate that the corresponding interstory drifts are between 0.7% and 1.4%, with a mean value of approximately 1.0%. The 1.0% interstory drift limit is roughly equal to the mean maximum interstory drift with a 975-year return period.

The California Building Code requires N-D (Life Safety for areas of public occupancy) performance for nonstructural components in the BSE-R (225-year return period) earthquake ground motion and does not require consideration of nonstructural performance in the BSE-C (975 year return period) ground motion. Since the mean window drift capacity corresponds to the 975-year return period drift, we find that the window system exceeds the minimum performance standards of the California Building Code.

PROBABILISTIC RESULTS AND CONCLUSIONS

Results of the risk analysis describe the likelihood that the curtain wall system experiences a given damage state over the next 30 years. We find that the existing windows are unlikely to affect the functionality of the laboratories. According to our calculations, there is less than a 7% chance that curtain wall damage exceeds the *Major Disruption* performance level. Given the low probability of experiencing significant damage, we find that the window system does not contribute significantly to the overall seismic risk of the Health Sciences buildings. The existing window glazing system need not be retrofitted to improve overall resilience.

Because damage to the curtain wall system has the potential to interrupt laboratory functionality and present a life-safety hazard, we incorporate window system fragility functions into our seismic risk analysis. Depending on the extent and severity of damage, the curtain wall system can affect the probability of exceeding the *Moderate Disruption*, *Major Disruption*, and *Life Risk* performance levels. Based on a review of window racking test data, we conclude that window system damage initiates with cracking and progresses with increasing drift until glass is dislodged and falls from the frame. We assign the *Moderate Disruption* performance level to our estimate of the median drift which would result in glass cracking. Both the *Major Disruption* and *Life Risk* performance levels are associated with the median drift at the onset of falling glass. The resulting curtain wall fragility functions are presented in Appendix K.

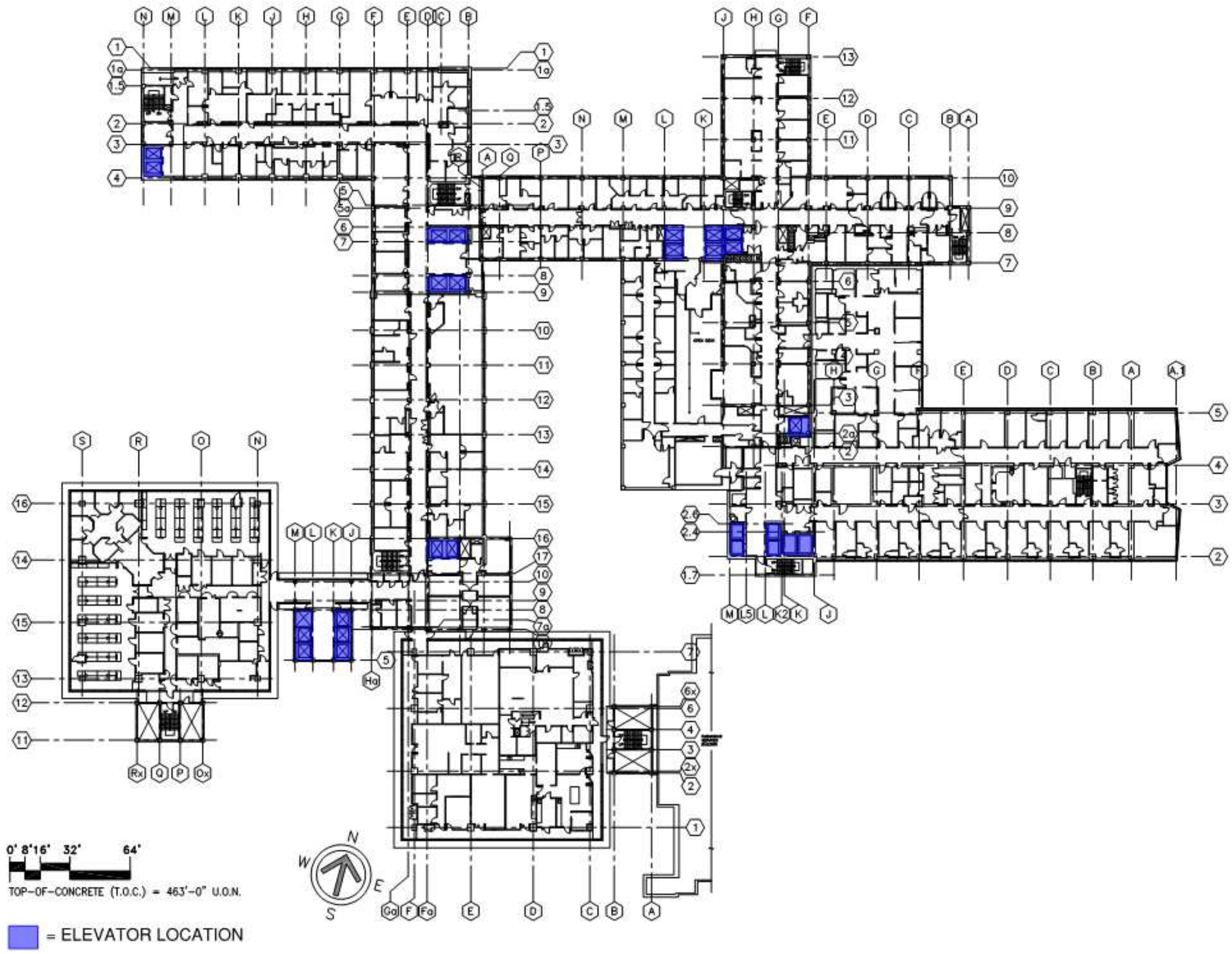


Figure 104: Typical floor plan showing the Health Sciences, Medical Sciences, Long Hospital, and Moffitt Hospital buildings. Common floor elevations connect the buildings. Elevator locations are shown highlighted.

9.2. ELEVATORS

Our evaluation of the Health Sciences elevators is based on a review of the Health Sciences, Medical Sciences, Long Hospital, and Moffitt Hospital existing building drawings maintained as part of the University's online drawing archive. We interviewed the University's elevator service provider at ThyssenKrupp, Jeff Roloff, to determine the condition of elevators at the Parnassus campus and the seismic safety devices currently in use. We researched the performance of similar elevators in past earthquakes. Additionally, we reviewed the evolution of elevator seismic safety standards under California law.

Our research identifies some vulnerabilities in the elevators serving the Health Sciences buildings. Nevertheless, the elevators provide seismic safety that is roughly equivalent to a new system. Additionally, the Health Sciences buildings are part of a complex of structures on the south side of Parnassus Avenue that share common, interconnected floors. This building configuration provides redundancy which can be leveraged to provide access to function elevators for the most critical activities following an earthquake.

HISTORICAL SEISMIC PERFORMANCE OF TRACTION ELEVATORS

Since the 1970's, post earthquake reconnaissance has shown that elevators in California have consistently provided life-safe seismic performance. There is no record of elevator fatalities during earthquakes. Over the same period, elevators have been shown to be vulnerable to interruptions in serviceability in moderate earthquakes (Filiatrault, Christopoulos, & Stearns, September 2001).

Elevator damage during the 1971 San Fernando earthquake was widespread, even among buildings without structural damage. Surveys identified at least 674 cases of derailed counterweights, along with damage related to toppling of unanchored motors and machines, and tangled ropes (Ding, et al., 1990).

During the 1989 Loma Prieta earthquake, the majority of the elevators in the affected region experienced only moderate shaking, with considerably less damage than was observed following San Fernando. Reconnaissance and surveys identified 98 cases of counterweight derailment. Typically, derailed counterweights were associated with the use of light, 8 lb rails or inadequate rail support brackets. The California Office of Statewide Health Planning and Development reported that there were 428 hospitals under its jurisdiction in the affected region. 282 of the facilities experienced intentional automatic elevator shut-down due to seismic switches triggered by the shaking, with 50 elevators requiring repair. The seismic switches include accelerometers triggered by either strong shaking or the P-waves that proceed strong shaking. When acceleration exceeds a predefined threshold, the device is intended to operate in a fail-safe mode, stopping at the next floor, opening the doors, and going out of service. It was observed that seismic switches provided unreliable performance, either not being triggered in damaged elevators or not operating in a fail-safe fashion (Ding, et al., 1990).

Following the 1994 Northridge earthquake, damage was observed in elevators in the greater Los Angeles area. Out of about 30,000 elevators in the affected region, survey results for 17,000 elevators identified 688 cases of counterweight derailment. Although the absolute number of affected facilities was greater than reported following the San Fernando earthquake, the larger number of installed elevators meant that the proportion of damaged facilities was reduced. (Reitherman, et al., 1995).

CALIFORNIA ELEVATOR REQUIREMENTS

Following the 1971 San Fernando earthquake, the State of California adopted the first elevator seismic safety code provisions. The Elevator Safety Orders require that machines are anchored to the primary structure, that heavier counterweight guiderails are used, and that elevators are provided with either a seismic switch or a derailment device. The seismic switch is triggered by an acceleration threshold. The derailment device activates when the counterweight displacement exceeds a preset limit. When the derailment device is activated, the car stops, moves away from the counterweight to the next floor, opens the doors, and goes out of service. In each case the elevator must be reset by a trained technician to re-enter service. At hospitals only, where a lack of elevator service can present a life safety hazard, elevators are allowed to operate at reduced speeds before they have been reset. The Elevator Safety Orders required retrofit of existing elevators to meet the minimum code provisions within seven years.

Recently, the California Building Code adopted the prescriptive ASME A17.1 Safety Code for Elevators and Escalators. The current 2010 edition of the standard requires that new and existing elevators be equipped with both seismic switches and derailment devices and provides guidance on rail design forces, and counterweight bracket spacing.

HEALTH SCIENCES ELEVATORS FINDINGS

The Health Sciences laboratory buildings are most immediately served by a central bank of five elevators located in the elevator connecting tower. The elevators serve the Third Floor to the Sixteen Floor of the laboratory buildings. Originally installed during the building construction in 1963, the elevators underwent a modernization program in the mid-1980's that included a series of seismic safety upgrades in compliance with the California Elevator Safety Orders. Retrofits included the installation of derailment sensors, spreader brackets, seismic anchorage of controllers, and installation of rope guards (Roloff, 2010). The elevator service provider expects that the retrofitted elevators in their current condition can be expected to provide seismic performance roughly equivalent to a new installation (Roloff, 2010).

The Health Sciences buildings are part of a complex including the Medical Sciences, Long Hospital, and Moffitt Hospital buildings on the south side of Parnassus Avenue. Each of the buildings shares common floor elevations, allowing continuous foot traffic throughout the complex. In total, 25 elevators serve the buildings, providing redundancy in case of a loss of some elevators following an earthquake. However, the top two stories of the Health Sciences laboratories are above the adjacent Medical Sciences building roof and are only served by the bank of elevators in the Health Sciences elevator tower.

Based on the performance of elevators in previous earthquakes, we expect that moderate shaking at the Parnassus campus could be expected to cause derailments or other interruptions in service for the some of the elevators serving the medical research complex south of Parnassus Avenue. Because elevator damage could affect facilities throughout the city, technician response times will be adversely affected. For an academic scenario study of an earthquake affecting the Los Angeles basin, the elevator risk consultant stated:

> *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 exacerbate the problem. This will even be a problem for critical facilities, such as hospitals, that are given high priority for service. The disruption of service associated with the loss of power and delays in restoring service that requires the inspection of an elevator service mechanic is the major problem* (Schiff, 2008).

We expect that this scenario is also applicable to restoration of elevator service in San Francisco following a moderate to significant earthquake. A detailed assessment of the seismic risk of the Health Sciences elevators is beyond the scope of this evaluation. We recommend that the University develop an elevator action plan for the medical research complex, which prioritizes critical activities to protect life safety and important research following an earthquake. The critical activities would be directed to have primary use of the functional elevators before complete service is restored. Such a plan would rank user priorities, presumably emphasizing hospital patient care and preservation of valuable research specimens. A single person on the campus could be charged with reporting the availability of elevators following an earthquake and assigning elevators to critical user-groups. A central list of critical research sample locations and requirements, including the need for cooling or other services could be maintained to facilitate protection of important research. Since the University has a number of elevators at the Parnassus campus, they could negotiate an earthquake response plan with ThyssenKrupp that prioritizes inspection and resumption of service at the campus.

Figure 105: Circulation diagram for a typical laboratory floor. The building utilities and infrastructure scope U0 protects the primary circulation path to the mechanical tower stairs.

9.3. Exiting and Circulation

As a part of the California Building Code compliance review and the architectural coordination process, we document the circulation and exiting pathways for each laboratory floor. A typical floor circulation diagram is shown in Figure 105 for the 11th Floor laboratories. A complete record of the circulation diagrams for all floors and a summary of the architectural code review are presented in Appendix J.

Based on our team's code analysis of the Health Sciences complex, we confirm the buildings as *H* occupancy due to the volume of hazardous laboratory chemicals present. The code-required exit pathway and exit widths are satisfied in the laboratory buildings. The maximum exit travel distance is less than the 400 ft required by code. For office and lab areas over 100 ft^2, the building code requires two exits, which are provided.

