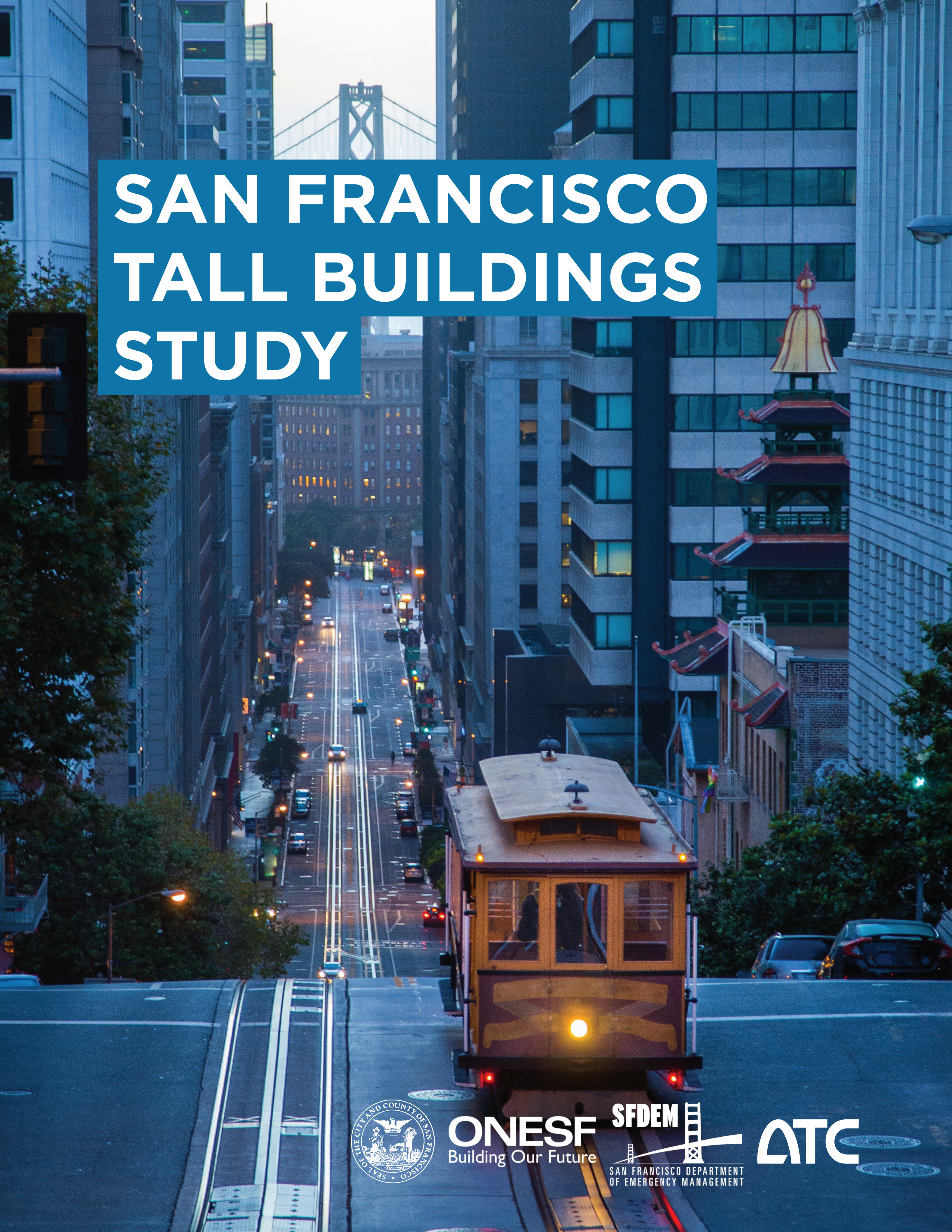
SAN FRANCISCO TALL BUILDINGS STUDY



SEAL OF THE CITY AND COUNTY OF SAN FRANCISCO

ONESF
Building Our Future

SFDEM
SAN FRANCISCO DEPARTMENT OF EMERGENCY MANAGEMENT

ATC

San Francisco Tall Buildings Study

Prepared by

APPLIED TECHNOLOGY COUNCIL
201 Redwood Shores Parkway, Suite 240
Redwood City, California 94065
www.ATCouncil.org

Prepared for

City and County of San Francisco Office of Resilience and Capital Planning
Brian Strong, Chief Resilience Officer and Director
Danielle Mieler, Principal Resilience Analyst
San Francisco, California

ATC MANAGEMENT AND OVERSIGHT

Ayse Hortacsu
Justin Moresco

PROJECT WORKING GROUPS

Carlos Molina Hutt
Preetish Kakoty
Anne McLeod Hulsey
Alireza Eksir Monfared
Max Rattie
Wen-Yi Yen

TASK REVIEW PANEL

Mark X. Haley
William Walton

PROJECT TECHNICAL COMMITTEE

John D. Hooper (Task Leader)
David Bonowitz
Gregory Deierlein
Shahriar Vahdani

CITY AND COUNTY OF SAN FRANCISCO TALL BUILDINGS EXECUTIVE PANEL

Naomi Kelly (Chair)
Mary Ellen Carroll
Kathryn How
Tom Hui
Brian Strong

December 2018

CITY AND COUNTY OF SAN FRANCISCO TALL BUILDINGS STAKEHOLDERS

Board of Supervisors
- District 3
- District 6

Private Engineers and Architects
- DBI Code Advisory Committee
- Heller Manus Architects
- Building Inspection Commission
- HOK
- Maffei Structural Engineering
- BXP
- SEAONC AB 82/83 Code Advisory Committee
- Fennie+Mehl Architects

Developers
- San Francisco Apartment Owners Association
- Tishman Speyer
- Boston Properties

Business
- San Francisco Chamber of Commerce

Real Estate
- Real Estate Advisory Services

Community and Non-profit Organizations
- SPUR
- BOMA
- The East Cut Community Benefit District

City and County of San Francisco Staff
- Department of Building Inspection
- Real Estate Division
- Department of Emergency Management
- Planning Department
- Fire Department
- SFPUC
- Public Works
- Office of Resilience and Capital Planning

Preface

In 2010, the Applied Technology Council (ATC) concluded the Community Action Plan for Seismic Safety (CAPSS) for the City and County of San Francisco, identifying risks from future earthquakes and developing recommendations to reduce the most significant earthquake impacts. As a result, in 2011, the City published the *CAPSS Earthquake Safety Implementation Program Workplan 2012-2042* that anticipates actions for broad groups of new and existing buildings throughout the city. In considering this broad workplan, it became clear that special consideration would be needed to address the unique characteristics of San Francisco's tall buildings. In 2017, ATC was awarded a contract to develop an inventory of tall buildings in San Francisco and to review the impact of earthquakes on such buildings.

ATC is indebted to the members of the Project Technical Committee who served as the principal authors of this report, including David Bonowitz, Greg Deierlein, John Hooper (chair), and Shariar Vahdani, and the members of the Working Groups consisting of Carlos Molina Hutt, Anne McLeod Hulsey, and Wen-Yi Yen. Preetish Kakoty, Alireza Eksir Monfared, and Max Rattie also contributed to the Working Groups. ATC gratefully acknowledges Mark Haley and Bill Walton for their review of Part 6 of this report. The names and affiliations of those who contributed to this report are provided in the list of Project Participants at the end of this report.

ATC also gratefully acknowledges guidance and support provided by Brian Strong and Danielle Mieler of the City's Office of Resilience and Capital Planning. The Tall Buildings Executive Panel, consisting of Mary Ellen Carroll, Kathryn How, Tom Hui, Brian Strong, and Naomi Kelly (chair) provided advice at key stages of the work. ATC staff members Justin Moresco and Carrie Perna provided project management support and report production services, respectively.

Ayse Hortacsu	Jon Heintz
ATC Director of Projects	ATC Executive Director

Summary Recommendations

In 2011, the City and County of San Francisco published the *CAPSS Earthquake Safety Implementation Program Workplan 2012-2042* as a result of the Community Action Plan for Seismic Safety (CAPSS) in response to Mayor Newsom's Executive Directive 10-022. *Workplan 2012-2042* anticipates programs for broad groups of new and existing buildings throughout the City. Looking ahead, the City recognized that within each broad group, some buildings would need special attention through exemptions, programmatic solutions, or specific technical criteria, to make the work feasible.

One such subgroup comprises the City's "tall buildings." In tall buildings, difficult exterior access, multiple tenants and uses within a building, and their sheer size complicate evaluation and retrofit. Their structural systems preclude generic performance assumptions and prescriptive engineering solutions. Damage to a tall building can pose risks well beyond its own footprint. Their high concentration downtown poses an aggregate risk to neighborhood and citywide recovery not presented by other building groups. Perhaps most important, San Francisco's new and existing tall buildings represent a dominant portion of the City's business sector, and increasingly house residents as well.

For these reasons, the City initiated the first project in the nation to consider the impact of earthquakes on tall buildings. The project conducted investigations in seven focus areas under separate tasks. This report documents the complete findings and recommendations of each task, and the recommendations are summarized below.

An inventory developed for the project identified 156 buildings that are 240 feet or taller, either constructed or permitted for construction, primarily located in San Francisco's northeast neighborhoods (Supervisorial Districts 3 and 6). Approximately 60% of these buildings contain primarily business uses, while the others are predominantly residential.

The recommendations presented here flow from a study of these tall buildings, but most are also applicable to a wider set of buildings supporting similar functions or posing similar risks. Tall buildings, even in downtown San

Francisco, are only part of a neighborhood's building stock, and from a public policy perspective, their earthquake performance is bound up with that of the shorter buildings around them.

Each recommended action identifies one or more City departments to lead its implementation. However, implementation of any new policy is assumed to involve appropriate coordination with other City departments, outside experts (as needed), and other stakeholders. Some recommended actions require enactment of legislation by the Mayor and Board of Supervisors or action by the Building Inspection Commission and can only commence after these approvals.

Sections 1 through 4 below summarize the findings from the study in issue statements and describe the associated recommendations to address the issue. Section 5 presents the recommendations in a table format showing different aspects of each recommendation, including potential implementation timeframe, relationship to *Workplan 2012-2042*, City department responsible for implementation, and the relationship of recommendations to Parts of the project report for reference.

The Summary Recommendations were originally published by the City as part of the *Tall Buildings Safety Strategy* on October 4, 2018. They have been updated in response to feedback from the structural engineering community. The specific changes are as follows:

- Recommendation 2A was modified to include a clarification that the repair provisions of the *San Francisco Existing Building Code* should be applied in accordance with a future Administrative Bulletin that defines appropriate triggers and other indicators of potential damage.
- Recommendation 2B was expanded to include the addition of triggers that apply when buildings are purchased or leases are renewed. In addition, the issue statement was expanded to include change of occupancy triggers.

1. Actions for Reducing Seismic Risk Prior to Earthquakes – New Buildings

1A. Develop Regulations to Address Foundation and Geotechnical Issues

Issue: The *San Francisco Building Code* sets minimum requirements for geotechnical site investigations and foundation design. Because they are minimum requirements, they do not fully address all of the geotechnical conditions found in San Francisco. Over the past several decades, the San Francisco geotechnical community has developed best practices for geotechnical evaluation and foundation design, but these are not yet codified. Many of the new tall building developments are challenging even these best practices due to unique soil conditions, the size and weight of the new buildings, and the sophisticated site investigation and the analysis approaches being used to assess overall building behavior, including building response to extreme earthquake ground motions.

Recommendation: To help reduce the risk associated with these geotechnical challenges, the Department of Building Inspection (DBI) should develop an Administrative Bulletin or Information Sheet (with building code amendments as needed) with acceptable practices on topics including the following:

- Settlement design and analysis criteria,
- Quality Assurance/Quality Control for foundation systems,
- Foundation design and other countermeasures for soil liquefaction and lateral spreading,
- Shoring and dewatering design and analysis criteria,
- Lateral earthquake resistance of deep and shallow foundations, and
- Site characterization and exploration.

In addition, to strengthen DBI procedures for assessing the completeness of the foundation and excavation design for tall buildings, two actions are recommended:

- Increase DBI's expertise on geotechnical issues related to tall buildings through enhanced training and staffing.
- Develop a geotechnical report checklist to help ensure the completeness of the submitted geotechnical investigation, design, and field monitoring reports.

1B. Establish Recovery-Based Seismic Design Standards

Issue: *San Francisco Building Code* requirements for earthquake design, including the performance-based requirements of Administrative Bulletin 083, *Requirements and Guidelines for the Seismic Design of New Tall Buildings using Non-Prescriptive Seismic-Design Procedures,* are primarily intended to provide acceptable safety in extreme earthquakes. Studies conducted in this project estimate that for a tall building designed to current standards, it might take 2 to 6 months to mobilize for and repair damage from a major earthquake, depending on the building location, geologic conditions, and the structural and foundation systems. Long downtimes in tall buildings can have disproportionate harmful effects on residents and businesses in San Francisco. By the City's tentative recovery goals, even three months of downtime is unacceptably long for major employers and other recovery-critical uses.

Recommendation: To shorten downtime in new tall buildings, DBI should develop an Administrative Bulletin (with building code amendments as needed) that supports the implementation of the City's tentative recovery goals and specifies recovery-based seismic design and construction requirements, including tighter drift limits under expected ground motions, enhanced design criteria for critical mechanical, electrical, plumbing, and elevator systems, enhanced detailing requirements for exterior cladding and interior partition walls, and measures to mitigate externalities that impede recovery. San Francisco's Building Occupancy Resumption Program (BORP) is designed to address some of these externalities (see recommendation 3B). BORP, or a program like it, should be required for all new tall buildings.

2. Actions for Reducing Seismic Risk Prior to Earthquakes – Existing Buildings

2A. Apply the Repair Provisions of the San Francisco Existing Building Code with Respect to Possible Loma Prieta Damage

Issue: The 1994 Northridge earthquake revealed unexpected damage to dozens of welded steel moment frame structures throughout the greater Los Angeles area (some of which were tall; most of which were not). In most cases, the structural damage did not reveal itself through evident architectural damage or noticeable changes under everyday use. Five years earlier, without the benefit of lessons later learned in Northridge, San Francisco's steel buildings were not consistently inspected after the Loma Prieta earthquake. Ground motion data in downtown San Francisco during the Loma Prieta earthquake are limited, and recorded data are lower than ground motions associated with connection damage in the Northridge earthquake. However, observational data suggest that some areas of San Francisco experienced higher levels of shaking, potentially causing some buildings to sustain connection damage that went undetected.

Recommendation: DBI should develop a new Administrative Bulletin to interpret provisions of the *San Francisco Existing Building Code* as they apply to post-earthquake inspection and evaluation of welded steel moment frames. The Administrative Bulletin should be developed in a community consensus process used to define appropriate ground motion triggers and other indicators of potential damage. This process should consider all currently available information, including FEMA 352, *Recommended Postearthquake Evaluation and Repair Criteria for Welded Steel Moment-Frame Buildings*, available recorded and observational data on ground motions from the Loma Prieta earthquake, and information from building inspections that have occurred since the Loma Prieta earthquake. (Development of the Administrative Bulletin should be coordinated with related work described in Recommendation 3F.)

Using the resulting Administrative Bulletin, DBI should apply the repair provisions of the *San Francisco Existing Building Code* with respect to possible Loma Prieta damage. Because the enforcement would be taking place so long after the damaging earthquake, it would likely benefit from a special program including notification, guidance to owners and tenants, appropriate deadlines, and consideration of voluntary inspection and repair work already performed. This Administrative Bulletin can also be used in the implementation of *Workplan 2012-2042* Task C.2.d, evaluation and retrofit of all pre-1994 welded steel moment frame buildings.

2B. Amend the San Francisco Existing Building Code Triggers

Issue: San Francisco has its own code provisions that trigger seismic upgrade when building alterations or changes of occupancy exceed a certain scope. Because they apply only when two-thirds of a building's floors are altered or the occupant load is significantly increased, they rarely apply to tall buildings, and are easily avoided. Therefore, even the most collapse-prone tall buildings almost never receive the scrutiny intended by the code. That said, since the retrofit of an occupied tall building is especially expensive and disruptive, a more aggressive trigger provision could discourage modernization, tenant improvement, or adaptive reuse. Thus, the *San Francisco Existing Building Code*'s generic provisions are problematic for tall and similarly large or complex facilities.

Federal and State government agencies, as well as some private institutions, also trigger seismic evaluation, and possibly retrofit, when a facility is purchased or leased. As contemplated in *Workplan 2012-2042*, San Francisco could supplement its rarely used alteration and change of occupancy triggers with such an acquisition trigger. Understanding that triggered retrofits of tall buildings can be unreasonably expensive and disruptive, acquisition-based triggers for just evaluation would at least ensure that buildings are properly valued with respect to the risks they pose to owners and tenants.

Recommendation: As the *San Francisco Existing Building Code* is amended to coordinate with the 2019 California codes, the Building

Inspection Commission (BIC) should consider revisions that would find the right balance between triggers and triggered scope from an alteration. This could include setting a lower trigger, such as evaluation and disclosure, for most of tall buildings and reserving a higher trigger, such as mandatory retrofits, for those most prone to collapse. BIC should also consider adding triggers that apply when buildings are purchased or leases are renewed. For purposes of resilience and recovery planning, triggered evaluations might be required to include an estimate of recovery time as well as safety.

2C. Recommend Minimum Levels of Earthquake Insurance or Other Collateral to Ensure Post-Earthquake Recovery

Issue: Available information suggests that earthquake insurance availability and market penetration for commercial and residential buildings are low. Furthermore, when available, the insurance coverage is often limited to a small fraction of the building replacement cost, raising questions about the ability to repair and recover after a damaging earthquake. Insurance or other resources to cover losses suffered by the neighbors of a tall building or costs to the City (for debris removal or emergency protective measures) are also unclear.

Recommendation: The City should identify potential limitations on the availability of financial capital after a damaging earthquake and recommend minimum levels of insurance (or other collateral) for tall building owners to ensure recoverability of their buildings and the neighboring community.

2D. Review Requirements for Post-Earthquake Fire Suppression and Evacuation Systems

Issue: Tall buildings rely on automatic fire suppression systems (typically sprinklers) to inhibit fire spread and allow time for evacuation. Automatic fire suppression will be particularly important following a significant earthquake, when risk of fire ignitions might be higher, and the response time of fire departments might be longer than usual. Normally, sprinkler systems in high-rise buildings use water from the City's municipal system, pressurized with pumps and emergency generators. Recognizing the risk that the City water supply may be disrupted by an earthquake, the *San Francisco Building Code* requires many buildings to have an in-building secondary water supply to operate the sprinkler system for 30 minutes.

Recommendation: The San Francisco Fire Department (SFFD) and the Department of Emergency Management (DEM) should coordinate a study to evaluate the adequacy of automatic fire suppression and occupant evacuation systems in tall buildings for conditions following a significant earthquake. The study should be coordinated with other City departments and within the broader context of the *San Francisco Emergency Response Plan* to evaluate whether: (1) the in-building secondary water supply for automatic fire suppression in tall buildings is sufficient to inhibit fire spread and allow safe evacuation; and (2) the building code provisions that rely on elevators for evacuation during a fire emergency will be effective following an earthquake. The study should develop requirements and recommendations for the *San Francisco Building Code* and *Emergency Response Plan* to address any significant limitations or risks that are identified.

3. Actions for Reducing Seismic Risk Following Earthquakes

3A. Develop New Policies and Procedures for Implementing the State's Safety Assessment Program

Issue: The Safety Assessment Program (SAP), through which volunteer inspectors "post" buildings with red, yellow, or green placards, is run by the California Office of Emergency Services (Cal OES). DBI is charged with implementing San Francisco's participation in the program. The SAP procedures and criteria are based on ATC-20, *Procedures for Postearthquake Safety Evaluation of Buildings*, and are generic and not well-suited to complex or recovery-critical facilities, including most tall buildings.

Recommendation: DBI should develop its own procedures suited to San Francisco's tall buildings (and otherwise unique building stock) regarding such topics as limits on exterior-only inspection, limits on rapid evaluation, damage estimates, placard use, and placard text.

In coordination with its implementation of SAP, DBI should also develop a plan to use specially-qualified SAP volunteers to inspect pre-selected groups of buildings, one of which might be tall buildings, especially those not covered by BORP. More generally, the building groups of interest should be related to the City's adopted recovery goals. These specially trained and assigned inspection teams would facilitate recovery of building types with fast recovery goals.

3B. Extend and Improve the Building Occupancy Resumption Program

Issue: BORP, created by DBI, allows building owners to arrange in advance for post-earthquake safety inspections using their own contracted inspectors. Participation is voluntary. DBI approves each participating building's application and pre-certifies the owner's inspection team. Most of the current BORP participants are downtown office buildings. BORP solves many of the problems associated with applying the general Safety Assessment Program to tall or otherwise complex or recovery-critical buildings. To enhance

BORP's effectiveness and to derive the most value from it, DBI should maintain and update the program.

Recommendation: DBI should enhance the BORP program with the following:

- Conduct simulation-based training to ensure readiness of building staff, BORP-certified inspectors, and DBI staff.
- Update the BORP instructions and procedures to improve consistency and practicality. In particular, material required for certification by DBI should be separate from material to be used by the BORP inspection team in the field. The field material should be organized to align with the ATC-20 evaluation procedures already adopted by BORP as its standard.
- Add specific criteria and pre-earthquake procedures to facilitate implementation of FEMA 352, *Recommended Postearthquake Evaluation and Repair Criteria for Welded Steel Moment-Frame Buildings*, for welded steel moment frame structures.
- Require BORP for all new tall buildings. The program should be extended through incentive programs, triggers, and possibly even mandates (see recommendations 1B and 3H) for certain tall or otherwise recovery-critical buildings. The purpose would be to require at least a basic recovery plan, if not full BORP documentation and implementation, for a larger set of existing buildings.
- Together with the Office of Resilience and Capital Planning (ORCP), extend the BORP scope to support functional recovery in addition to safe re-occupancy. For tall buildings, this might include additional procedures for individual tenant spaces.

3C. Clarify and Update Roles and Responsibilities Associated with Post-earthquake Emergency Response and Safety Inspection

Issue: Current procedures and practices for post-earthquake emergency response and safety inspection are inconsistent, and sometimes out of date, regarding the roles of certain City departments and their interaction with state-level programs and private sector plans (including BORP).

Recommendation: DEM, in coordination with DBI and the Department of Public Works (DPW), should update the Earthquake Annex of the *San Francisco Emergency Response Plan* regarding activation of the Cal OES Safety Assessment Program.

DBI and DPW should also update their SAP and BORP procedures regarding the division of responsibility and criteria for establishing cordons and barricades, in coordination with Lifelines Council recommendations for priority transit routes. (See also recommendation 3G.)

3D. Update and Amend the San Francisco Existing Building Code Triggers for Repair Projects

Issue: The *San Francisco Existing Building Code* triggers for seismic upgrade based on the extent of earthquake damage have fallen out coordination with the latest state code. In addition, while the latest requirements are rational for most buildings, for larger structures, they can become disproportionately expensive and disruptive; if repairs to many large buildings are triggered, the aggregate impact can affect the City's overall recovery. San Francisco's code amendments might exacerbate the problem by not allowing use of reduced loads typical for retrofits.

Recommendation: As the 2019 California code becomes effective, DBI should take the opportunity to update its traditional amendments and coordinate them with the state code.

Regarding repair-triggered retrofits, ORCP, together with appropriate BIC committees, should investigate whether San Francisco should relax its code provisions for certain buildings, especially regarding the "substantial structural damage" trigger. The study should consider typical San Francisco buildings, ideally with a scenario that considers the effects of multiple buildings on downtown recovery.

3E. Update Administrative Bulletin 099 and Clarify its Application to Tall Concrete Structural Systems

Issue: Administrative Bulletin 099, *Post-earthquake Repair and Retrofit Requirements for Concrete Buildings*, supplements the *San Francisco Existing Building Code* by implementing FEMA 306, *Evaluation of Earthquake Damaged Concrete and Masonry Wall Buildings*, published in 1998. Many of AB-099's references are due for updates. Further, while AB-099 applies to all concrete buildings, some of its provisions might prove difficult to apply to tall buildings.

Recommendation: DBI should update AB-099 to coordinate with newer building code provisions and reference standards. DBI should also develop a commentary to the updated AB-099 to guide its application to tall and otherwise complex concrete buildings.

3F. Develop a New Administrative Bulletin for Post-earthquake Inspection and Evaluation of Welded Steel Moment Frames

Issue: Many of San Francisco's tall buildings have welded steel moment frames as their structural systems. FEMA 352, *Recommended*

Postearthquake Evaluation and Repair Criteria for Welded Steel Moment-Frame Buildings, was developed after the 1994 Northridge earthquake specifically for this structure type, but it is not fully referenced by San Francisco codes and procedures.

Recommendation: DBI should develop an Administrative Bulletin to implement FEMA 352. If code amendments are necessary, DBI should work with BIC to approve them.

For post-earthquake safety evaluation, the recommended Administrative Bulletin should clarify the procedures and criteria that DBI will accept in coordination with Rapid and Detailed Evaluations using ATC-20. (As noted in recommendation 3B, FEMA 352 is already allowed by BORP, but the program procedures are incomplete regarding its use, so the recommended Administrative Bulletin could address that shortcoming as well.)

For post-earthquake application of upgrade triggers, just as AB-099 supplements the *San Francisco Existing Building Code* for concrete buildings, a similar bulletin should be developed for welded steel moment frames, which are even more common in San Francisco's tall buildings.

3G. Create Protocols and Procedures for Establishing Cordons around Damaged Buildings

Issue: Cordons or barricades are often needed to protect the areas around a damaged building. The cordoned area is generally based on the perceived level of damage and the risks posed by potential aftershocks, wind loading, time-dependent creep effects, or other factors. While cordons may be required around buildings of any height, the disruptive implications of current generic guidance for cordon distance increase dramatically with building height, potentially leading to unnecessary closure of neighboring buildings and infrastructure.

Recommendation: DPW, in coordination with DBI (see recommendation 3C) should develop protocols and procedures for cordoning around damaged buildings. The procedures should be based on current practices but should also account for tall building effects on neighborhood safety and recovery, as well as new information regarding aftershock risks and early warning.

3H. Require Existing Buildings to File Recovery Plans

Issue: Because aggressive retrofit triggers and mandates can be unreasonably expensive and disruptive for occupied tall buildings, thorough recovery planning is a more feasible alternative. BORP can facilitate recovery in a lightly damaged building, but as a voluntary program, it is not used widely enough to improve the recovery of a whole neighborhood like

San Francisco's downtown, where the City's tall buildings are most densely clustered.

Recommendation: ORCP, in coordination with DBI, should identify buildings critical to the City's recovery and mandate the filing of building information, and possibly a basic recovery plan, with the City. The building information would enhance existing databases and inform the City's response and recovery planning, and the building-specific recovery plan would be a way of extending the benefits of BORP without requiring ongoing expense from owners.

4. Actions to Improve the City's Understanding of its Tall Building Seismic Risk

4A. Maintain and Expand the Database of Tall Buildings

Issue: This project compiled a database with information about all buildings 240 feet or taller either constructed or currently permitted for construction in San Francisco. The database includes information on building location, height, occupancy, age, construction material, structural system, year of retrofit, and foundation type. Prior to the creation of this database, the City had no centralized, searchable repository with this information about all tall buildings in its jurisdiction. Following the completion of this project, the City will need to develop a mechanism for maintaining or expanding the database.

Recommendation: As the owner of the database, ORCP should work with DataSF and DBI to maintain and expand the tall building database. Specific recommended actions include:

- Develop mechanisms for harvesting building data from building permits or inspections administered by DBI or other agencies. For example, *San Francisco Existing Building Code* Chapter 4E will yield information on building façades as the provisions are implemented over the coming years.
- Make the database available for public review and use. This will encourage updates by building owners and will facilitate use of the data in studies to promote recovery and resilience planning.
- Expand the database to include the following:
    - All buildings taller than 75 feet. As described in the project report, the 240-foot height criterion for the initial database was somewhat arbitrary. To the extent that the *San Francisco Building Code* imposes elevator, fire safety, and other requirements on high-rise

buildings defined as those taller than 75 feet, it would be useful to expand the database to include at least all buildings above this height.

- Specific building uses by industry or employment sector. The data should be coordinated with development of the City's overall recovery goals. Ideally, similar data for non-tall buildings serving those same recovery functions would also be compiled, allowing a better understanding of the tall building effects in terms of neighborhood and citywide recovery.
- Foundation type of any building on a site mapped as susceptible to liquefaction.

4B. Develop a Comprehensive Recovery Plan for the Financial District and Adjacent Neighborhoods

Issue: The present study addresses the effect of tall building damage on the tall buildings themselves and, to a lesser extent, on the downtown neighborhoods where tall buildings are densely clustered and on the City overall. But it does not explicitly address the likely interactions between the tall buildings, the non-tall buildings that still comprise most of downtown, and the critical infrastructure that serves the neighborhood. Nor does it explicitly consider resource demands and capacities of the businesses, residents, workers, and other stakeholders.

Recommendation: A separate recovery plan, drawing on the present study's findings, would bring these ideas together in a practical way to support a neighborhood and its functions, as opposed to just individual buildings with certain characteristics. An interim recovery plan, which should be developed by the City Administrator, would need to make many assumptions. This should be followed by a more thorough recovery study to confirm or correct those assumptions and to fill in the most critical knowledge gaps. Such a study would address the combined effects of tall buildings, non-tall buildings, and infrastructure, including liquefaction effects. The study would develop a recovery curve estimating the level of immediate functional loss and the extent of re-occupancy and recovery over time following one or more scenario earthquakes.

The recovery plan should consider developing alternative habitability standards for tall buildings. In 2012, SPUR recommended relaxing normal habitability standards during post-earthquake recovery, gradually returning to a state of normalcy. But the SPUR recommendations were developed primarily for houses and wood-frame apartment buildings. Tall buildings, by contrast, rely on sophisticated systems for heating, ventilation, air-conditioning, elevators, and fire suppression, so the SPUR recommendations will not apply. DEM, together with ORCP, should develop alternative recovery-phase habitability standards for tall buildings, considering minimum requirements for fire barriers, suppression and safety systems, vertical transportation, water services, and electricity.

5. Summary Tables

Table S-1 presents the recommendations in terms of potential implementation timeframes. Short-term actions can be started with essentially no additional study and completed without a legislative process. Typical examples include administrative updates, development of Administrative Bulletins, and initiatives requiring only nominal interdepartmental coordination. Mid-term actions can normally be completed without substantial additional study and without a legislative process but are expected to involve substantial input from multiple stakeholder groups. Long-term actions are expected to require substantial additional technical study or a legislative process.

Table S-2 relates the recommendations to relevant to tasks *Workplan 2012-2042*. In some cases, this tall building study identified additional programs and initiatives not detailed in *Workplan 2012-2042*. In others, the tall building recommendations identify specific technical criteria, exemptions, or programmatic solutions suitable for tall and similarly complex buildings, even relaxing some of the *Workplan 2012-2042*'s broad directives.

Table S-1 Implementation Timeframe

Recommended Action	Short-Term	Mid-Term	Long-Term
1. Actions for Reducing Seismic Risk Prior to Earthquakes – New Buildings			
1A. Develop Regulations to Address Foundation and Geotechnical Issues			
Training and checklist	X		
Develop geotechnical regulations		X	
1B. Establish Recovery-Based Seismic Design Standards			X
2. Actions for Reducing Seismic Risk Prior to Earthquakes – Existing Buildings			
2A. Apply the Repair Provisions of the San Francisco Existing Building Code with Respect to Possible Loma Prieta damage		X	
2B. Amend the San Francisco Existing Building Code Triggers			
Alteration and change of occupancy triggers	X		
Acquisition triggers		X	
2C. Recommend Minimum Levels of Earthquake Insurance or Other Collateral to Ensure Post-Earthquake Recovery			X
2D. Review Requirements for Post-Earthquake Fire Suppression and Evacuation Systems		X	

Table S-1 Implementation Timeframe (continued)

Recommended Action	Short-Term	Mid-Term	Long-Term
3. Actions for Reducing Seismic Risk Following Earthquakes			
3A. Develop New Policies and Procedures for Implementing the State's Safety Assessment Program	X		
3B. Extend and Improve the Building Occupancy Resumption Program			
Conduct simulation-based training	X		
Update procedures	X		
Extend program		X	
3C. Clarify and Update Roles and Responsibilities Associated with Post-earthquake Emergency Response and Safety Inspection	X		
3D. Update and Amend the San Francisco Existing Building Code Triggers for Repair Projects		X	
3E. Update Administrative Bulletin 099 and Clarify its Application to Tall Concrete Structural Systems			X
3F. Develop a New Administrative Bulletin for Post-Earthquake Inspection and Evaluation of Welded Steel Moment Frames			X
3G. Create Protocols and Procedures for Establishing Cordons around Damaged Buildings	X		
3H. Require Existing Buildings to File Recovery Plans			X
4. Actions to Improve the City's Understanding of its Tall Building Seismic Risk			
4A. Maintain and Expand the Database of Tall Buildings	X		
4B. Develop a Comprehensive Recovery Plan for the Financial District and Adjacent Neighborhoods			X

Table S-2 Relationship of Recommended Actions to Workplan 2012-2042 Tasks

Recommended Action	Workplan 2012-2042 Task
1. Actions for Reducing Seismic Risk Prior to Earthquakes – New Buildings	
1A. Develop Regulations to Address Foundation and Geotechnical Issues	New action
1B. Establish Recovery-Based Seismic Design Standards	B.6.a Update code for new buildings to reflect desired performance goals
2. Actions for Reducing Seismic Risk Prior to Earthquakes – Existing Buildings	
2A. Apply the Repair Provisions of the San Francisco Existing Building Code with Respect to Possible Loma Prieta damage	B.4.b Develop post-earthquake repair and retrofit standards

Table S-2 Relationship of Recommended Actions to Workplan 2012-2042 Tasks (continued)

Recommended Action	Workplan 2012-2042 Task
2. Actions for Reducing Seismic Risk Prior to Earthquakes – Existing Buildings (continued)	
2B. Amend the San Francisco Existing Building Code Triggers	C.1.a Mandatory evaluation on sale or by deadline C.1.b Evaluation of buildings retrofitted prior to 1994 or building to non-conforming performance standards C.2.a Mandatory retrofit of older non-ductile concrete residential buildings C.2.d Mandatory evaluation and retrofit of pre-1994 welded steel moment frame buildings C.2.e Mandatory evaluation and retrofit of other low-performance buildings
2C. Recommend Minimum Levels of Earthquake Insurance or Other Collateral to Ensure Post-earthquake Recovery	A.1.b Provide information and assistance about insurance
2D. Review Requirements for Post-Earthquake Fire Suppression and Evacuation Systems	A.6.i Study fire-related earthquake resilience topics
3. Actions for Reducing Seismic Risk Following Earthquakes	
3A. Develop New Policies and Procedures for Implementing the State's Safety Assessment Program	A.4.f Update post-earthquake inspection (ATC-20) policies and procedures
3B. Extend and Improve the Building Occupancy Resumption Program	B.1.b Develop non-structural upgrade program for businesses
3C. Clarify and Update Roles and Responsibilities Associated with Post-Earthquake Emergency Response and Safety Inspection	Procedural update
3D. Update and Amend the San Francisco Existing Building Code Triggers for Repair Projects	B.4.b Develop post-earthquake repair and retrofit standards
3E. Update Administrative Bulletin 099 and Clarify its Application to Tall Concrete Structural Systems	A.4.d Adopt disproportionate damage trigger B.4.b Develop post-earthquake repair and retrofit standards
3F. Develop a New Administrative Bulletin for Post-Earthquake Inspection and Evaluation of Welded Steel Moment Frames	A.4.d Adopt disproportionate damage trigger B.4.b Develop post-earthquake repair and retrofit standards
3G. Create Protocols and Procedures for Establishing Cordons around Damaged Buildings	Program update, new action
3H. Require Existing Buildings to File Recovery Plans	B.1.b Develop non-structural upgrade program for businesses
4. Actions to Improve the City's Understanding of its Tall Building Seismic Risk	
4A. Maintain and Expand the Database of Tall Buildings	A.2.b Adopt façade maintenance regulations
4B. Develop a Comprehensive Recovery Plan for the Financial District and Adjacent Neighborhoods	A.4.a Develop and adopt Shelter-in-Place policies and procedures B.2.b Mandatory evaluation of 5+ dwelling unit residential buildings and hotels/motels C.2.b Mandatory evaluation and retrofit of critical stores, suppliers, and service providers C.2.c Mandatory evaluation and retrofit of larger (over 300 occupants) assembly buildings

Table S-3 presents the recommendations in terms of City departments responsible for implementation. The departments are identified as follows:

- BIC Building Inspection Commission
- DataSF
- DBI Department of Building Inspection
- DEM Department of Emergency Management
- DPW Department of Public Works
- ORCP Office of Resilience and Capital Planning
- SFFD San Francisco Fire Department

Table S-4 presents the relationship of Summary Recommendations to Parts of this report for reference.