The figure shows three levels of circulation area, ordered by exiting priority. The stairs are the highest priority, followed by the primary and secondary circulation areas. In this hierarchy, the spaces used by the greatest number of people as exit pathways are given the highest priority, and are used to focus equipment anchorage and building utility retrofit efforts. Based on the results of this analysis, we define the retrofit of plumbing and mechanical components at the separation joint adjacent to the stairs as the highest retrofit priority, required for compliance with the requirements of the CBC as documented in Section 5. Primary circulation areas are prioritized to protect the occupants of given floor by strictly requiring the anchorage of laboratory equipment that can present a risk to human life. Building utilities in the primary circulation areas are typically identified for retrofit under the *U1: Targeted Loss Reduction* scope. Laboratory research spaces are defined as secondary circulation areas under this scheme. These areas receive contents anchorage and building utility retrofits at a normal priority level.

10. RETROFIT SCHEMES AND COST ESTIMATES

Based on logical combinations of retrofit scopes for the Health Sciences structures, utilities, and laboratory contents, we define four overall retrofit schemes, with estimated construction costs (as of January 2012), as shown below:

- Scheme 0: $548,000 (Chapter 34 Minimum Code Compliance)
- Scheme I: $6,460,000
- Scheme II: $10,056,000 (Recommended)
- Scheme III: $22,453,000 (Recommended)

The retrofit schemes most consistent with the University's seismic performance objectives for the Health Science laboratories are Scheme II or Scheme III. Both schemes include:

- Retrofitting of column splices;
- Structural dampers across separation joints at the mechanical towers;
- Improved retrofitting of utilities;
- Improved anchoring of laboratory contents.

Additionally, Retrofit Scheme III adds the following resilience retrofits:

- Structural damper frames throughout the laboratory buildings;
- Further enhanced retrofitting of utilities;
- Further enhanced anchoring of laboratory contents.

The other retrofit schemes listed above would be suitable to pursue, possibly as interim measures, if the University is not able to obtain full funding for Retrofit Scheme II or Scheme III. **The minimum retrofit scope allowed according to the California Building Code is Scheme 0**. As a minimum, this retrofit is required to provide seismic life safety for occupants exiting the building.

A summary of the scope included in each retrofit scheme is described subsequently in this section. The resulting seismic performance implications of each scheme for UCSF are described in Section 11. The retrofit schemes that are less expensive than Scheme II are associated with a reduced scope of retrofit work, and such schemes result in an increased likelihood, compared to Scheme II, that the research work being carried out in the buildings will be disrupted by an earthquake.

Figure 106 illustrates the estimated construction cost of each retrofit Scheme. (Note that in this figure added dampers are categorized as a structural retrofit, but in fact they improve the seismic performance of utilities and contents as well as that of the structure.)

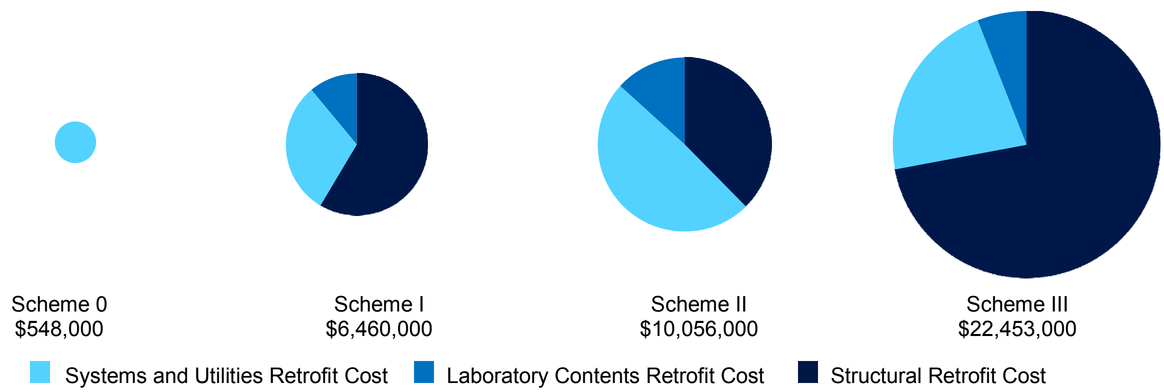


Figure 106: Relative contribution of structural, building utilities and laboratory contents costs to the total proposed retrofit scheme cost. The area of each chart and sector is proportional to the estimated construction cost for the retrofit scheme.

10.1. BASIS OF COSTS

The costs presented here are total construction costs for the complete Health Sciences complex—East and West laboratory buildings plus mechanical and elevator towers. The estimates include abatement of asbestos, but do not include soft costs such as occupant surge during construction, design costs, or the University's project management costs.

DPR Construction prepared cost estimates for the retrofit scopes proposed for the Health Sciences complex. The cost estimates address the retrofit scopes associated with the building structural system, building utilities, and laboratory contents. The cost estimates are based on the following information, provided by Rutherford & Chekene and the project sub-consultants:

- Structural drawings showing the proposed structural retrofit locations and scopes (10 sheets);
- Site visit with Rutherford & Chekene to review the structural retrofit scopes;
- Mechanical, plumbing, and fire protection drawings by Gayner Engineers (27 sheets);
- Mechanical, plumbing, and fire protection systems report section and retrofit scope matrix by Gayner Engineers;
- Electrical systems report section and retrofit scope matrix by Cammisa & Wipf;
- Information technology systems retrofit drawing by TEECOM (1 sheet);
- Information technology systems report section and retrofit matrix by TEECOM;
- Site visit with Rutherford & Chekene, Gayner Engineers, and Cammisa & Wipf to review the building utilities and systems retrofit scopes;
- HSE3 and HSE16 laboratory contents survey and map by Rutherford & Chekene;
- Laboratory contents report section by Rutherford & Chekene;
- Typical anchorage details for laboratory contents by Rutherford and Chekene (27 sheets).

The details of the cost estimate, with assumptions and exclusions, are provided in Appendix M.

10.2. COSTS OF INDIVIDUAL RETROFIT SCOPES

The costs for each retrofit scheme are assembled from the costs of individual retrofit scopes, which are shown in Table 33. (Details are provided in Appendix M.) The relative costs for individual retrofit scopes help guide the selection of effective combinations of retrofit work that we use to define the four retrofit schemes.

We defined the four retrofit schemes that we present here to cover the range of potential options that UCSF might choose to pursue for the Health Sciences buildings. It would also be possible, however, to define additional retrofit schemes based on different combinations of the individual scope items. Such a customization of retrofit scope could be readily done if in future project phases an adjustment of costs or seismic benefits was desired.

Each of the retrofit schemes is summarized in the sections below.

Table 33: Construction cost and disruption of individual retrofit scopes.

Structural Retrofit	Utilities and Systems Retrofit	Contents Retrofit
S0: CBC Chapter 34 Minimum (No retrofit required) **$0**	U0: CBC Chapter 34 Minimum Temporary disruption to utilities **$548,000**	C0: CBC Chapter 34 Minimum (No retrofit required) **$0**
S1: Column Splice Levels 5.5 & 7.5 Localized disruption in 16 locations **$1,677,000**	U1: Targeted Loss Reduction Temporary disruption to utilities **(Additional) $1,417,000**	C1: Targeted Loss Reduction Minor disruption **$713,000**
S2: Mechanical Tower Dampers Exterior work, minimal disruption **(Additional) $2,105,000**		C2: Life Safety + High Value Minor disruption **(Additional) $411,000**
S3: Laboratory Damper Frames Phased disruption ¼ floor per installation, must coordinate with architecture **(Additional) $12,397,000**	U3: Enhanced Resilience Temporary disruption to utilities **(Additional) $2,972,000**	C3: Enhanced Resilience Minor disruption **(Additional) $213,000**

10.3. Retrofit Scheme 0: California Building Code Minimum

This retrofit scheme defines the scope of work required to achieve full compliance with the requirements of Chapter 34 of the California Building Code. Our evaluation shows that the buildings in their current condition already meet the Chapter 34 requirements as related to the structures and the laboratory contents; only utilities and systems may need to be retrofitted to meet this requirement. Table 34 illustrates the scope of retrofitting included in Scheme 0.

The required retrofit of utilities and systems depends on the interpretation of ASCE 41-06 Hazards Reduced performance level. This performance level requires that there should not be a significant risk of life threatening injuring in areas of public assembly. Parts of the ASCE 41-06 standard imply (while other parts do not) that there should also not be such a risk in egress routes.

At this point, we have applied the more conservative interpretation, and we recommend that Scheme 0 include the retrofitting of potential hazards in egress routes, which for this project occur at the separation joints between the laboratory buildings and the stair/mechanical towers. The principal scope of such work is the retrofitting of piping systems that contain gas, hot water, and acid waste to be able to withstand differential movement across the separation joint.

As described in Section 3 of this report, Chapter 34 of the building code refers to performance levels in the ASCE 41-06 seismic retrofit standard. Our finding on the buildings' compliance with Chapter 34 for structures, utilities and systems, and laboratory contents is summarized below:

STRUCTURAL ASSESSMENT FOR SCHEME 0

Our structural analysis shows that the existing structural systems for the laboratory buildings, mechanical towers, and elevator towers meet the Life Safety Structural Performance Level (S-3 per ASCE 41-06) for the 225-year return period earthquake hazard level (BSE-R). The structural systems satisfy the Collapse Prevention Performance Level (S-5) for the 975 year return period earthquake hazard level (BSE-C). Based on these findings, structural retrofit is not required to meet the requirements of the California Building Code, and we recommend structural retrofit scope *S0*, which does not include any retrofit work or cost.

Table 34: Scope included in Retrofit Scheme 0 (Cost $0.6M).

Structural Retrofits	Utilities Retrofits	Contents Retrofits
S0: CBC Chapter 34 Minimum = no retrofitting required	U0: CBC Chapter 34 Minimum	C0: CBC Chapter 34 Minimum = no retrofitting required
S1: Column Splice Levels 5.5 & 7.5	U1: Targeted Loss Reduction	C1: Targeted Loss Reduction
S2: Mechanical Tower Dampers		C2: Life Safety + High Value
S3: Laboratory Damper Frames	U3: Enhanced Resilience	C3: Enhanced Resilience

UTILITIES AND SYSTEMS ASSESSMENT FOR SCHEME 0

ASCE 41-06 provides the requirements for non-structural seismic performance of existing buildings subject to Chapter 34 of the California Building Code. The standard requires that areas of public assembly, and possibly egress routes, shall meet the Life Safety Nonstructural Performance Level (N-C), which requires that hazardous materials and falling hazards should not result in a significant risk of life-threatening injury. Gayner Engineers has identified several piping systems that contain hazardous materials including gas, hot water, and acid waste which cross the separation joint between the laboratory building and mechanical towers in an exit pathway without adequate seismic bracing or flexible couplings to withstand differential movement across the joint. To satisfy the requirements of Chapter 34 and ASCE 41-06, retrofit scope *U0* includes retrofit work which will move these hazards from the exit pathway.

LABORATORY CONTENTS ASSESSMENT FOR SCHEME 0

During our review of typical laboratory floors, we have documented that the laboratory contents are already anchored in a manner that exceeds the minimum requirements of the ASCE 41-06 standard. This limited criterion requires that hazardous chemicals should be stored to preclude exposure to building occupants following an earthquake. We observed that acids, radioactive chemicals, and other hazardous chemicals were stored in designated areas. Based on these observations, the *C0* contents retrofit scope does not include any additional cost for contents anchorage.

10.4. RETROFIT SCHEME I

Retrofit Scheme I is designed to be a cost-effective solution to improving the seismic performance of the buildings beyond the minimum code requirements. The scheme does not provide the full level of post-earthquake resilience desired by UCSF, but seismic performance is significantly improved over the minimum code approach of Scheme 0. The retrofit scope included Scheme I is illustrated in Table 35.

STRUCTURAL RETROFIT SCOPE FOR SCHEME I

Scheme I includes the retrofitting of column splices in eight locations in each building (four columns each at Levels 5.5 and 7.5) as described in Section 4. The retrofitting reduces the likelihood that the buildings will be yellow tagged or red tagged after a large earthquake because of damage or residual deformation at these splices.

Scheme I also includes the addition of dampers across the separation joints between the laboratory building and the mechanical towers, as described in Section 4. This retrofit scope limits the movement at the separation joints and reduces the amount of retrofitting of utilities that cross or are within the joints. The retrofit also benefits the performance of the laboratory buildings, through reduced accelerations and deflections, and the mechanical towers, through reduce shear forces.

UTILITIES AND SYSTEMS SCOPE FOR SCHEME I

Retrofit Scheme I includes a prioritized scope for retrofitting utilities and systems that we call *U1: Targeted Loss Reduction.* Compared to the code minimum U0 scope, the U1 is designed to give additional protection to the systems which are most critical to the functionality of the Health Sciences laboratories. As described in Sections 5, 6, and 7, our subconsultants Gayner Engineers, Cammisa & Wipf, and TEECOM have identified a number of low cost retrofit measures that improve the resilience of the building utilities by addressing vulnerabilities in the most critical components. Details of the most critical vulnerabilities and the corresponding retrofit actions are presented in Sections 5 through 7 of this report.

LABORATORY CONTENTS RETROFIT SCOPE FOR SCHEME I

Retrofit Scheme I also includes a prioritized scope for retrofitting laboratory contents that we call *C1: Targeted Loss Reduction.* The C1 scope is designed to protect those laboratory contents that are most critical to the continued operation or quick resumption of research in the Health Sciences buildings. As described in Section 8, we have identified a scope of moderate cost retrofit measures that improve the resilience of the laboratories by addressing vulnerabilities in the most critical contents. Details of the most critical contents and the corresponding retrofit work are presented in Section 8 of this report.

The objective of the *C1: Targeted Loss Reduction* retrofit scope in this scheme to provide an economical way to increase the likelihood of the functionality of laboratory contents following an earthquake. The scope provides anchorage for the most dangerous and most valuable laboratory contents. The cost of this retrofit scope reflects that a substantial portion of the existing laboratory contents is already anchored to satisfy the requirements of this scope.

Table 35: Scope included in Retrofit Scheme I (Cost $6.5M).

Structural Retrofits	Utilities Retrofits	Contents Retrofits
S0: CBC Chapter 34 Minimum = no retrofitting required	U0: CBC Chapter 34 Minimum	C0: CBC Chapter 34 Minimum = no retrofitting required
S1: Column Splice Levels 5.5 & 7.5	U1: Targeted Loss Reduction	C1: Targeted Loss Reduction
S2: Mechanical Tower Dampers		C2: Life Safety + High Value
S3: Laboratory Damper Frames	U3: Enhanced Resilience	C3: Enhanced Resilience

10.5. RETROFIT SCHEME II

Retrofit Scheme II, like Scheme I, is designed to be a cost-effective solution to improving the seismic performance of the buildings beyond the minimum code requirements. It differs from Scheme I in that it includes more extensive retrofitting of utilities, systems, and laboratory contents. The scheme does not provide the full level of post-earthquake resilience desired by UCSF, but seismic performance is in improved of Scheme I and is significantly improved over the minimum code approach of Scheme 0. The retrofit scope included Scheme I is illustrated in Table 35.

STRUCTURAL RETROFIT SCOPE FOR SCHEME II

Scheme II has the same structural retrofit scope as Scheme I, including the retrofitting of column splices in eight locations in each building, and the addition of dampers across the separation joints between the laboratory building and the mechanical towers. These retrofit scopes are described in Section 4.

UTILITIES AND SYSTEMS SCOPE FOR SCHEME II

Retrofit Scheme II includes a more comprehensive scope for retrofitting utilities and systems that we call *U3: Enhanced Resilience.* Compared to the code minimum U0 and U1 scopes, the U3 scope gives more thorough protection to the utilities and systems of the Health Sciences laboratories. The U3 scope addresses the majority of vulnerabilities identified in the mechanical, plumbing, fire protection, electrical, and information technology systems. Details of the vulnerabilities addressed and the scope of retrofit actions are presented in Sections 5 through 7 of this report.

LABORATORY CONTENTS RETROFIT SCOPE FOR SCHEME II

Retrofit Scheme II also includes the most thorough scope for retrofitting laboratory contents, which we call *C3: Enhanced Resilience.* The *C3* contents retrofit scope provides anchorage for the majority of laboratory equipment and secondary protection for sensitive contents and samples. The scope also requires structural review of the adequacy of existing anchorage details and existing partition walls to support equipment anchorage. Details of this retrofit scope are presented in Section 8 of this report.

As with any retrofitting plan for building contents, the effectiveness of the effort will rely on having an ongoing program to enforce appropriate anchorage and storage of laboratory contents and to ensure appropriately resilient construction during future renovations to the building.

Table 36: Scope included in Retrofit Scheme II (Cost $10.0M).

Structural Retrofits	Utilities Retrofits	Contents Retrofits
S0: CBC Chapter 34 Minimum = no retrofitting required	U0: CBC Chapter 34 Minimum	C0: CBC Chapter 34 Minimum = no retrofitting required
S1: Column Splice Levels 5.5 & 7.5	U1: Targeted Loss Reduction	C1: Targeted Loss Reduction
S2: Mechanical Tower Dampers		C2: Life Safety + High Value
S3: Laboratory Damper Frames	U3: Enhanced Resilience	C3: Enhanced Resilience

10.6. Retrofit Scheme III

Retrofit Scheme III is a recommended retrofit scheme, which aims to provide the post-earthquake resilience desired by UCSF. The scheme is the same as Scheme II, except for the addition of a major scope of work to add damper frames throughout the laboratory building. The scope of work included in Scheme III is illustrated in Table 37.

A major benefit of Scheme III is that the added damper frames provide a substantial increase in protection for laboratory contents and the building utilities and systems. Even though this scheme also includes anchoring such contents and systems, there will always be damage to a percentage of contents and systems because anchoring schemes are not fool-proof. Also, small contents and items on laboratory benches are often (intentionally or not) left unanchored. The damper frames provide a reliable source of seismic protection because they directly reduce the shaking accelerations that are the cause of potential damage and disruption to the building's continued operation.

Table 37: Scope included in Retrofit Scheme III (Cost $22.4M).

Structural Retrofits	Utilities Retrofits	Contents Retrofits
S0: CBC Chapter 34 Minimum = no retrofitting required	U0: CBC Chapter 34 Minimum	C0: CBC Chapter 34 Minimum = no retrofitting required
S1: Column Splice Levels 5.5 & 7.5	U1: Targeted Loss Reduction	C1: Targeted Loss Reduction
S2: Mechanical Tower Dampers		C2: Life Safety + High Value
S3: Laboratory Damper Frames	U3: Enhanced Resilience	C3: Enhanced Resilience

11. SEISMIC PERFORMANCE RESULTS

To quantify the expected seismic performance of the Health Sciences buildings in their current conditions, we perform a nonlinear computer analyses that feed into a probabilistic seismic risk assessment. The methodology for this assessment is described in Section 3, while the details of the nonlinear analysis and the structural calculations behind it are described in Section 4.

Based on the results of the analysis, we identify vulnerabilities in the critical building systems that could affect the buildings and research function, and propose retrofit schemes to address the vulnerabilities. Then we repeat the nonlinear computer analyses for any of the four proposed retrofit schemes and, using probabilistic methods, we compute the risk of the building exceeding important performance levels due to earthquake shaking should any of these schemes be implemented.

Figure 107 presents a summary of seismic risk to the building over the next 30 years. The results of the probabilistic analysis can help guide the selection of the retrofit option that best meets the University's seismic performance and cost objectives for the buildings.

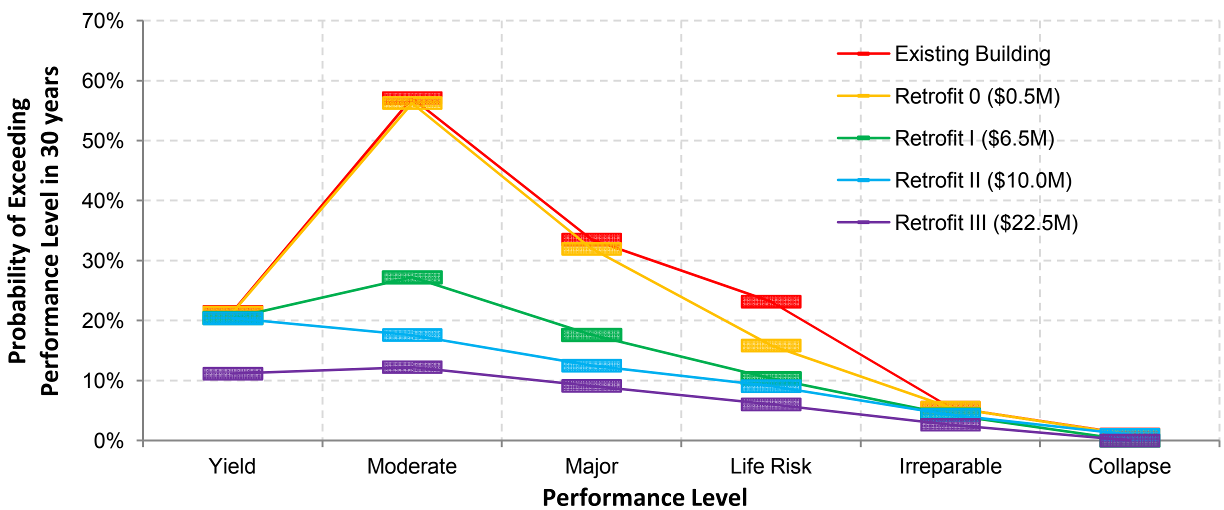


Figure 107: Probability of reaching or exceeding project-specific performance levels at least once over the next 30 years. The plot shows the cumulative probability considering structural, contents, mechanical, plumbing, fire protection, electrical, information technology, and window system damage. The existing building is shown as the base case, along with the results for each proposed retrofit scheme.

The figure shows that increasing levels of retrofitting provide progressively decreasing building risk for each of the performance levels. Without retrofitting, the existing building has almost a 60% chance over the next 30 years of experiencing at least once a *Moderate Disruption* or worse from earthquake damage, defined in Section 3 as an interruption to research activities lasting days to weeks. The building has greater than a 30% chance over the same period of experiencing at least once the *Major Disruption* performance state or worse. This damage state is defined as a significant interruption in research activities lasting longer than weeks in which irrecoverable work or essential samples are lost.

The results for Retrofit 0 (California Building Code Minimum) reveal that the proposed scheme is focused primarily on reducing the probability of experiencing a loss of life by protecting critical exit pathways. Retrofit III, the recommended scheme, offers the most complete set of seismic risk reduction measures, targeting all major building systems. This scheme reduces the likelihood of exceeding

Moderate Disruption by almost 80% and the likelihood of exceeding *Major Disruption* by more than 70%.

11.1. EXISTING BUILDING

The risk assessment of the existing Health Sciences East building establishes a baseline for the seismic performance of the complex. A plot showing the total seismic risk for all performance levels is included in Figure 108A. In the plot, the probability of exceeding a given performance level at least once over the next 30 years is represented by the vertical bars. Taller bars indicate a greater probability of exceedance, with the calculated probability shown above each bar.

The figure indicates that the existing building structure has only a 1% probability of *Collapse* and 6% probability of exceeding *Irreparable Damage* in the next 30 years. Both results indicate a building with acceptable seismic-structural performance. The probability of collapse is roughly consistent with that for a new building.

While the buildings are unlikely to collapse, they are vulnerable to disruption to their function. Over the 30-year period, the probability of experiencing at least once the *Moderate Disruption* performance state or worse is 57%. The probability of experiencing at least once the *Major Disruption* performance state (meaning possibly irrecoverable loss of research work or samples) or worse is 33%.

Life-safety risk is relatively high at a 23% probability over the next 30 years. This results from risk associated with laboratory contents and mechanical and plumbing systems, rather than structural risk.

Figure 108 also shows the contribution from each of the building systems to the total risk. The overall probabilities shown in part A of the figure result from a probabilistic combination of the worst case result for each of the building systems. Thus, for example, the 33% probability of major disruption is less than an additive combination of 25% probability of major disruption from mechanical systems and 27% probability of major disruption from laboratory contents, etc.

In this particular case, the figure shows that three systems (contents, mechanical, and electrical) each independently could create at least a 19% probability of major disruption. As a corollary, retrofit measures need to be applied to each of these systems if the probability is to be substantially reduced.

Figure 109A shows the deaggregated existing building risk for the *Onset of Moderate Disruption* performance level. Deaggregation is a process that displays the weighted contribution of individual components or systems to the overall seismic risk. The figure shows the contribution of the six building systems at each return period weighted by seismic hazard. Taller bars indicate a greater contribution to seismic risk. By locating the tallest bars in the deaggregation, we identify the system and earthquake return period that can be targeted by a retrofit to create the greatest reduction in overall risk. We use these results to guide our selection of retrofit schemes presented in Section 10. Similar results are shown in parts B, C, and D of the figure for the *Onset of Major Disruption*, *Life Risk, and Collapse* performance levels.

The deaggregation shows that *Moderate Disruption* is most likely to be caused by damage at smaller levels of earthquake shaking (72- and 142-year return period), with the main culprits being damage to laboratory contents and damage to mechanical systems. *Major Disruption* is most likely caused by shaking with a return period of 225 years and less. Even the *Onset of Life Safety* is governed more by the smaller shaking levels rather than the more rare events. *Collapse* on the other hand, is shown in Figure 109D to be governed by structural failure that occurs at a return period of 475 years and higher.