Table S-3 Responsible Department

Recommended Action	BIC	Data SF	DBI	DEM	DPW	ORCP	SFFD
1A. Develop Regulations to Address Foundation and Geotechnical Issues			X				
1B. Establish Recovery-Based Seismic Design Standards			X				
2A. Apply the Repair Provisions of the San Francisco Existing Building Code with Respect to Possible Loma Prieta damage			X				
2B. Amend the San Francisco Existing Building Code Triggers	X						
2C. Recommend Minimum Levels of Earthquake Insurance or Other Collateral to Ensure Post-Earthquake Recovery			X				
2D. Review Requirements for Post-earthquake Fire Suppression and Evacuation Systems			X				X
3A. Develop New Policies and Procedures for Implementing the State's Safety Assessment Program			X				
3B. Extend and Improve the Building Occupancy Resumption Program			X			X	
3C. Clarify and Update Roles and Responsibilities Associated with Post-Earthquake Emergency Response and Safety Inspection			X	X	X		
3D. Update and Amend the San Francisco Existing Building Code Triggers for Repair Projects	X		X			X	
3E. Update Administrative Bulletin 099 and Clarify its Application to Tall Concrete Structural Systems			X				
3F. Develop a New Administrative Bulletin for Post-Earthquake Inspection and Evaluation of Welded Steel Moment Frames	X		X				
3G. Create Protocols and Procedures for Establishing Cordons around Damaged Buildings			X	X	X		
3H. Require Existing Buildings to File Recovery Plans			X			X	
4A. Maintain and Expand the Database of Tall Buildings		X	X			X	
4B. Develop a Comprehensive Recovery Plan for the Financial District and Adjacent Neighborhoods				X		X	

Table S-4 Recommended Actions by Part

Recommended Action	Part 1	Part 2	Part 3	Part 4	Part 5	Part 6	Part 7
Actions for Reducing Seismic Risk Prior to Earthquakes – New Buildings							
1A. Develop an AB and Code Amendment that Addresses Foundation and Geotechnical Issues		X					
1B. Establish Performance-Based Seismic Design Standards			X				
Actions for Reducing Seismic Risk Prior to Earthquakes – Existing Buildings							
2A. Apply the Repair Provisions of the SFEBC with Respect to Possible Loma Prieta Damage						X	
2B. Amend the SFEBC Triggers							X
2C. Require Minimum Levels of Earthquake Insurance to Ensure Recovery					X		
2D. Increase Local Water Supply for automatic Fire Suppression Systems in Tall Buildings					X		
Actions for Reducing Seismic Risk Following Earthquakes							
3A. Develop New Policies and Procedures for Implementing the State's SAP					X	X	
3B. Extend and improve BORP						X	
3C. Clarify and Update Roles and Responsibilities Associated with Post-earthquake Emergency Response and Safety Inspection					X	X	
3D. Update and Amend the SFEBC				X			
3E. Update AB-099 and Clarify its Application to Tall Concrete Structural Systems				X			
3F. Develop a New AB to Implement FEMA 352 for Post-earthquake Inspection and Evaluation of Welded Steel Moment Frames				X		X	
3G. Create Protocols and Procedures for Establishing Cordons Around Damaged Buildings					X		
3H. Require Existing Buildings to File Recovery Plans							X
Actions to Improve the City's Understanding of its Tall Building Seismic Risk							
4A. Maintain and Expand the Database of Tall Buildings	X						
4B. Develop a Comprehensive Recovery Plan for the Financial District and Adjacent Neighborhoods						X	

Table of Contents

Part 2: Geotechnical Engineering for Tall Buildings

Part 5: Tall Building Effects on Post-earthquake Recovery

Part 6: Post-earthquake Occupancy and Safety Inspection

Part 7: Pre-earthquake Evaluation

List of Figures

List of Tables

List of Abbreviations

AB	administrative bulletin
ATC	Applied Technology Council
BIC	Building Inspection Commission
CAPSS	Community Action Plan for Seismic Safety
CEBC	California Existing Building Code
DBI	Department of Building Inspection
DD	disproportionate damage
DEM	Department of Emergency Management
DPW	Department of Public Works
FEMA	Federal Emergency Management Agency
IEBC	International Existing Building Code
ORCP	Office of Resilience and Capital Planning
SEAONC	Structural Engineers Association of Northern California
SFEBC	San Francisco Existing Building Code
SFFD	San Francisco Fire Department
SSD	substantial structural damage

Introduction to Tall Buildings Study

I.1 Background

In 2017, the City and County of San Francisco initiated the first project in the nation to consider the impact of earthquakes on a city's tall buildings. The project conducted investigations in seven focus areas under separate tasks. This project supplements the *CAPSS Earthquake Safety Implementation Program Workplan 2012-2042* published by the City in 2011 as a result of the Community Action Plan for Seismic Safety (CAPSS) in response to Mayor Newsom's Executive Directive 10-022. The *Workplan 2012-2042* anticipates programs for broad groups of new and existing buildings throughout the city.

In 2018, the City recognized that tall buildings may need special attention through exemptions, programmatic solutions, or specific technical criteria, to make the work described in the *Workplan 2012-2042* feasible. This is because difficult exterior access, multiple tenants and uses within a building, and their sheer size complicate evaluation and retrofit; their structural systems preclude generic performance assumptions and prescriptive engineering solutions; and San Francisco's new and existing tall buildings represent a dominant portion of the City's business sector. In addition, their high concentration downtown poses an aggregate risk to neighborhood and citywide recovery not presented by other building groups.

I.2 Project Objectives and Scope

The primary objective of this study is to examine the earthquake performance of San Francisco's tall buildings and develop recommendations to address building code requirements, policies and practices for the design of new buildings, assessment and retrofit of existing buildings, and post-earthquake inspection and response to promote the earthquake resilience of San Francisco. To achive this objective, work was conducted in seven focus areas under separate tasks. This report documents the findings and recommendations resulting from each of the focus areas. It is noted that the findings and recommendations here flow from a study of these tall buildings, but most are also applicable to a wider set of buildings supporting similar functions or posing similar risks. Tall buildings, even in downtown San Francisco, are only part of a neighborhood's building stock, and from a public policy perspective, their earthquake performance is bound up with that of the shorter buildings around them.

The initial task of the project was to quantify the tall buildings in the City in terms of height, age, usage, and structural system characteristics. The building inventory database is developed in a Geographical Information Systems (GIS) format to facilitate analysis and visualization of the tall building inventory.

The project team served as principal authors of the findings and recommendations of the focus areas. The findings were developed by means of a literature review and consultation with other professionals, where available. The City provided guidance and feedback through regular meetings with the Tall Buildings Executive Panel (TBEP). In addition, key stakeholders representing elected officials, private engineers and architects, developers, community organizations, and city officials were convened by the City to receive updates on the study's progress and provide feedback.

I.3 Intended Audience and Next Steps

This report presents the project team's findings and recommendations. While the Summary Recommendations of this report can be used for big picture implementation, a thorough review of the discussion presented in this report is critical for the success of any implementation activity.

Table I-1 presents the recommendations in terms of potential implementation timeframes. Short-term actions can be started with essentially no additional study and completed without a legislative process. Typical examples include administrative updates, development of Administrative Bulletins, and initiatives requiring only nominal inter-departmental coordination. Mid-term actions can normally be completed without substantial additional study and without a legislative process but are expected to involve substantial input from multiple stakeholder groups. Long-term actions are expected to require substantial additional technical study or a legislative process.

Table I-2 relates the recommendations to relevant to tasks in *Workplan 2012-2042*. This connection will ensure that the recommendations fit in the City's existing programs. In some cases, this study identified additional programs and initiatives not detailed in *Workplan 2012-2042*. In others, the tall building recommendations identify specific technical criteria, exemptions, or programmatic solutions suitable for tall and similarly complex buildings, even relaxing some of the *Workplan*'s broad directives.

Table I-3 presents the summary recommendations in terms of City departments responsible for implementation. Implementation of any new policy is assumed to involve appropriate coordination with other City departments, outside experts (as needed), and other stakeholders. Some recommended actions require enactment of legislation by the Mayor and Board of Supervisors or action by the Building Inspection Commission and can only commence after these approvals. In Table I-2, the departments are identified as follows:

- BIC Building Inspection Commission
- DataSF DataSF
- DBI Department of Building Inspection
- DEM Department of Emergency Management
- DPW Department of Public Works
- ORCP Office of Resilience and Capital Planning
- SFFD San Francisco Fire Department

I.4 Report Organization

This report documents findings of seven focus areas and related recommendations. Each focus area is covered in a "Part" of this report. Each Part consists of multiple chapters that document findings and recommendations, as well as necessary appendices and references. Although the focus areas are documented distinctly, where substantial overlaps are identified, references are made to relevant Parts.

- Part 1 presents the tall building inventory
- Part 2 summarizes best practices for geotechnical engineering
- Part 3 studies performance expectations for new buildings
- Part 4 studies post-earthquake structural evaluation
- Part 5 studies tall building effects on post-earthquake recovery
- Part 6 studies post-earthquake safety inspection of buildings
- Part 7 studies pre-earthquake evaluation of buildings

Table I-4 presents the relationship of Summary Recommendations to Parts of this report for reference.

Table I-1 Implementation Timeframe

Recommended Action	Short-Term	Mid-Term	Long-Term
1. Actions for Reducing Seismic Risk Prior to Earthquakes – New Buildings			
1A. Develop Regulations to Address Foundation and Geotechnical Issues			
Training and checklist	X		
Develop geotechnical regulations		X	
1B. Establish Recovery-Based Seismic Design Standards			X
2. Actions for Reducing Seismic Risk Prior to Earthquakes – Existing Buildings			
2A. Apply the Repair Provisions of the San Francisco Existing Building Code with Respect to Possible Loma Prieta damage		X	
2B. Amend the San Francisco Existing Building Code Triggers			
Alteration and change of occupancy triggers	X		
Acquisition triggers		X	
2C. Recommend Minimum Levels of Earthquake Insurance or Other Collateral to Ensure Post-Earthquake Recovery			X
2D. Review Requirements for Post-earthquake Fire Suppression and Evacuation Systems		X	
3. Actions for Reducing Seismic Risk Following Earthquakes			
3A. Develop New Policies and Procedures for Implementing the State's Safety Assessment Program	X		
3B. Extend and Improve the Building Occupancy Resumption Program			
Conduct simulation-based training	X		
Update procedures	X		
Extend program		X	
3C. Clarify and Update Roles and Responsibilities Associated with Post-Earthquake Emergency Response and Safety Inspection	X		
3D. Update and Amend the San Francisco Existing Building Code Triggers for Repair Projects		X	
3E. Update Administrative Bulletin 099 and Clarify its Application to Tall Concrete Structural Systems			X
3F. Develop a New Administrative Bulletin for Post-Earthquake Inspection and Evaluation of Welded Steel Moment Frames			X
3G. Create Protocols and Procedures for Establishing Cordons around Damaged Buildings	X		
3H. Require Existing Buildings to File Recovery Plans			X
4. Actions to Improve the City's Understanding of its Tall Building Seismic Risk			
4A. Maintain and Expand the Database of Tall Buildings	X		
4B. Develop a Comprehensive Recovery Plan for the Financial District and Adjacent Neighborhoods			X

Table I-2 Relationship of Recommended Actions to Workplan 2012-2042 Tasks

Recommended Action	Workplan 2012-2042 Task
1. Actions for Reducing Seismic Risk Prior to Earthquakes – New Buildings	
1A. Develop Regulations to Address Foundation and Geotechnical Issues	New action
1B. Establish Recovery-Based Seismic Design Standards	B.6.a Update code for new buildings to reflect desired performance goals
2. Actions for Reducing Seismic Risk Prior to Earthquakes – Existing Buildings	
2A. Apply the Repair Provisions of the San Francisco Existing Building Code with Respect to Possible Loma Prieta damage	B.4.b Develop post-earthquake repair and retrofit standards
2B. Amend the San Francisco Existing Building Code Triggers	C.1.a Mandatory evaluation on sale or by deadline C.1.b Evaluation of buildings retrofitted prior to 1994 or building to non-conforming performance standards C.2.a Mandatory retrofit of older non-ductile concrete residential buildings C.2.d Mandatory evaluation and retrofit of pre-1994 welded steel moment frame buildings C.2.e Mandatory evaluation and retrofit of other low-performance buildings
2C. Recommend Minimum Levels of Earthquake Insurance or Other Collateral to Ensure Post-earthquake Recovery	A.1.b Provide information and assistance about insurance
2D. Review Requirements for Post-earthquake Fire Suppression and Evacuation Systems	A.6.i Study fire-related earthquake resilience topics
3. Actions for Reducing Seismic Risk Following Earthquakes	
3A. Develop New Policies and Procedures for Implementing the State's Safety Assessment Program	A.4.f Update post-earthquake inspection (ATC-20) policies and procedures
3B. Extend and Improve the Building Occupancy Resumption Program	B.1.b Develop non-structural upgrade program for businesses
3C. Clarify and Update Roles and Responsibilities Associated with Post-earthquake Emergency Response and Safety Inspection	Procedural update
3D. Update and Amend the San Francisco Existing Building Code Triggers for Repair Projects	B.4.b Develop post-earthquake repair and retrofit standards
3E. Update Administrative Bulletin 099 and Clarify its Application to Tall Concrete Structural Systems	A.4.d Adopt disproportionate damage trigger B.4.b Develop post-earthquake repair and retrofit standards
3F. Develop a New Administrative Bulletin for Post-earthquake Inspection and Evaluation of Welded Steel Moment Frames	A.4.d Adopt disproportionate damage trigger B.4.b Develop post-earthquake repair and retrofit standards
3G. Create Protocols and Procedures for Establishing Cordons around Damaged Buildings	Program update, new action
3H. Require Existing Buildings to File Recovery Plans	B.1.b Develop non-structural upgrade program for businesses

Note: The Earthquake Safety Implementation Program (ESIP) Workplan 2012-2042 was published by the City and County of San Francisco in 2011 as a result of the Community Action Plan for Seismic Safety (CAPSS) in response to Mayor Newsom's Executive Directive 10-02.

Table I-2 Relationship of Recommended Actions to Workplan 2012-2042 Tasks(continued)

Recommended Action	Workplan 2012-2042 Task
4. Actions to Improve the City's Understanding of its Tall Building Seismic Risk	
4A. Maintain and Expand the Database of Tall Buildings	A.2.b Adopt façade maintenance regulations
4B. Develop a Comprehensive Recovery Plan for the Financial District and Adjacent Neighborhoods	A.4.a Develop and adopt Shelter-in-Place policies and procedures
	B.2.b Mandatory evaluation of 5+ dwelling unit residential buildings and hotels/motels
	C.2.b Mandatory evaluation and retrofit of critical stores, suppliers, and service providers
	C.2.c Mandatory evaluation and retrofit of larger (over 300 occupants) assembly buildings

Table I-3 Responsible Department

Recommended Action	BIC	Data SF	DBI	DEM	DPW	ORCP	SFFD
1A. Develop Regulations to Address Foundation and Geotechnical Issues			X				
1B. Establish Recovery-Based Seismic Design Standards			X				
2A. Apply the Repair Provisions of the San Francisco Existing Building Code with Respect to Possible Loma Prieta damage			X				
2B. Amend the San Francisco Existing Building Code Triggers	X						
2C. Recommend Minimum Levels of Earthquake Insurance or Other Collateral to Ensure Post-Earthquake Recovery			X				
2D. Review Requirements for Post-earthquake Fire Suppression and Evacuation Systems			X				X
3A. Develop New Policies and Procedures for Implementing the State's Safety Assessment Program			X				
3B. Extend and Improve the Building Occupancy Resumption Program			X			X	
3C. Clarify and Update Roles and Responsibilities Associated with Post-Earthquake Emergency Response and Safety Inspection			X	X	X		
3D. Update and Amend the San Francisco Existing Building Code Triggers for Repair Projects	X		X			X	
3E. Update Administrative Bulletin 099 and Clarify its Application to Tall Concrete Structural Systems			X				
3F. Develop a New Administrative Bulletin for Post-Earthquake Inspection and Evaluation of Welded Steel Moment Frames	X		X				
3G. Create Protocols and Procedures for Establishing Cordons around Damaged Buildings			X	X	X		
3H. Require Existing Buildings to File Recovery Plans			X			X	
4A. Maintain and Expand the Database of Tall Buildings		X	X			X	
4B. Develop a Comprehensive Recovery Plan for the Financial District and Adjacent Neighborhoods				X		X	

Table I-4 Recommended Actions by Part

Recommended Action	Part 1	Part 2	Part 3	Part 4	Part 5	Part 6	Part 7
Actions for Reducing Seismic Risk Prior to Earthquakes – New Buildings							
1A. Develop an AB and Code Amendment that Addresses Foundation and Geotechnical Issues		X					
1B. Establish Performance-Based Seismic Design Standards			X				
Actions for Reducing Seismic Risk Prior to Earthquakes – Existing Buildings							
2A. Apply the Repair Provisions of the SFEBC with Respect to Possible Loma Prieta Damage						X	
2B. Amend the SFEBC Triggers							X
2C. Require Minimum Levels of Earthquake Insurance to Ensure Recovery					X		
2D. Increase Local Water Supply for automatic Fire Suppression Systems in Tall Buildings					X		
Actions for Reducing Seismic Risk Following Earthquakes							
3A. Develop New Policies and Procedures for Implementing the State's SAP					X	X	
3B. Extend and improve BORP						X	
3C. Clarify and Update Roles and Responsibilities Associated with Post-earthquake Emergency Response and Safety Inspection					X	X	
3D. Update and Amend the SFEBC				X			
3E. Update AB-099 and Clarify its Application to Tall Concrete Structural Systems				X			
3F. Develop a New AB to Implement FEMA 352 for Post-earthquake Inspection and Evaluation of Welded Steel Moment Frames				X		X	
3G. Create Protocols and Procedures for Establishing Cordons Around Damaged Buildings					X		
3H. Require Existing Buildings to File Recovery Plans							X
Actions to Improve the City's Understanding of its Tall Building Seismic Risk							
4A. Maintain and Expand the Database of Tall Buildings	X						
4B. Develop a Comprehensive Recovery Plan for the Financial District and Adjacent Neighborhoods						X	

PART 1:
Inventory of Tall Buildings

Chapter 1

Introduction

1.1 Background

San Francisco's building inventory has grown significantly over the past century, with much of the current development rooted in the rebuilding following the 1906 earthquake and fire. This rebuilding included significant land development along the San Francisco waterfront. Since the late 1950s, San Francisco's skyline has changed dramatically with the construction of tall buildings in the downtown financial district. Significant growth over the past fifteen years has included high-rise residential and mixed-use buildings concentrated in the region south of Market Street. Among the multi-faceted earthquake risks facing the city, the concentration of tall buildings and infrastructure in the densely populated downtown neighborhoods is raising questions about the risks to life, property, and recovery from large earthquakes in San Francisco.

As a first step towards addressing these questions, this report describes the development of an inventory of tall buildings in San Francisco. The primary motivation is to quantify the tall buildings in terms of height, age, usage, and structural system characteristics of the buildings. Although not the only earthquake risk, tall buildings are of special concern due to their size and large occupant loads, where earthquake damage to one tall building can have disproportionate effects on its occupants, its neighbors, and the community at large. The inventory is intended to help inform policies and practices to manage risks associated with existing buildings and to improve the planning and design of future buildings. The inventory can further serve to inform planning for emergency response and recovery.

One of the key decisions in establishing the tall building inventory is to establish a definition of "tall." While the immediate emphasis of the tall building inventory development is on buildings taller than 240 feet (roughly 18 to 24 stories tall), this height is somewhat arbitrary insofar as all buildings are vulnerable to earthquake damage and have implications on the seismic resilience of San Francisco. The 240-foot value is based on a threshold for certain seismic design requirements for buildings in national standards that are adopted by the *San Francisco Building Code* (CCSF, 2016). Further discussion is provided in Part 5. While 240 ft is a convenient threshold to define the initial scope of this inventory, apart from this practical aspect, the 240-foot height does not have any empirical or scientific basis in how buildings respond to earthquakes. In this regard, any planning, policy, or other recommendations developed on the basis of the inventory would be equally relevant for shorter buildings.

This Part summarizes the development of the inventory and presents recommendations for maintenance and update of the inventory. This Part corresponds to Recommendation 4A presented in Summary Recommendations.

This Part is not specifically covered in *CAPSS Earthquake Safety Implementation Program's Workplan 2012-2042* (CCSF, 2011), but addresses aspects of the following task:

- Task A.2.b Adopt façade maintenance regulations

1.2 Inventory Overview

For this study, two databases were assembled. The first is a general database of all buildings in the downtown neighborhoods of San Francisco, which was compiled from building information available from DataSF (https://datasf.org/), the *San Francisco Property Information Map* (CCSF, 2017a), and other sources. Shown in Figure 1-1 is a map of buildings from this database in downtown San Francisco, including the Financial District and the South of Market neighborhoods. This map shows the concentration of tall buildings in the downtown region, distinguished between height groupings with thresholds of 75 ft (6 to 8 stories), 160 ft (12 to 16 stories) and 240 ft (18 to 24 stories). The 75-foot threshold is based on the fire safety code in Section 403 of the *California Building Code* (CBSC, 2016), which requires special provisions for "high-rises" over 75 ft. The 160-foot and 240-foot limits are based on building code requirements for seismic design. A subset of these buildings covering the blocks with the highest concentration of tall buildings was also produced and is discussed in Part 2 Section 2.2.

The second database is a more detailed inventory of buildings over 240 ft tall that includes information on building location, height, occupancy, age of construction, construction material, structural system, foundation type, façade system, and other relevant design information. This was compiled from the general data sources listed above, combined with surveys of building permit drawings for all buildings taller than 240 ft. The locations and schematic geometries of these tall buildings are shown in Figure 1-2. The locations are superimposed on a map where beige shading identifies areas that are susceptible to soil liquefaction during strong earthquakes, which is one of many risk considerations addressed in their design. The database includes 156 buildings taller than 240 ft, based on currently available data. The database also includes some additional buildings between 160 ft to 240 ft tall, although this portion of the list is incomplete and does not reflect the complete inventory in this height range.

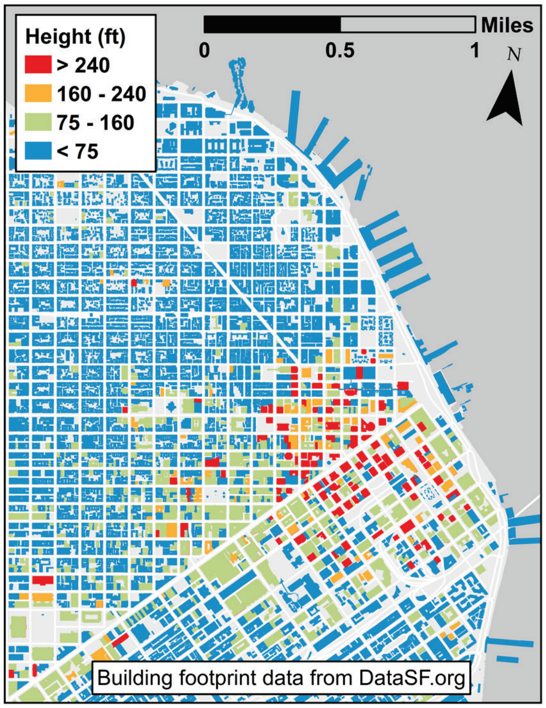


Figure 1-1 Spatial distribution of San Francisco's downtown building inventory mapped as a function of building height.

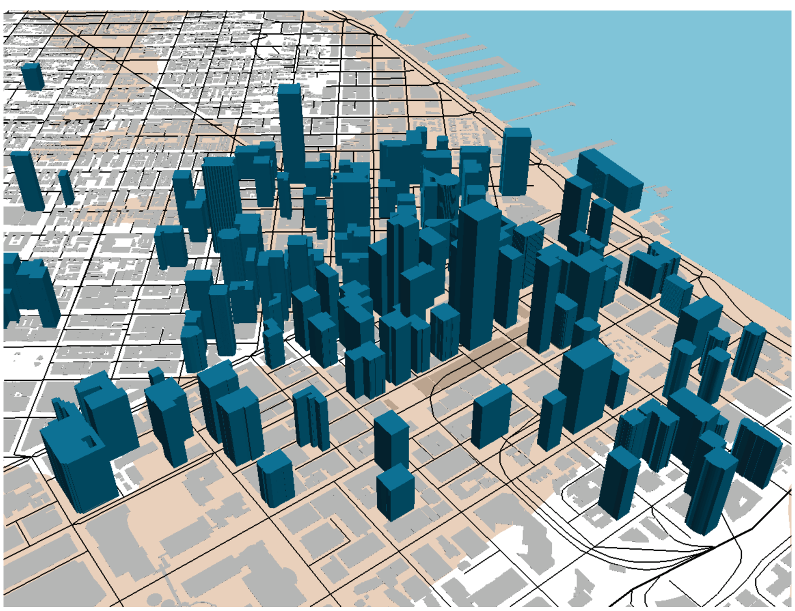


Figure 1-2 Spatial distribution of San Francisco's tall building inventory. Footprints of buildings not included in the inventory are shown for reference. Beige denotes liquefaction zone (per DataSF).

Further details and statistics from both databases are discussed later in Section 2. The two databases are compiled in an industry standard Geographic Information System (GIS) Shapefiles, which can be viewed and interrogated using a variety of open source and proprietary software systems.

1.3 Data Fields and Source

Damage to tall buildings has the potential to affect a large number of people and can have significant consequences on surrounding areas. Susceptibility to earthquake damage is attributed to several factors including: intensity of ground motion shaking, soil characteristics at the site, age of construction, building material, structural system, cladding, interior finishes, and mechanical and electrical components. Many of these features, which are identified in the database, can help understand the expected seismic performance of these building under varying intensities of ground motion shaking.

The data fields contained within the tall building inventory can be classified into the following broad categories:

- Building name and location
- Height (by feet and number of stories)
- Primary occupancy
- Date (permit or completion, depending on source)
- Retrofit information (if applicable/available)
- Construction material
- Structural system (i.e., lateral force resisting system)
- Foundation system
- Geotechnical information (e.g., soil class, depth to rock)
- Massing (e.g., overall dimensions, square footage)
- Irregularities (e.g., podiums, setbacks)
- Façade composition (e.g., pre-cast concrete panels, glass)
- Fire-suppression information (i.e., system type and rating)
- Presence of strong motion instrumentation
- Other remarks and observations

A detailed list of all the variables included within the database is included in Appendix A. Although not all parameters are available for all buildings, the available information provides a fairly comprehensive description of buildings taller than 240 ft and allows for the expansion of the database as additional information becomes available.

Numerous data sources enabled the development of the tall building inventory. Review of available information found that because these data sources are developed for different purposes, information for a single building parameter can be conflicting across sources. For instance, when reporting square footage, some sources may report gross square footage, while others may indicate usable square footage. When conflicting parameters were identified from different source documents, the most representative values were selected based on judgement for inclusion in the database. The following are the key sources of data used in the compilation of the tall building inventory:

- Construction permit documents at the San Francisco Department of Building Inspection (DBI)
- Public data from the City and County of San Francisco's Planning Department (CCSF, 2017)
- San Francisco Fire Department data
- Building Occupancy Resumption Program (BORP) reports (discussed further in Part 6)
- Existing literature, including a previous database developed by the Structural Engineers Association of Northern California (Molina Hutt et al. 2016, Almufti et al. 2018, USGS, 2018)
- Interviews and surveys with local professional engineers.
- Emporis (https://www.emporis.com/)

Appendix A identifies the major source of data for each data field included in the tall building inventory.

In addition to data fields discussed above, the GIS shape file also includes spatial location polygons representing the footprint of each building. Polygon footprint data for buildings that existed during a LIDAR flight, conducted on behalf of the City in 2010, are taken from the LIDAR dataset available on DataSF. For buildings constructed since 2010, the footprint is approximated. Note that these polygons are based on the footprints at ground level and include, for example, podium structures that are not as tall as the main tower of the building.

1.4 Maintaining an Updated Inventory

Up-to-date and comprehensive building databases are an important resource to better understand and quantify vulnerability of the city to earthquakes, fire conflagrations, bayshore flooding, and other hazards. When coupled with modern computational and information technology software, detailed databases enable reliable assessment and implementation of policies and other measures to reduce vulnerability and increase resilience of the city. By incorporating information on seismic retrofit of buildings, BORP participation, and other measures to improve resilience, the database can provide City administrators data on building owner behavior and the effectiveness of voluntary and mandatory policies. Chapter 3 of this Part includes information and suggestions of three categories of tall buildings where tracking and assessment of policy implementations could be particularly worthwhile.

To help facilitate maintenance and updating of the tall building inventory, Appendix B provides a recommended form for consideration by San Francisco DBI as a supplement to building permit applications (retrofit and/or new construction). The proposed supplement would enable maintenance of the database with information on new or retrofitted buildings, including the following key parameters: building name and location, height, occupancy, construction date, construction material, structural system, foundation system and façade composition. It is further recommended that San Francisco DBI consider developing mechanisms to collect information from other inspections and reports, such as those required by San Francisco's façade ordinance (CCSF, 2017b), to supplement the database. To the extent that all of the information in the database was obtained from publicly available sources, the database can be made publicly available.

1.5 Organization

Chapter 2 describes the characterization of building inventory features, presents a study of all buildings in a limited study area in downtown San Francisco for context-setting, and presents an overview of seismic codes and other considerations.

Chapter 3 describes tall building cohorts identified when reviewing the database of tall buildings.

Chapter 4 presents a summary of the inventory and recommendations.

Appendix A presents attributes present in the tall building inventory and descriptions.

Appendix B sets forth a supplementary form to be used during the building permit process to enhance inventory maintenance and expansion.

Appendix C presents the background for bedrock depth data in San Francisco.

A list of references is provided at the end of this Part.

Chapter 2

Characterization of Building Inventory Features

2.1 Inventory of Buildings Taller than 240 Feet

The tall building inventory includes detailed information on 156 buildings over 240-feet tall, which currently exist or have been permitted for construction in San Francisco. The specifics of the structural materials and systems for these buildings were identified by reviewing structural drawings on file in the building permit and Building Occupancy Resumption Program (BORP) report databases maintained by San Francisco Department of Building Inspection (DBI) (more discussion on the BORP program is in Part 6). The tall building inventory also includes information on an additional 22 buildings between 160 ft to 240-ft tall, which have been collected from other sources. The list of buildings below 240-feet tall, and details of their structural systems and materials, are not complete since the first priority for the database was to focus on buildings above 240 ft.

Summary statistics from the database of buildings 240-ft tall and above are provided below according to height, occupancy, construction date, construction material and structural configuration.

- **Height:** Roughly 10% of the inventory is below 20 stories, 70% of the inventory is in the 20- to 40-story range, 20% is above 40 stories.
- **Occupancy:** Approximately 60% of the buildings are commercial and just under 40% are residential or hotel.
- **Construction date:** Over 55% of the tall building inventory was constructed between 1960 and 1990. Tall building construction slowed down in the 1990s, but has resurged since then, with almost 25% of the inventory constructed since 2000.
- **Construction material:** Structural steel systems account for about 65% of the inventory, reinforced concrete systems account for about 20% of the buildings, and the remainder either incorporate mixed steel-concrete systems or, in a few instances, the systems were unidentified.
- **Structural configuration:** Approximately 50% of the buildings employ steel moment-resisting frames as the seismic-force-resisting system

(SFRS), 10% have steel braced frame systems, and about 30% have reinforced concrete shear wall systems. Consistent with the building codes and engineering practice of the 1960-1990 period, most of the steel braced frame and the older concrete shear wall systems are combined with a moment resisting frame, forming a dual system. Many concrete shear wall buildings constructed since the introduction of performance-based seismic design procedures around 2007 are shear wall only systems. About 10% of the older buildings in the inventory did not have enough available information to identify a structural system.

Figure 1-3 illustrates the fraction of buildings according to their SFRS, decade of construction, and number of stories. Superimposed on the figure are notes indicating two significant earthquake events (San Fernando in 1971 and Northridge 1994) and the advent of performance-based design procedures (2007) that relate to significant changes building code requirements and design practices, which are discussed further in the next section.

Figure 1-4 presents the inventory distinguished by building occupancy. Whereas the structural system and construction date are relative indicators of expected seismic performance, the occupancy type provides insights into the services that tall buildings within the inventory provide to the community.

Three noteworthy trends in the data in Figures 1-3 and 1-4 are: (1) the predominance of steel moment-resisting frame construction during the 1960s, 70s and 80s; (2) a transition from steel moment-resisting frames to steel braced frame dual systems in the 1990s; and (3) the emergence of concrete shear wall systems since 2000. The trend away from steel moment-resisting frame to braced frames is due to a combination of architectural preferences towards taller buildings with more irregular geometries, combined with advancements in computer methods that facilitated their analyses. The post-2000 emergence of concrete shear walls is in large part due to the construction of high-rise residential buildings, whose architectural configurations (smaller floor plate sizes and room layouts, and smaller occupant densities with lower elevator and HVAC requirements) make them more amenable to concrete construction.

Figure 1-5 presents an alternate view to illustrate the number of buildings as a function of construction date and number of stories. The red shaded portion reveals that about 35% of the tall building inventory is in the range of 15 to 29 stories, built in the 1960s through 1980s. The blue shaded portion highlights more recent construction with a clear trend toward modern buildings of 40 stories, representing just over 12% of the inventory, built since 2000.

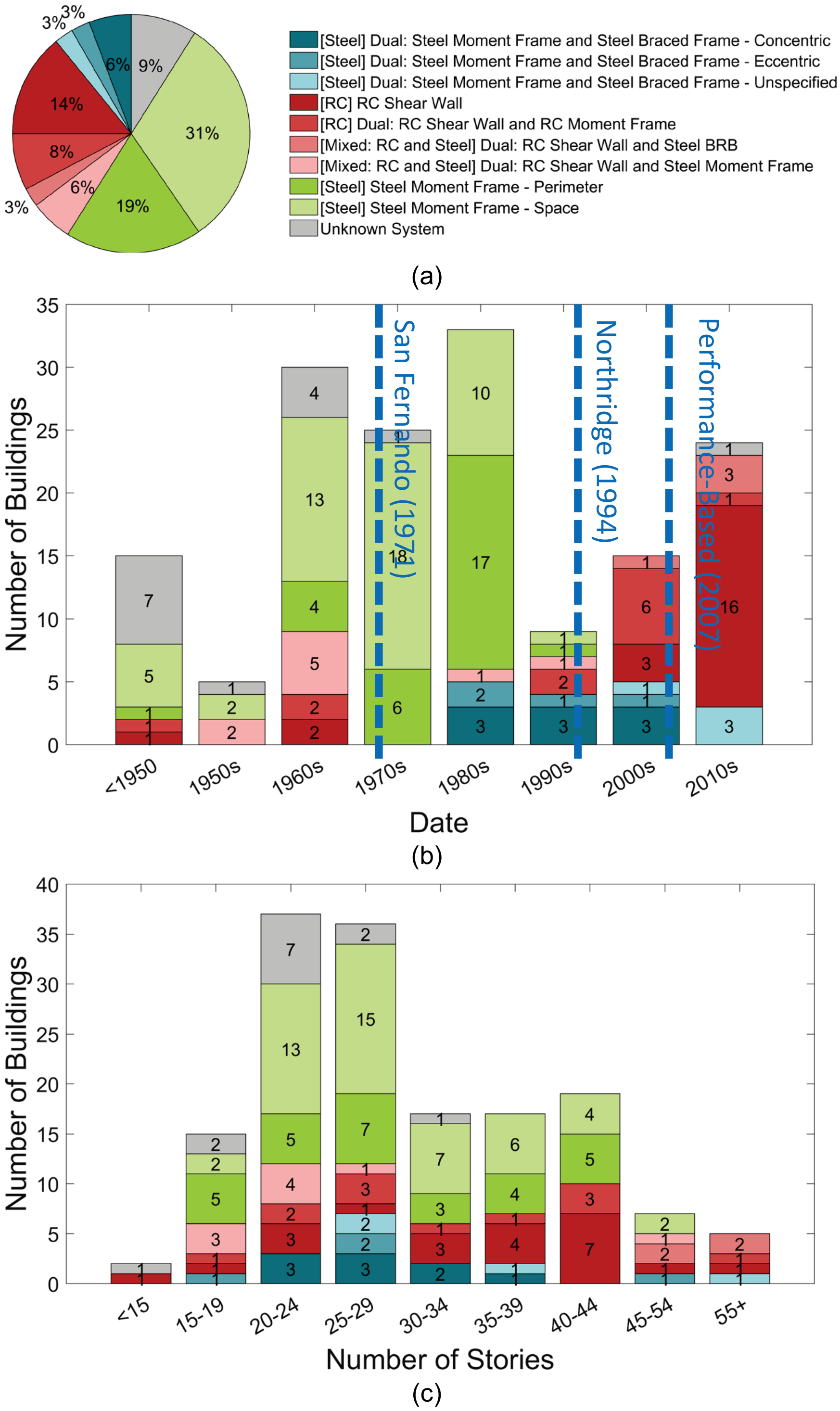


Figure 1-3 Figure showing 156 San Francisco buildings tall than 240 ft by: (a) percentage of buildings by SFRS type, (b) breakdown of SFRS by decade, and (c) distribution of SFRS by number of stories.

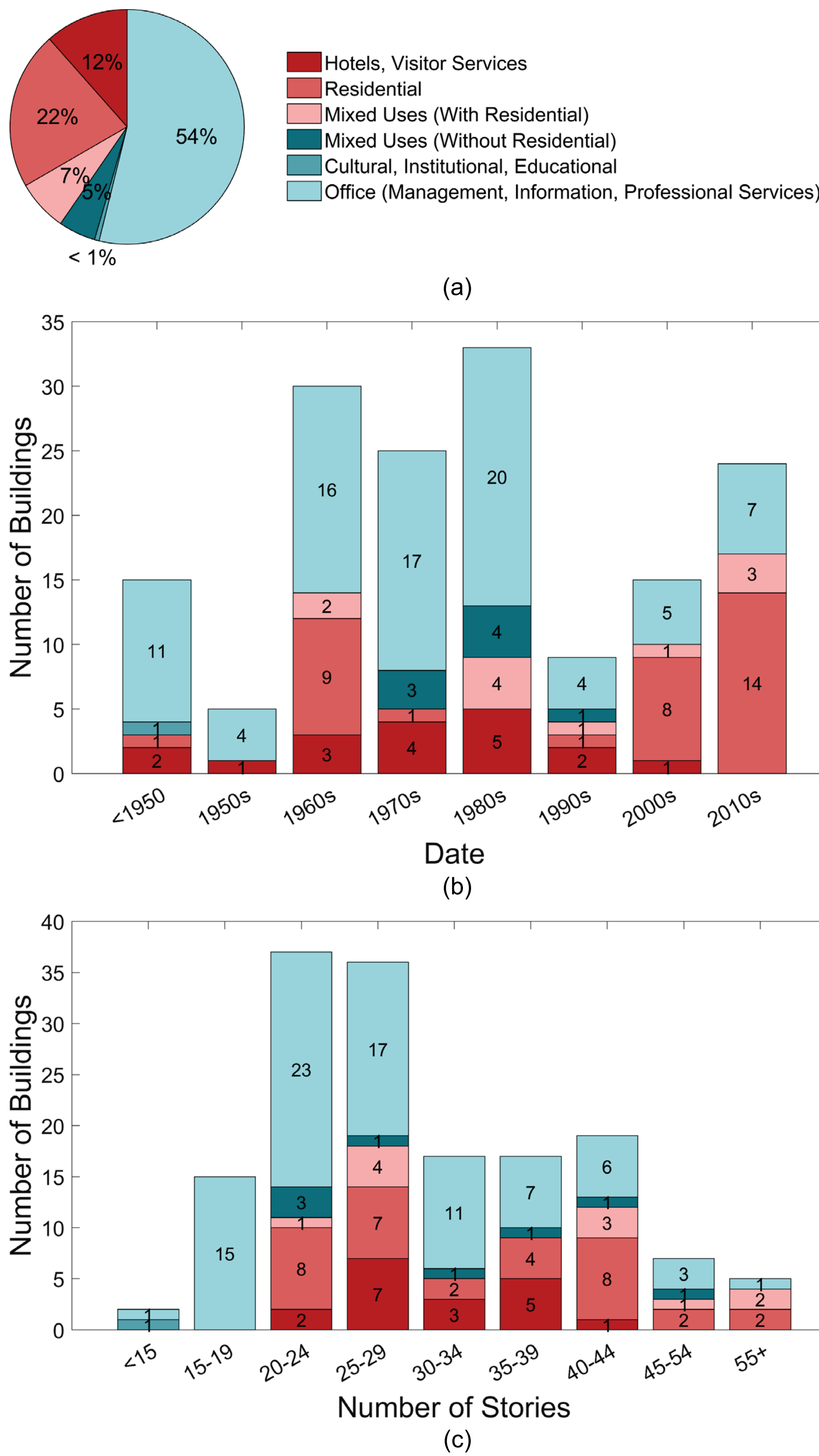


Figure 1-4 Figure showing 156 San Francisco buildings taller than 240 ft by: (a) percentage of buildings by occupancy type, (b) distribution of occupancies by decade, and (c) distribution of occupancies by number of stories.