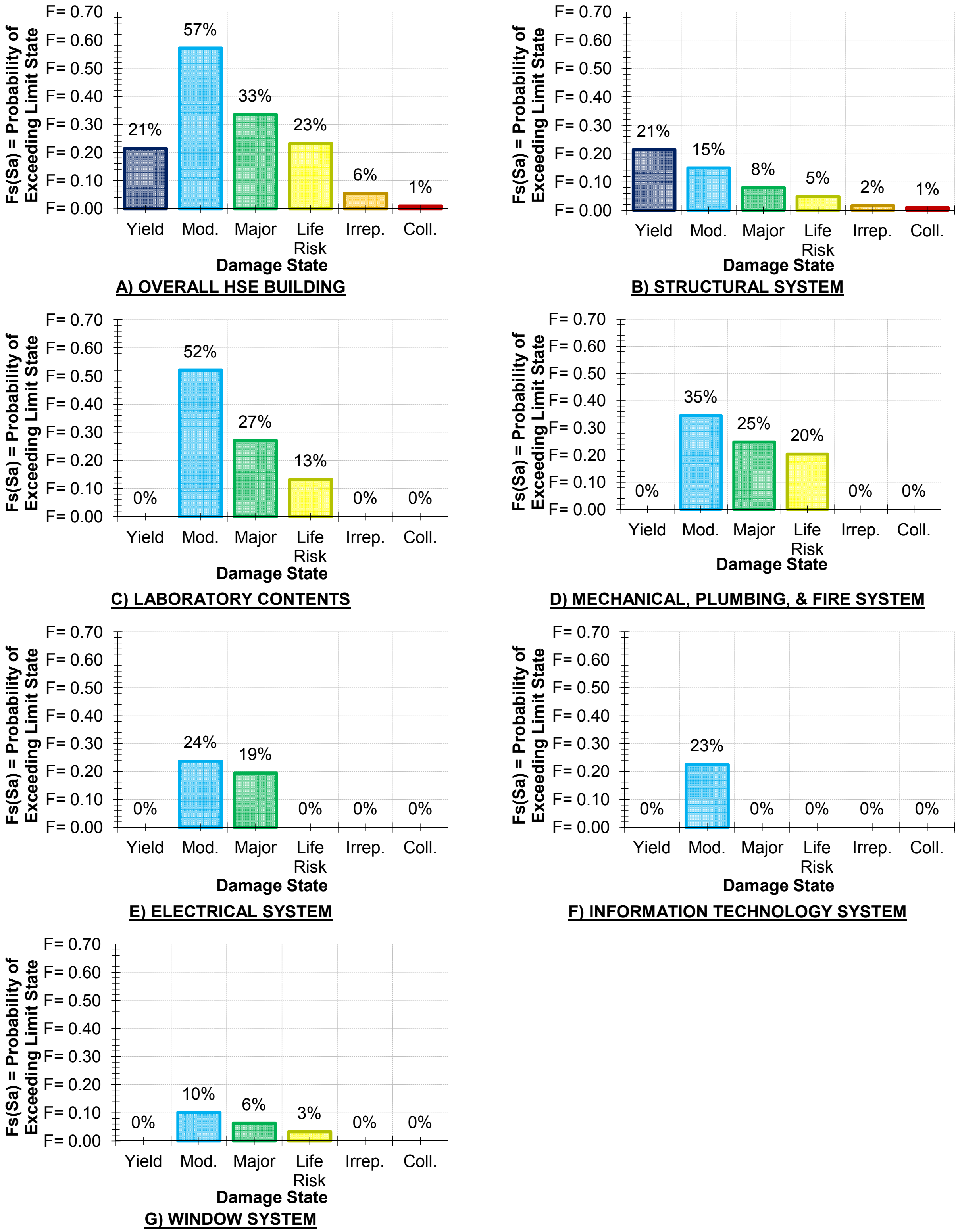


Figure 108A-G: Probability of exceeding project-specific performance levels at least once over the next 30 years for the overall HSE building and for the individual building systems.

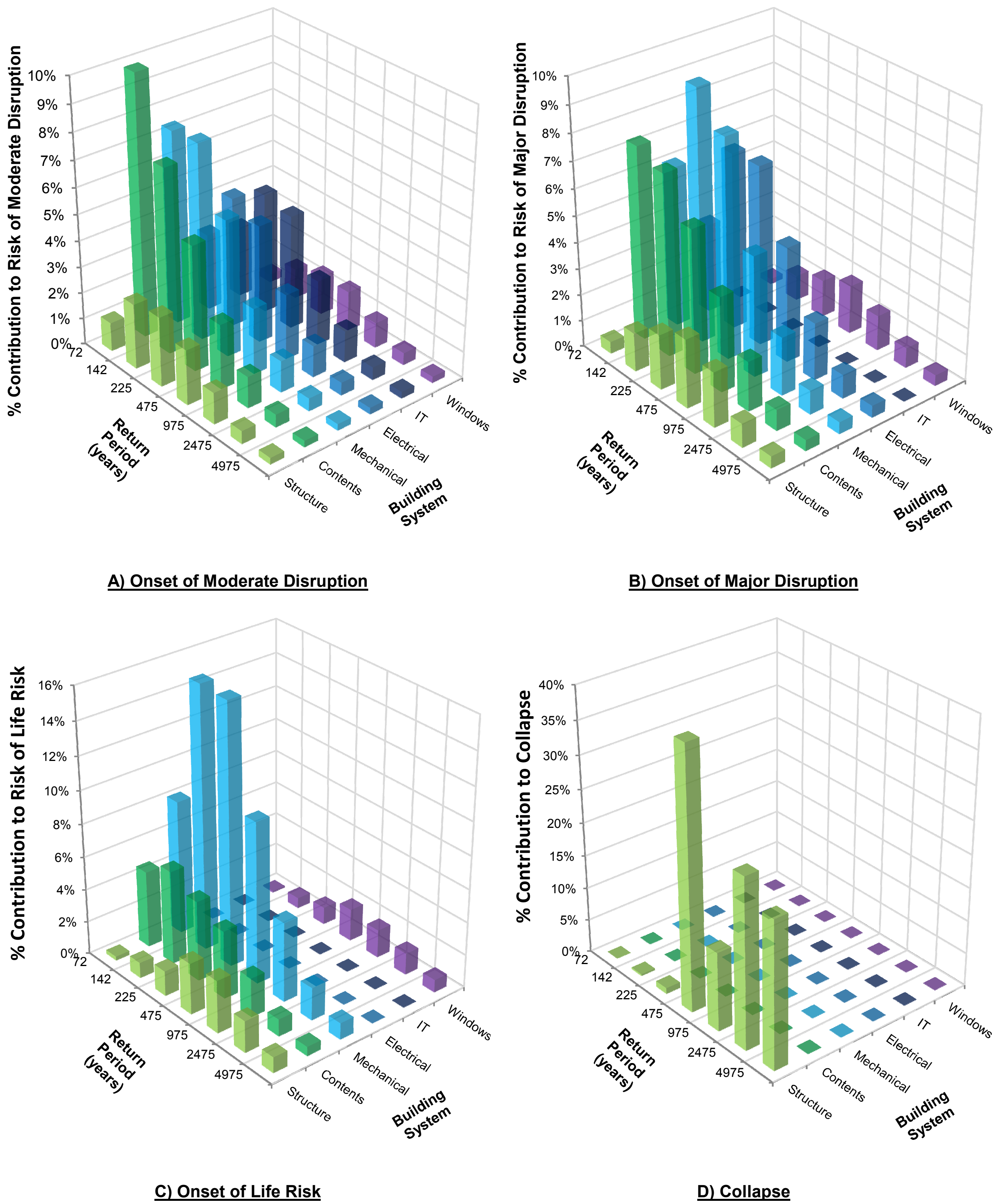


Figure 109A-D: Deaggregation of seismic risk for the several performance levels in the existing building, by building system and ground shaking mean return period. The height of each column indicates the relative contribution of each building system at each ground shaking mean return period to the total risk. Within each plot, taller columns imply that the system is more vulnerable.

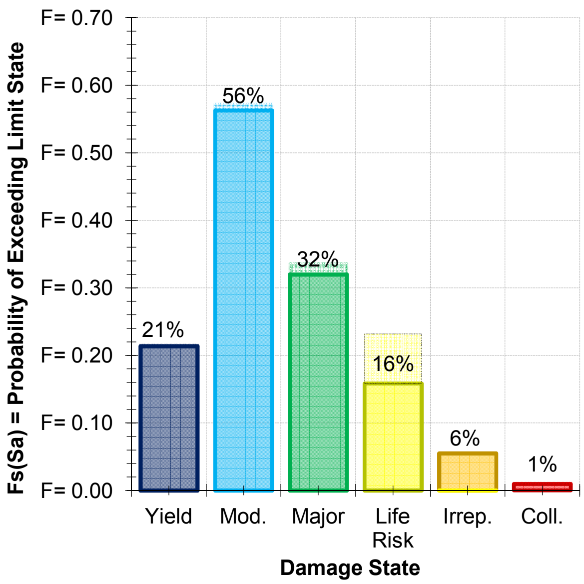


Figure 110: Probability of reaching or exceeding project-specific performance levels for Retrofit Scheme 0 at least once over the next 30 years. The results for the existing building are shown as a light shadow for comparison.

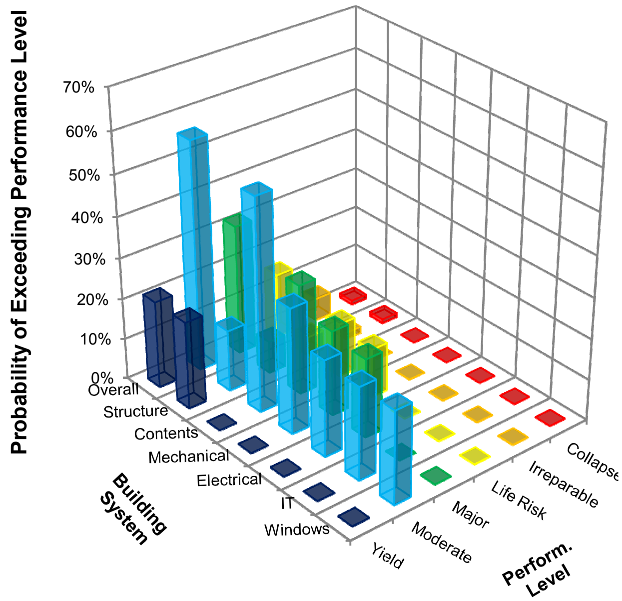


Figure 111: Deaggregation of seismic risk for reaching or exceeding the *Moderate Disruption* performance level for Retrofit Scheme 0.

11.2. RETROFIT SCHEME 0: CALIFORNIA BUILDING CODE MINIMUM

Retrofit Scheme 0 represents the minimum required scope of work to meet the requirements of the California Building Code Chapter 34. The retrofit scheme is described in Section 10.3. Based on our review of the building structure and laboratory contents, we have determined that both systems exceed the minimum code seismic requirements. Similarly, the electrical and information technology systems meet the minimum anchorage and bracing requirements of the CBC, as expressed in the ASCE 7-05 standard. The requirements of the standard are unclear regarding the need to protect exit passageways. Taking a conservative interpretation of the requirements, we interpret that retrofit measures to the mechanical, plumbing, and fire protection systems are required to protect the exit pathway to the mechanical tower stairs from overhead utilities, including gas and acid waste piping, which could be damaged by motion at the separation joint.

Figure 110 shows the seismic risk for all performance levels. Because the scope of Retrofit Scheme 0 is limited to critical mechanical components at exit pathways, the overall reduction in risk is small for most damage states. The greatest impact is on the probability of exceeding the Onset of Life Risk performance level in the next 30 years, which is reduced from a 23% probability to 16%.

The deaggregation in Figure 111 is also similar to the results for the existing building condition. Some impact of the mechanical systems retrofitting is evident by the reduction in risk for frequent ground shaking.

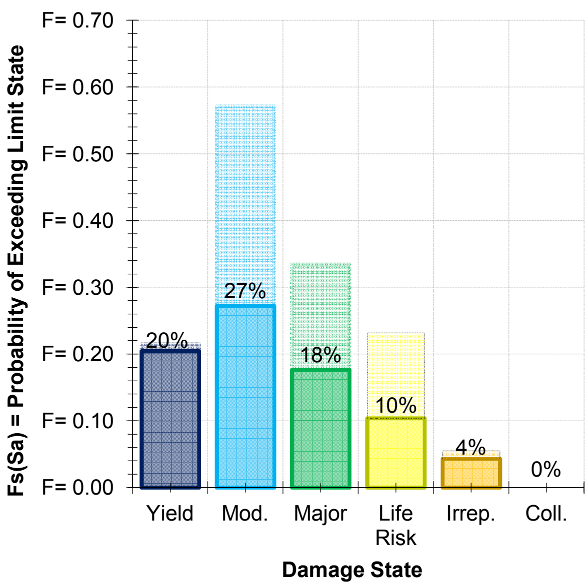


Figure 112: Probability of reaching or exceeding project-specific performance levels for Retrofit Scheme I at least once over the next 30 years. The results for the existing building are shown as a light shadow for comparison.

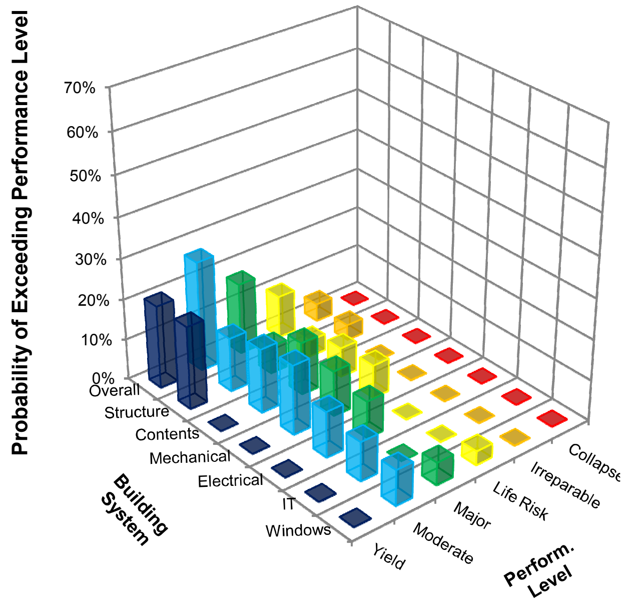


Figure 113: Deaggregation of seismic risk for reaching or exceeding the *Moderate Disruption* performance level for Retrofit Scheme I.

11.3. RETROFIT SCHEME I

The goal for Retrofit Scheme I is to provide improved seismic performance over a building meeting the requirements of Retrofit Scheme 0 and the California Building Code Chapter 34. The scope of retrofit is described in Section 10.4. Structural modifications include strengthened column splices and the addition of fluid viscous dampers between the mechanical towers and the laboratory buildings. Targeted loss reduction scopes are applied to the building utilities and to the anchorage of laboratory contents to improve nonstructural seismic performance.

The overall seismic risk results for Retrofit Scheme I are shown in Figure 112. In the Figure, the risk results for Retrofit Scheme 0 are shown as a lighter shadow for comparison. The proposed retrofit work results in significant reductions in the risk of reaching or exceeding both the *Onset of Moderate Disruption* (reduced by 29%) and the *Onset of Major Disruption* (reduced by 14%) performance levels. The probability of exceeding the *Onset of Life Risk* at least once in the next 30 years is reduced by 6%.

The deaggregation in Figure 113 illustrates that the retrofit measures target specific systems to reduce the overall seismic risk. The greatest risk reductions are for the laboratory contents and mechanical systems. The retrofit measures target the most vulnerable components, which have the potential to cause damage and disruption at low levels of ground shaking. Retrofitting these components and incorporating the beneficial effects of the mechanical tower dampers that reduce force and displacement demands result in improved overall performance. With implementation of this retrofit scheme, the height of the bars in the deaggregation is more uniform, indicating that risk is more evenly distributed and that the worst vulnerabilities have been addressed by the scheme.

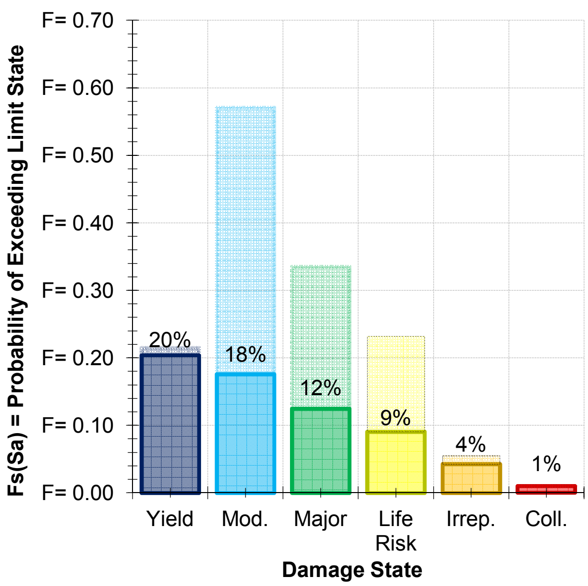


Figure 114: Probability of reaching or exceeding project-specific performance levels for Retrofit Scheme II at least once over the next 30 years. The results for the existing building are shown as a light shadow for comparison.

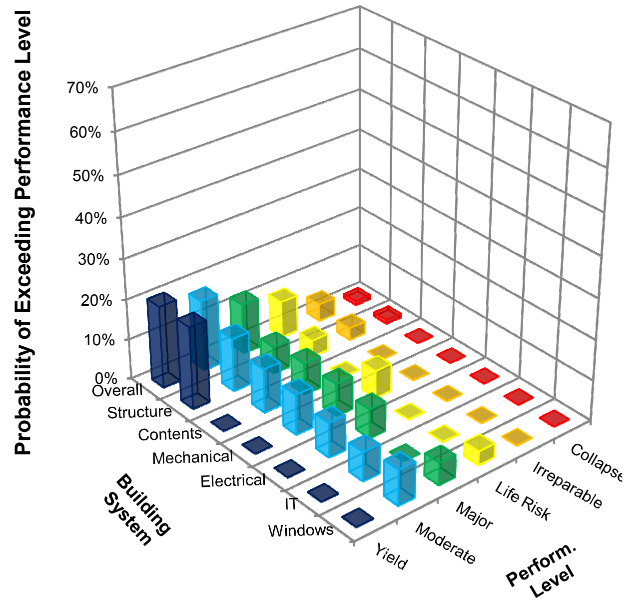


Figure 115: Deaggregation of seismic risk for reaching or exceeding the *Moderate Disruption* performance level for Retrofit Scheme II.

11.4. Retrofit Scheme II

Retrofit Scheme II includes the improvements of Retrofit Scheme I, with additional scope for the protection of building utilities and laboratory contents. A description of this scheme is presented in Section 10.5. Based on our understanding of the University's objectives, Scheme II is a recommended retrofit option.

Figure 114 shows that the seismic risk is further reduced by this retrofit scheme. The majority of the improvement occurs in the disruption performance levels, indicating that this retrofit contributes well to maintaining productivity following an earthquake. Although both Retrofit Schemes I and II include a structural retrofit with viscous dampers, little risk reduction occurs in the structural performance levels of *Yield, Onset of Irreparable Damage*, and *Collapse*. This is because, although the tower dampers can be considered a structural retrofit measure (because they become a new part of the structure) their real benefit is to protect mechanical systems and other utilities and the separation joints, and to generally reduce acceleration demands on nonstructural components.

In the deaggregation of Figure 115, we see that the relative contribution of structural damage to the overall seismic risk has increased. This increase reflects the improved performance of the various nonstructural systems, not a decrease in structural performance. Similarly, the relative risk contribution of the windows has increased as the vulnerability of the other systems is reduced.

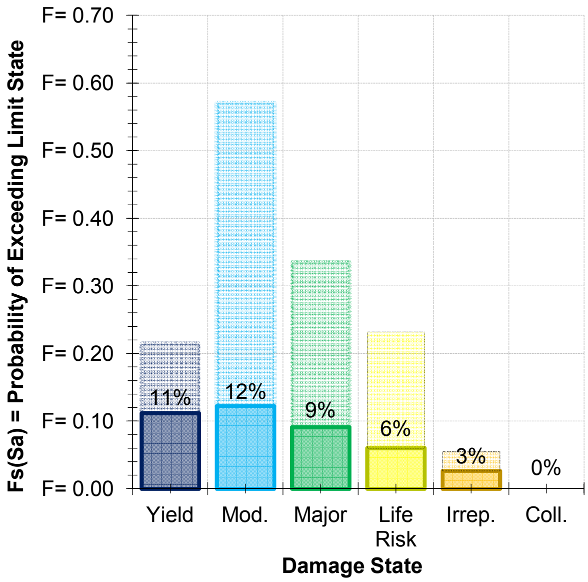


Figure 116: Probability of reaching or exceeding project-specific performance levels for Retrofit Scheme III at least once over the next 30 years. The results for the existing building are shown as a light shadow for comparison.

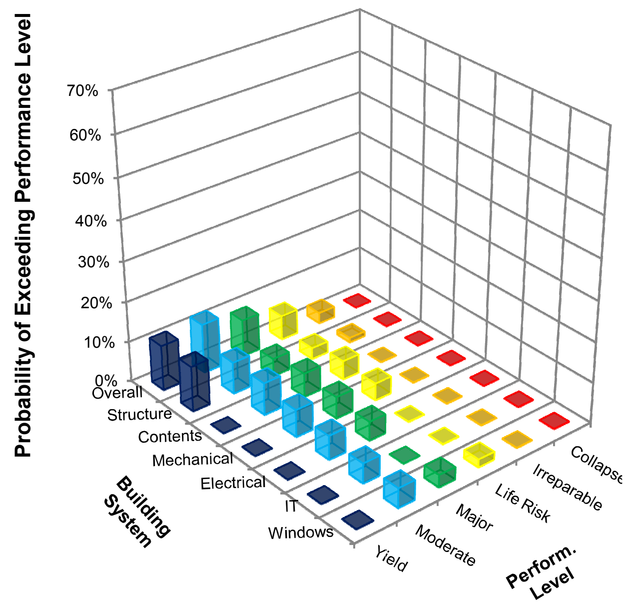


Figure 117: Deaggregation of seismic risk for reaching or exceeding the *Moderate Disruption* performance level for Retrofit Scheme III.

11.5. Retrofit Scheme III

Based on our understanding of the University's objectives, a recommended retrofit scheme is Scheme III. This option incorporates the most thorough retrofit measures for each building system to offer the best overall seismic performance. Retrofit Scheme III includes the improvements described for Scheme II, and adds further scope to add fluid viscous damper frames to each story and each elevation of the laboratory buildings. The damper frames serve to reduce building drifts and accelerations, which offers protection for both structural and nonstructural systems. The scope of the retrofit scheme is described in Section 10.6.

The improvements in seismic risk resulting from the application of the Scheme III retrofit scope are shown in Figure 116. In this scheme, we observe that the probability of structural components experiencing nonlinear behavior (the *Yield* performance level) in the next 30 years is reduced from 20% to 11%. Additional, improvements in overall seismic risk occur in the *Onset of Moderate Disruption*, *Onset of Major Disruption*, *Onset of Life Risk*, and *Irreparable Damage* performance levels.

The *Moderate Disruption* seismic risk deaggregation in Figure 117 shows that the largest contribution to seismic risk is due to remaining minor vulnerabilities in the structural system and laboratory contents anchorage. In Retrofit Scheme III, the majority of the lab contents are anchored, but the remaining risk is due to the fact that even with anchored components, some anchorage may fail or some contents may be damaged.

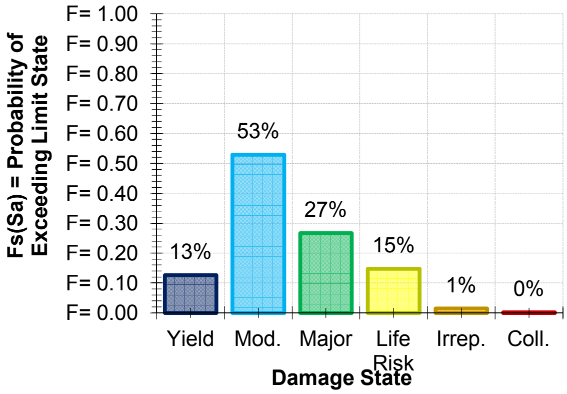


Figure 118: Probability of reaching or exceeding project-specific performance levels the existing building during the Hayward Fault ground motion scenario.

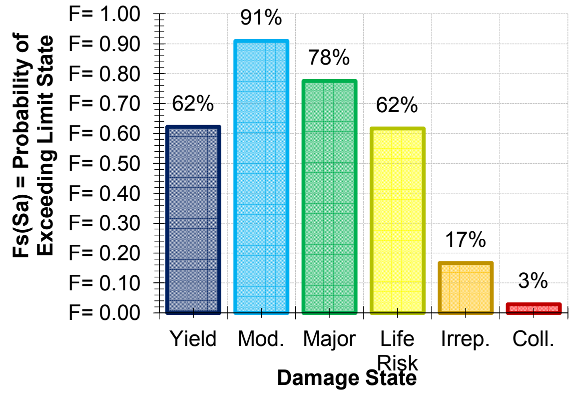


Figure 119: Probability of reaching or exceeding project-specific performance levels the existing building during the San Andreas Fault ground motion scenario.

11.6. Scenario Evaluation Results

Based on feedback from the peer review committee and UCSF, we also present seismic performance results for two earthquake ground motion scenarios. The reviewers requested that we describe the expected levels of damage and disruption to the Health Sciences complex due to two possible scenario events: an earthquake on the Hayward Fault and an earthquake on the San Andreas Fault. Figures 118 and 119 illustrate the probability of the existing building exceeding the project-specific performance levels for the two scenarios. The Hayward Fault scenario reflects ground motions resulting from a magnitude M_w = 6.6 to 7.3 earthquake on the Hayward-Rodgers Creek Fault system. The San Andreas Fault scenario represents a magnitude M_w = 7.4 to 8.1 earthquake on the Northern San Andreas Fault system.

The probability of experiencing an interruption in laboratory use due to the *Moderate Disruption* or *Major Disruption* performance levels is much greater in greater in the San Andreas Fault scenario. The higher likelihood of disruption is due to the greater earthquake magnitudes and closer proximity of the San Andreas Fault. In the Hayward scenario, the building has a 53% chance of experiencing at least *Moderate Disruption.* In the San Andreas scenario, the likelihood of experiencing at least *Moderate Disruption* increases to 91%. The results show that the existing building is vulnerable to the ground shaking associated with these scenarios.

We present scenario results for each of the proposed retrofit schemes in Figure 120A-H. The plots show increasing performance with increasing retrofit thoroughness. The risk of reaching or exceeding *Moderate Disruption* drops from 53% for the existing building to 5% for Retrofit Scheme III under the Hayward Scenario. For the San Andreas scenario, the probability decreases from 91% to 36%.

It is important to note that while the scenario results provide probabilistically robust descriptions of the expected building performance under two possible earthquake ground motion scenarios, they are less complete than the full probabilistic results presented in Sections 11-11.5. The scenario results describe two possible earthquake ground motions. The full probabilistic results describe the total expectation of disruption for all earthquake ground motions that could affect the Health Sciences complex, weighted by the likelihood that a given ground motion will occur. In this way, the probabilistic results combine both the expectation of building behavior and the likelihood of experiencing damaging earthquake ground motions.