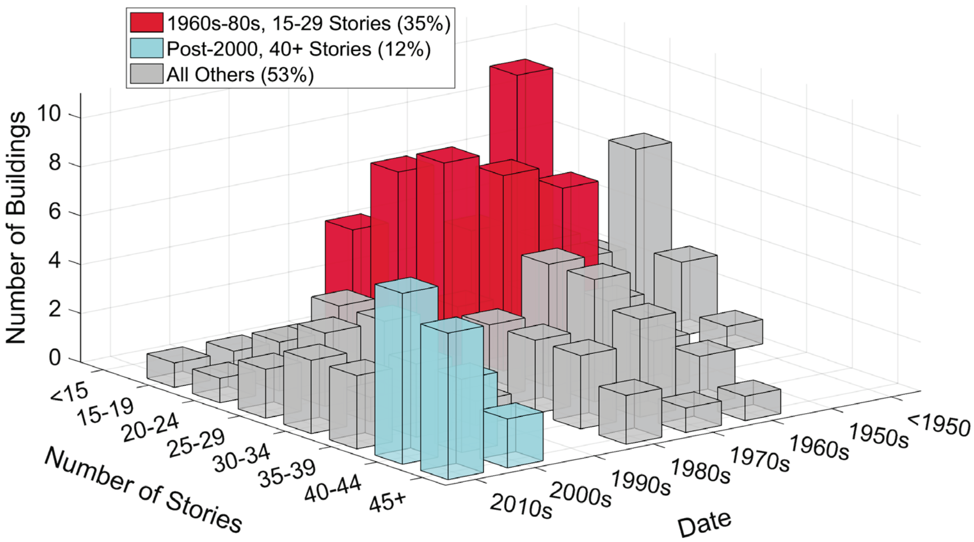
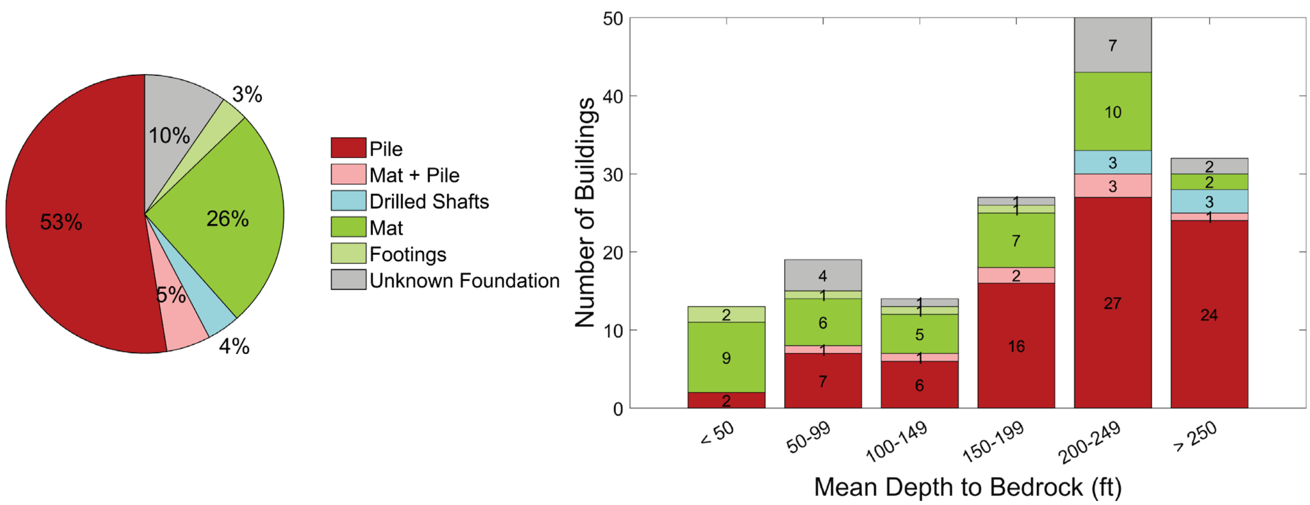


Figure 1-5 Figure showing distribution of the number of stories by decade.

Figure 1-6 provides a distribution of the foundation systems used, with over half of the buildings on pile foundations and about a quarter on shallow mat foundations. The remainder of foundations consist of shallow footings, pile supported mats, or drilled shafts. Drilled shaft foundation techniques were introduced relatively recently to San Francisco, and used in several tall buildings constructed in sites with depth to bedrock greater than about 200 ft.



(a) (b)

Figure 1-6 Figure showing: (a) percentage of buildings by foundation type, and (b) breakdown of foundation types by depth to bedrock.

2.2 A Closer Look: Financial District Building Inventory

To understand San Francisco's tall buildings in context, a limited study was conducted considering every building, of any height, in a 31-block portion of downtown, shown in Figure 1-7a.

The study area does not include all of San Francisco's tallest or most iconic buildings. The Transamerica Tower, Embarcadero Center, Millennium Tower, and Salesforce Tower are all outside the study area. However, the study area does cover the downtown neighborhoods with the highest concentration of tall buildings. Were the study area to be extended one block in each direction, the basic findings would not change significantly.

The study area contains 243 distinct buildings. The structural system, retrofit status, use/occupancy, and occupant load of each building are of interest to a comprehensive resilience-based inventory, but for this context study, only the most basic building data – height, building area, and age – were compiled. Figure 1-7b shows the distribution of height and dates of the buildings.

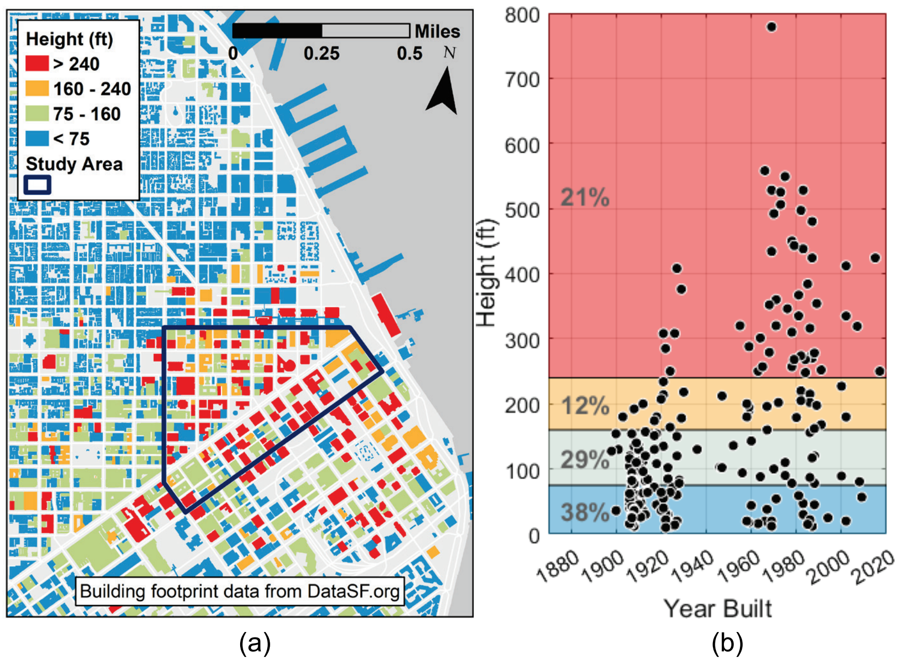


Figure 1-7 Map showing 243 buildings in a 31-block downtown San Francisco study area: (a) the study area, focused on the blocks with the highest concentrations of tall buildings; and (b) the height and dates of buildings in the study area, with the percent in each height category.

The height, area, and age data were compiled from Datasf.org, San Francisco Property Information Map (CCSF, 2017a), Sanborn maps, and satellite and street level imagery. In addition, limited information for the 243 buildings were found or confirmed from online searches by address, with data coming from websites produced by building owners, leasing agents, contractors, architects, or building historians. The resulting dataset, while not perfect, is more complete and reliable than the data from any single source.

Table 1-1 summarizes the data from Figure 1-7 in bins by height and age in count normalized to the total number of buildings. The height bins are based on the precedent building code limits (75 ft, 160 ft, and 240 ft) as well as an arbitrary distinction at 400 ft. The age bins are based on San Francisco's development history, as evident from Figure 1-7, and on the date of a key change in steel building design that was implemented following the 1994 Northridge earthquake.

Table 1-1 Height and Age of All Buildings in Study Area

Number of Buildings					
Height [ft]	*Count*	*Pre-1915*	*1915-1950*	*1951-1994*	*Post-1994*
401+	18	0	1	15	2
241 – 400	33	0	5	25	3
161 – 240	28	3	9	14	2
75 – 160	71	41	14	14	2
Up to 74	93	54	19	18	2
All	243	98	48	86	11
Normalized to total of 243					
Height [ft]	*All*	*Pre-1915*	*1915-1950*	*1951-1994*	*Post-1994*
401+	7%	0%	0%	6%	1%
241 – 400	14%	0%	2%	10%	1%
161 – 240	12%	1%	4%	6%	1%
75 – 160	29%	17%	6%	6%	1%
Up to 74	38%	22%	8%	7%	1%
All	100%	40%	20%	35%	5%

While the age bins in Table 1-1 are continuous, Figure 1-7 reveals trends within each bin:

- The Pre-1915 bin captures the decade of rebuilding that followed the 1906 earthquake and fire, but there is an obvious concentration in the first few years. Sixty-two of the 98 buildings in this group date from 1906 to 1908.
- The 1915-1950 bin includes the building boom of the 1920s, in which much of San Francisco's housing stock was built. Forty-three of the 48 buildings in this group were built before 1930.
- The 1915-1950 bin also shows the gap in development during the Great Depression and World War II, during which San Francisco added only a handful of buildings to its downtown.

- The 1951-1994 bin includes the mini-boom of tall buildings in the 1970s and 1980s. Within this 44-year period, 54 of the 86 buildings were built in those two decades.

Table 1-1 and Figure 1-7 show, unsurprisingly, a relationship between height and age. Very few of San Francisco's tallest buildings were built before 1960, and most of the shorter buildings (even up to 160 ft) were built before 1930. This trend will become stronger as each new tall building replaces several older and shorter ones.

Available data did not support a study of building area relative to height or age. Even so, it is obvious that the taller buildings have much larger areas; the 51 buildings taller than 240 ft likely contain more total square footage than the 192 shorter buildings combined. Therefore, a structurally deficient tall building represents a greater potential loss than a similarly deficient shorter building.

On the other hand, the total street frontage presented by buildings in different height categories is likely to include a higher percentage of lower buildings. Just as the tall buildings have greater volume, the more numerous short buildings front a larger share of downtown streets and sidewalks. Cladding failure and other nonstructural falling hazards can be as dangerous from 50 feet as from 250 feet. So where falling hazards or leaning buildings are of concern, it is possible that the shorter buildings, as a group, may pose greater risks to more blocks and more public spaces than the tall buildings, especially considering their age and obsolete materials. San Francisco's façade inspection requirements are likely to increase understanding on this issue.

2.3 Evolution of Seismic Building Codes

Changes to the seismic and other design requirements that have been made to the *San Francisco Building Code* over the past century offer important clues to interpreting the building databases. Building codes are living documents that are revised and updated over time, often in response to observations from damaging earthquakes, development of new building technologies, and most recently to advancements in building performance assessment methods. Building code requirements also evolve as the risk or the expected performance associated with such requirements is deemed unacceptable (FEMA, 2006). Therefore, older buildings may include deficiencies that were unrecognized by the building codes and practices under which they were designed and constructed but have since been acknowledged in newer building codes. This section provides a brief review of the evolution in seismic building codes as related to implications on the safety and performance of tall buildings.

The *Uniform Building Code* (UBC) was first published in 1927 to help promote public safety and standardized construction. The code was updated every three years until 1997, which was the final version of the code before the introduction of the *International Building Code* (IBC) in 2000. One significant change over the course of the UBC revisions was the inclusion of seismic design code provisions. From 1927 to 1961, the UBC had a non-mandatory appendix, until mandatory seismic design requirements were introduced via recommendations published in 1959 by the Structural Engineers Association of California (SEAOC) in *Recommended Lateral Force Requirements and Commentary*, or the "SEAOC Blue Book." This established performance goals that still underlie today's building codes. The code's minimum design provisions were intended to provide earthquake safety. Most were based on fundamental engineering principles, but many, including height limits and height-related details, were set by consensus engineering judgment. The requirements included minimum lateral strength provisions, simplified design methods and other prescriptive requirements to help avoid deficiencies that led to severe building damage and collapse in past earthquakes.

Over the course of time, the definitions for seismic requirements have been updated. In the 1970s, Cornell's (1968) principles of seismic hazard analysis were implemented to develop hazard maps that represented the intensity of earthquake ground shaking with a 10% chance of being exceeded occurring during the [assumed] 50-year design life of a building. Statistically, this probability of exceedance is also characterized as a ground motion with a 475-year return period (the average time over which the ground motion is typically exceeded). For many years, this level of intensity was the basis of the design basis earthquake (DBE) in building codes. As more data became available to characterize earthquake ground motion hazard and recognizing the nonlinear relationship between ground motion risk and intensity, the Maximum Considered Earthquake (MCE) was introduced into building codes in the mid-1990s. In downtown San Francisco, the MCE ground motion has roughly a 2% to 5% chance of being exceeded in 50 years, depending on the soil conditions (about 2% in rock sites and 5% in soft-soil sites). This level of ground motion is on the order of what could be expected by a magnitude-8 earthquake on a nearby segment of the San Andreas fault.

The ground motion hazard is one component of design, which defines the expected ground shaking. The other component involves many assumptions inherent in building code design and analysis requirements, which vary by structural materials and systems. The combined risk is based on the combined probability of experiencing strong ground motions and the chance of damage and collapse to those ground motions. The most recent building codes, based on ASCE/SEI 7-16 (ASCE, 2017), are more explicit as to the implied life safety risk to buildings, stating that a code-designed building should have less than 10% probability of collapse under the Maximum Considered Earthquake (MCE) ground motion. While this is a maximum permissible risk, it is generally accepted that the actual risk of collapse in modern code-designed tall buildings is less than the 10% rate under MCE ground motions, although this risk is difficult to rigorously quantify.

For most buildings, design methodologies are still based on simplified analysis methods and prescriptive requirements for proportioning structural members and connections. The predominant method of design entails calculating the lateral forces based on weight of the building modified by coefficients to account for soil conditions, building height, the type of structural system and other factors. Though these prescriptive methodologies do not explicitly measure the seismic performance of the design, they are generally accepted as providing the minimum requirement of life safety for conventional buildings. However, these types of prescriptive rules are limited to structural systems and configurations based on traditional design and construction practices and observations of building performance in past earthquakes. More recently, advanced performance-based analysis and design approaches have been introduced for tall buildings.

From a structural response perspective, tall buildings have unique seismic response characteristics including fundamental translation periods of vibration well in excess of 1 second, significant mass participation and lateral response in higher modes of vibration, as well as seismic resisting systems with slender aspect ratios (PEER, 2017). These characteristics result in distinct seismic behavior when compared to low- and mid-rise construction. The limitations of code prescriptive requirements to tall building design are apparent through height limitations associated with certain SFRS. For example, the standard building code does not allow prescriptively designed steel braced frame or concrete shear wall systems to be used in buildings over 160 ft tall (or with certain restrictions 240 ft), unless they are combined with moment-resisting frames (ASCE, 2017).

In response to the shortcomings and limitations of standard prescriptive seismic design requirements, over the past decade, performance-based seismic design guidelines have been developed and applied for tall building design. In 2008, San Francisco issued Administrative Bulletin 083 (CCSF, 2008), which formally permitted the use of performance-based design procedures and outlined several key design requirements. In 2010, the first edition of the *Guidelines for Performance-based Seismic Design of Tall Buildings* was published (PEER, 2010), which was updated in 2017 and is routinely employed for the design of tall buildings (greater than 240 ft tall) in San Francisco. The Los Angeles Tall Building Structural Design Council has published similar guidelines (LATBSDC, 2017), which have been adopted for the design of new tall buildings in Los Angeles.

The so-called performance-based seismic design procedures employ advanced nonlinear analysis to evaluate the expected building response to large earthquakes and, thereby, provide greater assurance that buildings meet the performance intended in current building codes to minimize life safety risks under extreme earthquakes, and help control damage under small frequent earthquakes. Moreover, by permitting exceptions to certain prescriptive design requirements, the performance-based design procedures allow opportunities to design more economical structural systems.

Further discussion of seismic design and performance for three predominant types of structural systems (older steel framed buildings, older non-ductile concrete buildings, and modern high-rise residential buildings) are included in Chapter 3 of this Part.

2.4 Other Considerations

The inventory process also revealed the following about San Francisco's tall buildings:

Geotechnical Site Conditions: Much of San Francisco's downtown is built on soil fill placed between the late 1800s to mid-1900s. Liquefiable soils are also within this area, usually in the top 30 ft of soil with denser soil (sand and clay layers) below. The soil type and foundation system type can influence the seismic response of tall buildings (see Part 2). The tall building foundation types, summarized in Figure 1-6, and reported in the inventory are selected based on the soil conditions at the site, combined with other design parameters, including:

- Depth to rock (typically 0 to > 250 ft in San Francisco)
- Soil type or stiffness (typical San Francisco soil profiles include recent marine deposits, Colma sands, and old bay clay).
- Building height and weight
- Number of basement levels
- Slope of soil surface or proximity to shoreline
- Ground water level

- Proximity to adjacent or underlying structures
- Available construction technologies, logistics, and economics

Fire Regulations: Since 1976, San Francisco has required fire sprinklers in all new buildings 75 feet high or taller. This threshold is based on limitations of firefighters reaching stories about this height. In 1993, San Francisco required retrofitting high-rise commercial buildings and hotels with sprinklers, but the requirement excluded residential buildings. Nevertheless, even many residential buildings have been retrofitted such that most buildings have at least some percentage of their interior sprinklered. The database reports the percentage of sprinklers that are installed in the 135 buildings that could be cross-referenced with the Fire Department's database. Of these buildings with known information, 84% of them are over 95% sprinklered (i.e., essentially fully sprinklered), 7% are reported to be less than 30% sprinklered, and 9% are in between 30% to 95% sprinklered.

Zoning Ordinances and Urban Planning: Zoning ordinances and urban planning can influence the development of high-rise buildings. For instance, prior to the mid-80s few tall buildings were located south of Market Street, whereas the majority of post-2000 tall buildings built in this area of the city. The *Downtown Area Plan* (CCSF, 1995), adopted in 1985, introduced a set of policies to guide land use decisions that created today's downtown. Urban planning and zoning changes can continue to shape future developments.

BORP: Following the 1989 Loma Prieta earthquake, San Francisco introduced the Building Occupancy Resumption Program (BORP) to facilitate inspection and recovery following an earthquake. Participation in the program is voluntary and requires an up-front investment by the building owner to underwrite the preparation of an engineering report of the building system. Fifty seven of the 156 buildings over 240-feet tall (37% of the inventory) currently participate in BORP.

Building Instrumentation: As specified in San Francisco's Administrative Bulletin 058, *Procedures for Seismic Instrumentation of New Buildings*, (CCSF, 2014) and the *California Building Code* (CBSC, 2016), all new buildings over 10 stories tall constructed since about 2008 are required to have seismic strong motion instrumentation. This instrumentation provides data on how the building responds to earthquakes, which can facilitate post-earthquake inspection and provide data on the performance of buildings, which can inform earthquake safety policies and building code provisions and ultimately lead to improved design and construction practices. Based on information from the California Strong Motion Instrumentation Program (CSMIP), an organization that records strong motion data, 8 of the 156 buildings over 240-ft tall have strong motion instrumentation. This number is contrasted with about 20 buildings constructed (or permitted) since 2010, suggesting that either some buildings are instrumented but not included in the CSMIP database or are otherwise out of compliance.

Chapter 3

Tall Building Cohorts

3.1 Identification of Tall Building Cohorts

The tall building inventory enables the identification of building clusters or "cohorts" that best represent the City's existing tall building stock. This section identifies the following three cohorts which have relevance to earthquake damage and recovery in San Francisco: older steel-framed buildings, older nonductile concrete buildings, and modern concrete residential buildings.

Table 1-2 summarizes the height range, occupancy type, construction dates, construction material and structural system for the three cohorts of interest. Sections 3.2 to 3.4 describe the rationale for the selection of each cohort (e.g., high risk, prominent building type, disproportionate consequences associated with poor performance, or unique design approach) and further observations.

Table 1-2 Predominant Features of Tall Building Cohorts of Special Interest

Cohort Name	Story Range	Occupancy Type	Construction Dates	Construction Material	Structural System
Older Steel Buildings	20-40	Commercial	Before mid- 1990s	Steel	Moment-resisting frame
Older Concrete Buildings	Low 20s	Commercial	Before mid- 1970s	Reinforced Concrete	Shear Wall & Dual System
Modern Concrete Building	>40	Residential	Post 2000	Reinforced Concrete	Shear Wall

3.2 Older Steel Buildings

Approximately 50% of the buildings within the inventory have steel moment-resisting frame seismic-force-resisting systems (SFRS), and almost 90% of these were constructed in the 1960s through the 1980s. These are identified as a building cohort of primary interest owing to: (1) their prominence as the most common system type in the tall building inventory; and (2) concern regarding deficiencies in their design, including the potential for fracture prone welded connections, which came to light following the 1994 Northridge earthquake.

The heatmaps in Figure 1-8 illustrate the number of steel moment-resisting frame buildings as related to construction date, number of stories, and occupancy type. Almost 70% of the steel framed structures are office buildings, followed by about 15% that are hotels, with the remainder being residential or mixed use. The predominance of the steel moment-resisting frames of 20 to 39 stories is shown in Figure 1-8a, and the predominance of office buildings constructed during the 1960-80s is evident in Figure 1-8b.

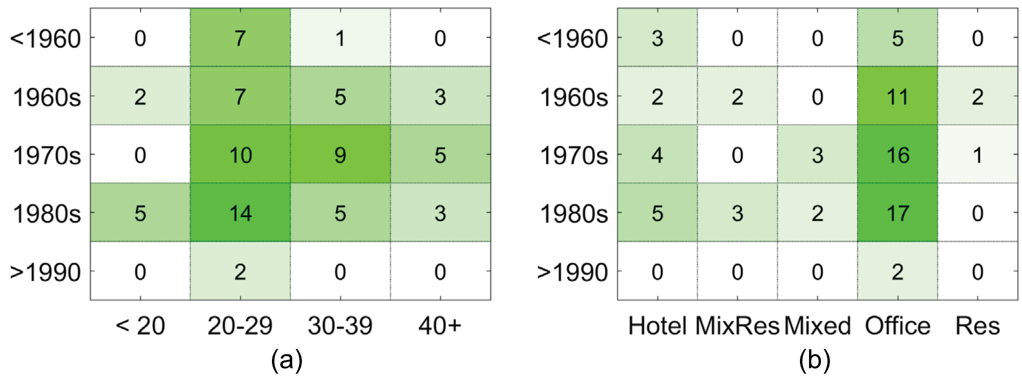


Figure 1-8 Joint distribution of pre-1994 steel moment-resisting frame tall buildings for (a) building date versus number of stories, and (b) date versus occupancy type (values and color intensity denote the number of buildings in each category).

Since the 1906 earthquake, structural engineers have generally regarded steel moment-resisting frame systems as being among the most ductile and reliable SFRS for buildings (FEMA, 2000a). The common view was that when subjected to earthquake shaking, moment-resisting frames would experience only localized damage due to ductile yielding of members and connections. This led to widespread construction of this system, particularly in the high seismic regions of the western United States (FEMA, 2000a). The 1994 Northridge earthquake dramatically changed existing perceptions, when post-earthquake inspections revealed cracking in the beam-to-column joint welds in several dozens of low- and mid-rise buildings.

In 1995, the Los Angeles City Council passed an ordinance mandating connection inspections and repairs in steel moment-resisting frame buildings that may have experienced damage. The ordinance only applied to regions that experienced strong ground shaking in the Northridge earthquake, excluding downtown Los Angeles and other parts of the city (FEMA, 2000b). The LA ordinance was primarily targeted at detecting and repairing damage that had occurred in the Northridge earthquake, as opposed to proactively identifying and addressing buildings that may be at risk to future earthquakes.

Outside of Southern California, the Northridge earthquake damage prompted investigations of some steel moment-resisting frame buildings in San Francisco that had been subject to the 1989 Loma Prieta earthquake (FEMA, 2000b). Although the building code would normally have required repair of any Loma Prieta damage, the City did not retroactively require additional investigation five years later, in part because the Loma Prieta ground motions in downtown San Francisco were relative low (see Part 6 Section 6.2.2, for additional discussion). Even where Loma Prieta damage did not occur or was nominally repaired, the risk of damage in future earthquakes remains. As illustrated in Figure 1-8b, the majority of 'older steel buildings' within the tall building inventory were constructed prior to the Northridge earthquake. Furthermore, typical construction details of these buildings, as observed during the compilation of the database (and previous studies), are similar to the beam-to-column connections that fractured in the Northridge earthquake. There are also concerns about the integrity of welded column splices in these systems.

When first introduced into construction practice, it was customary for steel moment-resisting frame systems to be configured as "space frame systems," whereby all of the beams and columns in the building are engaged to resist lateral loads by providing rigid moment-resisting connections at all beam-to-column intersections. Motivated by improved construction economics and architectural configurations, this practice later evolved to one where the moment-resisting frames are concentrated in a subset of building frame. For example, a common system is to concentrate the seismic-force-resisting frames at the building perimeter of the building. Both the space frame and perimeter frame configurations are observed within this building cohort with roughly a 50-50 split between them.

While steel moment-resisting frames are clearly more vulnerable than originally envisioned, the significance of the risk in these buildings is likely to be highly variable, depending on the specific characteristics of the building. A recent study by Molina Hutt et al. (2018) suggests that the older (pre-1994) steel moment-resisting frames with fracture prone connections could have a mean annual frequency of risk of collapse 25 times higher than new code-conforming buildings and up to 2 times higher average annual risk of economic loss and downtime. A currently ongoing study funded by the National Institute of Standards and Technology (NIST) is further exploring these issues to help inform policy decisions based on the risks posed by existing tall steel buildings, including those associated with cordons around damaged tall buildings.

3.3 Older Concrete Buildings

Damage and collapses observed to concrete buildings during the 1971 San Fernando earthquake and other earthquakes have raised serious concern about the safety of older non-ductile reinforced concrete buildings. In response to the San Fernando earthquake experience, major changes to building codes were instituted in the late 1970s for seismic design of reinforced concrete structures. Prior to these changes, seismic design requirements did not require sufficient steel reinforcement or capacity design provisions to resist story collapse mechanisms and other types of failure. While it is generally recognized that older concrete buildings do not provide the same level of safety and damage control as modern buildings, there are continuing debates as to what policies should be adopted to implement detailed risk assessment and mitigation (e.g., Liel and Deierlein 2013 and http://www.concretecoalition.org).

In Southern California, cities such as Los Angeles, Santa Monica and West Hollywood have recently mandated a retrofit policy for non-ductile reinforced concrete frame buildings. In San Francisco, there are an estimated 3,300 concrete buildings constructed prior to 1980, which may pose high collapse risks (http://www.concretecoalition.org).

As indicated in Figure 1-3b, older concrete buildings account for only about 10% of the buildings in the tall building database, the majority of which were constructed in the 1960s and are in the lower (20-story) height range. Most of these are either concrete shear wall or dual wall-frame systems. Further work is needed to better understand the risk and develop appropriate policies for this tall building cohort in San Francisco, as well as with the vast majority of low and mid-rise older concrete buildings. New approaches identified by FEMA can enable screening assessments to identify the most vulnerable nonductile concrete buildings.

3.4 Modern Concrete Buildings

Many of the recently constructed (post-2000) tall buildings are residential concrete shear wall systems, typically over 40 stories in height. These comprise over 10% of the tall building inventory, a number that is likely to continue to increase to address San Francisco's housing needs. Roughly half of this cohort is composed of shear wall only systems and half are dual shear wall and moment-resisting frame systems. As discussed in Chapter 2 of this Part and noted in Figure 1-3b, many modern (post-2000) tall buildings are engineered following performance-based seismic design guidelines. It is estimated that about half of this cohort is designed using the performance-based approach, which permits use of shear wall only systems. Otherwise, for regular shaped buildings over 240 ft, the prescriptive building code requirements would mandate a dual wall-frame configuration.

This cohort is identified as one of special interest because of the prevalence of these tall buildings for residential occupancies, which pose risks to San Francisco's residents that are quite different from office buildings. In particular, while the modern high-rise residential towers are generally considered to pose acceptably small life safety risks, damage from moderate to large earthquakes may result in extended repair times and building closures that displace residents. The *Guidelines for Performance-based Seismic Design of Tall Buildings* (PEER, 2017) explicitly state that they are "written with the intent that a building properly designed in accordance with these Guidelines should be capable of achieving the seismic performance objectives intended by ASCE 7". In other words, the current design requirements for tall buildings are intended to achieve parity with the performance implied by the current *San Francisco Building Code*. As an example of what might occur, the recent Haywired Scenario (USGS, 2018) included an assessment of a modern 42-story concrete residential tower located in Oakland. Under the magnitude-7.0 Hayward scenario earthquake, the building is estimated to incur damage resulting in median times of 4 months for re-occupancy and 8 months for functional recovery. While the risks do not carry over directly to buildings in San Francisco, the potential clearly exists for large scale displacement of residents that can severely impact the city.

Further study of this cohort will highlight the expected seismic performance of newer tall buildings and cost implications associated with enhanced seismic design performance objectives (see Part 3).

Chapter 4

Summary and Recommendations

In an effort to understand the impact that tall building performance may have on the recovery of the City after a major earthquake, an inventory of the city's tall buildings was developed. The building inventory database is developed and implemented in a Geographical Information Systems (GIS) format to facilitate analysis and visualization of the tall building inventory and identification of locations where building damage is most likely to have disproportionate impacts on the urban community, and the broader socio-economic factors that affect resilience (e.g., services provided to the community by the affected buildings).

The database tabulates building characteristics by location, height, occupancy, age of construction, construction material, structural system, foundation type, façade system and other relevant design information, as available (such as geometry and irregularities). An overview of the inventory highlights the following statistics:

- The inventory includes 156 buildings with heights 240 ft or taller and 22 buildings with heights between 160 ft to 240 ft, either constructed or permitted for construction in San Francisco.
- Buildings constructed in the 1960s, 1970s, and 1980s represent roughly 55% of the tall building inventory, and buildings constructed since 2000 represent about 25% of the inventory.
- The breakdown of structural materials and systems types in the tall building inventory are as follows:
  - 50% - steel moment resisting frame systems
  - 10% - steel dual braced frame-moment frame systems
  - 30% - reinforced concrete shear wall systems
  - About 10% of the older buildings in the inventory did not have sufficient documentation to identify a structural system
- 10% of the inventory is below 20 stories, 70% of the inventory is in the 20 to 40 story range, 20% is above 40 stories.

- Approximately 60% of the buildings house commercial occupancies and just under 40% are residential (multi-family housing) and hotel occupancies.

Based on this review of the tall building inventory and considering the impact of the evolution of seismic design, past earthquake observations, and other considerations, the following tall building cohorts of special interest are identified as worthy of further study:

- **Older Steel Buildings:** Steel-framed buildings constructed in the 1960s through the early 1990s have many known deficiencies, relative to current seismic building codes, including welded connection details similar to those that fractured in buildings subjected to strong shaking during the 1994 Northridge Earthquake. The vulnerability of these buildings warrants further study of the risks they pose to their occupants and neighboring buildings.
- **Older Concrete Buildings:** Concrete buildings constructed prior to 1980 are likely to have non-ductile detailing and other deficiencies that resulted in building collapses during the 1971 San Fernando Earthquake. Although the number of older, tall concrete buildings is small relative to those with other structural systems, the vulnerability of these systems poses a significant risk. Vulnerable older concrete buildings represent a much larger percentage of low- and mid-rise buildings and are well recognized to pose a significant risk. This has prompted mandatory assessment and retrofit requirements in other jurisdictions in California, which should be evaluated for implementation in San Francisco.
- **Modern Concrete Buildings:** Newer concrete shear wall buildings that meet modern (post-2000) building code requirements are expected to have low life-safety risks from falling hazards and collapse. However, modern building codes do not provide minimum requirements for controlling earthquake damage that may require extensive repair with extended downtime. Given the emergence of modern high-rise residential buildings in San Francisco, it is recommended that the implications of adopting seismic performance objectives beyond the code-minimum be evaluated.

The appendix to this report summarizes the metadata fields of the digital GIS database. To help maintain and expand the database, it is recommended to develop a checklist of information that could be collected from building permit applications for new and/or retrofitted buildings, façade inspection reports, and other similar programs managed by the City of San Francisco.

Given the relatively small number of older concrete buildings in the tall building database (6 concrete and 7 mixed concrete-steel older buildings in the 17 to 25 story range), the non-ductile concrete issue is less driven by tall building concerns as compared to the overall high risk posed to the large number (estimated 3,300) of low- to mid-rise older concrete buildings in San Francisco. The modern (post-2000) residential high-rise buildings are generally considered to be safe. The concern with these relates to their potential risk to damage and extended downtime, causing displacement of residents, in a moderate to severe earthquake.

The continued maintenance and expansion of this database is considered to be an essential resource for San Francisco to assess and manage risks to the large building stock that houses residents and businesses of the city.

Appendix A

Tall Building Inventory Data Fields, Definitions, and Notes

This appendix lists all variables included within the tall buildings database. For all basic attributes, such as date, height, number of stories, occupancy, if permit drawings located at the San Francisco Department Building Inspection (DBI) or Building Occupancy Resumption Program (BORP) reports were unavailable or insufficient, the data were taken from the Emporis building information database (https://www.emporis.com/). Sources of data for the various data fields in Table 1-3 are indicated through the footnotes to the field descriptions in the table.

Table 1-3 Tall Building Inventory Data Fields

Field	Field Type	Descriptions
OBJECTID	Numeric	Object ID
Shape	Geometry: Polygon	Footprint of the building at ground level[i]
Name	Text	Name of the building
Address	Text	Street address of the building
MapBlockLot	Text	The block and lot number of the representative parcel for the building[ii]
MBL_Unique	Text	The MapBlockLot (MBL) number, modified with a cardinal direction if multiple buildings share the same MBL
Date	Numeric	Building's year of construction[iii]
Retrofit_Date	Numeric	Year the building was seismically retrofit, if known[iv]
Description	Text	Description of basic building characteristics
Height_ft	Numeric	Height of the building, in feet[v]
Stories_Above_Grade	Numeric	Number of stories above ground level[v]
Stories_Below_Grade	Numeric	Number of stories below ground level[v]
Occupancy	Text	Occupancy type of building, consistent with San Francisco's Land Use categorization[vi]
Structural_Material	Text	Structural material of the building (steel, reinforced concrete, or mixed)[v]
Structural_System	Text	Lateral Force Resisting System (LFRS) of the building[vii]
Structural_Types	Text	Broad categorization of the LFRS (steel moment frame, steel braced frame, reinforced concrete shear wall)
Façade_Material	Text	Façade material, classified by visual identification
Foundation_System	Text	Type of foundation system used[vii]
BORP_Report	Text	Is the building part of BORP?

Table 1-3 Tall Building Inventory Data Fields (continued)

Field	Field Type	Descriptions
Instrumented	Text	Is the building seismically instrumented to record earthquake motion? [viii]
Square_Footage	Numeric	Square footage of the building[ix]
Fire_Resistence_Type	Text	Fire resistance of building materials[ix]
Percent_Sprinklered	Numeric	Percentage of building with sprinklers installed[ix]
Liquefaction_Potential	Text	Classification of the liquefaction potential at the building's centroid[x]
BedrockDepth_MEAN	Numeric	Mean depth to bedrock over the building's footprint[xi]
BedrockDepth_MAX	Numeric	Maximum depth to bedrock over the building's footprint[xi]
BedrockDepth_MIN	Numeric	Minimum depth to bedrock over the building's footprint[xi]
Permit_Date_1	Numeric	Year of any date stamps found on the permit documents
Permit_Date_2	Numeric	Year of any date stamps found on the permit documents
Permit_Date_3	Numeric	Year of any date stamps found on the permit documents
Completion_Date	Numeric	Year of building completion[xii]
Building_Code_Year	Numeric	The date of the applicable design code, if noted in permit documentation
Base_Plan_Size	Text	Dimensions of the plan at ground level[xiii]
Tower_Plan_Size	Text	Dimensions of the plan for the tower[xiii]
Typ_Story_Height	Text	Height of typical story[xiii]
Attyp_Story_Height	Text	Any unique story heights[xiii]
Architectural_Notes	Text	Relevant architectural features, such as setbacks or plan shape[xiii]
Atrium_Location	Text	Story on which an atrium is located[xiii]
MEP_levels	Text	Story on which mechanical, electrical, or plumbing (MEP) equipment is located[xiii]
Site_Class	Text	Site class, if noted in permit documentation
Foundation_Info	Text	Additional description of the foundation[vii]
Collected_on	Text	Date the data was collected
Primary_Source	Text	Primary source the data was collected from (BORP reports, DBI permits, Emporis, or Peer Reviewers)
Shape_Length	Numeric	Footprint geometry info (in feet)
Shape_Area	Numeric	Footprint geometry info (in sqft)

i Extracted from DataSF Building Footprints or hand-drawn for newer buildings
ii Identified based on SF Property Information Map
iii In order of preference: certificate of final completion, latest permit date, or Emporis
iv Discussions with Engineer of Record or design peer reviewers
v Permit application drawings or Emporis (Emporis's height may be measured directly or calculated as ~12.5ft × number of stories)
vi Informed by "main usage" field in Emporis
vii Only from permit applications or discussions with engineers/peer reviewers (not Emporis)
viii Inferred from locations on the Strong Motion Center map (https://www.strongmotioncenter.org/LoaderNC.html). Anecdotal evidence suggests more buildings are instrumented but how/where they are managed is unknown.
ix From SF Fire Department records (February 2018). Not all buildings have a match.
x From USGS maps (https://pubs.usgs.gov/of/2000/of00-444/)
xi Based on 1961 USGS bedrock map and current day topographic data (Part 1, Appendix C)
xii Certificate of Final Completion (CFC) or Emporis (only if no permit dates are available)
xiii Only from permit applications

Appendix B

Building Permit Supplement for Inventory Maintenance

In order to maintain and expand the tall building inventory, it is recommended that the form shown in Figure 1-9 is used as a starting point for the development of a supplemental data collection form required in the process of permit applications. The fields in Figure 1-9 are based directly on the main attributes of the digital database prepared in this study and are identified in Figure 1-10. Further development of the supplemental form should consider coordination with other data sets and feasibility of implementation and maintenance.

In addition to the list of information items, it is recommended to request from the building owner/design team, an electronic submission of a .shp, or .kml file that contains the building footprint. This information can be incorporated in the database to facilitate visualization of the building location and geometry.

City and County of San Francisco
Department of Building Inspection

London Breed, Mayor
Tom C. Hui, S.E., C.B.O., Director

PERMIT SUPPLEMENT – FOR MAINTENANCE OF BUILDING INVENTORY

TO BE SUBMITTED ALONG WITH A DIGITAL BUILDING FOOTPRINT

Building Address	Name of Building	Block and Lot #	Completion Date

Brief Description of the Building	Building Design Code / Year
Total Height Above Grade	Total Square Footage
Number of Stories Above Grade	Number of Stories Below Grade
Primary Structural Material ☐ Steel ☐ Reinforced Concrete ☐ Mixed	Detailed Structural System (Appendix A.1) Category #: , or Other:
Occupancy Type (Appendix A.2) Category #: , or Other:	Façade Material
Foundation System (Appendix A.3) Category #: , or Other:	Additional Foundation Notes
Design Soil Type / Vs30 Depth to Bedrock (average, max, min) Liquefaction Potential (per USGS map)	Architectural Notes (e.g. typical story height, extent/location of atrium, setbacks, atypical story heights, etc.)
Is Seismic Instrumentation Installed? ☐ Yes ☐ No	Structural Engineering Firm Geotechnical Engineering Firm

Figure 1-9 Proposed permit supplement form (page 1).

Categories for Structural Systems

Coarse Structural System Type Categories		Detailed Structural System Categories	Category Number
Coarse Structural System Type Categories	Steel Moment Frame	Steel Moment Frame - Perimeter	1
		Steel Moment Frame - Space	2
	Steel Braced Frame	Steel Moment Frame and Braced Frame – Buckling Restrained Brace	3
		Steel Moment Frame and Braced Frame - Eccentric	4
		Steel Moment Frame and Braced Frame - Concentric	5
		Steel Moment Frame and Brace Frame - Unspecified	6
	RC Shear Wall	RC Shear Wall	7
		RC Shear Wall and RC Moment Frame	8
		RC Shear Wall and Steel Moment Frame	9
		RC Shear Wall + Steel Braced Frame – BRB (or Mega Braces)	10
			11

Categories for Occupancy

Occupancy Categories	Category Number
Office (Management, Information, Professional Services)	1
Hotels, Visitor Services	2
Mixed Uses (With Residential)	3
Mixed Uses (Without Residential)	4
Residential	5
Medical	6
Retail, Entertainment	7
Cultural, Institutional, Educational	8

Categories for Foundations

Foundation Categories	Category Number
Pile	1
Mat + Pile	2
Drilled Shafts	3
Mat	4
Footings	5

Figure 1-10 Proposed permit supplement form (page 2).

Appendix C

Reference for Bedrock Depth Data

The tall building inventory includes three fields related to the depth of the bedrock beneath the buildings: the average, maximum, and minimum depth across the footprint. These values were calculated based on two sources: one for the elevation at ground level and one for the elevation at bedrock.

The elevation at ground level is provided as a raster dataset (a single elevation value for every square in a grid across the location of interest, similar to pixels in an image) by the U.S. Geological Survey (USGS). The dataset was published in 2000 and can be downloaded at https://pubs.usgs.gov/of/2000/of00-444/.

The elevation at bedrock also comes from the USGS in the form of a contour map from 1961 (Figure 1-11). The contours were traced and spatially interpolated to obtain a smooth surface for comparison with the ground level data. Using the footprint location of the buildings as a template, the bedrock elevation was then subtracted from the ground level elevation for the average, maximum, and minimum depth to bedrock for each building.

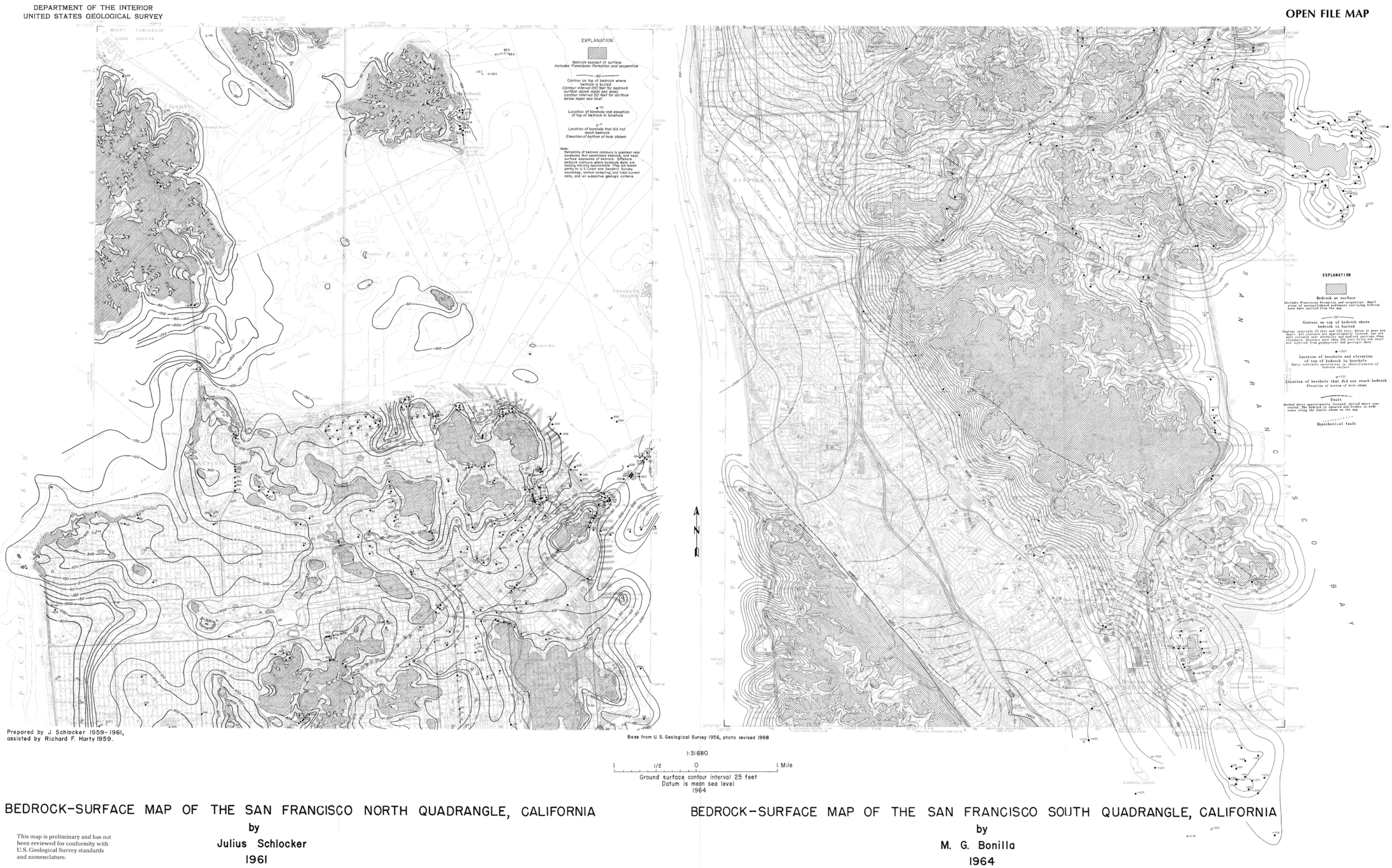


Figure 1-11 USGS map (1961) used for determining the depth to bedrock.

Appendix D

References

Almufti, I., Molina Hutt, C., Mieler, M.W., Paul, N.A., and Fusco, C.R., 2018, *The HayWired Earthquake Scenario - Engineering Implications: Case Studies of Tall-Building Structural Analyses, Downtime and Loss Assessment for the HayWired Scenario Mainshock*, United States Geological Survey, Menlo Park, California.

ASCE, 2017, *Minimum Design Loads and Associated Criteria for Buildings and Other Structures*, ASCE/SEI 7-16, American Society of Civil Engineers Structural Engineering Institute, Reston, Virginia.

CBSC, 2016, *2016 California Building Code*, California Code of Regulations, Title 24, California Building Standards Commission, Sacramento, California.

CCSF, 1995, *Downtown Area Plan*, Planning Department, the City and County of San Francisco, California.

CCSF, 2008, *Requirements and Guidelines for the Seismic Design of Tall Buildings Using Non-Prescriptive Seismic-Design Procedures*, Administrative Bulletin 083, Department of Building Inspection, the City and County of San Francisco, California.

CCSF, 2011, *CAPSS Earthquake Safety Implementation Program Workplan 2012-2042*, the City and County of San Francisco, California.

CCSF, 2014, *Procedures for Seismic Instrumentation of New Buildings,* Administrative Bulletin 058*, Department of Building Inspection*, the City and County of San Francisco, California.

CCSF, 2016, *The San Francisco Building Code*, the City and County of San Francisco, California.

CCSF, 2017a, *San Francisco Property Information Map, Planning Department*, the City and County of San Francisco, California. Available at: http://propertymap.sfplanning.org/, last accessed: August 2018.

CCSF, 2017b, *Building Façade Inspection and Maintenance*, Administrative Bulletin 110, Department of Building Inspection, the City and County of San Francisco, California.

Cornell A., 1968, "Engineering seismic risk analysis," *Bulletin of the Seismological Society of America*, Vol. 58, No. 5, pp. 1583-1606.

FEMA, 2000a, *State of the Art Report on Systems Performance of Steel Moment Frames Subject to Earthquake Ground Shaking*, FEMA 355-C, prepared by the Structural Engineers Association of California, Applied Technology Council, and Consortium of Universities for Research in Earthquake Engineering (SAC) Joint Venture for the Federal Emergency Management Agency, Washington, D.C.

FEMA, 2000b, *State of the Art Report on Past Performance of Steel Moment-Frame Buildings in Earthquakes*, FEMA 355-E, prepared by the Structural Engineers Association of California, Applied Technology Council, and Consortium of Universities for Research in Earthquake Engineering (SAC) Joint Venture for the Federal Emergency Management Agency, Washington, D.C.

FEMA, 2006, *Next-generation Performance-based Seismic Design Guidelines*, FEMA 445, prepared by the Applied Technology Council for the Federal Emergency Management Agency, Washington, D.C.

LATBSDC, 2017, *An Alternative Procedure for Seismic Analysis and Design of Tall Buildings Located in the Los Angeles Region*, Los Angeles Tall Buildings Structural Design Council, Los Angeles, California.

Liel, A., and Deierlein, G.G., 2013, "Cost-benefit evaluation of seismic mitigation alternatives for older reinforced concrete frame buildings," *Earthquake Spectra*, Vol. 29, No. 4, pp. 1391-1411.

Molina Hutt, C., Rossetto, T., and Deierlein, G.G., 2018, "Comparative risk-based seismic performance assessment of 1970s vs modern tall steel moment resisting frames," manuscript submitted for publication.

PEER, 2010, *Guidelines for Performance-based Seismic Design of Tall Buildings*, PEER Report 2010/05, Pacific Earthquake Engineering Research Center, University of California, Berkeley, California.

PEER, 2017, *Guidelines for Performance-based Seismic Design of Tall Buildings*, PEER Report 2017/06, Pacific Earthquake Engineering Research Center, University of California, Berkeley, California.

SEAOC, 1959, *Recommended Lateral Force Requirements and Commentary*, Seismology Committee, Structural Engineers Association of California, Sacramento, California.

USGS, 2018, *The HayWired Earthquake Scenario - Engineering Implications*, USGS Scientific Investigations Report 2017-5013-I-Q, US Geological Survey, Menlo Park, California.

PART 2:
Geotechnical Engineering for Tall Buildings

Chapter 1

Introduction

1.1 Background

In San Francisco, geotechnical engineering and foundation design varies considerably among practitioners because best practices are not yet fully codified. This Part summarizes the state-of-the art and state-of practice for geotechnical engineering for tall buildings and corresponds to Recommendation 1A presented in Summary Recommendations.

The purpose of this Part is to put forward recommendations that can be established as guidelines to bring consistency and uniformity to geotechnical practice for foundation design. Accordingly, this Part presents the following information for each type of foundation considered for support of a tall building:

- What are the key technical issues?
- How should key technical issues be addressed?
- What are the reference documents?
- What is the state-of-practice and what is the state-of-the-art?
- Considering the best practices at the national and international levels, what are the shortcomings of the state-of-practice in our region and how can it be advanced to the state-of-the-art?

1.2 Intended Audience and Use of this Report

This Part summarizes the state-of-the art and state-of practice for geotechnical engineering for tall buildings based on experience in San Francisco and cities with tall buildings built on similar soils (Chicago and Boston), as well as guidance from New Zealand. The material was developed for review by persons with advanced geotechnical engineering knowledge and understanding. References are provided within each section to allow for further review of technical details.

Administrative Bulletin (AB) 083, *Requirements and Guidelines for the Seismic Design of New Tall Buildings using Non-Prescriptive Seismic Design Procedures*, (CCSF, 2014) sets requirements for structural design of tall buildings, and it is recommended that the recommendations of this report be implemented in a similar AB for geotechnical design following a thorough review of relevant sections by local practitioners.

In addition, to strengthen the Department of Building Inspection's (DBI) procedures for assessing the completeness of the foundation and excavation design for tall buildings, two additional actions are recommended:

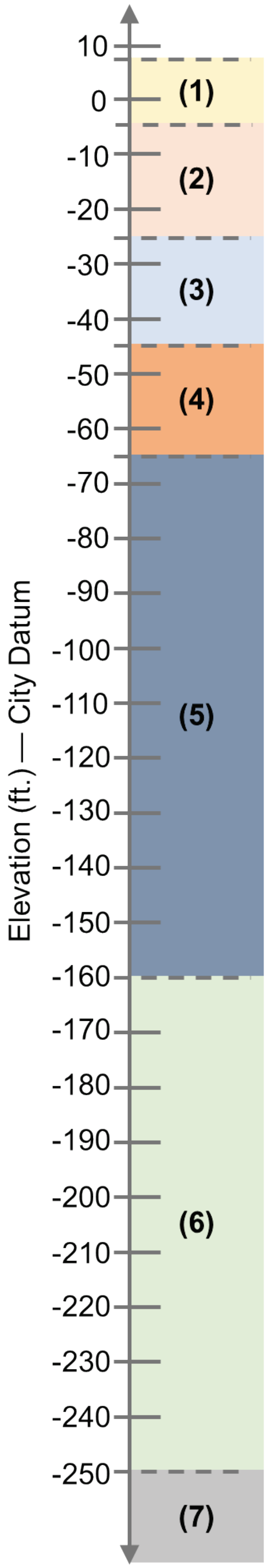
- Increase DBI's expertise on geotechnical issues related to tall buildings through enhanced training and staffing.
- Develop a geotechnical report checklist to help ensure that submitted geotechnical investigation, design, and field monitoring reports are complete.

Geotechnical design of tall buildings was not covered in the ESIP Workplan (CCSF, 2011).

1.3 San Francisco Conditions

Figure 2-1 (developed as part of the inventory described in Part 1) illustrates the types of foundations for tall buildings in downtown San Francisco, superimposed on a map that identifies regions that are susceptible to soil liquefaction during strong earthquakes. Figure 2-2 shows a typical soil profile illustrating the varying soil layers.

Figure 2-1 Map showing foundation types of tall buildings superimposed where beige shading identifies regions that are susceptible to soil liquefaction during strong earthquakes (soil information from https://datasf.org/).



Descriptions of Layers

(1) FILL –
Sand and Silty Sand, loose to medium dense (liquefiable below water table)

(2) DUNE SAND –
Sand, Sand with Silt (liquefiable below water table)

(3) YOUNG BAY MUD –
Soft to medium stiff w/loose sand lenses

(4) COLMA SAND –
Silty and clayey sand, dense to very dense

(5) OLD BAY CLAY –
Stiff to very stiff clay, OCR of 2 to 4

(6) ALLUVIUM / COLLUVIUM –
Interbedded layers of clay, clayey sand, gravelly clay, sand and silty sand. Very stiff to hard clayey soils and very dense sandy soils

(7) FRANCISCAN COMPLEX BEDROCK –
Matrix of Shale, Siltstone and Sandstone with shear wave velocity >2,500 ft/s

Figure 2-2 Typical soil profile for a downtown San Francisco site.

Foundation types of tall buildings vary due to the following factors:

- Depth to rock: This may range from 0 ft to more than 250 ft. Figure 2-2 shows a typical soil profile from a downtown site with the Franciscan bedrock formation located at 250 ft depth.
- Soil type and stiffness: The stiffness of the soil may vary depending on the site's soil profile, as illustrated in Figure 2-2.
- Building height and weight
- Number of basement levels
- Slope of site
- Proximity to shoreline
- Adjacent and underlying structures
- Ground water level
- Available construction technologies, logistics, and economics

1.4 Organization

Chapter 2 presents a discussion on acceptable limits of total and differential settlement for tall buildings.

Chapters 3 and 4 present discussions on geotechnical engineering for new tall buildings on deep foundations and shallow foundations, respectively.

Chapter 5 presents a discussion on evaluation and retrofit of foundations of existing tall buildings.

Chapter 6 presents a discussion on foundation design of tall buildings near shoreline.

Chapter 7 presents a discussion on shoring and dewatering considerations.

Chapter 8 presents a discussion on sea level rise and its effects on foundation design.

Chapter 9 presents summary recommendations for evaluating the adequacy of geotechnical reports.

Appendix A presents recommendations regarding settlement considerations based on case histories.

References and full citations are incorporated into each chapter for ease of use.

Chapter 2

Total and Differential Settlement

The current *California Building Code* does not provide criteria for maximum allowable limits for total and differential settlement of foundations supporting commercial or residential tall buildings. However, many practitioners and officials with building departments of various cities consider acceptable a total settlement of less than 4 inches and a differential settlement limited to an angular distortion of 1/500 for composite steel or concrete core buildings and 1/750 for reinforced concrete towers (rigid structures) acceptable. The structural system, cladding system, partitions, and other nonstructural components may also control the acceptable differential settlement.

It is noted that some buildings, such as base-isolated structures, are more sensitive to differential settlement than other conventional buildings and would require a tighter limit on differential settlement. However, base-isolation systems are seldom used in tall buildings.

Building settlement should be monitored during construction and to at least 10 years after construction. Monitoring and reporting requirements should follow guidelines presented on Information Sheet S-18, *Interim Guidelines and Procedures for Structural, Geotechnical, and Seismic Hazard Engineering Design Review for New Tall Buildings*, issued by the San Francisco Department of Building Inspection (SFDBI) on December 2017.

Chapter 3

New Tall Buildings on Deep Foundations

When ground conditions at a site are not suitable for shallow spread or raft foundation systems, especially for high-rise buildings where the vertical and lateral loads imposed on the foundation are significant, it is necessary to support the building on deep foundation or piles. Drilled shaft, driven pile, or auger cast pile foundations can be used either as a single unit or in groups and are generally located beneath columns and load bearing walls.

Deep foundations provide support through skin friction, end bearing, or combination of skin friction, and end bearing. In the latter case, displacement compatibility between mobilized ultimate capacity in friction and end bearing should carefully be evaluated.

Due to the presence of liquefiable fill and soft Bay mud, the majority of the existing tall buildings in downtown San Francisco are supported either on drilled shafts extending to Franciscan bedrock formation or on driven 12-inch or 14-inch precast, prestressed concrete piles gaining support through end bearing within the dense Colma sand layer below Young Bay mud.

3.1 Deep Foundation Types

Most common types of deep foundation used in the San Francisco Bay Area for support of tall buildings are drilled shafts, driven concrete or steel piles, and auger cast piles. An alternative deep foundation used frequently in Europe is Barrettes, or known on the east coast of the United States as LBEs (Load Bearing Elements, which is an isolated slurry vertical diaphragm with rectangular elements 3 feet to 4 feet wide by 9 feet to 10 feet long. The application of this foundation in the western United States has been very limited with only one tall building in San Francisco supported on Barrettes. However, on the east coast, they have been used on many tall buildings since the late 1980s.

3.1.1 Drilled Shafts

Diameter of drilled shafts or caissons commonly ranges from 30 inches to 72 inches; however, larger diameter drilled shafts have also been successfully constructed and tested in the San Francisco Bay Area (e.g., 300 feet deep drilled shafts with a diameter as large as 10 feet were constructed and successfully tested for support of the high viaduct at Presidio Parkway in San Francisco in early 2012).

Depending on the local soil conditions, drilling fluid (slurry) or steel casing is used to stabilize the drilled hole before placement of rebar cage and tremie placed structural concrete. High capacity drilled shafts (axial capacity in the range of 20,000 kips) with rock socket into Franciscan bedrock formation have been successfully installed and tested in the recent past.

Due to the high groundwater table in the San Francisco Bay Area, drilled shafts are commonly constructed using the slurry method, and concrete is placed using the tremie method for concrete mixes having a slump of 7 to 9 inches. Tremie pipes should be at least 10 inches in diameter for pumped and gravity-fed methods, respectively. If used for stability of the drilled hole, to avoid contamination of concrete through caving soils, the temporary casing is pulled out with concrete surface being several feet above the bottom of casing at all times.

It is not practical to perform pile load test on high capacity (large diameter and very long) drilled shafts using conventional pile load test conducted from ground surface. In these cases, pile load tests are performed by placing and activating hydraulic jacks placed near the bottom of drilled shafts. Osterberg (O-Cell) or equivalent testing devices have successfully been used to measure ultimate skin friction and end bearing of high capacity drilled shafts in soil and rock.

Ultimate skin friction capacity obtained through O-Cell testing in bedrock of Franciscan bedrock formation ranges from 4 kips per square feet (ksf) to 20 ksf. Ultimate end bearing in bedrock of Franciscan bedrock formation greatly varies, depending on the contractor’s procedure and the effort in cleaning of the bottom of the shaft prior to placement of concrete. It varies from almost zero (a recent test at a project site in downtown San Francisco has encountered no end bearing resistance for vertical shaft movement at the tip of up to 2 inches) to ultimate end bearing of 100 ksf and greater.

There are various methods for cleaning the bottom of drilled shafts. These include the use of cleanout bucket or air lifting. Inspection of high capacity drilled shaft is performed by various methods including measurement of drilled shaft height immediately before placement of concrete using a weighted tape measure (least reliable method), a Mini-SID (Shaft Inspection Device), or DID (Ding Inspection Device). For smaller diameter drilled shafts

(3 feet or less) where the bottom of the shaft could not be inspected / tested, end bearing capacity is normally ignored. Shaft verticality is checked with automated monitoring equipment on the drill rig by using Kelly bar or cutting auger, downhole sonic caliper, or stringed plumb bobs from a centered hole frame.

For drilled shafts socketed in bedrock, the minimum rock socket length should generally be 15 feet or twice the shaft diameter, whichever is greater. The minimum rock socket length requirement could be relaxed for sites where the depth to bedrock is greater than 25 times the shaft diameter and hard rock with substantial lateral extension and depth (i.e., the hard rock boulder is the size of a house and not a compact car) is encountered below the bottom of drilled shafts.

Typically, for sound rock, 50% of axial capacity of rock socket is due to skin friction. For softer rocks, the contribution of skin friction increases to 75%.

Until concrete has gained sufficient strength (i.e., set for 48 to 72 hours), newly installed drilled shafts should be protected against construction related vibration (e.g., pile driving) or installation of other drilled shafts within six shaft diameters or 25 feet, whichever is less. The above criteria should be revised if admixtures (retarders) are used.

The most relevant references are the following:

- FHWA, 2010, *Drilled Shafts: Construction Procedures and LRFD Design Methods*, FHWA Publication No. FHWA-NHI-10-016, U.S. Department of Transportation Federal Highway Administration.
- For installation guidelines: ACI, 2014, *Report on Design and Construction of Drilled Piers*, report by by ACI Committee 336, American Concrete Institute, Farmington Hills, Michigan.

3.1.2 Driven Concrete or Steel Piles

When precast, prestressed concrete piles are used, the typical allowable axial capacity of 12-inch-square and 14-inch-square piles driven to refusal within Colma sand layer are 100 and 150 tonnes, respectively.

For sites where either Colma sand does not exist, the layer is not thick enough to prevent punching through while pile driving, or the Franciscan bedrock formation is within 100 to 200 feet below ground surface, steel H piles with driving shoes are used. The spatial variation of rock quality and variation in bedrock surface elevation makes steel H piles an attractive alternative as the length of pile could be increased through field welding or adjusted by cutting extra length of pile if refusal occurs before the pile is fully driven to the design cut-off evaluation. If steel pile splices are required, they should be full penetration welds with no flange or web plates allowed in seismic regions.

The main disadvantages of driven concrete piles are human factors, such as objection to noise and vibration that are problematic in congested and well-developed areas of the City. In addition, due to presence of debris in the surficial fill layer, predrilling of pile locations would normally be required. The main disadvantage of steel pipe or H piles is the potential for corrosion.

The axial capacity of steel piles driven to refusal within Franciscan bedrock formation is controlled by structural capacity of the pile; however, the ultimate capacity may be controlled by buckling failure mode if soft Young Bay mud or liquefiable soils are present.

There are three alternatives for addressing corrosion issues related to steel piles. These include the following:

- Adding a sacrificial thickness to the pile section (normally 1/8-inch or 1/16-inch, depending on soil corrosivity). This option is the least reliable as reduction in thickness of pile may not occur uniformly along the pile as assumed.
- Applying coating material (factory applied epoxy coating) to the top portion of piles to a depth where there is no potential for oxygen to reach the pile (this depth is normally taken at 10 feet below the surface of Young Bay mud). This option is more reliable than adding sacrificial thickness to pile, but the coating may be damaged during pile driving and if a section of coating is removed during pile driving resulting in scratches on the pile, very weak spots vulnerable to corrosion are created.
- Applying a Cathodic protection system. This is the most reliable system to address corrosion issues related to steel piles, but is also a costly option and that requires long term maintenance.

The standard of practice is summarized in the following reference:

- FHWA, 2016, *Design and Construction of Driven Pile Foundations*, FHWA Publication No. FHWA-NHI-16-009, U.S. Department of Transportation Federal Highway Administration.

3.1.3 Auger Cast Piles

Due mainly to their cost effectiveness, auger cast piles are gaining popularity in recent decades. Typical diameter of auger cast piles range from 18 inches to 48 inches. Auger cast piles have successfully been installed to a depth of 200 feet.

The most suitable soil classification for installation of auger cast piles is medium stiff to stiff clays and medium dense to dense sands. The most challenging soil classification is loose sand (especially below ground water table) and soft clays, as caving of soil could result in necking in the pile section.

In addition, installation of long and small diameter auger cast piles in soft clays requires checking for buckling mode of failure under static and seismic loading conditions.

Until concrete has gained sufficient strength, newly installed auger cast piles should be protected against construction related vibration (e.g., pile driving) or installation of other auger cast piles within six shaft diameters (see discussion on this topic presented in Section 3.1 under heading of "Drilled Shafts").

The most relevant reference for design of auger cast piles is given in the following reference:

- FHWA, 2007, *Design and Construction of Continuous Flight Auger Piles*, FHWA Publication No. FHWA-HIF-07-03, U.S. Department of Transportation Federal Highway Administration.

3.1.4 Other Deep Foundation Types

Specialty constructed piles include torque-down piles, Tubex piles, Franki piles, micropiles, and Fundex piles. Due to the limited axial capacity provided by these piles (maximum allowable axial capacity is normally limited to about 600 kips or slightly higher with structural design calculations and load testing), their use is not common for supporting new tall buildings, which have a high demand on axial pile capacity.

These specialty piles are designed and constructed by specialty design-build (DB) contractors using proprietary tools and techniques. In addition to developing foundation performance criteria and reviewing plans and specification for foundation construction and testing, the geotechnical engineer of record (GEOR) should ask for and review the design submittal by the DB contractor, as the GEOR is ultimately responsible for geotechnical aspects of foundation design.

3.2 Piles Going Through Soft or Liquefiable Soils and Firm Soil Interface

Lateral reinforcing of concrete piles normally extends down below the point of fixity (about 12 pile diameters below the pile top). However, in some cases, lateral reinforcement should continue well below the interface between soft or liquefiable soils and the underlying firm, competent soils. This is because large shear strains are developed at this interface as seismic waves propagate from competent soils at depth to the ground surface.

For example, if subsurface conditions at a site consist of 35 feet of liquefiable sand and soft Young Bay mud underlain by alluvium consisting of dense sands or stiff clays, lateral reinforcement of a 24-inch drilled shaft should extend down 40 feet below the top of the shaft as opposed to stopping below point of fixity at a distance of about 25 feet below the top of the shaft.

3.3 Downdrag Loads on Piles

Often, sites are raised by placing new fill over existing ground surface. The added load would cause long term settlement within saturated clayey soils, imposing downdrag loads on piles. Downdrag loads applied to a pile group depends on pile geometry and layout, undrained shear strength of soil, effective vertical stress, and soil plasticity. General information regarding downdrag loads on piles are presented in the following publications:

- Kuwabara, F., Poulos, H.G., 1989, "Downdrag forces in group of piles," *Journal of Geotechnical Engineering*, Volume 115, No. 6.
- Lee, C.J., and Ng, C.W., 2004, "Development of downdrag on piles and pile groups in consolidating soil," *Journal of Geotechnical and Geoenvironmental Engineering*, Volume 130, No. 9.

Useful information regarding case histories and methods for evaluation and mitigation of downdrag loads are presented in the following reference:

- Fellenius, B.H., 1998, "Recent advances in the design of piles for axial loads, dragloads, downdrag, and settlement," *ASCE and Port of NY&NJ Seminar*, April 22-23, 1998.

In case of San Francisco Young Bay mud, downdrag loads are computed by multiplying vertical effective stresses by a factor between 0.25 to 0.35 for a single pile and by a factor between 0.1 to 0.2 for a typical pile group (3 by 3, 4 by 4, or more). The corresponding range of values for sandy fill are 0.3 to 0.55 for a single pile and 0.20 to 0.35 for a typical pile group (3 by 3, 4 by 4, or more). The smaller values are used for drilled shafts and auger cast piles and the larger values are used for driven piles.

To arrive at allowable axial pile capacity, downdrag load is subtracted from ultimate axial pile capacity before applying a factor of safety (as opposed to dividing the ultimate axial capacity by a factor of safety and then subtracting the downdrag load).

3.4 Integrity Testing of Deep Foundations

Integrity testing of high capacity drilled shafts are performed through cross-hole sonic, gamma-gamma, or thermal tests. Smaller diameter drilled shafts could be tested through conventional static or dynamic pile load test conducted from the ground surface.

To check the potential for necking of auger cast piles, field observation of installation should be performed using a fully automated data acquisition system. In addition, pile integrity tests should be performed for 5 to 10 percent of production piles and for piles with questionable recorded installation data. Pile integrity tests could be performed through static or dynamic pile load tests, cross-hole sonic, gamma-gamma, or thermal tests. Cross-hole sonic and gamma-gamma tests cannot be performed for piles with a diameter less than 24 inches (i.e., for 18-inch diameter auger cast piles, only thermal test could be performed). It is noted that performing cross-hole sonic, gamma-gamma, and thermal tests would require attachment of the test instrument to the rebar cage before the cage is inserted into the fully grouted drilled hole.

Other dynamic tests for confirmation of compression load capacity of production or suspect deep foundations should be performed in accordance with ASTM D4945[1] using proprietary software Statnamic[2] or GRL Apple[3]. The test could be performed in a stepped manner and up to twice the static force acting on the deep foundation element. The CAPWAP[4] analysis is often performed using the GRL Apple system that involves a falling weight striking the head of piles or shafts. A concrete filled steel casing is placed above the top of the pile to prevent damage caused by impact loading.

3.5 Allowable Tolerance for Installation of Deep Foundations

Deep foundations should meet plumbness criteria. Typical specifications require a tolerance of less than 1 inch for every 10 feet of depth (1 in 75 to 1 in 100 ratios have also been specified in other practice areas). In addition,

---

1 ASTM, 2018, *Standard Test Method for Shrinkage Factors of Cohesive Soils by the Water Submersion Method*, ASTM International, West Conshohocken, Pennsylvania.
2 Berminghammer, Canada.
3 Load testing system, GRL Engineers, Inc., Cleveland, Ohio.
4 Case Pile Wave Analysis Program, Pile Dynamics, Inc., Cleveland, Ohio

the center of deep foundation is normally specified to be within an inch from center of a column and 3 inches from the design location.

3.6 Pile Load Test Procedure and Interpretation of Results

Axial pile compression load tests are performed in accordance with the following ASTM standards:

- ASTM, 2013a, *Standard Test Methods for Deep Foundations Under Static Axial Compressive Load*, ASTM International, West Conshohocken, Pennsylvania.
- ASTM, 2017, *Standard Test Method for High-Strain Dynamic Testing of Deep Foundations*, ASTM International, West Conshohocken, Pennsylvania.

Lateral pile load tests are performed less frequently in practice than axial pile load test for design of new foundations. Instead, design engineers rely on interpretation of lateral pile load test results and development of p-y springs by others as presented in the literature. The following ASTM standard corresponds to lateral pile load tests of deep foundations:

- ASTM, 2013b, *Standard Test Methods for Deep Foundations Under Lateral Load*, ASTM International, West Conshohocken, Pennsylvania.

In the San Francisco Bay Area, where liquefiable fill and Young Bay mud are present, the contribution of the liquefiable sand layer and Young Bay mud to axial pile capacity should be carefully evaluated and subtracted from ultimate pile capacity obtained through conventional pile load tests conducted from ground surface. In addition, down drag loads should be subtracted from ultimate axial pile capacity (see discussion related to downdrag loads in Section 3.3).

Comparison on the same piles show that the ultimate axial pile capacity obtained from a dynamic load test in clayey soils is normally 1.3 times that obtained from a static load test. This means that strain rate effects increase the measured axial capacity of piles by 30 percent in clayey soils present in the San Francisco Bay Area. However, the ultimate axial capacity of piles in cohesionless soils are not affected by strain rate effects. Similar results are obtained from static and dynamic test conducted in sandy soils.

3.7 Soil Liquefaction-Related Design Issues

Presence of liquefiable soils within saturated loose to medium dense sand within surficial fill layer or sand lenses within Young Bay mud would impact design of deep foundations. The standard of practice for evaluation of soil liquefaction potential is based on the following reference:

- Idriss, I.M, and Boulanger, R.W., 2008, *Soil Liquefaction During Earthquakes*, EERI Monograph, Earthquake Engineering Research Institute, Oakland, California.

The procedures and recommended methods for evaluation of soil liquefaction potential presented in the above references are very similar for loose to medium dense sands but are noticeably different for denser sands (with $N_{1(60)}$ between 20 and 30, indicating SPT blow count normalized to 1 tsf (tonne/sqft) confining pressure with 60% free fall energy of a 140 pound hammer dropping 20 and 30 inches) at high cyclic stress ratios. Most practitioners consider the difference between the methods as "epistemic uncertainty," and both methods are used to provide a range of solutions.

Most practitioners would agree that the effects of soil liquefaction (defined as pore water pressure ratio of 100 percent) does not need to be considered for sand with $N_{1(60)}$ greater than 25. This is because sands at this density are dilative and therefore, soil has limited potential for developing large shear strains or undergo appreciable loss of shear strength. Accordingly, the effects of soil liquefaction could be ignored for sands with $N_{1(60)}$ of 25 or greater.

Issues related to soil liquefaction are discussed below:

3.7.1 Temporary Loss of Axial Support

Soil liquefaction would result in temporary loss of axial support within the liquefied soil layer. Therefore, no axial support in friction is assigned to this layer.

3.7.2 Reduction of Lateral Support

The lateral support of piles is reduced as a result of soil liquefaction. The standard of practice is to develop *p-y* springs (from L-PILE[5] analysis) prior to liquefaction (static condition) and reduce the static p-y curve by a factor between 5 and 10, i.e., post liquefaction p-y being 10 % to 20% of static values.

Other practitioners would assign an undrained shear strength to liquefied soil based on $N_{1(60)}$ and use post liquefaction undrained shear when developing p-y springs using soft clay criteria.

[5] ENSOFT, Inc., Austin, Texas.

3.7.3 Downdrag Loads

Post-earthquake settlement of liquefied soil can cause downdrag loads being applied to deep foundation (piles and shafts) through negative skin friction within soil layers above the liquefied soil layer. Downdrag loads are applied slowly (compared to duration of an earthquake); therefore, they are combined with static gravity and sustained live loads. Therefore, downdrag loads are not combined with the loads caused by the inertial response of the building.

Evaluation of magnitude of downdrag loads are discussed in Section 3.3.

3.7.4 Effect on Ground Motion

Soil liquefaction has a significant effect on ground motion intensity and characteristics, especially for cases where liquefiable soil layer is at depth with relatively high pre-earthquake confinement pressure and corresponding shear strength (as compared with liquefiable layer being at a shallow depth).

According to the *California Building Code*, sites with liquefiable sand layers or deep soft clays (Young Bay mud in San Francisco Bay Area) fall into Site Category F, requiring site response analysis, using such computer programs as SHAKE[6], DEEPSOIL[7], or FLAC[8], except where soft clays and liquefiable soil layers are completely removed as part of basement construction (i.e., basement mat is supported directly on competent soil / rock).