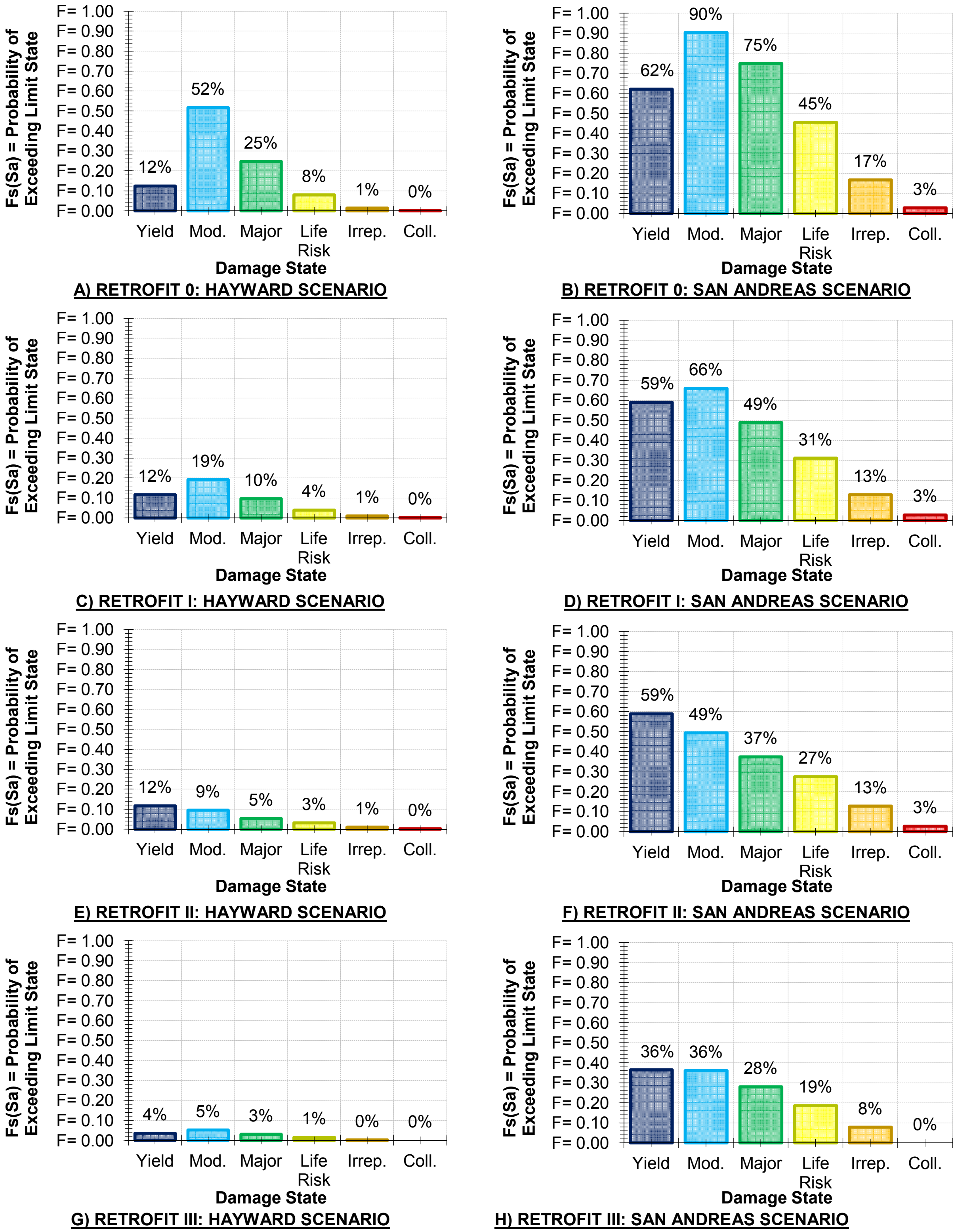


Figure 120A-H: Probability of exceeding project-specific performance levels for scenario Hayward and San Andreas ground motions

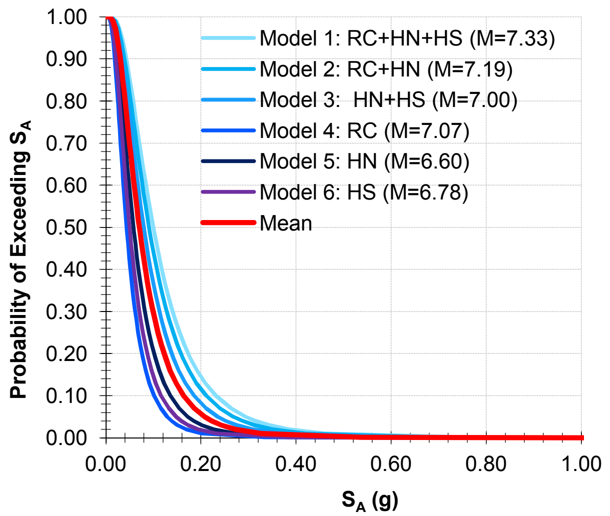


Figure 121: Probability of reaching or exceeding a spectral acceleration at the building first mode period for the Hayward Fault scenario ground motion.

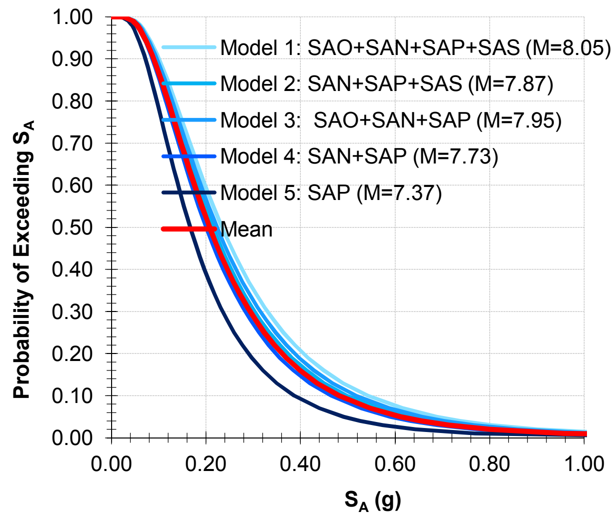


Figure 122: Probability of reaching or exceeding a spectral acceleration at the building first mode period for the San Andreas Fault scenario ground motion.

Earthquake Ground Motion Scenarios

The peer review committee requested that we define building performance for earthquake ground motions resulting from the maximum earthquakes that can be generated on the Hayward and San Andreas Faults. The USGS defines each fault as a series of smaller segments. Forecasts by the California Geological Society and others (2007 Working Group on California Earthquake Probabilities, 2008) describe the maximum earthquake magnitude likely to be generated for each individual segment or for a series of segments rupturing together. We use the predicted maximum magnitudes and segment rupture combinations to define the earthquake ground motion hazard for the scenario events.

For the Hayward Fault scenario, we use the USGS description of three fault segments: Hayward North, Hayward South, and Rodgers Creek (see Figure 120). Each segment has an associated maximum earthquake magnitude. Additionally, the USGS defines that a rupture could connect the Hayward South and Hayward North segments, the Hayward North and Rodgers Creek segments, or all three segments, resulting in six total rupture combinations. We use the four Next Generation Attenuation (NGA) ground motion prediction equations, weighted equally, to establish the median spectral acceleration at the Health Sciences site resulting from each possible rupture (and considering earthquake magnitude, source-to-site distance, and local soil characteristics). Since the NGA models assume that earthquake ground shaking is lognormally distributed around the median value, we use the model standard deviation to define a lognormal distribution of spectral accelerations for each rupture. We find the mean scenario hazard curve by equally weighting the hazard curves resulting from each possible rupture. The individual rupture hazard curves and the mean Hayward Fault scenario hazard curve used in our scenario analysis is shown in Figure 118.

We perform a similar analysis to define the San Andreas Fault scenario hazard curve. In this case, the forecasts describe four segments: the San Andreas Santa Cruz Mountains segment, the San Andreas Peninsula segment, the San Andreas North Coast segment, and the San Andreas Offshore segment. The earthquake magnitude and rupture combinations considered are shown in Figure 119.

Figure 123: Fault segments considered in the Hayward and San Andreas ground motion scenarios. Key for Hayward-Rodgers Creek scenario: HS = Hayward South, HN = Hayward North, RC = Rodgers Creek. Key for San Andreas scenario: SAS = Santa Cruz Mountains segment, SAP = Peninsula segment, SAN = North Coast segment, SAO = Offshore segment (not shown). © 2012 Google. © 2012 TerraMetrics

CALCULATION OF SCENARIO RISK

We compute the risk of damage and disruption for the earthquake ground motion scenarios using a technique similar to the calculation of the full probabilistic results. We use the results of the Monte Carlo simulation to generate cumulative probability distributions which describe the probability of damage for each building system as a function of ground motion intensity. We numerically convolve the cumulative damage distribution with the scenario earthquake hazard curves to calculate the probability of reaching or exceeding the project-specific performance levels.

Additional details of the Monte Carlo simulation process are presented in Appendix L. Descriptions of the fragility functions used in the analysis are presented in Appendix K.

11.7. UC SYSTEM SEISMIC RATING

The University of California Seismic Rating system has been recently revised (Comartin, Pers. Comm.) to relate more specifically to earthquake evaluation levels of Chapter 34 of the California Building Code. R&C has studied the revised rating system in detail and has attempted to correlate the definitions to a probabilistic basis, while remaining consistent with the previous definitions and interpretations of the ratings. This work is ongoing.

The UC rating system is designed to only consider life safety and protection against collapse. It does not address continued operation, recoverability, or resiliency. For the health sciences buildings, we estimate the UC System rating based on:

1. The probability of collapse resulting from our analysis, in comparison to our estimates of this probability for new "code" structures and for other UC structures that we have evaluated.

2. A comparison by judgment of the expected performance of the Health Sciences buildings.

In our opinion, if the code mandated Retrofit Scheme 0 is carried out, the Health Sciences buildings will have seismic performance consistent with a Level II (Fair) rating according to the UC System. If the rating were based on the structural performance alone, a Level III (Good) rating would be justified—it is the life safety risk of contents and system that cause us to judge the rating as Fair.

To achieve a Level III (Good) rating overall, we believe that Retrofit Scheme II is required. Retrofit Scheme III not only would achieve a Level III (Good) rating but also would provide substantial additional resilience to the Health Sciences buildings. It might be possible to show that Retrofit Scheme I provides a Level III (Good) rating, but this would require further study.

12. BUILDING INSTRUMENTATION

Table 38: Health Sciences building instrumentation conceptual schemes and approximate costs.

Instrument Scheme	System 1 Cost: Digitexx Data Systems	System 2 Cost: Canterbury Seismic Instruments
Basic Scheme: 9 accelerometer channels (3 x 3-component accelerometers)	$28,000	$14,000
Intermediate Scheme: 15 or 16 accelerometer channels	$45,000 (15 x 1-component accelerometers)	$41,000 (5 x 3-component accelerometers)
Full Scheme: 32 or 33 accelerometer channels	$58,000 (32 x 1-component accelerometers)	$67,600 (11 x 3-component accelerometers)
Full Scheme with Displacement Monitoring: 87 (29 x 3-component) accelerometer channels, 9 linear voltage displacement transducer	---	$170,000

Based the University's interest in the possibility of adding seismic instruments to the Health Sciences buildings, we obtained informal proposals from two leading instrumentation providers. Potential instrumentation plans for the buildings are presented in Table 38. The configurations provide monitoring options ranging from recording basic structural responses to providing real-time data, including:

- Monitoring of reference input accelerations;
- Detailed monitoring of horizontal and vertical accelerations at up to 28 total locations in the two laboratory buildings and mechanical towers;
- Information on building drifts at up to 10 locations;
- Information on relative motion across the mechanical tower separation joints at up to 9 locations.

12.1. INSTRUMENTATION BACKGROUND

It is possible to outfit buildings with a range of sensor systems to monitor, record, and transmit data relevant to structural performance. The systems we describe have the capability to process information on the engineering demand parameters identified as having the greatest effect on the resilience of Health Sciences buildings: floor acceleration, building drift, and relative motion across the building separation joints. Floor accelerations affect a range of building components including mechanical, electrical, and information technology systems as well as valuable laboratory contents. A detailed discussion of acceleration sensitive systems is provided in Appendix K. Floor acceleration is typically measured using strong motion accelerometers, which can record acceleration data on up to three axes simultaneously. Accelerometers can be located in a range of locations]. A sensor located outside the building can provide a reference station that records accelerations acting at the site interface to the building foundation. Accelerometers at the basement level can be used to calculate base slab averaging effects which may filter high frequency vibrations which are transmitted to the building superstructure. Accelerometers can also be distributed over several levels of the building, to record the variation over height of acceleration.

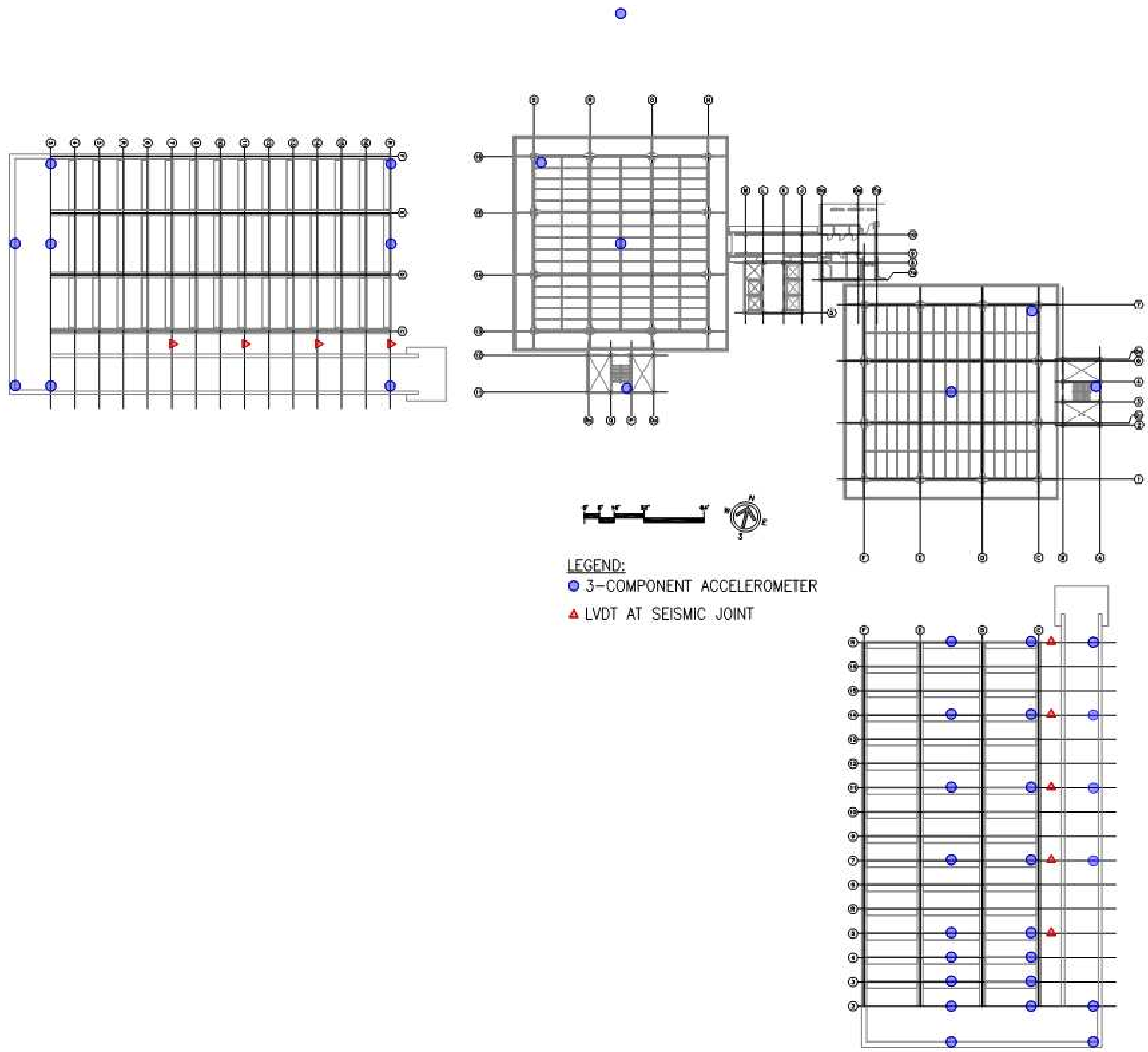


Figure 124: Plan and elevation diagram for the full instrumentation scheme with displacement monitoring. Floor plans indicate location of accelerometers on typical floors. Elevations show distribution of accelerometers over the building height and locations for LVDTs at separation joints.

Our computer analytical model outputs building drift, which is correlated to damage in the seismic lateral force-resisting system. Additionally we have identified a range of nonstructural components affected by the relative motion of adjacent floors including fume exhaust and plumbing risers. Analytically, it is possible to compute building displacement by integrating the processed acceleration time history twice. Specific mathematical techniques and preferred instrument configurations for recovering displacement data vary between instrument providers. Some systems use two three-component accelerometers per floor when displacement data is desired. Computing the displacement of individual sensors and synchronizing the displacement results over a range of sensors, it is possible to compute the relative displacement between two locations. Such calculations make it possible to estimate building drifts, torsional floor rotations, and foundation rocking effects.

Our analysis has identified that critical utilities crossing the building separation joints make the greatest contribution to the risk of disruption to laboratory research activities. The full scheme with displacement monitoring includes sensors to directly measure the motion between laboratory buildings and mechanical towers. The sensors, linear voltage displacement transducers (LVDTs) are wheatstone bridge circuit instruments that measure changes in electrical resistance as the instrument lengthens or shortens.

12.2. DESCRIPTION AND OBJECTIVES OF INSTRUMENTATION SCHEMES

Strong motion instrumentation installations can serve a range of goals. We define a series of schemes that involve various levels of initial investment which address a range of possible goals.

Basic Instrumentation Scheme

We present two options for basic acceleration instrumentation schemes in Table 38. Both schemes consist of nine total acceleration channels, with sensors located throughout the building. The information provided by the basic instrumentation scheme is roughly equivalent to CESMD instrumentation scheme. Output data from the scheme could be used to calibrate analytical models or for preliminary assessments following more severe ground shaking.

Intermediate Instrumentation Scheme

The intermediate instrumentation scheme options include the sensors described in the basic instrumentation scheme and additional accelerometers located throughout the complex. These sensors can be used to calculate foundation rocking, basement slab input accelerations, floor accelerations at several levels, and building displacements at several levels. The additional sensors provide additional resolution for calibrating building analytical models or predicting structural damage following an earthquake.

Full Scheme with Displacement Monitoring

This configuration, which could be called a “structural health monitoring” system, provides comprehensive real-time data on the seismic performance of the building structure, which can be used to facilitate expedited review and re-occupancy of the building following an earthquake. The system data output could be included as part of a building occupancy resumption program (BORP), which could qualify the building for immediate re-occupancy if preset performance thresholds are not exceeded. In larger, damaging events, the output could be used to help estimate the extent and locations of damage to the building.

The configuration includes monitoring of both the Health Sciences East and West buildings and mechanical towers. The HSE building, modeled as a part of this study, is more extensively instrumented, with sensor locations tailored to the specific vulnerabilities we have identified. As a result, the sensor locations will vary slightly based on any retrofits implemented following this study. The instrumentation configuration for the Scheme III retrofit is shown in Figure 124.

The configuration includes a single three-component accelerometer located outside the building, possibly in Saunders Court. Since it is impossible to find a free field site near the Health Sciences buildings, this exterior sensor will provide reference acceleration data which can be compared to the basement accelerations recorded in either the HSE or HSW buildings. At the HSE building, sensors will include accelerometers in the basement and at intermediate levels over the height of the structure. We include two three-component accelerometers in the basement to accurately record accelerations acting on the slab, to observe base slab averaging effects, and to estimate foundation rocking. Placing two three-component accelerometers at selected levels of the superstructure will allow monitoring of building accelerations, torsional displacements, and lateral displacements. Calculating the relative displacements between adjacent instrumented levels allows the calculation of building drifts. The instrumentation configuration locates additional sensors in the bottom levels of the building, where drifts are expected to be largest. Additional accelerometers are located in the HSE mechanical tower. Since our analysis identified that the most significant vulnerability of the Health Sciences buildings is associated with critical utilities crossing separation joints without flexible couplings to account for building movement, the instrumentation plan includes linear voltage displacement transducers (LVDTs) to measure the relative motion of the mechanical tower and laboratory building. Similar instrumentation is included in the HSW building.

The data from the instrumentation system can be used for a variety of purposes. The system provides real-time access to the output via an internet connection. Data from moderate ground shaking can be used to calibrate the building model, ensuring that the assumptions used in the creation of the analytical model match the behavior of the actual structures. Following more severe ground shaking, the output can be used to estimate whether damage has occurred in the structure. If the dynamic properties of the structure are changed, the output can be used to predict locations of damage within the building to limit the damaging removal of gypsum wallboard and to reduce the time required for structural investigation.

As part of an integrated BORP, the sensor output can be used to facilitate re-occupancy of the structure. An engineer can review the building performance using predetermined fragility functions, like those used in this study, to estimate the likelihood of damage to the structure. Below threshold values, the building can be deemed structurally safe for re-occupancy without further investigation. Similar systems were used following the recent Canterbury, New Zealand earthquakes. Users particularly commented that the systems were useful to reassure building tenants and occupants that structures were seismically safe.

12.3. CESMD Instrumentation Program

A minimum cost option for instrumenting the Health Sciences building complex would involve applying to have the buildings added to the Center for Engineering Strong Motion Data (CESMD) network. CESMD acts a clearinghouse, organized by the U.S. Geological Survey (USGS) and the California Geological Survey (CGS), providing uniformly processed data for several strong motion networks.

CESMD themselves fund some instrumentation installations. However, because the School of Nursing and the Medical Center at Parnassus have already been instrumented by the network, it is uncertain whether CESMD would fund the installation of additional instruments at the Health Sciences buildings. If University funding were available to pay for the instruments, it is likely that the building could be added to the network. It is important to note that all seismic data recovered as a part of the CESMD program is publically available.

Adding instruments via the CESMD system would serve to provide basic analytical modeling information for the University and would provide a public good by adding to database of information available to scientists and engineers. Typical CESMD installations include instruments at two or three locations over the height of a structure, commonly with two accelerometers at the selected floors. According to the network, using their data, it is possible to recover floor accelerations, floor displacements, and foundation rocking effects. Since the instruments are not densely installed, output from the system could be used to verify basic modeling assumptions under moderate ground shaking. Information from more severe shaking could be used as inputs to analytical modeling to predict the severity of damage to the structural system. According to CESMD, the system could be designed to provide access to the building owner or engineer immediately following an earthquake, without waiting for the public release of data. However, the information is not available in real-time.

12.4. R&C RECOMMENDATION FOR INSTRUMENTATION

Seismic instrumentation in the buildings can help speed the University's recovery and re-occupancy of the laboratories following a damaging earthquake. If sufficient funding can be secured for instrumentation, we recommend an instrumentation scheme costing approximately $100,000. This equates to a system that includes displacement monitoring, but is somewhat less extensive than the full scheme outlined above.

13. CONCLUSIONS AND POTENTIAL FUTURE REFINEMENTS

This report defines four possible Retrofit Schemes, from minimal code compliance to more complete measures, to improve the seismic performance of the Health Sciences buildings. Based on our understanding of the University's objectives and the value and importance of the research that takes place in the buildings, we recommend implementation of Scheme II or Scheme III.

Seismic instrumentation in the buildings can help speed the University's recovery and re-occupancy of the laboratories following a damaging earthquake. If funding can be secured for instrumentation, we recommend an instrumentation scheme costing approximately $100,000.

As the next phases of retrofit planning proceed, refinements to the chosen retrofit scheme can be made to maximize its value to the University and to minimize the cost and disruption of its construction. This includes plans for phasing of construction and development of further details of evaluation and retrofitting. If needed, this probabilistic risk analysis can be used in a more detailed way to define likely damage scenarios that can further demonstrate the value of the proposed retrofitting.

Refinements to the study reported here can be made in the categories of structural evaluation, development of retrofit schemes, and risk analysis methodology, as described below.

13.1. POTENTIAL REFINEMENTS TO STRUCTURAL EVALUATION

One of the capabilities that our risk analysis provides is an ability to optimize and "tweak" a retrofit scheme to maximize benefit versus cost. As the next steps of retrofit planning proceed, it may be worthwhile to investigate in further detail two aspects of the structural evaluation and proposed retrofitting. The first is the shear-governed behavior of the mechanical and elevator towers, and the second is refining the benefit and need related to retrofitting column splices. Further detail on these potential refinements is given below. One potential refinement that was suggested to us is to evaluate the effect of vertical ground motions on the structure, which could affect the long span girders. Per section 4.6, we conclude that the structure is not particularly sensitive to vertical acceleration. An additional refinement would be to explicitly consider impact effects when the building separation joints close.

MECHANICAL AND ELEVATOR TOWERS

The mechanical towers are shown in our analysis to be likely to perform well structurally. However, if all of the measures of Retrofit Scheme III are implemented, we find that one of the principal remaining sources of potential structural risk is related to the mechanical towers in very large ground motions. While Scheme III addresses all major deficiencies in the buildings and greatly reduces overall the earthquake risk to the complex, when we analyze the residual risk that remains, a significant portion is related to potential shear failure or irreparable damage of the walls of the mechanical (or elevator) towers.

We have considered possible measures of improving the resilience of the structural performance of these walls. Such measures might include application of horizontally oriented fiber reinforcement (carbon fiber or glass fiber in epoxy) to the exterior of the towers at the lower levels. Shotcrete or a combination of shotcrete and fiber could also be considered. We have not included these measures in any of the retrofit schemes, but it may be worth evaluating their cost and potential benefit to performance, if there were a chance that funding could be obtained for work beyond that required for Scheme III.

Evaluation of this potential additional retrofit measure could also include a more detailed assessment of the mechanism of shear resistance for the unique structural combination of concentric steel braced frames and concrete walls, and a more detailed evaluation of the differences between the elevator tower compared to the mechanical towers.

COLUMN SPLICE RETROFITTING

Retrofitting of column splices has been included as a structural measure in Retrofit Schemes I, II, and III. It is logical to include in the schemes because of its relatively low cost compared to other measures. While the measure is economical to implement, our final risk analysis results showed less resilience benefit to the facility from this measure than we might have expected. This is partly because the column splice only yields in quite large earthquake motions, with a low probability of occurrence. A potential refinement would be to further investigate the benefits of Schemes I, II, and III if this measure were eliminated (either on both stories 7.5 and 5.5, or just on story 7.5). The costs and benefits of this measure could be compared to that of providing shear strengthening of the mechanical towers as discussed above.