In the first step of performing site response analysis, ground motion is defined at the surface of competent soil or rock at depth. Then, a one-dimensional soil profile is constructed based on conventional geotechnical site investigation and laboratory test results and site-specific shear wave velocity measurement. Soil degradation curves that show the variation of shear modulus and damping with shear strain amplitude are based on either: (1) site-specific resonant column, cyclic triaxial, cyclic direct simple shear tests, or a combination of these tests (seldom performed for design of foundation of tall buildings); or (2) results of tests that have been performed on similar soil units in the past (in San Francisco Bay Area, a number of high quality tests were performed as part of design and construction of the new east span of Bay Bridge and expansion of runways at SFO). Finally, site response analysis is performed in frequency domain (SHAKE) or in time domain (DEEPSOIL or FLAC).

---

[6] Lysmer, J, Schnabel, P.B., and Seed, H.B.
[7] University of Illinois at Urbana-Champaign and Yousef Hashash
[8] Itasca Consulting Group, Inc., Minneapolis, Minnesota

According to California Building Code, the ground motion computed at the ground surface through site response analysis cannot fall below 80 percent of Site Class E.

3.7.5 Liquefaction Related Ground Settlement

Liquefied soils undergo volumetric strain resulting in ground settlement. The magnitude of settlement would depend on soil density before the earthquake, earthquake magnitude, and induced cyclic stress ratio. Magnitude of settlement could be obtained from the following reference:

- Tokimatsu, K., and Seed, H.B., 1987, "Evaluation of settlement in sands due to earthquake shaking," *Journal of Geotechnical and Geoenvironmental Engineering*.

Although the above reference is three-decades old, it correctly and closely predicted the magnitude of ground settlement observed in San Francisco as a result of 1989 Loma Prieta earthquake.

Flexible connections should be used at the interface of utility lines and pile supported structures to safely accommodate differential settlement caused by liquefaction related ground settlement.

3.8 Floating Deep Foundations

In some cases, bedrock is very deep (>300 feet) and it would not be economical or practical to extend deep foundations to gain support within bedrock at depth. In these cases, if required, floating deep foundations are used to reduce the total and differential settlement to acceptable limits discussed in Chapter 2 of this Part.

Floating foundations normally consist of pile-supported mat or pile-supported grade beam systems. Unlike mat or grade beams supported on end bearing piles, in case of floating foundations, the mat or grade beams will always remain in contact with the soil, and therefore, both the piles and the soil below the mat or grade beams will contribute to the support of structural loads.

Design of floating foundations requires careful and detailed sampling and consolidation testing of clayey soils adjacent and below the pile tips and three-dimensional settlement analyses using programs, such as 3D-PLAXIS[9] (preferred) or 3D-SETTLE[10]. For preliminary estimates of foundation settlement, a hand calculation of settlement is performed using one-

[9] Plaxis, Delft, the Netherlands
[10] Rocscience

dimensional consolidation analysis and equivalent raft foundation model. In this method, axial loads carried by piles are distributed on an imaginary raft foundation located at a depth equal to 2/3 of pile length. The vertical pressure is then distributed assuming a load spread based on a 2V (vertical): 1H (horizontal) distribution, spreading out in two orthogonal directions.

The procedure for design of pile-supported floating foundation is more complicated than design of pile-supported structures where deep foundations gain support within competent soil / rock at depth. This is because in this case, pile head stiffness would depend on long term (consolidation related) foundation settlement. In turn, foundation settlement would depend on pile head axial load based on pile head stiffness. Therefore, foundation design of pile-supported floating foundations is performed using an iterative procedure.

The steps taken in an iterative foundation design procedure consist of the following:

- Long-term foundation settlement is estimated and axial pile head stiffness is calculated by dividing pile loads by the magnitude of estimated settlement.
- The structural engineer develops a model of the mat or grade beam system, and represents axial stiffness of piles using soil spring constants. The pile head axial loads are obtained from this model.
- The geotechnical engineer performs three-dimensional settlement analysis using computer programs 3D-PLAXIS (preferred) or 3D-SETTLE and recalculates the long-term foundation settlement.
- The iterative process continues until the assumed pile head axial stiffness becomes close to (within 5 percent of) the calculated values.

It is noted that in case of large (3 to 4 inches) consolidation related foundation settlement, dynamic axial pile head stiffness is much higher than, and is very different from, those corresponding to the static loading conditions. This means that axial pile and soil stiffness values during seismic loading conditions could be higher than static values by an order of magnitude.

3.9 Pile Group Effects

To avoid reduction in axial capacity of a pile in a pile group, piles should have a minimum center-to-center spacing of three pile diameters. Reduction in lateral stiffness for pile with center-to-center spacing between 3 and 6 pile diameters would depend on pile group geometry, pile layout, location of the pile within pile group, and type / stiffness of soil between piles. Bending moment of a pile in a pile group is somewhat higher than that of a single pile subjected to the average pile head lateral load within the pile group.

Lateral stiffness of a single pile could be estimated using the computer program LPILE. Various methods for evaluation of pile group effects are presented in the following reference:

- Poulos, H.G., 2017, *Tall Building Foundation Design*, CRC Press.

The hybrid method for evaluation of pile group effects has been developed by the following reference:

- Ooi, P.S.K., and Duncan, J.M., "Lateral load analysis of groups of piles and drilled shafts," *Journal of Geotechnical Engineering*, Vol. 120, No. 6.

3.10 Modeling of Pile-Supported Structures and Soil-Structure Interaction

The following report presents three methods for inclusion of seismic soil-structure interaction effects:

- PEER, 2017, *Guidelines for Performance-Based Seismic Design of Tall Buildings*, Version 2.03, Report No. 2017/06, Pacific Earthquake Engineering Research Center, Berkeley, California.

The three methods are as follows:

- Ignoring presence of soils adjacent to the basement
- Bathtub modeling (soil springs and dashpots are attached to basement walls) with a single set of ground motions applied to the ends of soil springs and dashpots that are not connected to the basement walls
- Bathtub modeling, with location-specific (depth-varying) ground motions applied to the back of soil springs and dashpots

However, according to the present state of practice, interaction between the structure and foundation is typically ignored and seismic analysis is performed in two steps:

- In the first step, a fixed base model of the structure is analyzed and the design base shear force and maximum overturning moment are calculated at the foundation level. If present, basements are included in the model; however, interaction between basement walls and surrounding soils is ignored (i.e., soil springs and dashpots are not included in the model). The number, location, diameter, and length of pile foundations are determined based on the magnitude of calculated base shear force and axial demand due to gravity, sustained live loads, and maximum overturning moment.

- In the second step, a model is constructed that includes mat or grade beams and soil springs representing axial and lateral pile stiffness. The model is then subjected to base shear, gravity load, sustained live load, and overturning moments to calculate axial force, lateral force, and bending moment developed at the pile tops and shear and bending stresses developed in the mat or grade beam system.

It should be noted that not including the surrounding soil in the numerical model may be conservative when basement walls are in contact with relatively soft soils (i.e., passive soil springs are not stiff enough to impact the magnitude of axial or in-plane forces developed in the basement diaphragms). However, in cases where basement walls are in contact with very stiff soil or rock, eliminating passive soil springs may result in underestimation of axial (in-plane) loads developed in the ground floor diaphragm through redistribution of shear force in the core between ground surface and the basement slab.

The current analysis procedure ignores the kinematic soil-structure interaction (SSI) effects and elongation of structural period due to lateral flexibility of the foundation. It is thought by the structural engineering community that these assumptions (i.e., ignoring SSI effects) are conservative.

The current analysis procedure described above is in contrast with practice in the offshore and bridge industries. For more than half a century, foundation of offshore platforms and bridges are included in the dynamic model of the structure and both kinematic SSI effects and effects of foundation flexibility on dynamic performance of the structure are accounted for in a more rigorous manner. Perhaps, the contrast between state of practice in building foundation design and design of offshore / bridge foundations stem from the fact that for both offshore and bridge structure, kinematic motion (ground motion transmitted to the structure) could substantially be different from ground motion at the ground surface (mudline).

Kinematic motions could be obtained by either: (1) modeling of individual piles, use of p-y springs to represent lateral soil stiffness, and use of depth-varying free-field ground motion behind the soil springs (obtained through one-dimensional site response analysis); or (2) modeling piles using beam elements, modeling soil using two or three dimensional solid elements, and attaching the soil mesh with the beam elements with p-y springs. The mass of above-ground structure is not included when performing kinematic SSI analysis. There are a number of commercially available computer software appropriate for kinematic SSI analysis including SASSI[11], FLAC, and LS-DYNA[12], among others. SASSI performs analysis in frequency domain and is only applicable to linear systems. Its practical application is limited to deeply embedded structures with very rigid below-ground structural stiffness (BART San Francisco Transition structure and Transbay Terminal Center structure were analyzed using SASSI). FLAC is capable of accounting for soil and structural nonlinearities; however, only the two-dimensional version of this program is used in practice in dynamic mode. LS-DYNA has successfully been used in two-, and three-dimensions to perform kinematic SSI analysis.

The simplest way to include pile foundation in analytical model of structure is through the use of "dummy cantilever" beams. The process involves the following numerical steps: (1) linearize the pile head lateral stiffness. This is accomplished by assuming a certain pile head deflection and calculating pile force based on L-PILE analysis results for either fixed pile head or free pile head, depending on pile head fixity condition; and (2) calculate the length of a dummy cantilever beam which has the same pile head stiffness. The cross-section area of the dummy cantilever beams is adjusted to capture actual pile axial stiffness values. Because the pile head lateral and axial stiffness in each iteration are constant, the model is suitable for use in linear modal SSI analysis.

The next step is to compare the pile head displacements obtained from the SSI analysis to the assumed pile head displacement used for linearization of the pile head stiffness. If the assumed and calculated values are different by more than 5 percent, the process is repeated until the difference between assumed and calculated pile head lateral displacements obtained from two consecutive iterations becomes smaller than 5 percent.

This procedure is very well described in the following reference:

- PoLam, I., Kapuskar, M., Chaudhuri, D., 1998, *Modeling of Pile Footings and Drilled Shafts for Seismic Design*, MCEER-98-0018, Multidisciplinary Center for Earthquake Engineering Research, University at Buffalo, New York.

According to the present standard of practice, if two towers share a common mat foundation (whether the mat is pile supported or not), the interaction between the two towers through structure-foundation-structure interaction is

[11] *System for Analysis of Soil-Structure Interaction*, Lysmer, J., Tabatabaie-Raissi, M., Tajirian, F., Vahdani, S., Ostadan, F.

[12] Livermore Software Technology Corporation

ignored, i.e., both towers are subjected to the same ground motion. More studies are needed to validate this approach.

3.11 Expected Performance of Piles During the Design Seismic Event

Because it would be difficult to inspect and repair damage to piles after a major seismic event, although some yielding of the pile at the connection points with the pile caps / mat foundation may be acceptable, piles are generally designed to suffer no appreciable structural damage, with bending moment and shear stress well below critical limits, i.e., plastic deformations are allowed at the base of the tower, but not below the ground surface. According to the current standard of practice, geotechnical engineers are normally asked to use 0.3 to 0.7 times the EI of concrete pile section when performing lateral pile analysis.

3.12 Use of Battered Piles

Use of battered piles for support of tall buildings should be avoided. If needed to resist unbalanced lateral soil pressure in case of sloping ground, shear keys are commonly used.

Chapter 4

New Tall Buildings on Shallow Foundations

A tall building is supported on a shallow foundation in case of one of the following conditions: (1) the foundation is directly placed over bedrock; (2) the foundation is underlain by dense to very dense Colma sand or alluvium consisting of stiff to very stiff clays and dense to very dense sands; (3) the building has a deep basement extending well below groundwater table where highly compressible Young Bay mud or liquefiable soils are excavated as part of basement construction; or (4) the foundation is placed over improved ground extending to the surface of firm soil at depth.

In case of improved ground supporting the foundation of a tall building, integrity of ground improvement elements (e.g., deep soil mixing - DSM) during both static and seismic loading conditions should be carefully evaluated. To satisfy integrity of DSM during a major seismic event, DSM is typically placed in a grid pattern with center to center spacing of 15 feet or less and wall thickness of 3 feet (replacement ratio of 30% or greater).

4.1 Bearing Capacity and Short or Long Term Settlement

If shallow foundation is not directly placed over bedrock, the bearing capacity and short or long term settlement of soils below the foundation and above the bedrock should be carefully evaluated. The density of sandy soil should be evaluated by performing standard penetration tests or cone penetration test (CPT) soundings. Sample of stiff to hard clayey soils (e.g., Old Bay clay) below the foundation should be obtained using pitcher barrel sampler or equivalent and high-quality tests should be performed on undisturbed samples to measure shear strength and consolidation characteristics of clayey soil layers. Use of Shelby tube for obtaining soil samples within stiff to very stiff clayey soil is not desirable due to sample disturbance and uncertain consolidation and strength characterization.

A very useful test often performed in the East Coast of United States and other countries is the pressuremeter test. The results of this test provide insitu soil stress-strain relations that can be used in computer modeling of soil-structure systems. However, pressuremeter tests are seldom performed for projects in the San Francisco Bay Area.

Settlement calculations should be performed using computer programs such as 3D SETTLE[1] or 3D PLAXIS[2] (preferred). Timeline of settlement should be plotted using snapshots at various stages of construction including placement of shoring, dewatering, completion of excavation, placement of foundation, termination of dewatering, completion of construction, 5 years after completion of construction, and 10 years after completion of construction.

When performing foundation settlement or bearing capacity analysis, effects of hydrostatic uplift pressure in reducing foundation pressure may be accounted for. However, the time required for recharge after termination of dewatering should be accurately estimated or measured and considered. For example, if it takes three years for water pressure under the foundation to become equal to water pressure outside of the excavation and the construction is completed in two years, accounting for full hydrostatic uplift pressure as part of settlement or foundation bearing capacity calculation could lead to excessive foundation settlement or bearing failure.

The factor of safety (FOS) against bearing failure should be greater than 3.0 for static plus sustained live loads and greater than 2.0 against total loads including wind and seismic loads. Gain in shear strength within clayey soils due to consolidation under dead plus sustained live loads may be accounted for when calculating FOS against total loads.

When shallow foundations are placed directly over or within short distance from stiff to very stiff clayey soils (e.g., Old Bay clay), in addition to maintaining a FOS against global bearing failure mechanism, punching shear failure mechanism should also be considered and FOS against this failure mechanism should be evaluated. In this case, friction between basement walls and shoring system may be accounted for provided that the basement walls are designed to safely resist the resulting tensile forces and reduction in frictional capacity between basement walls and shoring walls due to presence of waterproofing membrane is accounted for.

Finally, if a shallow foundation is directly supported on ground improvement elements such as deep soil mixing (DSM) or jet grouting (JG), the internal stability, as well as global stability of the DSM or JG system, should be evaluated during both static and seismic loading conditions. The geotechnical engineer of record (GEOR) shall be responsible for design of

[1] Rocscience
[2] Plaxis, Delft, the Netherlands

DSM or JG system and this responsibility shall not be passed on to the specialty Design Build (DB) contractors. DSM or JG shall be placed in grid pattern with limited size (normally center to center spacing of 15 feet or less) to adequately resist tensile, shear, and compressional stresses developed under application of total loads (dead plus sustained live loads plus seismic loads). Using individual columns of improved ground shall be avoided. The GEOR shall perform adequate studies to make certain that the ground improvement system will perform satisfactorily during the design earthquake, i.e., the ground improvement shall be considered to be part of the structural system. This is because brittle failure of the ground improvement system and its partial or total loss of support could result in excessive settlement of the structure.

A properly designed and constructed ground improvement system could change site classification from Site Class F (liquefiable soils or deep soft clays such as Young Bay mud) to Site Class D.

4.2 Foundation-Soil Interaction and Foundation Design Against Lateral Loads

A general discussion on state-of-the-art and state-of-practice related to methods for accounting for seismic soil structure interaction (SSI) effects was discussed previously in Chapter 2 of this Part.

To safeguard against migration of moisture through mat foundations directly placed over competent soil below the permanent ground water table, a waterproofing membrane is installed below the mat. Presence of a waterproofing membrane will result in very low frictional resistance between the base of the mat and the subgrade. The project team should consult with waterproofing manufacturers to obtain a safe allowable friction coefficient and check sliding of the structure under application of design base shear. If the base shear force is larger than the frictional capacity of the interface between the bottom of the mat and the waterproofing membrane or between the waterproofing membrane and the subgrade, passive soil pressure against the face of the mat and basement walls (if they exist) will be mobilized. In this case, displacement compatibility between the base resistance and the passive resistance should be evaluated and the basement walls should be designed to safely resist the resulting passive forces.

4.3 Effects of Foundation Flexibility on Dynamic Response of a Tall Building

Base rocking increases the structural period, and hence, reduces the seismic demand (in the long period range where spectral acceleration decreases with an increase in period). Because of this, some practitioners conservatively ignore foundation flexibility and do not include rotational soil springs in the structural model, i.e., a fixed based analysis is performed. Although this is a reasonable approach, it should be noted that rotation at the base will increase later movement (drift) of the structure. For this reason, some practitioners attach four vertical bi-linear springs to the base of the mat. The stiffness of these springs and distance between them are selected such that the rocking stiffness of the mat is reasonably modeled. If fixed-base analysis is performed, it would be prudent for the geotechnical engineers to estimate rocking stiffness of the foundation and the structural engineers to estimate the magnitude of drift due to Inclusion of foundation flexibility and collectively make a decision as to whether vertical soil springs should be included in the structural model.

Chapter 5

Evaluation and Retrofit of Foundations of Existing Tall Buildings

Evaluation and retrofit of foundation of existing tall buildings pose unique challenges including:

- The soil report and detailed information about the foundation type and its installation may or may not exist. In addition, the quality or quantity of data in older reports may not meet current standard of practice.
- For tall buildings designed before 1970, soil reports (if they do exist) are silent on design issues related to soil liquefaction, as the phrases "soil liquefaction" and "cyclic mobility" were conceived in early 1970s (by Kenji Ishihara of Tokyo University and Harry Seed of University of California at Berkeley) following the 1964 Niigata and 1964 Alaska earthquakes.
- Based on damage observed during the 1971 San Fernando earthquake, the building code changed substantially after 1975. For tall buildings designed in accordance with building codes prior to 1975, design base shear is lower than that of a tall building designed in accordance with the current version of the building code.
- Access to the building and overhead clearance in the basement or ground floor is usually very limited. As such, conventional soil investigation equipment cannot be mobilized to conduct a thorough geotechnical field investigation program according to modern standards.
- Retrofit of existing tall buildings normally requires installation of new shear walls, energy absorbing devices, or new columns to carry gravity loads. Therefore, either existing shallow spread footings are enlarged, new shallow spread footings are installed, or micropiles are installed to augment and strengthen the existing foundation system.

The most challenging aspect of retrofit of foundation of an existing tall building is to maintain compatibility of stiffness or capacity between existing foundation and new foundation elements. If compatibility between response of existing and new foundation elements cannot be maintained, this could result in overstressing and thus possible failure of new foundation elements and inadequate foundation support of the building.

5.1 Geotechnical Investigation Methods Operating from Basement of an Existing Tall Building

Information regarding the elevation of the interface between soft and firm soils in combination with knowledge of subsurface conditions and local geology is very useful in evaluation of the capacity of an existing foundation and design of new foundation elements.

Methods for geotechnical investigation from the basement of an existing tall building include:

- Excavation of test pits for exposing footings or pile caps and observing soil and pile conditions below the existing foundation. Depending on the depth of footings and groundwater level, excavating test pits may require shoring and dewatering.
- Excavation of shallow test borings and obtaining soil samples below the base of the existing foundations using hand augers.
- Performing cone penetration test (CPT) soundings using mini CPT equipment small enough to allow transport through doorways and entrances and operable in confined spaces with limited vertical clearance. Mini CPT equipment is anchored to the existing floor slab to develop reaction needed for pushing the cone through subsurface soils. The maximum axial compression load applied would be limited to tensile capacity of anchors. Experience working in San Francisco indicates that mini CPTs are able to go through fill and soft Bay mud, but not through dense Colma sands.

5.2 Use of Micropiles for Strengthening the Foundation of an Existing Tall Building

Micropiles are commonly installed to add axial capacity to the existing foundation of buildings being seismically retrofitted.

A micropile is a small diameter (typically less than 12 inches), drilled and grouted, replacement pile that is typically reinforced by placing a rebar at the center of the pile. It is constructed by drilling a borehole, placing reinforcement, and grouting the hole. Micropiles penetrating 40 to 60 feet into competent soils have axial capacity of up to 600 kips and axial stiffness of up to 400 kips/in. Post grouting is often used to increase the axial capacity of micropiles.

The most relevant reference for design, construction, and testing of micropiles is presented below:

- FHWA, 2000, *Micropile Design and Construction Guidelines*, Publication No. FHWA-SA-97-070, U.S. Department of Transportation Federal Highway Administration.

Proper design of micropiles for strengthening of foundation of an existing tall building would require detailed numerical modeling of the following: (1) axial stiffness of existing foundation elements (spread footing or pile cap); and (2) new micropiles. It is critically important to determine the percentage of axial gravity and seismic loads carried by micropiles both in compression and tension by properly modeling relative axial stiffness of the existing foundation system and axial stiffness of new micropiles.

Chapter 6

Foundation Design of Tall Buildings Near Shoreline

If a tall building is planned near the San Francisco Bay waterfront, foundation design should consider potential for shoreline instability and its effects on performance of foundation during a major seismic event.

Based on observations during the 1906 and 1989 earthquakes, detailed recent studies performed at Treasure Island using nonlinear numerical modeling (2D FLAC[1] and 2D PLAXIS[2] analyses), and similar detailed studies performed at waterfront sites in San Francisco, the magnitude of maximum lateral spreading along the shoreline during a major seismic event is estimated to be on the order of 10 to 15 feet. These studies indicate that, if large (10 to 15 feet) lateral spreading occurred at the shoreline, the magnitude of lateral ground movement will decay from its maximum value at the shoreline to less than 6 inches at a distance of about 300 feet. It is also judged that "pile pinning effects" would reduce the free field displacement of less than 6 inches to much smaller values through kinematic soil structure interaction. Therefore, well-designed and constructed pile-supported structures are expected to perform satisfactorily at distances equal or greater than 300 feet from the shoreline during a major seismic event. Thus, if a tall building is planned at a distance greater than 300 feet from the shoreline, foundation design could proceed without considering the potential for shoreline instability. However, the effects of sea level rise should be considered as discussed in Chapter 8 of this Part.

For sites located within 300 feet from the shoreline, seismic shoreline instability has to be carefully evaluated through literature survey and review of available data and reports with information related to seismic seawall and shoreline stability. If only limited information is available regarding seismic shoreline stability, performing simplified slope stability calculations alone would not provide adequate or defensible estimates of variation of lateral ground displacement with distance away from the shoreline. Thus, detailed,

[1] Itasca Consulting Group, Inc., Minneapolis, Minnesota
[2] Plaxis, Delft, the Netherlands

two-dimensional, time-dependent numerical analysis should be performed utilizing software such as 2D FLAC.

Ground motions should be developed at the bedrock or surface of competent soil at depth and nonlinear behavior of soft and liquefiable soils should be modeled considering generation of pore water pressure within sandy soils and strain softening within clayey material. If the magnitude of computed free-field lateral displacement is less than 6 inches at the pile location, foundation design could proceed without considering potential for shoreline instability. However, if the magnitude of free-field lateral displacement is calculated to be larger than 6 inches at the pile location, the foundation design should consider lateral loading on the foundation due to shoreline instability. In this case, the most economical way to address foundation design might be to mitigate the shoreline instability through the use of a suitable ground improvement technique, such as deep soil mixing, jet grouting, stone columns, compaction grouting, deep dynamic compaction, vibro-compaction, vibro-replacement, rapid-impact compaction. If ground improvement is used to mitigate seismic shoreline instability issues, the geotechnical engineer of record (GEOR) should perform adequate numerical modeling and analysis to make certain that the improved soil block will maintain its global stability as well as internal stability. Individual columns of improved ground shall not be used, deep soil mixing or jet grout system should be placed in a grid pattern. The responsibility for design of ground improvement system shall not be passed on to a design-build specialty contractor.

It should be noted that evaluation of the time history of lateral ground movement using a nonlinear, time-dependent analytical procedure follows a performance-based design approach. As such, the risk-baced MCE_R ground motion should be used to check against collapse of the structure as a result of foundation failure.

Chapter 7

Shoring and Dewatering

In cases where excavation cannot be completed by slopping back the ground surrounding the excavation, shoring systems with vertical walls are often used. Commonly used shoring systems in the San Francisco Bay Area include sheet pile walls, soldier pile and lagging, secant piles, and deep soil mixing (DSM) walls. Secant and DSM walls are reinforced by placing steel H-beams in regular spacings of 4 to 8 feet on center while the mix is in a fluid state. Information regarding design and construction of shoring systems could be found in the following reference:

- FHWA, 2008, *Earth Retaining Structures*, Publication No. FHWA-NHI-07-071, U.S. Department of Transportation Federal Highway Administration.

In case of DSM walls, the soil-cement mix acts as lagging between soldier beams. The stresses developed in DSM due to apparent earth pressure and deformation of soldier piles should be checked against design strength of soil-cement mix (typically 300 psi in unconfined compressive strength).

Installation of soldier pile and lagging system in areas with high groundwater table would result in lowering of groundwater table and settlement of ground outside of excavation which would be problematic when used in well-developed and congested areas. Overlapping secant piles and DSM shoring system would minimize (if not eliminate) lowering of groundwater table outside of the excavation. Therefore, overlapping secant piles and DSM shoring systems are often used in case of a high groundwater table and when excavation could have an adverse effect on the foundation of adjacent structures. It is noted that lowering of the groundwater table outside of excavation to historical low groundwater table would be acceptable as the soil has already experienced the higher effective vertical stresses associated with the lower historical groundwater table. Due to the cycles of drought in the San Francisco Bay Area, a drop of 5 feet in groundwater table when measured in rainy years is judged to be acceptable.

It is not practical to have a cantilever shoring system with a height of more than 12 feet. Therefore, a shoring system with a height exceeding 12 feet would require either internal bracing or tiebacks.

Geotechnical investigation requirements for design and installation of tiebacks; apparent earth pressure in sand, stiff clays, and soft to medium stiff clays; gross and net water pressure diagrams; accounting for surcharge loads; unbounded length requirements and design of anchor bond length; tieback slope and spacing requirements; tieback testing procedure including performance test, proof test, and creep test (creep test is specially required in case of tiebacks installed in soft to medium stiff clays); lock-off load requirements, anchor lift-off testing procedure; corrosion protection requirements (in case of permanent shoring walls), depth of penetration below excavation; design of soldier beams against axial loads due to inclined tieback loads; expected lateral deformation of well-designed and well-constructed shoring systems using tiebacks; inspection and short term / long term monitoring requirements; and other pertinent information are presented in the following reference:

- FHWA, 1999, *Ground Anchors and Anchored Systems*, Publication No. FHWA IF-99-015, U.S. Department of Transportation Federal Highway Administration.

A common failure mode during installation of soldier pile and lagging system is flow of saturated loose sands into the excavation from areas below the lowest installed lagging board, creating a cavity behind the wall that could extend upwards to the ground surface. If this occurs, dewatering of soils behind shoring wall could mitigate this failure mode.

A common failure mode during installation of tieback system is the flow of loose saturated sands around the auger and creation of a cavity between the auger and the ground surface. To investigate whether this failure mode is occurring, the volume of soil being extracted should be carefully checked against auger size and rate of advancement of the auger. In one case, ground surface settlement was observed below a building due to a cavity developed when the auger was spinning but not advancing with a rate proportional to the volume of soil being extracted.

In cases of deep excavation in soft to medium stiff clays, special attention should be given to basal heave and calculation of factor of safety (FOS) against blow out conditions discussed in the above reference. In the San Francisco Bay Area, there are sand lenses and layers within Young Bay mud and Old Bay clay layers. Dewatering of these layers may be required to prevent basal heave instability or blow out conditions. Additional information for calculation of FOS against basal heave instability is provided in the following reference:

- Wu, S.-H., Ou, C.-Y., Ching, J., 2014, "Calibration of model uncertainties in base heave stabilit for wide excavations in clay," *Soils and Foundations*, Vol. 54, No. 6.

It should be noted that for deep excavations, maximum ground settlement may occur away from the shoring wall due to deep seated mode of ground deformation. For example, for a 75 foot deep excavation in downtown San Francisco, the maximum ground settlement was calculated to be about 50 to 70 feet away from the face of the excavation.

A conservative safe distance from the face of an excavation is twice the depth of excavation. However, a more realistic safe distance is where the ground deformation is reduced to less than 1/4 inch. In case of a properly designed and constructed shoring system, ground deformation criteria of less than 1/4 inch would generally be satisfied at a distance greater than or equal to the height of the excavation.

The following reference provides additional information regarding design and construction of DSM walls:

- FHWA, 2013, *Deep Mixing Embankment and Foundation Support*, FHWA Design Manual, FHWA-HRT-13-046, U.S. Department of Transportation Federal Highway Administration.

To prevent stress relaxation and excessive lateral movement of an internally braced shoring system, struts are normally preloaded to 75% to 80% of loads calculated based on the apparent earth pressure.

When DSM walls are installed within zone of influence of foundation of adjacent structures, consideration should be given to potential lateral movement of the DSM wall and resulting settlement of adjacent structures while DSM remains in fluid state. This condition is especially critical when DSM walls extend below the tip of pile foundations supporting adjacent tall buildings.

Empirical ground settlement data for internally braced or tied-back shoring walls is presented in the following reference:

- Peck, R.B., 1969, "Deep excavation and tunneling in soft ground," *Proceeding of the 7th International Conference on Soil Mechanics and Foundation Engineering*, Mexico City, pp. 225-290.

In this reference, ground settlements are given at various distances from the excavation. Both ground settlement and distance from excavation are normalized to the depth of excavation.

Maximum lateral movement of internally braced or tied-back walls as a function of system stiffness and FOS against basal heave is presented in the following reference:

- Clough, G.W., and O'Rourke, T.D., 1990, "Construction induced movements of in-situ walls," *Design and Performance of Earth Retaining Structures*, No. 25, pp. 439-470.

Some practitioners use finite element or finite difference computer codes to estimate the magnitude of ground deformation in case of complicated or critical projects. The behavior of saturated clayey soils is generally modeled considering short term loading (undrained) conditions using total stress method of analysis. Although this method is appropriate for modeling of the stiffness and capacity of clayey soils, if the construction duration is extended, additional ground deformation may occur as the behavior of saturated clayey soils change from undrained conditions to drained conditions. As such, the analysis should employ the effective stress method as opposed to the total stress method.

Chapter 8

Sea Level Rise

The following reference presents a pilot study conducted by FEMA regarding sea level rise for San Francisco County:

- FEMA, 2016, *Sea Level Rise Pilot Study, Future Conditions Analysis and Mapping San Francisco County,* California, prepared by BakerAECOM for the Federal Emergency Management Agency, Washington, D.C.

This study developed the following equations to estimate mid- and high-range sea level rise (SLR):

- SLR (inches) = $0.0024 \times t^2 + 0.12 \times t$, where t = years from 2000
- SLR (inches) = $0.00366 \times t^2 + 0.2936 \times t$, where t = years from 2000

Accordingly, the mid-range estimate for sea level rise in San Francisco County in the years 2050 and 2100 are 12 inches and 36 inches, respectively. The high-range estimate for SLR in the years 2050 and 2100 are 24 inches and 66 inches, respectively. Groundwater table elevation at sites close to the Bay will rise with sea level rise. However, the effect of sea level rise on groundwater table elevation for sites away from the shoreline (especially for site at higher elevations) diminishes as the distance from the shoreline increases. This is relevant to geotechnical engineering because sea level rise will increase the thickness of saturated sandy fills, and will increase the potential for soil liquefaction related lateral spreading. Other effects of sea level rise include an increase in hydrostatic pressure on basement walls, an increase in hydrostatic uplift pressure, which would impact design of ground anchors (tiedowns), and an increase in uplift demand on deep foundations.

Chapter 9

Recommendations

Best practices are not fully codified and implementation of best practices varies considerably between practitioners. AB083 set the bar for *structural* design of tall buildings; need to develop a companion AB for *foundation* design. It is essential to bring consistency and uniformity to geotechnical practice for foundation design of tall Buildings by establishing guidelines for best practices w/o adversely impacting the construction cost or project schedule. It is recommended that SFDBI develop an Administrative Bulletin (AB) or Guidelines to address the following:

1. Acceptable limits of total and differential settlement
2. Evaluation / mitigation of geologic hazards including soil liquefaction and lateral spreading
3. Site characterization (field investigation and laboratory testing)
4. Foundation analysis and design
5. Numerical modeling, especially those dealing with bearing capacity calculation and evaluation of foundation settlement
6. Design of shoring / dewatering systems
7. Development of design ground motions and incorporation of soil-structure interaction effects
8. Design consideration related to sea level rise

The proposed AB / Guidelines will strengthen SFDBI in assessing the appropriateness / completeness of foundation design recommendations presented in geotechnical reports for tall buildings

The following 15 recommendations are extracted from material presented in previous chapters of this Part. These recommendations should not be followed without a thorough review of the relevant sections.

- **Recommendation 1 - Total and Differential Settlement.** There are no universally accepted criteria for maximum allowable total and differential settlement of foundation for support of commercial / residential tall buildings. However, many practitioners and officials with building departments of various cities consider the total settlement of less than 4 inches and limiting differential settlement to an angular distortion of 1/500 for composite steel or concrete core buildings and 1/750 for reinforced concrete towers (rigid structures) to be acceptable. The structural system, cladding system, partitions, and other nonstructural components may also control the acceptable differential settlement. Some buildings (e.g., base-isolated structures) are more sensitive to differential settlement than other conventional buildings and would require a tighter limit on differential settlement. However, base-isolation systems are seldom used in tall buildings. Detailed discussion is presented in Chapter 2 of this Part.

- **Recommendation 2 - Monitoring of Building Settlement.** Building settlement should be monitored during construction and to at least 10 years after construction. Monitoring and reporting requirements should follow SFDBI guideline presented on Sheet S-18.
- **Recommendation 3 - Tall Buildings Supported on Large Diameter (≥ 4 feet) Drilled Shafts Extending to or into Bedrock.** The ultimate skin friction in soil and skin friction / bearing capacity within rock socket should be evaluated using Osterberg (O-Cell) or equivalent test method. If end bearing is accounted for in the design, the bottom of drilled shaft should be cleaned out using "one-eye" bucket or flat bottom clean out bucket or airlifting method and verified by Mini-SID (Shaft Inspect Device) or DID (Ding Inspection Device). For drilled shafts socketed in bedrock, the minimum rock socket length should generally be 15 feet or twice the shaft diameter, whichever is greater. The minimum rock socket length requirement could be relaxed for sites where: (1) depth to bedrock is greater than 25 times the shaft diameter; and (2) hard rock with substantial lateral extension and depth below the bottom of drilled shafts is encountered (i.e., the hard rock bolder is a size of a house and not a compact car). Detailed discussion is presented in Section 3.1 of this Part.
- **Recommendation 4 - Tall Buildings Supported on 24-inch or Larger Auger Cast Piles.** The use of Auger cast piles (ACP or CFA) should either be avoided or carefully monitored if the subsurface conditions include layers of soft clay (e.g., Young Bay mud) or saturated loose sand. Installation of auger cast piles in all soil types should be installed by an experienced foundation contractor, preceded by load testing, and production pile installations monitored by skilled field inspectors or engineers and automated monitoring equipment (AME) data acquisition system. All Auger cast piles should include tubes for possible gamma-gamma, cross hole sonic, or thermal testing. Ten percent of auger cast piles should be tested for integrity as directed by the geotechnical

engineer of record (GEOR) using the above-mentioned methods. In addition, all auger cast piles with questionable installation record should be integrity tested. Static pile load test in combination with high strain dynamic pile load test should be performed to verify the axial capacity of the piles. Detailed discussion is presented in Section 3.1 of this Part.

- **Recommendation 5 - Tall Buildings Supported on Specialty Deep Foundations.** If used, specialty piles (torque-down piles, Tubex piles, enlarged base (Franki) piles, Micropiles, Fundex piles, etc.) are designed and installed by specialty design build (DB) contractors. The GEOR should review design submittals by DB contractors, including review of both static and seismic loading conditions and load paths. The GEOR is ultimately responsible for geotechnical aspects of foundation design. Detailed discussion is presented in Section 3.1 of this Part.
- **Recommendation 6 - Allowable Tolerance for Installation of Deep Foundations.** Deep foundations should meet plumbness criteria of less than 1.25 inch for every 10 feet of depth. Detailed discussion is presented in Section 3.5 of this Part.
- **Recommendation 7 - Soil Liquefaction Design Issues.** The potential for soil liquefaction and its effects on ground motion and foundation (temporary loss of axial support, reduction in lateral support, development of downdrag loads, ground settlement) shall be carefully evaluated and addressed. Detailed discussion is presented in Section 3.7 of this Part.
- **Recommendation 8 - Tall Building on Floating Deep Foundations.** In case of tall buildings supported on deep foundations that don't extend to bedrock, the potential for short-term and long-term total and differential settlement should be carefully evaluated. Undisturbed soil samples of clayey soil should be obtained using Pitcher Barrel sampler and high-quality consolidation tests should be performed on adequate number of soil samples. Three-dimensional settlement analysis should be performed using computer program 3-D PLAXIS (preferred) or 3-D SETTLE. It is recognized that long-term, consolidation related foundation stiffness is much lower than foundation stiffness during seismic loading conditions. Therefore, both long-term and seismic foundation stiffness should be evaluated and used in structural design. Pressure meter tests are used in the eastern United States and other countries to provide very useful information regarding the insitu stress-strain relationships, but unfortunately is not being performed in the San Francisco bay Area. Geotechnical engineers practicing is the Bay Area are encouraged to perform pressure meter test and develop a data base correlating the

pressure meter test results to laboratory test result. Detailed discussion is presented in Section 3.8 of this Part.

- **Recommendation 9 - Pile Group Effect.** Pile group effects on axial pile capacity and axial / lateral pile head stiffness depends on pile diameter, pile spacing, and soil characteristics. These effects should be evaluated as part of foundation design. Detailed discussion is presented in Section 3.9 of this Part.
- **Recommendation 10 - Soil-Structure Interaction Effects.** Evaluation of soil-structure interaction (SSI) effects should follow TBI's guidelines for performance-based seismic design of tall buildings. Accounting for kinematic SSI effects could potentially lead to economic design of the buildings. Computer programs such as SASSI, FLAC, LS-DYNA, among others could be used to perform two, or three-dimensional kinematic SSI analysis. Detailed discussion is presented in Section 3.10 of this Part.
- **Recommendation 11 - New Tall Buildings on Shallow Foundation.** The bearing capacity and short-term / long-term settlement of foundation should be carefully evaluated. In-situ density of sandy soil should be obtained using SPT or CPT, as appropriate. Samples of stiff to hard clayey soils (Old Bay clay) below the foundation and above bedrock should be obtained using Pitcher Barrel sampler and high-quality consolidation tests should be performed on adequate number of samples. Use of thin wall Shelby tube sampler in stiff clayey soils should be avoided. Three-dimensional settlement analysis should be performed using computer program 3-D PLAXIS (preferred) or 3-D SETTLE. Foundation bearing capacity calculation should consider both punching shear failure mechanism and global failure mechanism. Effects of hydrostatic uplift pressure on reducing foundation bearing pressure may be accounted for; however, time required for recharge and increase of water pressure to a level outside of the excavation should be accurately estimated or field measured during construction. Detailed discussion is presented in Section 4.1 of this Part.
- **Recommendation 12 - Evaluation and Retrofit of Foundation of Existing Tall Buildings.** Micropiles are commonly used to supplement the axial compression / tensile capacity of the existing foundations. It is critical to evaluate the axial capacity and stiffness of both the existing foundations and new foundations and include axial stiffness of these elements accurately in the computer model to evaluate a realistic distribution of static and seismic loads between the existing and new foundations. All micropiles (cased on uncased) shall have an upper pile section (> 20 ft long) of segmental step-tapered joint, flush couples steel,

API rated pipe casing and be pile load tested regardless of pile load capacity. Detailed discussion is presented in Chapter 4 of this Part.

- **Recommendation 13 - Foundation Design of Tall Buildings Near Shoreline.** For sites within 300 feet from the shoreline, effects of shoreline instability (lateral spreading) on performance of the foundation should be evaluated by a geotechnical engineer experienced in design of near-shore structures and mitigation measures be implemented to avoid overstressing and failure of the foundation during the design seismic event. The shoreline stability evaluation could be performed in phases consisting of literature survey, simplified slope stability and deformation analysis, detailed two-dimensional finite element / finite difference modeling, with each phase of evaluation becoming more detailed, time consuming, and costly than the previous phase. It is believed that if the magnitude of free-field lateral spreading is six inches or less at the closest pile location to the shoreline, the presence of pile foundation could mitigate the lateral spreading through kinematic SSI effects (otherwise referred to as pile pinning effects). Detailed discussion is presented in Chapter 6 of this Part.
- **Recommendation 14 - Shoring and Dewatering.** Critical issues related to shoring and dewatering include potential for settlement of adjacent streets and structures, basal heave if shoring system is installed in soft to medium stiff clays, loss of ground by flow of saturated loose sandy soil behind the shoring into the excavation from areas below the lowest installed lagging board, loss of ground in saturated sandy soils while installing tie-backs, and increase in ground deformation adjacent to a shored excavation with time as the behavior of saturated clayey soils changes from undrained to drained conditions. The shoring should be designed by qualified professional California Civil Engineer or California Structural Engineer retained by the contractor. Detailed discussion is presented in Chapter 7 of this Part.
- **Recommendation 15 - Sea Level Rise.** Potential for sea level rise should be accounted for as part of design of foundation and basement walls of a new tall building. Detailed discussion is presented in Chapter 8 of this Part.

Appendix A

Case Histories: Settlement of Tall Buildings

The following reference presents case histories comparing predicated and measured settlement of tall buildings:

- Poulos, H.G., 2017, *Tall Building Foundation Design*, CRC Press.

The case histories include La Azteca Building in Mexico, Emirates Twin Towers in Dubai, Burj Khalifa in Dubai, Incheon 151 Tower in South Korea, and a tower on karstic limestone in Saudi Arabia. The subsurface conditions of the Incheon 151 Tower in South Korea are very similar to those in the San Francisco Bay Area.

The following is a summary of the conclusions from the reference:

- Characterizing the ground conditions accurately is more critical than the method of analysis employed to carry out the settlement calculations, provided that the analysis method is reasonably sound and properly reflects the mechanism of behavior.
- The following applies to immediate settlement of tall buildings on unsaturated soils: When there are large number of piles in a pile-supported mat foundation of a tall building, the calculation of interaction among piles can have a large influence on group settlement, i.e., overestimating group effects could result in overestimation of foundation settlement by a large factor. Also, assuming constant soil stiffness with depth could result in overestimation of foundation settlement (because of reduction in soil stress and strain, soil modulus at depth is much higher than soil modulus at shallow depth).
- Foundation settlement calculation performed in case of tall buildings supported on drilled shafts socketed into soft rock using pile load test data and finite element analysis compare fairly well with measured data, with predictions often being somewhat larger than recorded data.

Lessons learned from the Incheon 151 Tower with subsurface conditions similar to the San Francisco Bay Area indicate that commercially available computer programs could accurately predict foundation settlements consisting of drilled shafts socketed into soft rock by accounting for pile-soil-pile interaction effects.

PART 3:
Performance Expectation for New Buildings

Chapter 1

Introduction

1.1 Background

This Part examines the design requirements and expected seismic performance of new buildings designed and constructed according to the current *San Francisco Building Code* (SFBC) (CCSF, 2016). The Part describes the seismic assessment of two archetype buildings—a reinforced concrete shear wall (RCSW) residential building and a steel buckling restrained braced frame (BRBF) office building—the results of which are used to present the potential costs and benefits of higher performance goals for new tall building construction. This Part corresponds to Recommendation 1B presented in the Summary Recommendations.

This Part addresses aspects of the following tasks described in the *CAPSS Earthquake Safety Implementation Program's Workplan 2012-2042* (CCSF, 2011):

- Task B.6.a: Update codes for new buildings to reflect desired performance goals and acceptable confidence levels in meeting them

1.2 Seismic Performance Objectives

The primary objective of modern seismic codes is to ensure life safety under extreme earthquake events. Building code requirements for seismic design are primarily aimed at limiting the risk of structural collapse under severe earthquake ground motions and damage to structural and nonstructural components that can pose life-safety risks (e.g., falling hazards and blocked egress). Even when the code is satisfied, damage to structural, architectural, or mechanical components can still require extensive and costly repairs and impact building functionality after a large earthquake.

In San Francisco and other West Coast cities with high seismic hazards, most tall buildings constructed since about 2008 have been designed using a performance-based seismic design approach, which helps ensure reliability of building response under strong earthquakes. This is in contrast to conventional seismic design methods, which rely on prescriptive procedures and criteria that are less transparent in terms of the expected building performance. Although the performance-based procedures provide a greater degree of confidence in engineering design, their use in San Francisco (and other West Coast cities) is rarely intended to provide an enhanced level of performance. Rather, they are calibrated to provide performance similar to that achieved with conventional prescriptive design methods.

About 10% of the San Francisco tall building inventory (see Part 1) has been constructed or permitted since 2000, including a significant number of residential buildings. The continuing high demand and prices for both residential and commercial office space is likely to fuel continued development of tall buildings in San Francisco for the foreseeable future. In addition to the overall amount of new building construction, the increasing density of residential and commercial occupancies raises concerns that building damage and downtime due to earthquakes can have disproportionate effects on building residents and urban communities (see Part 5).

In addition to direct economic losses, building damage can lead to indirect economic losses due to downtime, defined as the time required to achieve a recovery state after an earthquake. Bonowitz (2011) defines three recovery states: (1) reoccupancy; (2) functional recovery; and (3) full recovery. Reoccupancy occurs when the building is deemed safe enough to be used for shelter, although functionality may not be restored. Functional recovery occurs when the building is repaired to restore most of its primary function, (i.e., it is operational). Full recovery occurs when the building is restored to its pre-earthquake condition. This Part focuses its evaluation on the intermediate functional recovery criteria to align with the City's tentative recovery goals (ATC, 2018).

Two recent studies of a hypothetical reinforced concrete shear wall residential building, representative of recently constructed buildings in San Francisco, indicate that the building may incur damage requiring repairs costing about 15% of building replacement cost under a design earthquake (Tipler, 2014) and 5% under a magnitude-7 Hayward fault earthquake (Almufti et al., 2018). The percent economic losses are not uncommon for earthquake loss assessment studies of modern buildings. The same studies indicate that the buildings may experience downtimes to functional recovery on the order of 84 weeks under the design earthquake and 33 weeks under the magnitude-7 Hayward earthquake. The studies highlight how damage that results in moderate repair costs could result in excessive downtimes, leading to displacement of residents with associated indirect costs.

Motivated by issues highlighted from these and other recent studies, this Part further examines the expected seismic performance of modern tall buildings and how the performance relates to current building code requirements.

1.3 Organization

Chapter 2 describes the designs of the archetypes buildings, the seismic assessment methodology, and results.

Chapter 3 summarizes seismic performance results from other studies that have investigated tall buildings.

Chapter 4 presents recommendations and potential cost implications for achieving improvements in the expected seismic performance of new tall buildings.

A list of references is provided at the end of this Part.

Chapter 2

Archetype Tall Buildings

2.1 Design Basis and Objectives

The archetype tall buildings investigated in this study represent trends in modern tall building construction: one is a reinforced concrete shear wall (RCSW) residential building and the second is a steel buckling-restrained braced frame (BRBF) office building. Two predominant forms of recent tall office building construction in San Francisco are: (a) concrete shear wall core with steel gravity framing (e.g., Salesforce Tower); and (b) steel BRBF systems (e.g., 181 Fremont and Oceanwide Center Buildings). In addition, the tall building inventory in Part 1 demonstrates that 17 buildings adopting a RCSW system have been constructed or permitted for construction in San Francisco since the year 2000. In this study, the BRBF system is selected as representative of office buildings because the RCSW system is partially represented by the residential concrete archetype. Both archetype buildings were designed to comply with current *San Francisco Building Code* (SFBC) seismic design requirements.

In San Francisco, new buildings are designed using either a code-prescriptive or a performance-based seismic design approach. Prescriptive seismic designs adhere to the design methods and materials prescribed by the national standard ASCE/SEI 7-10, *Minimum Design Loads for Buildings and Other Structures* (ASCE, 2010), which is adopted by the SFBC. Performance-based seismic design employs advanced nonlinear structural analysis to design structural building systems that are capable of the same (or better) seismic performance as a code-prescriptive structural design. In addition to providing more reliable assessment of seismic performance, the performance-based method allows designers to employ structural systems that are not allowed by traditional prescriptive requirements. The performance-based methods are permitted by SFBC Section 104A.2.8, which allows alternative materials and methods of construction as approved by the building official. Despite these different analysis methods and design criteria, buildings constructed using code-prescriptive or performance-based designs are considered to be equivalent in their structural safety and adherence to minimum building code standards. San Francisco's Administrative Bulletin (AB) 083, *Requirements and Guidelines for the Seismic Design of New Tall*

Buildings using Non-Prescriptive Seismic-Design Procedures (CCSF, 2008a) provides basic requirements and guidelines for the seismic design of tall buildings following performance-based seismic design procedures, which are typically required to have an independent, third-party peer review to help confirm that the proposed building's structural system meets the minimum code safety standards and requirements. AB 082, *Guidelines and Procedures for Structural Design Review* (CCSF, 2008b), establishes guidelines for the design peer review process. Most, if not all, projects that are permitted by AB-083 will follow *Guidelines for Performance-Based Seismic Design of Tall Buildings* (PEER, 2017).

As outlined in AB 083, the performance-based seismic design approach entails evaluations under three levels of earthquake ground shaking: a Service Level Earthquake (SLE), a code-level earthquake, referred to as a Design Earthquake (DE), and a Maximum Considered Earthquake (MCE). The implicit goals are to provide "collapse-prevention" under MCE motions, life-safety in a DE event, and limited structural damage under SLE. These checks and the associated acceptance criteria are intended to represent the earthquake performance implied by national standards, such as ASCE/SEI 7-10, that are referenced by the SFBC.

The SLE evaluation is required to demonstrate acceptable seismic performance for moderate earthquakes. It evaluates anticipated seismic performance under a seismic hazard level that can be reasonably expected during the service lifetime of a building (nominally, ground motions with a 50% probability of being exceeded in 30 years).

The DE evaluation is used to identify the exceptions being taken to the prescriptive requirements of the SFBC and to define the minimum required strength and stiffness for earthquake resistance. Minimum strength and stiffness requirements are defined according to SFBC. Providing a non-prescriptive seismic design with minimum strength and stiffness comparable to code-prescriptive designs helps produce seismic performance at least equivalent to the code. Minimizing the number of exceptions to prescriptive requirements also helps achieve this aim. As specified in ASCE/SEI 7-10, the DE-level ground motions are defined as two-thirds of the MCE-level ground motions. For a Site Class D location in downtown San Francisco, the DE-level motions have roughly a 10% probability of being exceeded in 50-years.

The MCE evaluation is intended to verify that the structure has an acceptably low probability of collapse under severe earthquake ground motions. The evaluation uses nonlinear response-history analysis to demonstrate an acceptable mechanism of nonlinear lateral deformation and to determine the maximum forces to be considered for structural elements and actions designed to remain elastic. As determined according to requirements of ASCE/SEI 7-10, the MCE-level ground motions for a Site Class D location in downtown San Francisco have roughly a 4% probability of being exceeded in 50-years (similar in shaking intensity to extreme ground motions experienced in the 1906 magnitude-7.9 San Francisco earthquake).

Table 3-1 provides an overview of seismic design requirements at the three levels of seismic evaluation per AB-083. The table provides an overview of the objective of each evaluation, analysis methods, drift limits, acceptance criteria, strength reduction factor and relevant code or guidance documents. More complete details of the design requirements can be found in SFBC and *Guidelines for Performance-based Seismic Design of Tall Buildings* (PEER, 2010).

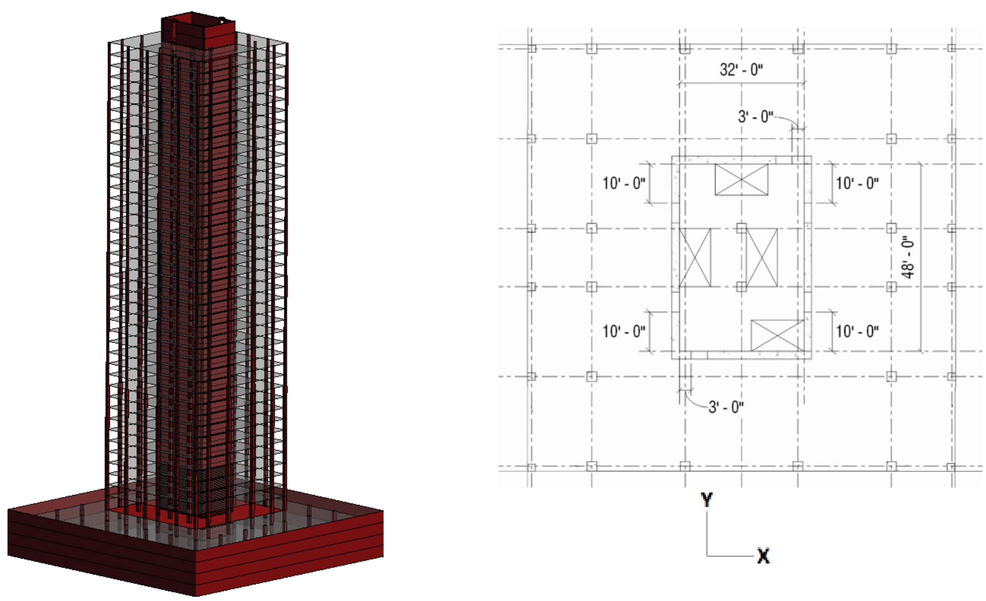
Table 3-1 Performance-Based Design Criteria

	Service Level Earthquake[1] (SLE)	Design Earthquake[2] (DE)	Maximum Considered Earthquake[2] (MCE)
Overall Objective	*Limited Damage*	*Code Compliance (Life-Safety)*	*Collapse Prevention*
Analysis Method	Linear (Response Spectrum)	Linear (Response Spectrum)	Nonlinear Response History
Response Modification Factor	---	Code-prescribed *R*-factor	---
Story Drift Limits	< 0.5%	< 2%	Mean Transient < 3% Mean Residual < 1%
Component Acceptance Criteria	Demand < 1.5 Nominal Strength[3]	Demand < Design Strength	Force and Deformation-Controlled Component Checks
Code or Guidance Document	*PEER Guidelines* (2017)	ASCE/SEI 7-10	*PEER Guidelines* (2017)

1. In *PEER Guidelines* (2017), the SLE ground motion is specified based on a mean annual return period of 43 years, corresponding to a 50% probability of exceedance in 30 years.
2. In ASCE/SEI 7-10, the MCE ground motion is determined by a combination of criteria. For a Site Class D soil site in downtown San Francisco, the MCE ground motion intensity is generally governed by the 84th percentile hazard from a magnitude-8 earthquake on the San Andreas fault, corresponding to a return period on the order of 1,200 years (~4% probability of exceedance in 50 years). For a Site Class B rock site, the MCE intensity is generally governed by the risk targeted ground motion, corresponding to a return period on the order of 2,500 years (~2% probability of exceedance in 50 years). The DE ground motion is specified as two-thirds of the MCE intensity, which equates to about a 500-year return period (~10% probability of exceedance in 50 years).
3. In *PEER Guidelines* (2017): Demand < 1.5 Nominal Strength (i.e., no strength reduction factor is applied at the service-level check). In *PEER Guidelines* (2010): Demand < 1.5 Design Strength (i.e., strength reduction factor is applied).

2.2 Reinforced Concrete Shear Wall Residential Building

The reinforced concrete shear wall (RCSW) structure evaluated in this study is a core wall residential building designed to comply with the requirements outlined in Section 2.1. The structure was originally designed by Magnusson Klemencic Associates (MKA) for a site in Los Angeles (PEER, 2011) and later re-designed for a site in San Francisco by Tipler (2014). The structure consists of 42 stories above grade and four basement levels. Story heights are approximately 10 ft, resulting in an overall height of 457 ft above the ground (including the roof bulkhead). The gravity system consists of 8-inch-thick slabs (measuring ~108 ft × 107 ft in the superstructure plan) supported by reinforced concrete columns with sections typically ranging from 36 in × 36 in near the base to 18 in × 18 in at the roof. The seismic-force-resisting system consists of coupled shear walls 32 inches thick up to floor 13 and 24 inches thick from floor 13 to the roof. The coupling beams are 30 inches in depth in the superstructure and 34 inches in the basement. Nominal compressive strength of concrete in the core is 8.0 ksi, and nominal yield strength of steel in the core and coupling beams is 60 ksi and 75 ksi, respectively. Figure 3-1 shows an isometric view of the building and its superstructure plan view. The dynamic vibration periods of the RCSW building are summarized in Table 3-2.



(a) (b)

Figure 3-1 Reinforced concrete shear wall residential building (a) isometric view and (b) superstructure plan view. Adapted from Almufti et al. (2018).

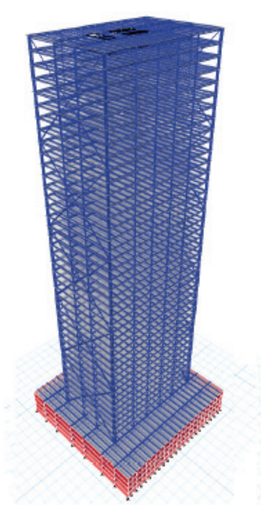
Table 3-2 Dynamic Vibration Periods of the RCSW Building

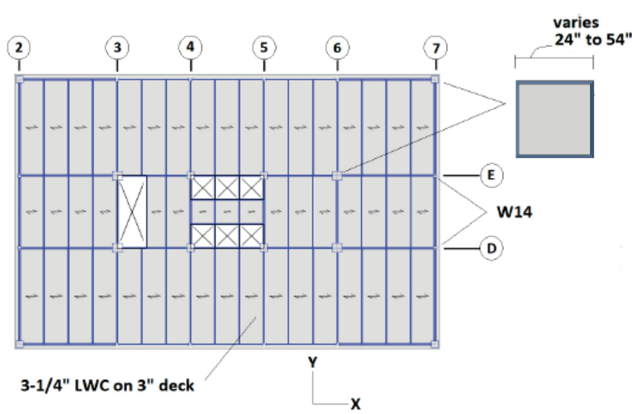
Period (s)	
Strong Direction (Y-axis)	Weak Direction (X-axis)
$T_1 = 4.37$	$T_1 = 5.27$
$T_2 = 0.93$	$T_2 = 1.10$

The core-only RCSW system is designed to dissipate energy through plastic hinging at the base of the wall piers and at the ends of the coupling beams. A 3-dimensional model of this structure, developed by Almufti et al. (2018) for the recently completed USGS HayWired study (USGS, 2018), was used to evaluate the response of the structure at DE and MCE by means of nonlinear response history analysis (NLRHA) in LS-DYNA (LSTC, 2009). In this model, the wall piers are modeled with distributed plasticity fiber beam-column elements, with fibers representing steel reinforcement and concrete with different levels of confinement. The coupling beams are modeled with concentrated plasticity elements, where the hinge response is validated against testing at the University of California Los Angeles (Naish et al., 2009) to reproduce their hysteretic behavior. Linear elastic beam-column elements were used to model the columns and linear elastic shell elements to model the slabs. More details of the archetype building design and modeling approach can be found in Tipler (2014) and Almufti et al. (2018).

2.3 Steel Buckling-Restrained Braced Frame Office Building

The steel buckling-restrained braced frame (BRBF) structure was designed following the requirements outlined in Section 2.1. The building has 40 stories above the grade and four basement levels. Story heights are typically 13.5 feet, resulting in an overall height of 545 feet above the ground. The gravity system consists of a composite floor slab (3.25-inch slab on 3-inch deck) supported by simply supported steel beams and wide flange steel columns. The seismic-force-resisting system consists of a buckling-restrained brace (BRB) core in the center of the building and mega-brace BRB configuration on the perimeter in the Y-direction, as shown in Figure 3-2.





(a) (b)

Figure 3-2 Buckling-restrained braced frame office building (a) isometric view and (b) superstructure plan view.

The core areas of the BRBs of the interior frames ranges from 6 in^2 to 12 in^2, and the core area of the exterior mega-brace BRBs ranges from 50 in^2 to 9 in^2. Most of the columns are box shaped with concrete fill. Nominal compressive strength of concrete fill is 10 ksi, and the steel for framing and braces are A992 and A36, with yield strengths of 50 ksi and 36 ksi, respectively. The dynamic properties of the steel BRBF building are summarized in Table 3-3.

Table 3-3 Dynamic Vibration Periods of the Steel BRBF

Period (s)	
Strong Direction (Y-axis)	Weak Direction (X-axis)
T_1 = 4.08	T_1 =4.53
T_2 = 1.35	T_2 =1.54

The steel BRBF system dissipates energy primarily by yielding of braces, while all other elements in the system are capacity designed to remain essentially elastic. Planar (2-dimensional) nonlinear models of each principal building direction were developed to evaluate the response of the structure at DE and MCE by means of NLRHA in OpenSees (McKenna et al., 2000). In these models, frame elements were simulated using concentrated plasticity elements, with hinge properties calibrated per the recommendations of Lignos and Krawinkler (2010, 2011) for W-section and tubular square columns. The truss elements used to simulate the BRBs were calibrated per the recommendations of Terashima (2018), using BRB test data. Finite-rigid offsets are modeled at ends of BRBs to represent the non-yielding zone, and rigid offsets are applied to connections between BRBs and frame elements to consider the presence of gusset plates. The planar frame models incorporated leaning columns to represent P-Δ effects, but otherwise, the gravity systems were not modeled.

2.4 Assessment Methodology

The RCSW residential building (Section 2.2) and steel BRBF office building (Section 2.3) comply with seismic design requirements for two locations in San Francisco with Site Classes D and B. The Site Class is a classification assigned to a site based on the soils present and their engineering properties to a depth of 100 feet. Site Class D is representative of a site with stiff soil (with shear wave velocities in the top 100 feet of soil in the range of 600 to 1,200 ft/s), and Site Class B is representative of a rock site (with shear wave velocities in the top 100 feet of soil in the range of 2,500 to 5,000 ft/s). Figure 3-3 illustrates the archetype building site locations superimposed with the City's existing tall building inventory (See Part 1). While the majority of the

City's tall buildings are located on Site Class D, the comparison to Site Class B is useful to: (1) show the variability of performance that can occur in San Francisco; and (2) provide insight on the performance of a building that is essentially over-designed for its site, in other words a "better than code" building.

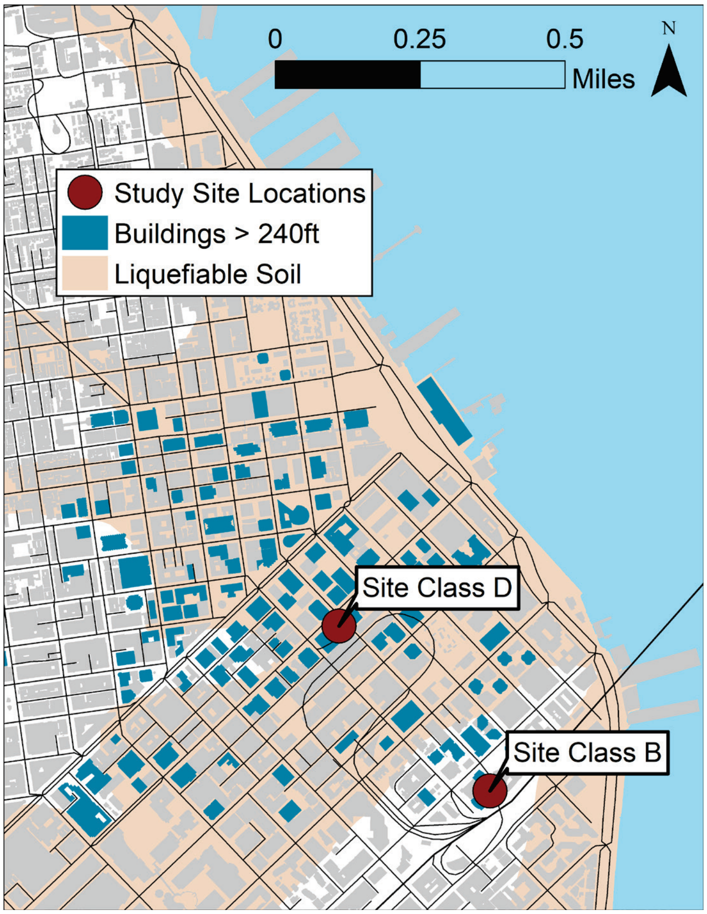
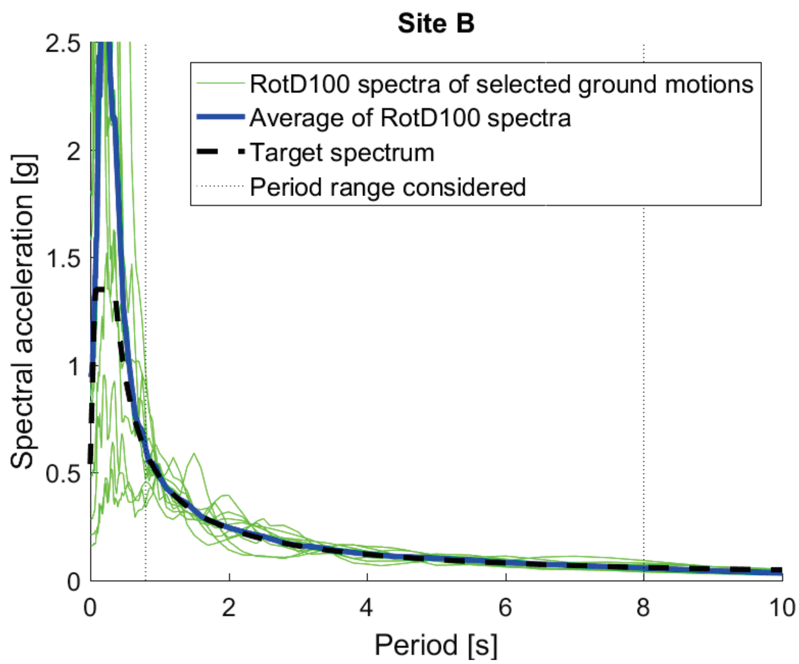


Figure 3-3 Site locations of archetype buildings in downtown San Francisco.

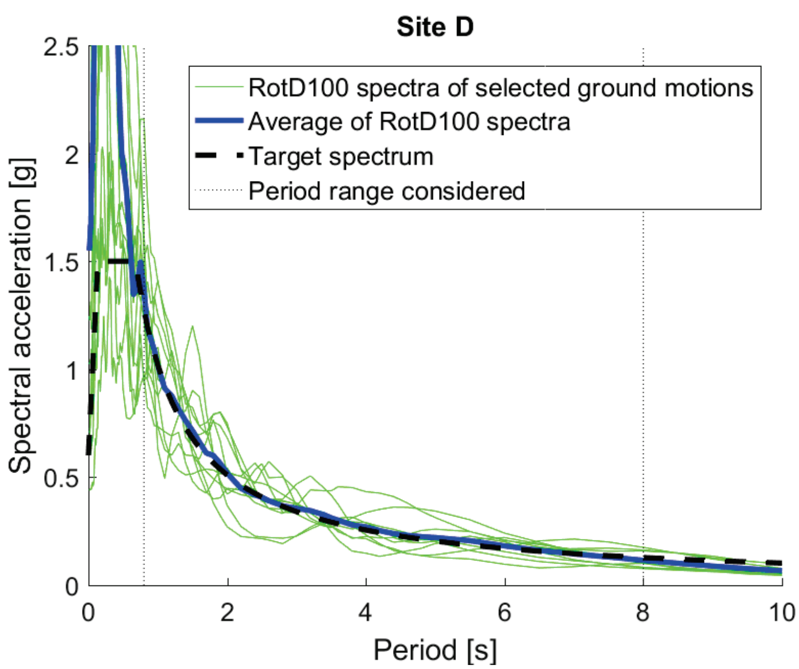
Since soft soils tend to amplify earthquake ground motion shaking, the location with Site Class D has significantly higher shaking intensity than the location with Site Class B. In spite of their distinct seismic hazard, the designs of the archetype RCSW and BRBF buildings, introduced in Sections 2.2 and 2.3 respectively, are similar for both site classes due to minimum code-prescribed base shear requirements and other design constraints (i.e., wind loads that control the design of the BRBF system). To the extent that the designs could be further optimized, the archetype buildings are slightly overdesigned for the ground motions at Site Class B. Nevertheless, the performance assessment for the buildings at the two sites (B and D) are considered to be representative of what could be expected from new code-conforming buildings at each site. In addition, for the purpose of evaluating the possible benefits of designing buildings with more stringent seismic design criteria (e.g., smaller drift limits, more rugged elevator systems), the relative performance of the buildings across the two sites provides insights into how the building performance could be improved.

To demonstrate compliance with current performance-based seismic design requirements, building drifts and member force and deformation demands were evaluated at MCE shaking. Two suites of eleven ground motions were selected and linearly scaled for each of the Site Classes, B and D. Figure 3-4 shows the target MCE spectrum, individual ground motion spectra (noted RotD100 in the figures), as well as the average ground motion suite spectra, which closely follows the target spectrum across the period range of interest from 1 to 8 seconds. Following *ASCE/SEI 7-10*, the SFBC defines DE shaking as two-thirds of MCE. To evaluate performance at DE shaking, the MCE target spectrum and associated ground motion suite, as illustrated in Figure 3-4, were scaled accordingly by a factor of two-thirds. In the figures, RotD100 denotes the maximum direction spectral acceleration, which is the greatest unidirectional acceleration value for any possible orientation of a pair of horizontal ground motions and forms the basis of code design spectra. A unique ground motion identifier (per the PEER 2013 ground motion database), the earthquake event, its magnitude and distance (from the seismic source to the site of the recording), as well as the corresponding scale factor applied to match the target spectrum are summarized in Tables 3-4 and 3-5 for Site Classes B and D, respectively.

The expected damage and associated economic losses in the archetype buildings are evaluated based on the FEMA P-58 (FEMA, 2012) methodology as implemented in the *Seismic Performance Prediction Program* (SP3) (HBrisk, 2018). This method is based on a building performance model, which is a collection of structural and nonstructural building components that are susceptible to seismic damage. Definition of the structural components is based on the structural design of each building, as presented in Sections 2.2 and 2.3 for the RCSW and the BRBF, respectively. The nonstructural components are defined using the *Normative Quantity Estimation Tool* of FEMA P-58, which is based on a database of typical nonstructural components in approximately 3,000 buildings of different occupancies. The resulting nonstructural component definitions were reviewed for consistency with previous studies on expected seismic performance of tall buildings similar to those under evaluation (Tipler, 2014; PEER, 2011). Certain nonstructural components required the calculation of parameters to characterize their seismic resistance. These were calculated automatically within SP3, which follows ASCE/SEI 7-10, Chapter 13: Seismic Design Requirements for Nonstructural Components.



(a)



(b)

Figure 3-4 Maximum Considered Earthquake (MCE) target spectrum superimposed with the average and individual ground motion spectra for (a) Site Class B; and (b) Site Class D locations.

Table 3-4 Site Class B Selected Ground Motions

Sequence Number	Earthquake Event	Scale Factor	Earthquake Magnitude	Distance (km)
286	Irpinia (Italy)	1.99	6.90	21.26
1511	Chi-Chi (Taiwan)	0.74	7.62	2.74
1521	Chi-Chi (Taiwan)	1.30	7.62	9.00
1541	Chi-Chi (Taiwan)	0.97	7.62	12.38
3750	Cape Mendocino (USA)	1.02	7.01	25.91
3947	Tottori (Japan)	1.83	6.61	5.86
3964	Tottori (Japan)	2.15	6.61	11.29
4852	Chuetsu-oki (Japan)	4.00	6.80	32.54
4882	Chuetsu-oki (Japan)	2.04	6.80	23.44
6928	Darfield (New Zealand)	2.32	7.00	25.67
8166	Duzce (Turkey)	2.10	7.14	3.58

Table 3-5 Site Class D Selected Ground Motions

Sequence Number	Earthquake Event	Scale Factor	Earthquake Magnitude	Distance (km)
139	Tabas (Iran)	4.00	7.35	13.94
143	Tabas (Iran)	0.81	7.35	2.05
286	Irpinia (Italy)	4.00	6.90	21.26
1165	Kocaeli (Turkey)	3.12	7.51	7.21
1511	Chi-Chi (Taiwan)	1.64	7.62	2.74
1521	Chi-Chi (Taiwan)	2.75	7.62	9.00
1541	Chi-Chi (Taiwan)	3.18	7.62	12.38
3750	Cape Mendocino (USA)	2.49	7.01	25.91
4848	Chuetsu-oki (Japan)	3.51	6.80	17.93
4882	Chuetsu-oki (Japan)	4.00	6.80	23.44
8166	Duzce (Turkey)	4.00	7.14	3.58

In the FEMA P-58 methodology, performance estimates of the structural and nonstructural building components are mathematically represented with fragility functions. A fragility function is a statistical distribution that indicates the conditional probability of incurring a damage state at a given value of demand. From each damage state, the associated repair costs and times are estimated by means of consequence functions. In addition to the family of fragilities recommended by the *Normative Quantity Estimation Tool*, user defined components were implemented to more accurately characterize damage to elevators, mega braces, and façades.

The fragility function provided in FEMA P-58 to predict damage to the elevator cabin is a function of peak ground acceleration developed based on a review of elevator performance in low- and mid-rise buildings. To better represent conditions in tall buildings, an additional fragility function was introduced to estimate elevator shaft rail damage as a function of residual story drift as outlined in Almufti et al. (2018). Residual story drift ratios were derived from peak transient story drifts as outlined in Appendix C of FEMA P-58.

User-defined fragility functions were developed to predict damage to the interior BRBs (Almeter et al., 2018) and exterior mega BRBs (Hooper, 2018) The latter functions are intended to recognize the differences between BRB mega-braces and single story BRBs for which fragility functions in FEMA P-58 were originally developed.

The parameters required to define fragility functions for the curtain wall system are based on those outlined in Almufti et al. (2018), since the standard curtain wall functions of FEMA P-58 where developed for low- and mid-rise construction. While damage state definitions and consequence data are consistent with that provided by FEMA, the drift values associated with each damage state are defined in accordance with the performance-based building design requirements.

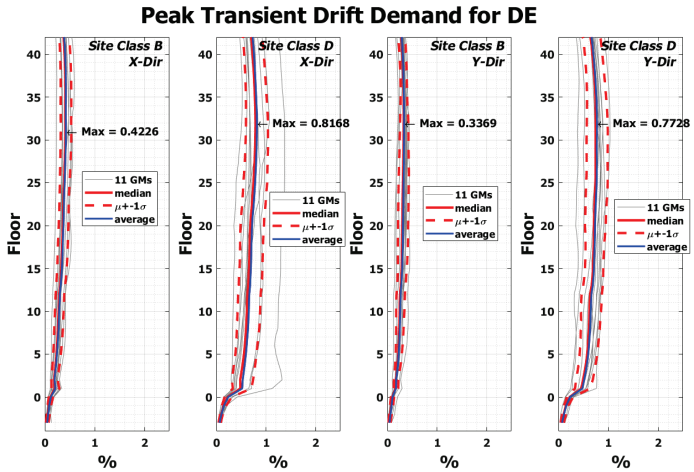
The input demands to fragility functions are generally referred to as engineering demand parameters (EDP), and are obtained from analysis. Different EDPs are used for different components, depending on their ability to predict damage (e.g., damage to acceleration-sensitive components can be estimated by peak floor accelerations).

The following EDPs are used to evaluate performance in the RCSW residential building: story drifts, residual drift, damageable wall drift, racking drift, coupling beam rotation, and floor acceleration. Table 3-6 summarizes the maximum value (throughout the building height) of the average set of results (from the eleven ground motions) in each story for each of the EDPs considered in the performance evaluation of the RCSW in Site Class B and D under DE and MCE. There is a subtle difference in the way the residual drifts are computed. The residual drifts noted in Table 3-6 are calculated by obtaining the maximum residual drift (throughout the building height) in a single ground motion simulation and then taking the average from the eleven ground motions (a single residual drift parameter is input into the loss model for each ground motion in each building direction). Figure 3-5 illustrates the distribution of maximum transient story drifts, racking story drifts, and floor accelerations up the building height.