13.2. REFINEMENT OF RETROFIT DETAILS IN SUBSEQUENT DESIGN PHASES

There are other refinements to the retrofit scopes that will naturally occur as more detailed designs for the retrofit measures are developed. It is worth mentioning them here as a reminder of tasks that might be included in the scope of schematic retrofit design. Items for possible refinement include:

- Further optimizing tower damping locations and sizes, as wells as optimizing damper properties for the wall damper scheme. Our risk assessment analysis that is set up provides a useful tool for optimization according to the University's goals.
- In laying out the locations of damper frames, more explicitly evaluate the trade-off between architectural preference and column axial load.
- For the damper frames, re-investigating the potential benefits of leveraged damper configurations or wall dampers.
- Evaluate the potential to seismically isolate the mechanical bridge on the rooftop of the HSE building.

13.3. POTENTIAL REFINEMENTS TO SEISMIC RISK ANALYSIS

As retrofit planning for the Health Sciences buildings proceeds, the seismic risk assessment analysis will be available as a tool for evaluating refinements to retrofit scope or decision making. Depending on how it will be used, the tool itself could be refined. Of particular value would be refinement of (a) how the movement of each of the building separation joints is considered, and (b) how ground motions are selected and how many analyses are run. Further refinement and validation of fragility functions could also be carried out.

SEPARATION JOINTS BETWEEN BUILDINGS

Damage to utilities and systems at the separation joints between buildings is a major source of vulnerability for the HSIR complex. Five such joints occur, as summarized in Table 39. Currently we use two categories of EDPs for the five joints—one for the J1 and J5 joints, which are locations of the tower damper scheme, and one for the J2, J3, and J4 joints, at which retrofitting is associated with improving the utilities located in or across the joint. For Joints J1 and J5, the relevant joint movement is explicitly computed by our analysis model. For now, the movement of the other joints in approximated based on the J1 results.

At the time when defined retrofit measures are chosen for the building separation joints, it would be useful to improve the input to the risk analysis that is related to the joints. Possible improvements to the parameters that serve as input includes:

- Considering separately opening, closing, and parallel joint movement.
- More explicitly determining the movement at each of the joints J2, J3, and J4.
- Defining the utility retrofit measures at each joint in a more detailed way, coordinating the utility retrofit scope with the tower damping retrofit, and updating fragility functions related to the added parameters.

Table 39: Characteristics of the separations joints between buildings that are relevant to the seismic risk analysis.

Separation Joint	Between	Associated retrofitting	EDPs used	How EDP values determined	Potential revision to EDPs used
J1	HSE and Mechanical Tower	Tower dampers, Protection of utilities within and crossing the joint.	"Mech Gap" (Maximum and 4^{th} Worst). SRSS of relative motion	Directly from Nonlinear analysis results	Consider parallel and perpendicular movement as separate EDPs. Possibly make opening a separate EDP from closing
J2	HSE and Medical Sciences	Protection of utilities within and crossing the joint.	"Elev Gap" Maximum and 4^{th} Worst.	Assumed nearly equal to EDP for J1 for analyses without tower dampers	Reduce amount of movement relative to J1 to account for stiffness of MedSci building
J3	Medical Sciences and Elevator Tower	Same as J2	Same as J2	Same as J2	Reduce amount of movement relative to J1 to account for stiffness of MedSci building
4	Elevator Tower and HSW	Same as J2	Same as J2	Same as J2	Same as J1
5	HSW and Mechanical Tower	Same as J1	Same as J1	Equal to that for J1	Same as J1

Refinements to Ground Motion Selection

For refined runs of the probabilistic risk assessment, it may be worth considering the following enhancements to the selection of ground motions:

- Include base slab averaging effects, for example per the requirements of FEMA 440. This will improve the estimate of demands and damage related to higher mode effects in the buildings. The Loma Prieta motions recorded at the nearby UCSF school of nursing, reported in Section 3.10, demonstrate that such effects are likely to occur at the HSIR site.
- Select a larger number of ground motions so that correlation of EDPs (discussed in Section 3.11) can be calculated at two or three different earthquake ground motion intensity levels. The increased accuracy from having more analysis runs would have to be balanced against the time and effort required to conduct the analyses and manage the output.
- Consider motion selection with an intensity measure other than spectral acceleration at the building period, for example vector-valued hazard.

14. REFERENCES

2007 Working Group on California Earthquake Probabilities. (2008). *The Uniform California Earthquake Rupture Forecast, Version 2 (UCERF 2): U.S. Geological Survey Open-File Report 2007-1437 and California Geological Survey Special Report 203* . [http://pubs.usgs.gov/of/2007/1437/].

2010 California Building Code. (2010). *California Code of Regulations, Title 24* . Sacramento, CA: California Building Standards Commission/International Code Council.

Abrahamson, N. (2000). Effects of rupture directivity on probabilistic seismic hazard analysis. *Proceedings of the Sixth International Conference on Seismic Zonation (6ICSZ).* Palm Springs, CA: Earthquake Engineering Research Institute.

Applied Technology Council. (May 2011). *Seismic Performance Assessment of Buildings: Volume 1 - Methodology, ATC 58-1 75% Draft.* Redwood City, CA.

Applied Technology Council. (May 2011). *Seismic Performance Assessment of Buildings: Volume 2 - Implementation Guide, ATC 58-1 75% Draft.* Redwood City, CA.

ASCE/SEI. (2006). *ASCE/SEI 41-06: Seismic Rehabilitation of Existing Buildings.* Structural Engineering Institute of American Society for Civil Engineers.

ASCE/SEI. (2005). *ASCE/SEI 7-05: Minimum Design Loads for Buildings and Other Structures: Including Supplement No. 1.* Structural Engineering Institute of American Society of Civil Engineers.

ATC. (1999). *FEMA 306: Evaluation of Earthquake Damaged Concrete and Masonry Wall Buildings: Basic Procedures Manual.* Redwood City, CA: Applied Technology Council.

ATC. (2005). *FEMA 440: Improvement of Nonlinear Static Seismic Analysis Procedures.* Redwood City, CA: Applied Technology Council.

Baker, J. W. (2011). Conditional Mean Spectrum: Tool for Ground Motion Selection. *Journal of Structural Engineering* (137(3)), 322-331.

Bazzuro, P. C. (September 2006). *Advanced Seismic Assessment Guidelines.* College of Engineering, University of California, Berkeley.

Bouwkamp, J. G. (1965). An Investigation for the Earthquake Resistant Design of Large-Size Welded and Bolted Girder to Column Connections. *Proceedings of the Third World Conference on Earthquake Engineering. Volume III*, pp. IV-90 - IV-122. Auckland and Wellington, New Zealand: New Zealand National Committee on Earthquake Engineering.

Chaudhari, S. R., & Hutchinson, T. C. (June 2006). Fragility of Bench-Mounted Equipment Considering Uncertain Parameters. *Journal of Structural Engineering* , 884-898.

Comartin, C. (2010). Personal Communication - UC System Seismic Rating Scale (J. Maffei, Interviewer).

Comerio, M. C. (November 2005). *PEER Testbed Study on a Laboratory Building: Exercising Seismic Performance Assessment.* Pacific Earthquake Engineering Research Center. College of Engineering, University of California, Berkeley.

Comerio, M. C., Stallmeyer, J. C., Holmes, W. H., Morris, P., & Lau, S. (December 2001). *Nonstructural Loss Estimation: The UC Berkeley Case Study.* Pacific Earthquake Engineering Research Center. College of Engineering, University of California, Berkeley.

Der Kiureghian, A. (2005). Non-ergodicity and PEER's Framework Formula. *Earthquake Engineering and Structural Dynamics* (34), 1643-1652.

Ding, D., Arnold, C., Lagorio, H., Tobriner, S., Rihal, S., Mangum, R., et al. (1990). Architecture, Building Contents, and Building Systems. *Earthquake Spectra , 6*, 339-377.

Environment, Health & Safety (EH&S) Laboratory Safety Design Guide: Second Edition. (September 2007). Univeristy of California Industrial Hygiene Program Manangement Group.

FEMA 306: Evaluation of Earthquake Damaged Concrete and Masonry Wall Buildings. (1999). Redwood City, CA: Applied Technology Council.

FEMA 440: Improvement of Nonlinear Static Seismic Analysis Procedures. (2005). Redwood City, CA: Applied Technology Council.

Filiatrault, A., Christopoulos, C., & Stearns, C. (September 2001). *Guidelines, Specifications, and Seismic Performance Characterization of Nonstructural Building Components and Equipment.* Pacific Earthquake Engineering Research Center. College of Engineering, University of California, Berkeley.

Holmes, W. T., & Burkett, L. A. (2006). Seismic Vulnerability of Hospitals Based on Historical Performance in California. *Proceedings, 8th U.S. National Conference on Earthquake Engineering.* San Francisco, CA: Earthquake Engineering Research Institute.

Holmes, W. T., & Comerio, M. C. (October 2003). *Implementation Manual for the Seismic Protection of Laboratory Contents: Format and Case Studies.* Pacific Earthquake Engineering Research Center. College of Engineering, University of California, Berkeley.

Hutchinson, T. C., & Chaudhari, S. R. (2006). Bench-shelf system Dynamic Characteristics and their Effects on Equipment and Contents. *Earthquake Engineering and Structural Dynamics* (35), 1631-1651.

Jayaram, N., Lin, T., & Baker, J. W. (2011). A Computationally Efficient Ground-Motion Selection Algorithm for Matching a Target Response Spectrum Mean and Variance. *Earthquake Spectra* (In press).

Konstantinidis, D., & Makris, N. (January 2005). *Experimental and Analytical Studies on the Seismic Response of Freestanding and Anchored Laboratory Equipment.* Pacific Earthquake Engineering Research Center. College of Engineering, University of California, Berkeley.

Kulak, G. L., Fisher, J. W., & Struik, J. H. (2001). *Guide to Design Criteria for Bolted and Riveted Joints* (Second Edition ed.). Chicago , IL: American Institute of Steel Construction.

Lopez Garcia, D., & Soong, T. T. (2003). Sliding Fragility of Block-Type Non-Structural Components. Part 1: Unrestrained Components. *Earthquake Engineering and Structural Dynamics* (32), 111-129.

Lopez Garcia, D., & Soong, T. T. (2003). Sliding Fragility of Block-Type Non-Structural Components. Part 2: Restrained Components. *Earthquake Engineering and Structural Dynamics* (32), 131-149.

Maffei, J. R. (2000). Suggested Improvements to Performance-Based Seismic Guidelines. *Proceedings of the 12th World Conference on Earthquake Engineering.* Auckland, NZ.

Mander, J. B., Dhakal, R. P., & Mashiko, N. (n.d.). *Incremental Dynamic Analysis Applied to Seismic Risk Assessment of Bridges.* Retrieved 2011 2-August from http://ir.canterbury.ac.nz/bitstream/10092/43/1/12603963_Main.pdf.

Purvance, M. D., Anooshehpoor, A., & Brune, J. N. (2008). Freestanding Block Overturning Fragilities: Numerical Simulation and Experimental Validation. *Earthquake Engineering and Structural Dynamics* (37), 791-808.

Purvance, M. D., Anooshehpoor, R., & Brune, J. N. (December 2009). *Fragilities of Sensitive Geological Features on Yucca Mountain, Nevada.* Pacific Earthquake Engineering Research Center. College of Engineering, University of California, Berkeley.

Rea, D., Bouwkamp, J. G., & Clough, R. W. (1966). *The Dynamic Behavior of Steel Frame and Truss Buildings: A Report of an Investigation to the American Iron and Steel Institute, New York.* Structures and Materials Research, Department of Civil Engineering, Division of Structural Engineering and Structural Mechanics, Berkeley, CA: Office of Research Services, University of California Berkeley.

Reitherman, B., Sabol, T., Bachman, R., Bellet, D., Bogen, R., Cheu, D., et al. (1995). Nonstructural Damage. *Earthquake Spectra , 11*, 453-514.

Roloff, J. (2010 22-July). ThyssenKrup Elevators (L. Burkett, Interviewer).

Schiff, A. (2008 May). *The Shakeout Scenario Supplemental Study: Elevators.* Retrieved 2011 йил 31-August from http://books.google.com/books?id=E_PWT_bv6KAC.

Schoeneck, F. (1971). *Stanlock Glazing Gasket Test Report: Evaluation of the Effects of Earthquake Racking Stresses on a Stanlock Preformed Gasket.* Portland, OR: Griffith Rubber Mills.

Somerville, P. S. (1997). Modification of Empirical Strong Ground Motion Attenuation Relations to Include the Amplitude and Duration Effects of Rupture Directivity. *Seismological Research Letters* , 199-222.

UCSF Facilities Design Guidelines. (Revised 11/04/2003). San Francisco, CA: University of California San Francisco, Capital Projects & Facilities Management.

Vamvatsikos, D., & Cornell, C. A. (March 2002). Incremental Dynamic Analysis. *Earthquake Engineering Structural Dynamics , Volume 31* (Issue 3), 491-514.

Yang, T. Y., Moehle, J., Stojadinovic, B., & Der Kiureghian, A. (2006). An Application of PEER Performance-Based Earthquake Engineering Methodology. *Proceedings 8th US National Conference on Earthquake Engineering.* San Francisco, CA.