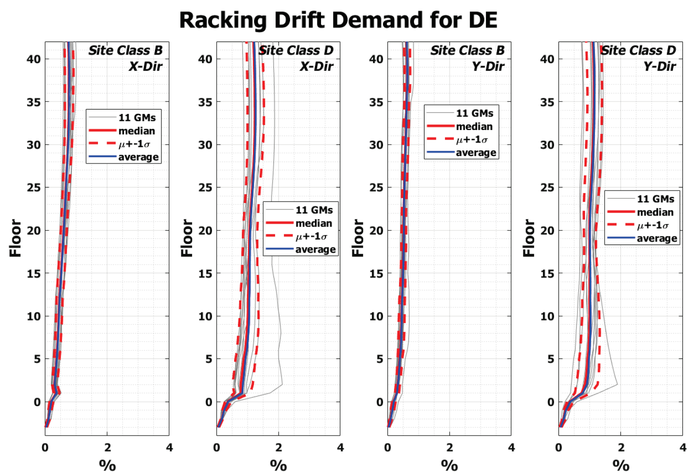
Table 3-6 Engineering Demand Parameters for Evaluation of the Reinforced Concrete Shear Wall Residential Building

Engineering Demand Parameters	Site Class	Ground Motion Intensity	Mean of Max x-Direction	Mean of Max y-Direction
Transient Story Drift	D	DE	0.82 %	0.77 %
		MCE	1.18 %	1.14 %
	B	DE	0.42 %	0.34 %
		MCE	0.58 %	0.49 %
Residual Story Drift	D	DE	0.10 %	0.09 %
		MCE	0.24 %	0.24 %
	B	DE	0.00%	0.00%
		MCE	0.04 %	0.02 %
Damageable Wall Drift	D	DE	0.21 %	0.24 %
		MCE	0.64 %	0.66 %
	B	DE	0.09 %	0.11 %
		MCE	0.14 %	0.17 %
Racking Story Drift	D	DE	1.27 %	1.16 %
		MCE	1.71 %	1.66 %
	B	DE	0.78 %	0.64 %
		MCE	1.01 %	0.86 %
Coupling Beam Rotation	D	DE	0.012 rad	0.012 rad
		MCE	0.020 rad	0.019 rad
	B	DE	0.006 rad	0.005 rad
		MCE	0.009 rad	0.008 rad
Peak Floor Acceleration	D	DE	1.60 g	1.68 g
		MCE	2.31 g	2.24 g
	B	DE	1.14 g	0.88 g
		MCE	1.62 g	1.31 g

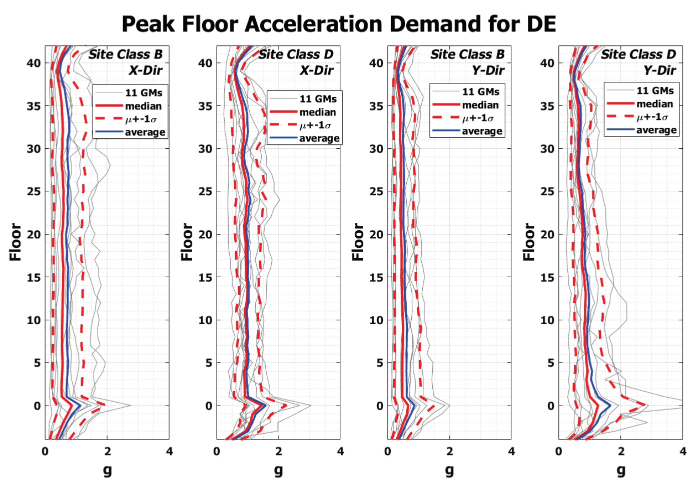
The following EDPs are used to evaluate performance in the BRBF office building: transient story drift, residual story drift, and floor acceleration. Table 3-7 summarizes the maximum value (throughout the building height) of the average set of results (from the eleven ground motions) for each of the EDPs for the two sites (Site Class B and D) and earthquake intensities (DE and MCE). As with the RCSW, there is a subtle difference in the way the residual drifts are computed. The residual drifts noted in Table 3-7 are calculated by obtaining the maximum residual drift (throughout the building height) in a single ground motion simulation and then taking the average from the eleven ground motions (a single residual drift parameter is input into the loss model for each ground motion in each building direction). Figure 3-6 illustrates the responses of the BRBF for the DE.



(a)



(b)



(c)

Figure 3-5 Earthquake demands from non-linear response history analysis of the reinforced concrete shear wall residential building under the design earthquake: (a) transient story drifts; (b) racking story drifts; and (c) floor accelerations.

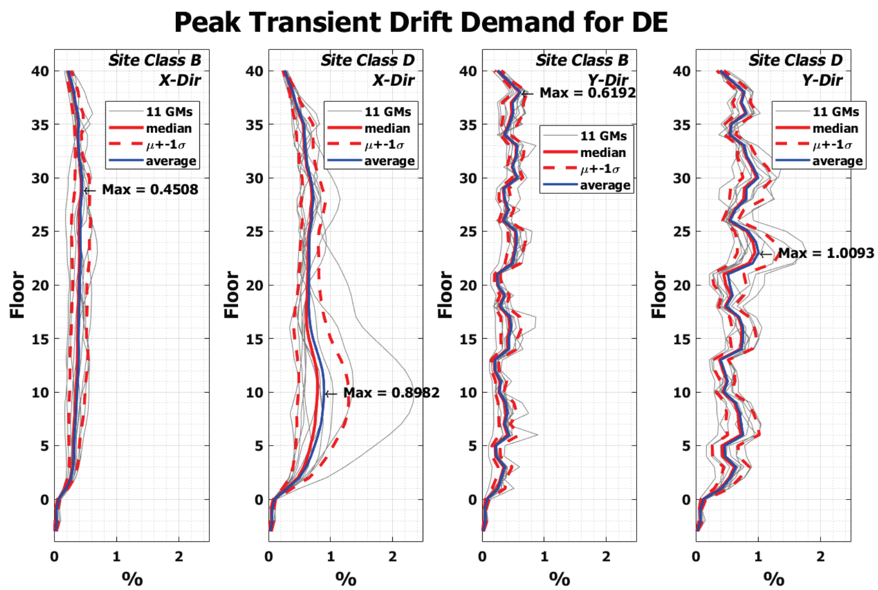
Table 3-7 **Maximum Engineering Demand Parameters for Evaluation of the Steel Buckling-Restrained Braced Frame Office Building**

Engineering Demand Parameters	Site Class	Ground Motion Intensity	Mean of Max x-Direction	Mean of Max y-Direction
Transient Story Drift	D	DE	0.90 %	1.01 %
		MCE	1.82 %	1.28 %
	B	DE	0.45 %	0.62 %
		MCE	0.58 %	0.75 %
Residual Story Drift	D	DE	0.05 %	0.06 %
		MCE	0.40 %	0.14 %
	B	DE	0.00 %	0.00 %
		MCE	0.00 %	0.01 %
Floor Acceleration	D	DE	0.71 g	1.12 g
		MCE	1.06 g	1.67 g
	B	DE	0.56 g	0.93 g
		MCE	0.78 g	1.33 g

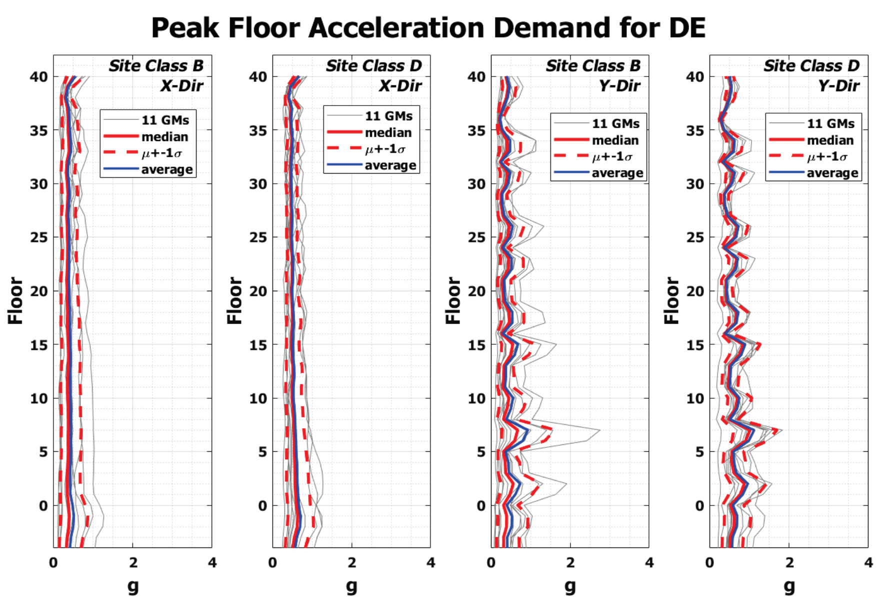
The methodology assigns a repair class to each damage state of the structural and nonstructural building components. The repair classes identify damage levels that hinder achieving a certain recovery state, where the building recovery is predicated on all the components achieving the associated recovery state. The repair class associated with each damage state for all components in the building are based on Almufti et al. (2018).

The methodology also considers a streamlined sequence of repairs on a floor-per-floor basis. The necessary repair activities are grouped by trade, including structural, façade (exterior partitions and cladding), egress (stairs and elevators), MEP, and office fitouts. Once structural repairs are complete, other repair activities can be carried out in parallel with limits to the number of workers per trade and the total number of workers on site.

In addition to repairs, the methodology identifies a series of impeding factors that may delay the initiation of repair work, such as post-earthquake inspection, engineering mobilization, contractor mobilization, financing, permitting and long-lead-time components. These impeding factors are grouped into three delay sequences, as illustrated in Figure 3-7, the longest of which controls the impeding factor delays. All delay sequences begin with post-earthquake inspection. Following post-earthquake inspection, the first sequence of delays considers time to mobilize an engineer, to carry out a detailed structural evaluation and any necessary design work, as well as delays associated with permitting such work. The second sequence considers the time associated with the mobilization of a contractor to carry out repair work as well as the procurement of specialty items (i.e., long-lead components). The last sequence relates to the financing of repair work. The median impeding factor delay estimates for each impeding factor assumed in this study are summarized in Table 3-8. Note that in addition to the baseline case, a recovery-planning option is considered to assess the benefits of implementing measures to minimize different impeding factor delays.



(a)



(b)

Figure 3-6 Earthquake demands from non-linear response history analysis of the steel buckling-restrained braced frame building under the design earthquake: (a) transient story drifts and (b) floor accelerations.

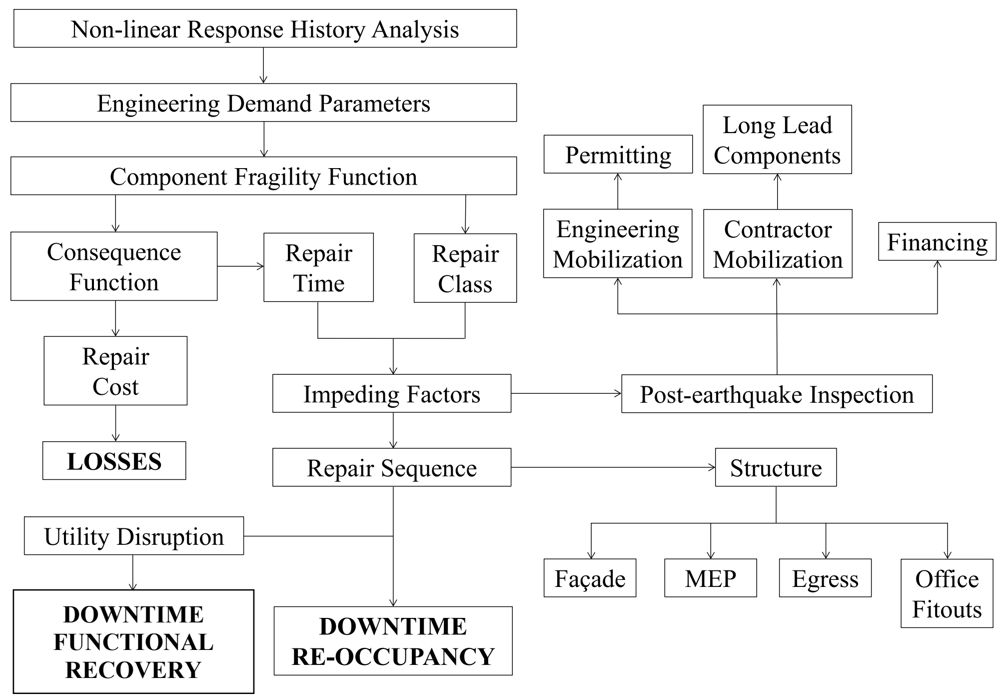


Figure 3-7 Loss and downtime assessment methodology. Adapted from Molina Hutt et al. (2016).

Table 3-8 Median Impeding Factor Delay Estimates for the Baseline Case and with Recovery-Planning Measures in Place

Impeding Factor Delays	Baseline Case	Recovery Planning
Inspection	5 days	1 day
Engineering Mobilization weighted by [cosmetic/significant] damage	[6/12] weeks	[2/4] weeks
Contractor Mobilization weighted by [cosmetic/significant] damage	[28/40] weeks	[3/7] weeks
Permitting binary for [cosmetic/significant] damage	[1/8] weeks	[0.5/4] weeks
Financing [only triggered for >10% costs]	15 weeks	1 week

The overall loss and downtime assessment methodology is graphically illustrated in Figure 3-7. The total building downtime is calculated as the sum of: (1) the time to repair damaged components that hinder achieving a recovery state; and (2) the longest sequence of impeding factor delays that control the overall downtime. Apart from repairs to the building itself, important contributors to the functional recovery time of the building are disruptions in water supply, electricity or natural gas distribution systems. However, delays associated with utility disruption or long-lead components are not evaluated in this study.

While the downtime assessment methodology outlined in REDi (Arup, 2013) was used as the basis to evaluate the time to functional recovery in this study, a number of modifications were introduced where limitations to the method were identified in its application to tall buildings. Key changes are as follows:

- **Mobilization Estimates.** Impeding factor delays associated with engineer and contractor mobilization are calculated by weighting the expected delays according to the percentage of components that experience minor (cosmetic) damage versus more severe damage that would prevent functionality. This change is introduced to scenarios where severe damage to a single component would otherwise trigger unrealistic impeding factors (e.g., one coupling beam in the RCSW building triggering 12 weeks of impeding time for engineering mobilization and 40 weeks for contractor mobilization).
- **Financing.** The methodology requires conducting realizations, where each realization represents one possible performance outcome for the building considering a single combination of possible values of each variable considered. For realizations where the loss ratio is less than 10% of building replacement cost, it is assumed that building owners would have sufficient capital to cover repair costs, as opposed to extensive delays associated with financing (on the order of 15 weeks).

Because there are many factors that can affect performance, such as intensity of ground shaking, building construction quality, building response, or vulnerability of contents, there is significant uncertainty in the predicted performance of the building. This uncertainty can be accounted for by means of a Monte Carlo simulation by conducting thousands of realizations. Following this process, results can be expressed as a performance function (i.e., probability of losses or downtime being less than or equal to a specified value as a result of an earthquake). The results presented in Section 2.5 focus on mean, median, and 90th percentile estimates of response.

2.5 Loss and Downtime Results

The methods discussed in Section 2.4 to estimate building losses were used to evaluate the expected performance of the RCSW residential and steel BRBF office buildings across the two sites, Site Class B and D, and two ground motion shaking intensities, DE and MCE. Table 3-9 provides a summary of the median loss results associated with each archetype building, Site Class and intensity level. The maximum story drift value (throughout the building height) of the average set of results (from the eleven ground motions) is also shown to gage the relationship between drift demands and expected losses. Loss results are normalized over total building replacement costs, which are estimated at $215 million ($312/square feet) for the RCSW residential building and $280 million ($341/square feet) for the BRBF office building. The results indicate that expected losses in the BRBF are considerably lower than in the RCSW. For instance, at Site Class D under DE shaking, the expected losses in the RCSW building are 8.1% versus 2.8% in the BRBF building. While these differences are due in part to the differences in construction costs, the expected absolute losses in the residential building are still approximately twice those of the office building, as illustrated in Figure 3-8.

Table 3-9 Summary of Median Loss Results and Associated Drifts

Archetype Building	Site Class	Drift* (% story height)		Median Loss (% replacement cost)	
		DE	*MCE*	*DE*	*MCE*
RCSW	B	0.4 (0.8)	0.6 (1.0)	2.8	4.9
	D	0.8 (1.3)	1.2 (1.7)	8.1	15.5
BRBF	B	0.6	0.8	0.9	1.8
	D	1.0	1.8	2.8	5.8

* Racking drifts shown in paranthesis.

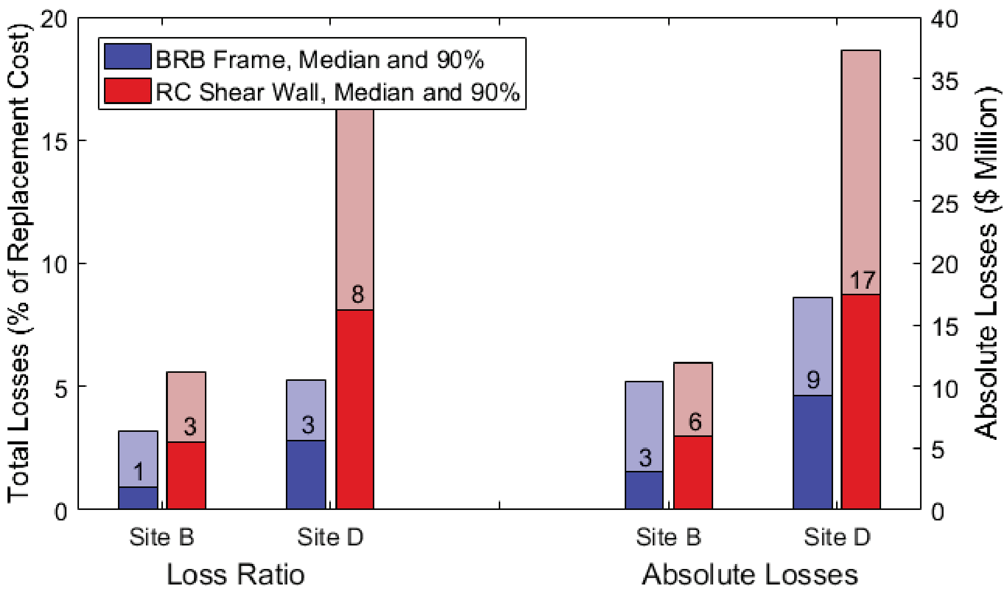


Figure 3-8 Summary of median and 90th percentile loss results (loss ratio and absolute losses) under DE shaking.

The loss methodology enables deaggregating losses into their constituent parts, which can help inform loss mitigation strategies. Figure 3-9 illustrates the average contribution of different building components to the overall expected loss under DE shaking. Losses in the RCSW building are dominated by damage to structural components and interior finishes, particularly those components that are sensitive to racking drifts, which are amplified by about 1.5 to 2 times over story drifts. Racking drift deformations occur in the RCSW structure due to differences in axial deformation (elongation) between the concrete walls and the gravity framing. The amplified racking cause damage and losses associated with interior partition wall finishes and slab-to-column connections. Losses in the steel BRBF, which are about half of those for the RCSW, are dominated by damage to cladding components (minor damage primarily associated with gasket seal failure), plumbing, and HVAC. Note that while elevator damage is a relatively small contributor to damage in both buildings, it is a more significant contributor to downtime.

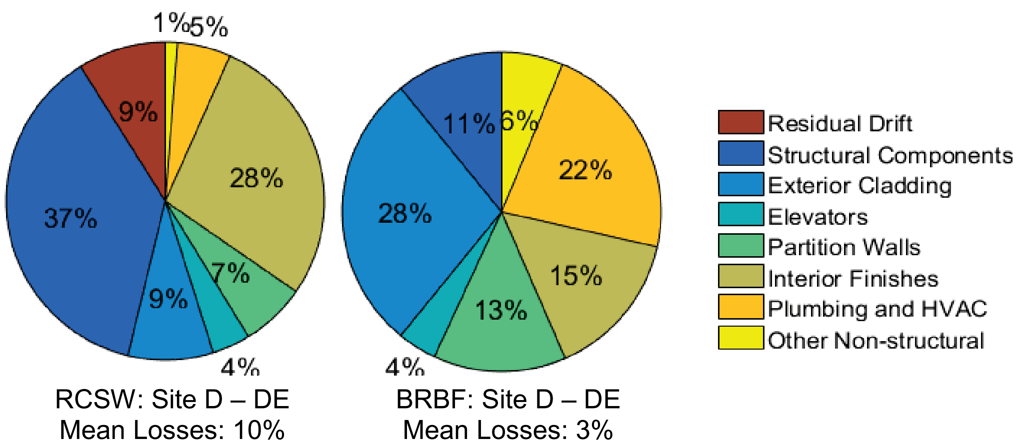


Figure 3-9 Contribution of different building components to overall loss at DE for Site Class D.

Figure 3-9 shows that residual drift contributes to the overall losses in the RCSW, but not in the steel BRBF. While the probability of residual drift rendering the building irreparable for the RCSW building in Site Class D under DE is less than 1%, because such a realization results in total building replacement cost, the contribution to the mean loss statistics, as seen in Figure 3-9, is significant.

Collapse realizations would also represent a total loss. However, based on the nonlinear response history analysis results under MCE, the probability of collapse under the ground motion shaking intensities considered is assumed to be zero for both buildings and site classes considered.

The repair costs represented in Figures 3-8 and 3-9 are one type of direct loss, downtime is another. The time needed to achieve reoccupancy and make the repairs needed for functional recovery is good measure of the impact of tall buildings on the recovery of San Francisco and its downtown neighborhoods. The median repair times (for components that inhibit functional recovery), and functional recovery times (which include delays associated with different impeding factors) are summarized in Table 3-10.

Table 3-10 Summary of Median Repair Times and Functional Recovery Time

Archetype Building	Site Class	Median Repair Time (days)		Median Time to Functional Recovery (days)		Reduced Median Time to Functional Recovery (days)	
		DE	*MCE*	*DE*	*MCE*	*DE*	*MCE*
RCSW	B	24	34	113	154	52	80
	D	50	280	194	464	129	408
BRBF	B	13	20	49	101	21	43
	D	29	118	134	245	68	171

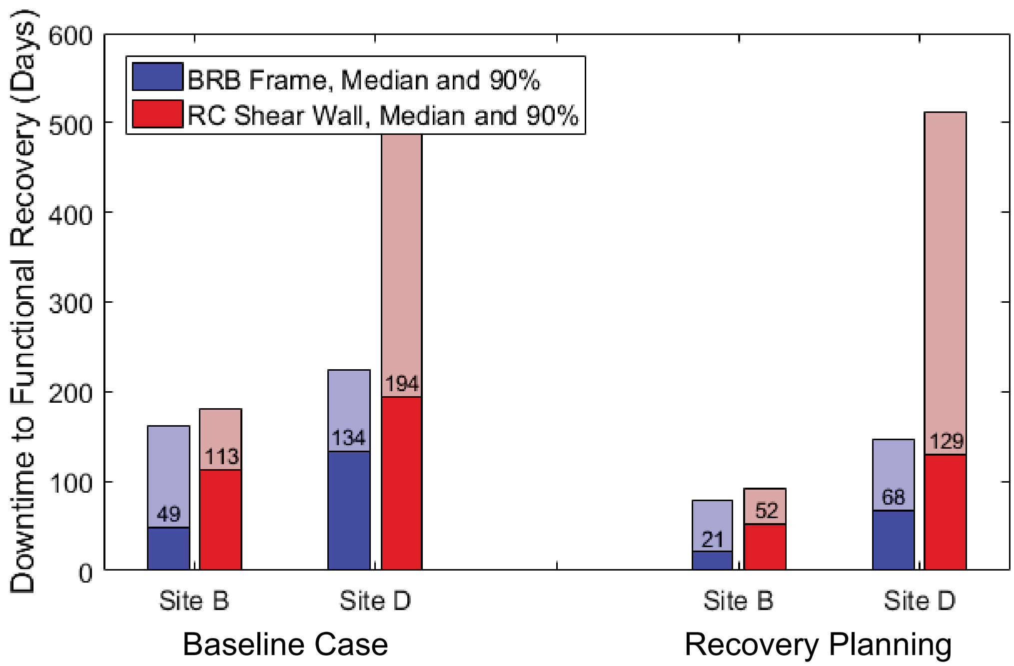


Figure 3-10 Summary of median and 90th percentile functional recovery time considering the baseline case and recovery planning under DE shaking.

As evident from Table 3-10, in spite of moderate economic loss (repair cost) levels, the associated repair and downtimes are extensive. For instance, at Site Class D under DE shaking, the median losses in the RCSW residential building of 8.1% result in repair times of 1.5 months, with functional recovery extending to 6.5 months. Similarly, at Site Class D under DE shaking, the expected losses in the BRBF office building of 2.8% corresponds to a repair time of 1 month and a time to functional recovery of 4.5 months.

Table 3-10 shows significantly shorter repair and recovery times at Site Class B than at Site Class D. As discussed above, the Site Class B results can be understood as estimates of Site Class D performance of a “better than code” building, specifically one designed with tighter drift limits. Another way to reduce downtime is to incorporate recovery considerations into the initial design. The benefits of designing with recovery in mind are shown in the two right-hand columns of Table 3-10., labeled, “Reduced median time to functional recovery.” Recovery-focused measures can include: (1) expediting post-earthquake inspection; (2) pre-earthquake arrangements to have an engineer on contract to minimize delays associated with engineering mobilization; (3) pre-earthquake arrangements to have a general contractor on retainer to minimize delays associated with contractor mobilization; and (4) expediting permits for building repairs. Schemes (1) and (2) already exist, through the Building Occupancy Resumption Program (BORP), and through arrangements that building owners frequently hold with engineers to carry out certain activities as needed. See Part 6 for further discussion on BORP. Schemes (3) and (4), while not routinely available, are evaluated to explore their impact on expediting recovery. The impact of these measures on the median recovery times are summarized in the last two columns of Table 3-10, and the median and 90th percentile results are illustrated in Figure 3-10 for both buildings under DE shaking. To the extent that effective pre-earthquake plans can be implemented and carried out, they can significantly reduce the building downtimes—potentially cutting downtime in half for many cases.

Although deaggregating the contributors to downtime is not as straightforward to deaggregating losses, it is possible to identify the major contributions to repair and functional recovery times. As described previously, depending on the component, the influence of component damage and repairs on downtime is amplified when impeding factors are triggered, so the relationship between repair and functional recovery times is not one-to-one.

For the RCSW building subjected to DE ground motions at Site Class D, the major controlling contributors to functional repairs and downtime are: elevators, structural repairs, and mechanical equipment. For the BRBF building subjected to DE ground motions at Site Class D, the major controlling contributors to the repair and functional recovery times are: elevators, curtain wall, and mechanical equipment.

Figures 3-11 and 3-12 summarize median and 90th percentile estimates of functional recovery time, which includes the impeding factor estimates previously discussed in Section 2.4, such as post-earthquake inspection, engineering mobilization, contractor mobilization, permitting and financing for the RCSW and BRBF buildings. The figures also illustrate the different sequences of delays, which are quantified in the recovery "paths," termed engineering, contractor, and financing.

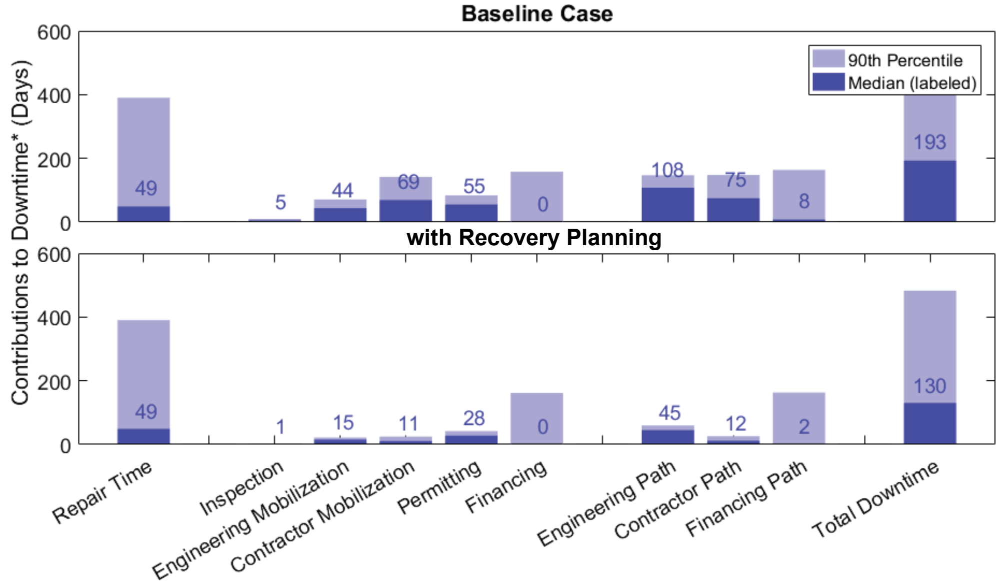


Figure 3-11 RCSW functional recovery time deaggregation at Site Class D under DE: baseline (top) and with recovery planning (bottom).

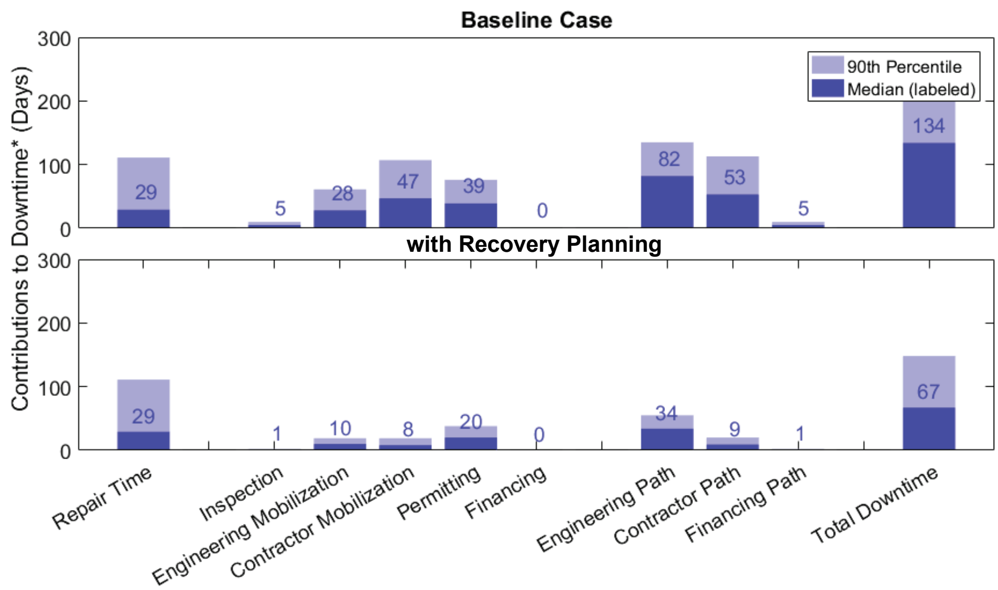


Figure 3-12 Steel BRB functional recovery time deaggregation at Site Class D under DE: baseline (top) and with recovery planning (bottom).

All three recovery paths begin with the time for inspection. Beyond this, the engineering path includes time for mobilizing engineering services and permitting of the proposed repairs; the contractor path includes contractor mobilization and procurement of replacement items with long lead times; and the financing path includes time to arrange financing of repairs. The total downtime estimate is essentially the combination of the longest of these three recovery paths plus the repair time itself. Note that the median values of the recovery paths for the total downtime are close to but not equal to simple additions of the constituent components, since the combined totals are statistical realizations of the medians that incorporate variability of the component parts. For the RCSW building (Figure 3-11), the longest sequence of delays comes on the engineering path, due to engineering mobilization and permitting. Comparing the upper and lower plots, implementation of recovery-planning strategies can reduce functional recovery time from approximately 6.5 months (194 days) to just over 4 months (128 days). Similar trends are seen for the BRBF building (Figure 3-12), where the engineering path is the longest sequence and implementation of resilience strategies can reduce functional recovery time from 4.5 months (134 days) to just over 2 months (68 days). The functional recovery time estimates in Figures 3-11 and 3-12 are for realizations that do not require total building replacement in the event of demolition due to excessive residual drifts. The probability of requiring demolition due to excessive residual drifts is zero in all cases except for the RCSW at Site Class D, with a 0.9% probability under DE and a 2.4% under MCE. In these cases, the downtime is assumed to be the building replacement time of 3 years.

Note that the functional recovery time estimates in Figure 3-11 do not include realizations where residual drifts render the building irreparable, which have a 0.9% probability of occurrence at Site Class D under DE. Note that the functional recovery time estimates in Figure 3-12 do not include realizations where residual drifts render the building irreparable, which have a 0.0% probability of occurrence at Site Class D under DE.

2.6 Variations in Archetype Tall Building Design Criteria

To evaluate how story drift demands can affect the damage, repair cost loss, and downtimes, sensitivity analyses were conducted for varying story drift demands. This was done by scaling the story drifts from NLRHA, as illustrated in Section 2.4, based on maximum of mean story drift ratios ranging from 1% to 3% in 0.5% increments, and re-running the SP3 loss analyses. In the re-evaluation of loss and downtime, the distributions of peak ground accelerations and floor accelerations are assumed to remain consistent with the baseline case, since the accelerations are assumed to be less sensitive to changes in the building stiffness. Thus, only repair costs and repair time associated with drift sensitive components varied, whereas those associated with acceleration sensitive components remained unchanged.

Figures 3-13, 3-14, and 3-15 illustrate the results of the sensitivity analyses for cost, repair time, and functional recovery time. The 90th percentile values are included in addition to the median values to indicate how the risk of large losses increases with increasing mean drift demands. The results suggest that to reduce the risk of large expected losses, the mean drift demands in the steel BRBF should be limited to about 2%. This is based on the dramatic increase in the trend in 90th percentile losses at the 2% story drift demand. Similarly, results suggest that the mean drift demands in the RCSW would need to be limited to about 1%.

The smaller limit for the RCSW system is due to two factors. First, due to wall elongation associated with flexural cracking, shear wall structures experience racking drifts that can double the damaging effects of story drifts, and thus, the RCSW story drifts of 1% can impose damaging racking drifts on the order of 2%. Second, structural damage tends to be experienced sooner in RCSW structures due to the larger localized deformation in coupling beams and wall-floor slab connections.

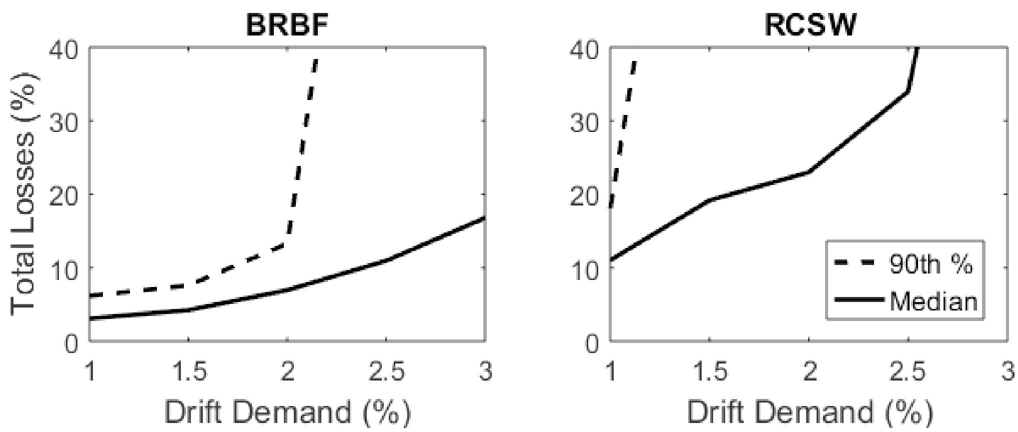


Figure 3-13 Mean story drift demand versus repair cost losses.

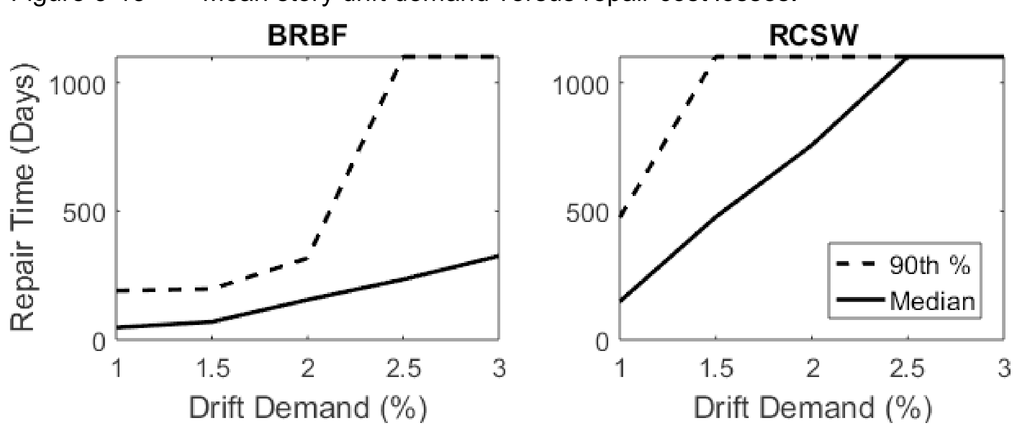


Figure 3-14 Mean story drift demand versus repair times.

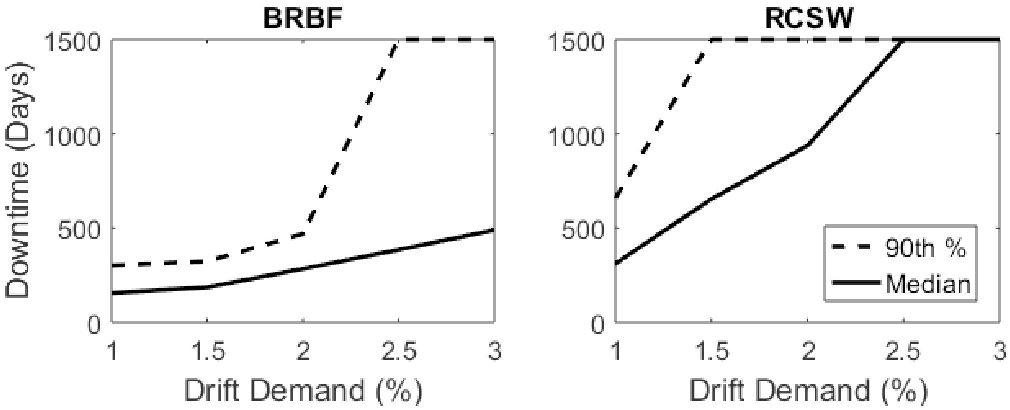


Figure 3-15 Mean story drift demand versus functional recovery times.

Chapter 3

Other Tall Building Studies

3.1 Recent Tall Building Projects

In addition to evaluating the archetype residential and office buildings presented in Chapter 2, the project team collected information from over 30 recently completed performance-based seismic design tall building projects in San Francisco, Los Angeles, Seattle, San Diego, and other locations. The purpose of collecting this information was twofold: (1) to benchmark the design of the archetype buildings against real project data; and (2) to understand how modifications to drift limits would affect the state of practice. Figure 3-16 shows the mean story drift demands for MCE evaluations in the principle directions for the 30 buildings.

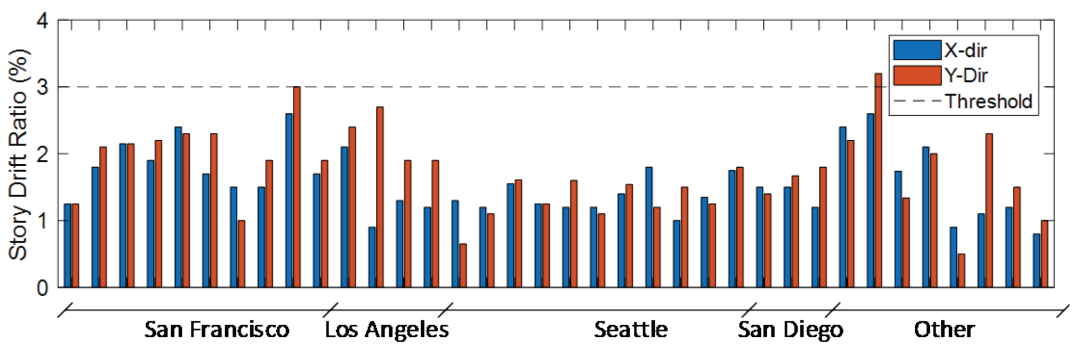
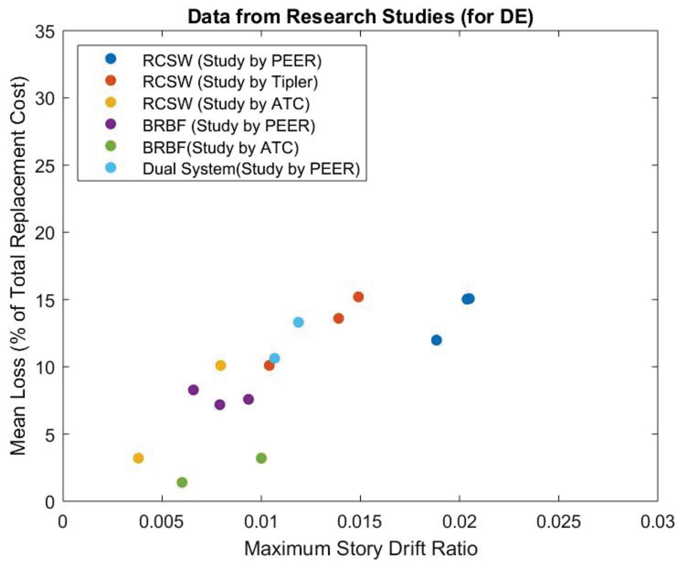


Figure 3-16 Mean MCE-level story drift demands from recently completed tall buildings designed using performance-based methods.

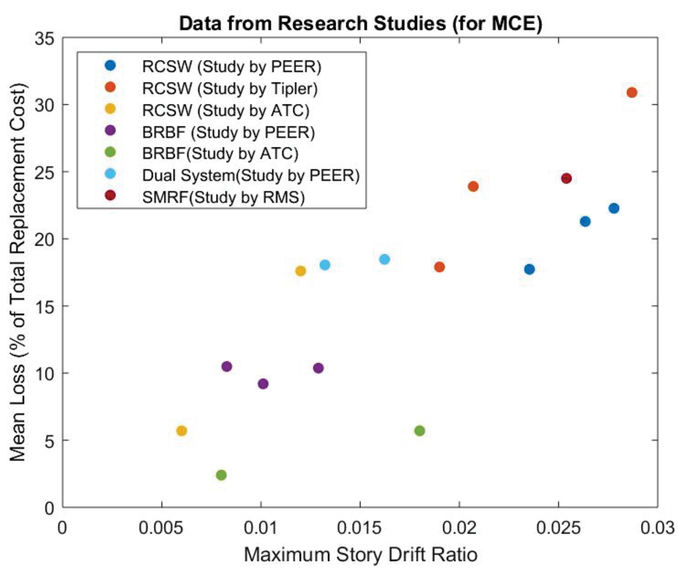
These results suggest that despite the 3% drift limit at MCE, currently prescribed by PEER TBI guidelines, about two-thirds of the buildings have maximum drifts less than 2% and half have drifts less than 1.5%. These data indicate that the two archetype buildings are fairly representative of the average stock of buildings, and further, that limiting MCE drifts to values less than the current 3% limit is not inconsistent with current design practice.

3.2 Research Publications

A literature review was carried out to compare the performance results from this study against those of similar tall building research projects. Data were collected primarily from the following studies: PEER (2011), Tipler (2014), and Jayaram et al. (2012), including loss results for buildings consistent with the steel buckling-restrained braced frame (BRBF) and reinforced concrete shear wall (RCSW) archetype buildings presented in Chapter 2, as well as for dual systems including a reinforced concrete core wall and perimeter reinforced concrete frame, and steel moment resisting frames. Expected losses at Design Earthquake (DE) and Maximum Considered Earthquake (MCE) levels found in the literature, as well as those previously reported for the archetype buildings, are shown in Figure 3-17.



(a)



(b)

Figure 3-17 Loss versus drift data for different structural systems at (a) DE and (b) MCE ground motion intensities.

The data shown in Figure 3-17 indicate that the loss estimates for the archetype buildings in the current study, while at the lower range of estimates from the other studies, are generally consistent with those from past studies.

The data also confirm a clear trend between drift demand and expected losses, suggesting that losses are about 10% of building replacement value for story drift demands of 1%.

3.3 Mid-Rise Building Loss Studies

To further investigate the trends identified in Section 2.6, FEMA P-58 *Performance Estimate Tool* (PET) (FEMA, 2018) was utilized to evaluate the losses and repair times for a 12-story steel BRBF office building and 12-story RCSW building. Note that FEMA P-58 PET does not include data for buildings taller than 12-stories, and the building systems considered in FEMA P-58 PET do not include the same features as the two archetype buildings (e.g., the RCSW in the study consists of a coupled shear wall building, whereas the RCSW in FEMA P-58 PET consists of a system with single shear walls). Nevertheless, the FEMA P-58 PET results can help substantiate the trends identified in Section 2.6.

Loss and downtime results of the FEMA P-58 PET analyses are summarized for the 12-story BRBF and RCSW systems in Figures 3-18 and 3-19, respectively. In these analyses, losses and downtime are calculated for alternate building designs that were designed to have DE drift demands of 0.5% to 2%. Therefore, in contrast to the plots from the archetype study (Section 2.6), in these plots, the horizontal axes correspond to the DE drift limits used in the design, rather than the calculated drift demands. To the extent that the drift demands are consistent with the drift limits, then the loss results for the DE shaking (blue lines in the figures) should be roughly consistent with the drift limits. On the other hand, the larger losses for MCE drifts would presumably be related to larger drift demands, as compared to the DE design drift limit.

In the FEMA P-58 PET analyses, the repair cost losses and downtimes seem remarkably low for the 12-story RCSW building, as compared to both the BRBF building and the 42-story RCSW archetype building presented in Chapter 2. On the other hand, the results for the 12-story BRBF building are comparable to those shown previously for the 40-story BRBF archetype of Chapter 2.

Similar to the trends observed previously with the 40-story BRBF, the 12-story FEMA P-58 PET BRBF design shows a distinct increase in losses for buildings with design drift limits above about 1.3%. This is about two-thirds of the 2% DE limit specified in current building codes. Assuming a linear relationship between DE and MCE drifts, the two-thirds reduction would correspond to an MCE drift limit of 2%, instead of the 3% permitted by current codes. While the overall losses for the 12-story FEMA P-58 PET RCSW are relatively small, they similarly show an increase at design drifts of about 1.3%.

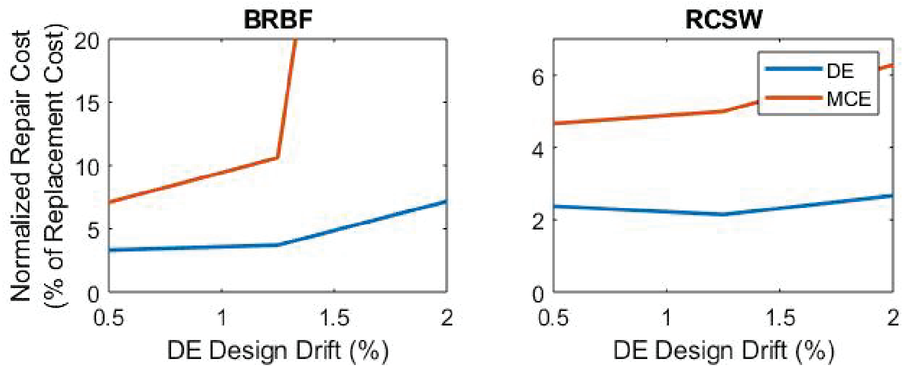


Figure 3-18 FEMA P-58 PET drift versus loss sensitivity results.

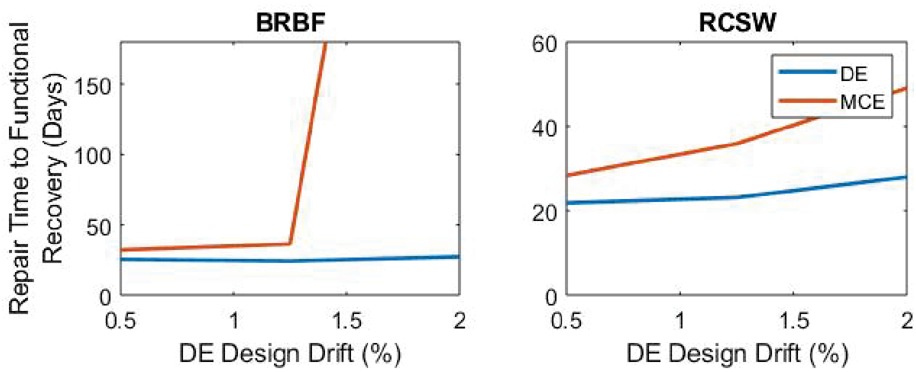


Figure 3-19 FEMA P-58 PET drift versus repair time sensitivity results.

Chapter 4

Summary, Recommendations, and Cost Implications

4.1 Summary

Current design requirements for tall buildings in San Francisco are primarily intended to protect life safety by minimizing the risk of collapse or significant structural or façade damage under extreme (i.e., Maximum Considered Earthquake) ground motions. While the requirements also include serviceability requirements to limit damage to the building structural system, this check is made under frequent earthquake ground shaking (with a 50% chance of exceedance in 30 years), whose intensity is much lower than that of the Design Earthquake (DE) ground motions (with roughly a 10% chance of exceedance in 50 years).

One of the key design criteria that controls the level of building damage under earthquake ground motions is the story drift ratio. Current building codes limit the maximum story drift ratios to 3% under MCE ground motions, which is roughly equivalent to story drift ratios of 2% under DE ground motions. While these are the maximum story drifts permitted by the building code, the drift ratios of many new tall buildings are significantly less than this limit due to other design constraints and considerations. For example, a survey of recently constructed buildings in San Francisco, Los Angeles, Seattle and San Diego reveals their MCE-level drift ratios to range from 1% to 3%, with typical values between 1.5% to 2% (see Figure 3-16). For comparison, the drift levels under DE ground motions would typically be about two-thirds of the MCE level drifts; hence the surveyed MCE drift ratios of 1% to 3% would relate to DE drift ratios of 0.7% to 2%.

Studies of two archetype tall buildings designed to meet current *San Francisco Building Code* requirements indicate that both would experience functional recovery times significantly longer than the tentative recommended recovery goals for occupancies common in San Francisco tall buildings (ATC, 2018). Those goals call for a functional recovery time of “weeks” for hotels, major employers, and most multi-family residential buildings, while allowing “months” for other business uses typical in office towers.

A performance evaluation for a 42-story concrete shear wall residential building indicates that damage resulting from DE ground motions will take on the order of 1.5 months to repair, with a functional recovery time extending to 6.5 months, including time for inspections, mobilization of engineering and contractor services, and permitting. By employing Building Occupancy Resumption Program (BORP) and other measures to reduce the impeding factors, estimates suggest that the recovery times can be reduced to about 4 months, which is still longer than the City's tentative recovery goal of "weeks" for multi-family residential buildings (ATC, 2018). These long repair and recovery times are in spite of the fact that this building is estimated to experience moderate story drift ratios of 0.8%, which is less than half the building code limit, although wall elongation creates amplified racking drifts of about 1.3%. The calculated repair costs of about 8% of the building replacement value are not out of line with expectations for standard repair cost loss analyses.

Damage and downtime analyses of the 40-story steel braced-frame office building yield slightly better performance, with an estimated repair time of 1 month and functional recovery times of 2.5 to 4.5 months, depending on measures to reduce impeding factors. This office building has estimated story drifts of 1%, again about half of the maximum permitted by the building code, and repair costs equal to about 3% of the building replacement value.

Studies of these same two buildings for sites with lower ground motions and lower drifts indicate that the repair and recovery times would significantly reduce with tighter story drift limits. For example, for the residential building, the repair and functional recovery times would reduce to about 3/4 month and 2.5 months, respectively, by limiting the building story drifts to 0.4% (about 20% of the code limit) and employing measures to reduce impeding factors. For the office building, the repair and functional recovery times would reduce to about 1/2 month and 3/4 month, respectively, by limiting the building story drifts to 0.6% (about 30% of the code limit). These are approaching San Francisco's tentative recommended recovery goals, although the economics and practicality of achieving these tight drift limits is unclear for sites with high ground motions (e.g., sites with Site Class D).

Parametric studies to examine the performance of buildings with drifts larger than those observed in the archetype studies indicate that story drift ratios up to the building code limit of 2% under DE ground motions would lead to extended downtimes on the order of a year or more.

4.2 Recommendations

Available evidence indicates that tall buildings designed per current minimum building code requirements may experience earthquake damage that will result in functional recovery times significantly in excess of San Francisco's tentative recommended recovery goals under DE ground motions.

Fully achieving the recommended functional recovery goal for functional recovery of "weeks" for multi-unit residential tall buildings is not economical with standard construction technologies, even when measures are included to reduce impeding factors. (The goal could potentially be achieved with seismic isolation or supplemental damping, but this is beyond the scope of the current investigation).

The target of 1 month to functional recovery may be achievable by limiting story drift ratios of tall buildings under the DE ground motions to less than 1%. Assuming that DE drift demands are roughly two-thirds of MCE drift demands shown in Figure 3-16, many recently completed tall buildings come close to meeting this reduced drift demand. In addition to the reduction in seismic drift limits, achieving the 1 month to functional recovery goal would require: (1) enhanced design and specification of critical MEP/elevator systems; and (2) implementation of BORP and other measures to mitigate impeding factors for recovery. With regard to the latter, Part 6 of this report and Lang et al. (2018) outline suggestions for improving the BORP program to better achieve functional recovery.

4.3 Cost Implications

Although there are cost premiums associated with the different strategies proposed in this study to achieve enhanced seismic performance, in the long term, these initial costs result in long term savings associated with the reduction in economic losses and downtime. Studies such as Tipler (2014) suggest benefit-to-cost ratios on the order of two to three when considering direct losses only, and five to twenty when considering both direct losses and downtime.

The cost of pursuing an enhanced design can vary significantly whether these are achieved through enhanced structural design (e.g., thicker walls or supplemental damping devices) or enhanced nonstructural components (e.g., seismic detailing to accommodate larger deformations and/or accelerations prior to damage). PEER (2011) evaluated the costs associated with performance-based versus code-based design of tall buildings (consistent with the RCSW and BRBF in this study) and estimated construction cost premiums for the performance-based designs of less than 2% of the building construction cost. A study by NIST (2013) showed that the benefits associated with improved seismic design can be significant and cost-beneficial (though that study was about areas of moderate study and not specifically about tall buildings). Tipler (2014) also estimated the construction costs of an archetype building consistent with the RCSW in this study, as well as variations in design that included enhanced structural solutions (i.e., damped outriggers and base isolation) along with enhanced nonstructural components. The study estimated premiums within 3% of the baseline construction costs. This is consistent with a report by Almufti et al. (2016) which noted that premiums associated with enhanced structural designs generally range from 0 to 5%.

The costs associated with participation in programs such as BORP are estimated at $30,000 to $50,000 as a one-time fee to prepare the required inspection plan (BORP Submittal Phase). This fee is likely to be less if the report is prepared in conjunction with the design of a new building. In the event that emergency inspection services are needed following an earthquake, these would generally be billed separately under a time and material pricing model. The cost of having an engineering design firm on retainer to minimize engineering mobilization details is estimated at $5,000 to $10,000 per year. (These estimates were derived in consultation with design firms that provide such services.)

The costs associated with other strategies such as having a contractor on retainer or earthquake insurance are not easily estimated. Having a contractor on hold to carry out specific repair activities, as needed, is not generally a service offered by general contractors. Furthermore, the retainer costs for contractors could vary significantly depending on the nature of the repair work (e.g., light partition wall damage versus severe structural damage). Similarly, while earthquake insurance is available to building owners, developing transparent estimates of earthquake insurance premiums is difficult because: (1) earthquake insurance is often lumped with other perils; (2) owners do not necessarily insure their entire asset value; and (3) premiums are derived based on portfolio level assessments (not individual buildings). As a result, it is difficult to disaggregate the corresponding costs associated with earthquake insurance alone.

While not many, there are examples of modern tall buildings that have adopted some of the strategies presented in this study. For instance, the 181 Fremont Building, located in downtown San Francisco next to the Transbay Transit Center, was reportedly designed to exceed the minimum seismic performance objectives of the *San Francisco Building Code* at “little-to-no cost premium” (Almufti et al., 2016). The building was designed to achieve immediate occupancy and limited disruption to functionality after a 475-year return period ground motion (approximately equal to the DE earthquake in this study). This was achieved by means of different strategies, many of which align with those presented in this study:

- Structural Design: Elements to remain essentially elastic under a DE earthquake.
- Nonstructural Design: Enhanced component design including elevators (consistent with requirements for California hospitals), staircase (to accommodate more movement and sustain less damage), façade system (air-tight and water-tight up to drift limits of 2%), and enhanced anchorage of nonstructural components.
- Recovery Planning: Participation in the BORP program, inclusion of back-up system for the building to remain functional in the event of utility disruption, training of personnel to re-start elevators after an earthquake. and the development of an owner's guideline to earthquake resilience.

Appendix A

References

Almeter, E., Wade, K.F., Haselton, C.B., Saxey, B., 2018, "Rapid risk assessment of buckling restrained braced frames, with focus on residual drifts," *Proceedings*, 11th National Conference on Earthquake Engineering, Earthquake Engineering Research Institute, Los Angeles, California.

Almufti, I., Krolicki, J., and Crowther, A., 2016, "The resilience-based design of 181 Fremont Tower," *Structure Magazine*, June, pp. 42-46.

Almufti, I., Molina Hutt, C., Mieler, M. W., Paul, N.A., and Fusco, C.R., 2018, *The HayWired Earthquake Scenario—Engineering Implications: Case Studies of Tall-Building Structural Analyses, Downtime and Loss Assessment for the HayWired Scenario Mainshock*, US Geological Survey, Menlo Park, California.

Arup, 2013, *Resilience-based Earthquake Design Initiative (REDi) for the Next Generation of Buildings*, Arup, San Francisco, California.

ASCE, 2010, *Minimum Design Loads for Buildings and Other Structures*, ASCE/SEI 7-10, American Society of Civil Engineers Structural Engineering Institute, Reston, Virgina.

ATC, 2018, *Recommended Earthquake Performance Goals for San Francisco's Buildings (ESIP Task A.6.h)*, final report submitted to the City and County of San Francisco prepared by the Applied Technology Council, Redwood City, California.

Bonowitz D., 2011, *Resilience Criteria for Seismic Evaluation of Existing Buildings*, a 2008 Special Projects Initiative Report to the Structural Engineers Association of Northern California, San Francisco, California.

CCSF, 2008a, *Requirements and Guidelines for the Seismic Design of New Tall Buildings using Non-Prescriptive Seismic-Design Procedures*, Administrative Bulletin 083, Department of Building Inspections, San Francisco, California.

CCSF, 2008b, *Guidelines and Procedures for Structural Design Review*, Administrative Bulletin 082, Department of Building Inspections, San Francisco, California.

CCSF, 2011, *CAPSS Earthquake Safety Implementation Program Workplan 2012-2042*, the City and County of San Francisco, California.

CCSF, 2016, *The San Francisco Building Code*, the City and County of San Francisco, California.

FEMA, 2012, *Seismic Performance Assessment of Buildings*, FEMA P-58, prepared by the Applied Technology Council for the Federal Emergency Management Agency, Washington, D.C.

FEMA, 2018, *Seismic Performance Assessment of Buildings, Volume 3—Supporting Electronic Materials and Background Documentation*, Second Edition, FEMA P-58-3, prepared by the Applied Technology Council for the Federal Emergency Management Agency, Washington, D.C.

HBrisk, 2018, *Seismic Performance Prediction Program*, SP3, Haselton Baker Risk Group, Chico, California.

Hooper, 2018, Personal communication with Peter Morris who previously provided cost estimates for earthquake repairs as part of the FEMA P-58 development process.

Jayaram, N., Shome, N., and Rahnama, M., 2012, "Development of earthquake vulnerability functions for tall buildings," *Journal of Earthquake Engineering and Structural Dynamics*, Vol. 41, No. 11, pp. 1495-1514.

Lang, A., Buckalew, J., Mayes, R., Wade, K., 2018, "FORWARD: Second generation B2B/BORP and resilient planning for existing buildings," SEAOC 2018 Convention, Palm Springs, California.

LSTC, 2009, *LS-DYNA*, Livermore Software Technology Corporation, Livermore, California.

McKenna, F., Fenves, G.L., Scott, M.H., and Jeremic, B., 2000, *Open System for Earthquake Engineering Simulation*, OpenSees, University of California, Berkeley, California.

Molina Hutt, C., Almufti, I., Willford, M., and Deierlein G.G., 2016, "Seismic loss and downtime assessment of existing tall steel-framed buildings and strategies for increased resilience," *Journal of Structural Engineering*, Vol. 142, No. 8.

Naish, D., Wallace, J.W., Fry, J.A., and Klemencic, R., 2009, *Reinforced Concrete Link Beams: Alternative Details for Improved Construction*, UCLA-SGEL Report 2009-06, Structural & Geotechnical Engineering Laboratory, University of California, Los Angeles, 103 pp.

NIST, 2013, *Cost Analysis and Benefit Studies for Earthquake-Resistant Construction in Memphis, Tennessee*, NIST GCR 14-017-26, prepared by the NEHRP Consultants Joint Venture, a partnership of the Applied Technology Council and the Consortium for Universities for Research in Earthquake Engineering, for the National Institute of Standards and Technology, Gaithersburg, Maryland.

PEER, 2010, *Guidelines for Performance-based Seismic Design of Tall Buildings*, Version 1.0, Report No. 2010/05, Pacific Earthquake Engineering Research Center, University of California, Berkeley, California.

PEER, 2011, *Case Studies of the Seismic Performance of Tall Buildings Designed by Alternative Means*, PEER Report 2011/05, Task 12 Report for the Tall Buildings Initiative, Pacific Earthquake Engineering Research Center, University of California, Berkeley, California.

PEER, 2013, *NGA-West2 Database*, PEER Report 2013/03, Pacific Earthquake Engineering Research Center, University of California, Berkeley, California.

PEER, 2017, *Guidelines for Performance-based Seismic Design of Tall Buildings*, Version 2.03, Report No. 2017/06, Pacific Earthquake Engineering Research Center, University of California, Berkeley, California.

Terashima, M., 2018, in review, *Ductile Fracture Simulation and Risk Quantification of Buckling Restained Braces under Earthquakes*, Dissertation (PhD), Stanford University, Stanford, California.

Tipler, J., 2014, *Seismic Resilience of Tall Buildings—Benchmarking Performance and Quantifying Improvements, Dissertation* (MSc), Stanford University, Stanford, California.

USGS, 2018, *The HayWired Earthquake Scenario—Engineering Implications*, USGS Scientific Investigations Report 2017-5013-I-Q, US Geological Survey, Menlo Park, California.

PART 4:
Post-earthquake Structural Evaluation

Chapter 1

Introduction

1.1 Background

This Part reviews and explains current San Francisco policy for post-earthquake structural evaluation, especially as it relates to the City's existing tall buildings. It recommends modifications to the current policy and proposes additional study or development of new regulations to address identified issues. This Part corresponds to Recommendations 2A, 3D, 3E, and 3F presented in the Summary Recommendations.

Current San Francisco policy requires the repair of earthquake damage in nearly all cases and checks three conditions, called triggers, under which required repair must be supplemented by a full-building seismic evaluation and, if warranted, a seismic retrofit.

The three triggers represent policy judgments about when retrofit, which adds cost and recovery time, is nevertheless viewed as beneficial to the community and reasonable for owners and tenants. This judgment, which applies to all San Francisco buildings except one- and two-family dwellings, should be reviewed as it relates to tall buildings, for which recovery time can be critical and retrofit costs impractical.

This Part addresses aspects of the following tasks described in the *CAPSS Earthquake Safety Implementation Program Workplan 2012-2042* (CCSF, 2011):

- Task A.4.d: Adopt disproportionate damage trigger
- Task B.4.b: Develop post-earthquake repair and retrofit standards

1.2 Organization

Chapter 2 reviews current San Francisco policy for post-earthquake structural evaluation.

Chapter 3 presents recommendations for improving San Francisco policy for post-earthquake structural evaluation.

Appendix A provides an annotated set of current San Francisco Existing Building Code provisions related to this topic.

A list of references is provided at the end of this Part.

Chapter 2

Current Policy

The *San Francisco Existing Building Code* (SFEBC) (CCSF, 2016a) regulates most aspects of post-earthquake structural evaluation of buildings. SFEBC provisions combine the *California Existing Building Code* (CEBC) (CBSC, 2016), itself an amended version of the *International Existing Building Code* (IEBC) (ICC, 2018), and San Francisco amendments.

The SFEBC is organized by project type, recognizing five categories: additions, alterations, repairs, changes of occupancy, and relocations. Because post-earthquake evaluation involves the assessment of damage, it is addressed by the code's provisions for repairs. With exceptions for one- and two-family dwellings, the provisions are the same for buildings of any height.

2.1 Evaluation, Repair, and Triggered Retrofit

2.1.1 2016 SFEBC Provisions

The SFEBC regulates the task of restoring a damaged building's pre-earthquake condition and, in some cases, improving it.

SFEBC Section 404.1 requires earthquake damage to be repaired. The remaining question is whether the code, as public policy, should also require the structure to be seismically improved at the same time. The answer has evolved over decades and is represented in the current SFEBC by four ideas:

- When earthquake damage to the seismic-force-resisting system (the wall and frame elements expected to resist earthquake effects) is substantial, the repair work will already be extensive, so the repair presents an opportunity to evaluate and improve an obsolete structure in anticipation of several more decades of service. The triggering level of damage is defined in SFEBC Chapter 2 as *substantial structural damage* to the lateral system (lateral SSD), and the trigger provision is in SFEBC Section 404.2.
- When earthquake damage to the gravity system is substantial, it indicates that the building's seismic-force-resisting system is grossly deficient, since the purpose of the seismic-force-resisting system is to protect the gravity system from collapse. Returning such a deficient building to

decades of service would be inconsistent with what is expected from new buildings. This triggering level of damage is a special case of *substantial structural damage* to the gravity system (gravity SSD caused by lateral load), and the trigger is in SFEBC Section 404.3.1.

- When earthquake damage to the seismic-force-resisting system is surprisingly high, given a low level of shaking, it indicates a grossly deficient seismic-force-resisting system prone to collapse in a larger earthquake. Again, returning such a deficient building to decades of service would be inconsistent with policy for new buildings. This triggering level of damage, together with the low hazard, is defined as *disproportionate damage* (DD), and the trigger is in SFEBC Section 404.4.1. The concept of DD was pioneered in San Francisco, but it has been adopted by the 2018 IEBC model code and will appear, slightly modified, in the 2019 edition of the statewide CEBC.
- Whenever triggering SSD or DD exists, the pre-earthquake structure should be evaluated, but not with the full earthquake loads (or forces) that would be used for the design of a similar new building. Rather, the evaluation should be allowed to use *reduced* loads; this ensures that any building failing the evaluation is unquestionably deficient, not a marginal case. If the building fails the evaluation, it must be retrofitted as well as repaired. The retrofit, like the evaluation, is allowed to use reduced loads. San Francisco has amended this CEBC allowance to require the larger of the reduced seismic loads and the loads from "the code under which the building or structure was designed" (SFEBC Section 404.1.1).

These four codified ideas comprise three repair-based retrofit triggers and one set of evaluation and design criteria. The triggers represent policy judgments about when retrofit, which adds cost and recovery time, is nevertheless viewed as beneficial to the community and reasonable for owners and tenants. These policy judgments should be reviewed as they relate to building groups for which recovery time is critical or retrofit costs are impractical.

Retrofit triggered by repair is more controversial as policy than retrofit triggered by other project types. Other projects—additions, alterations, or changes of occupancy—are essentially voluntary. If the building owner does not want the additional cost of retrofit, she has the option to change the scope of the project. Repairs, by contrast, are mandatory. Requiring a retrofit at the same time as the owner is forced to make unplanned repairs (and deal with tenants and insurers) adds a burden that is likely to delay the building's recovery. Further, if many buildings face similar delays, especially if those buildings are clustered geographically or serve the same function, the aggregate effect might impact recovery on a neighborhood or city scale.

Of the three repair-based triggers, lateral SSD is the most controversial. Unlike DD and gravity SSD caused by earthquake, lateral SSD is not necessarily an indicator of a collapse-prone structure; in a large earthquake, even some new buildings are expected to see significant structural damage. Thus, the lateral SSD trigger could catch some buildings that would pass a structural evaluation.

The question, then, is whether the additional time and expense needed to make these evaluations—which, for complex or tall structures, can be significant—are worth the benefit. The question is amplified when applied to dozens of tall buildings (and possibly hundreds more non-tall buildings) at the same time. On one hand, there is clear value in bringing dozens of tall office buildings back into service more quickly by eschewing triggered evaluations and retrofits. On the other hand, if dozens of tall buildings have already sustained substantial structural damage, just making repairs will not be quick either; downtime is likely to be on the order of months even in the more permissive case.

2.1.2 Administrative Bulletins

Two of the code's three triggers rely on a measurement of damage in terms of lost "lateral load-carrying capacity." A 33% capacity loss defines lateral SSD, while a 10% loss, together with a low shaking level, defines DD.

Building code provisions should have quantitative, enforceable terms, and this sometimes results in bright-line definitions, such as those of lateral SSD and DD. Even so, the triggering values in these two definitions are entirely subjective.

First, there is nothing technically special about a 10% or 33% capacity loss; that is, a building with a 30% loss is not clearly in a different category than a building with a 35% loss. Rather, SSD and DD should be understood to have conceptual meanings despite their quantitative definitions.

- The 10% figure in the DD definition is meant to convey a small but not insignificant level of damage. The intent of the DD provisions is to identify structural damage in cases where a properly designed and constructed building would have none, so the 10% level is meant as "something noticeable," as opposed to "practically nothing."
- The 33% figure in the lateral SSD definition is meant to convey a level of damage that is serious but still well short of collapse. When this figure

was debated in building code hearings, proponents suggested that a loss of half the lateral system on one side of a building—that is, a 25% loss—should not trigger an upgrade. By consensus judgment, 33% was agreed to as a more appropriate number representing a substantial and somewhat unexpected loss even in cases where some structural damage is anticipated.

Second, there is no agreed definition of capacity loss and no standard procedure for measuring it, given the observed damage. As with many code provisions, in-house rules and procedures will be used until a consensus emerges, perhaps through a formal standards process, but more likely through informal sharing of practices between implementing agencies. Until then, it is expected that San Francisco, like every other jurisdiction, will interpret and apply these definitions with ample discretion.

As the first jurisdiction to codify the concept of disproportionate damage, San Francisco has made these interpretations through a series of Administrative Bulletins (ABs) that provide procedures for calculating capacity loss or replace the somewhat-arbitrary values with more practical and definable damage descriptions. AB-099, *Post-Earthquake Repair and Retrofit Requirements for Concrete Buildings* (CCSF, 2012a), defines lateral SSD and DD for concrete shear wall and moment frame structures, including those in tall buildings. AB-098, *Post-Earthquake Repair and Retrofit Requirements for Wood-Frame Residential Buildings with Three or More Dwelling Units* (CCSF, 2012b), and AB-100, *Post-Earthquake Repair and Retrofit Requirements for One- and Two-Family Units* (CCSF, 2012c), define lateral SSD and DD for wood-frame multi-unit buildings and wood-frame dwellings; they do not apply to tall buildings.

AB-099 uses two methods to categorize damage as meeting the lateral SSD or DD thresholds:

- A quantitative method, involving structural analysis, based on the guideline document FEMA 306, *Evaluation of Earthquake Damaged Concrete and Masonry Wall Buildings* (FEMA, 1998), and
- A mostly qualitative method based on observed damage patterns, crack orientation, and crack width.

AB-099 also allows for nonstructural or cosmetic damage that need not be repaired at all.

AB-099 is mostly applicable to concrete shear wall structures. It should apply also to dual systems with concrete shear walls and concrete moment frames; for the moment frames, only the qualitative method is required, and the cosmetic damage level is omitted. Presumably, AB-099 would apply to dual systems with concrete shear walls and steel moment frames, but the AB is not completely clear on this point.

Currently, there are no ABs for other structural systems common to San Francisco's tall buildings. The San Francisco Department of Building Inspection's guidelines for its Building Occupancy Resumption Program refer to FEMA 352, *Recommended Postearthquake Evaluation and Repair Criteria for Welded Steel Moment-Frame Buildings* (FEMA, 2000), which addresses some of the same questions for steel structures that FEMA 306 does for concrete, but FEMA 352 is not required and is not written into enforceable provisions through an AB.

2.2 Relation to Post-Earthquake Safety Inspection

The structural evaluations that guide repairs are different from post-earthquake safety inspections. Repairs are planned, even if the damage being repaired was not. The applicable code provisions are concerned with regulating the building stock over the long term. Repairs therefore are regulated by the code as defined projects, along with administrative provisions for documentation, permitting, fees, and quality assurance.

By contrast, post-earthquake safety inspection is performed with a more immediate interest in short-term safety. As such, the code's repair provisions offer no rules or procedures for accessing or occupying a potentially damaged building, finding or ruling out damage, or posting or removal of placards (Red, Yellow, or Green). Post-earthquake safety inspection is addressed in Part 6.

The intersection of the SFEBC with safety inspection is limited to the code's general provisions regarding *dangerous* conditions. Whether a building is identified as dangerous will often be related to the findings made during post-earthquake safety inspection. Immediate measures to stabilize the structure might be made to eliminate dangerous conditions and to allow further assessment or reoccupancy. This is all SFEBC Section 401.3 contemplates; the code still requires permits for these "emergency repairs" but allows the paperwork to be completed after the fact. Those immediate measures are likely to be different from the code-required repairs, however. How structural damage will be repaired, and specifically whether a seismic upgrade will be required, is unrelated to the building's status as dangerous.

Indeed, by the time planned repairs are able to begin, the structure will have been stabilized and any Red or Yellow placards will have been removed or modified. In cases of light damage, it is even likely that general reoccupancy will be allowed before the repair-guiding evaluations are complete and before the repair work is begun.

2.3 Relation to Other Code-Regulated Work

Repairs (and the structural evaluations that guide them) are distinct from maintenance, alteration, and retrofit projects, as noted in SFEBC Section 404.1.

In general, all buildings must be maintained in serviceable and habitable conditions. In recent code cycles, the *International* family of codes, which includes the IEBC, has begun to clarify the intended distinction between maintenance and repair, both of which are required. While the resolution is not yet complete, a fair statement of the intent is that maintenance preserves an acceptable condition, while repair corrects an unacceptable condition. In any case, earthquake damage is a condition likely to require repair, not maintenance.

Related to general maintenance requirements is San Francisco's façade inspection and maintenance program for pre-1998 buildings taller than four stories. Program requirements are given in SFEBC Chapter 4E and AB-110, *Building Fa*çade *Inspection and Maintenance* (CCSF, 2017). In addition to deadlines for regular inspection (the first of which, for buildings constructed before 1910, is not until the end of 2021), the program requires inspection when façade elements "exhibit significant damage or failure" due to an earthquake (Section 403E.3). By responding to apparent damage only, the requirement is not proactive and is thus redundant to the code's general requirements to repair damage and to eliminate dangerous conditions. Also, the earthquake provision does not trigger any evaluation or retrofit of the structural system.

Alteration, addressed by SFEBC Section 403, is distinct from repair and unrelated to post-earthquake evaluation. Model code revisions since 2009 have made this distinction clearer, and SFEBC Section 404.1 further clarifies that any work necessary to complete a repair is to be treated as part of the repair, not as an alteration. Thus, earthquake damage should not invoke, or trigger, any requirements that normally attach to alteration projects. Nevertheless, this distinction is worth noting because past codes covered alterations and repairs with the same set of provisions, and local practices in some jurisdictions continue to conflate these project types. Current policy and recommendations regarding alteration projects are addressed in Part 7.

Retrofit, whether voluntary, mandated by city policy, or triggered by a substantial project, is a type of alteration. Like all alterations, retrofit is distinct from repair and unrelated to post-earthquake evaluation. This distinction is worth noting, however, because many owners and tenants refer to retrofit as “fixing” a deficient building, leading to possible confusion.

Chapter 3

Issues and Recommendations

3.1 Revisions and Amendments to the SFEBC

3.1.1 Policy on Repair-Triggered Retrofit

San Francisco policy regarding seismic retrofit triggered by damage and repairs is embodied in three SFEBC provisions. The three triggering provisions represent judgments about when the community's benefit from retrofit outweighs the expected cost and possible delays in recovery. Of the three triggers, two specifically target highly deficient structures. The third, known as lateral SSD, is more likely to affect buildings that time-consuming and costly structural evaluations will often find acceptable. In concept, if tall buildings represent a community function for which recovery time is critical or retrofit cost is impractical, then the lateral SSD trigger might be waived or adjusted for buildings in this group. (Current provisions gravity SSD caused by earthquake, however, should not be waived or relaxed.) Since the issue here is loss of community function, not building height, any reconsideration of current policy should apply to all buildings that support that function, or at least to large and complex buildings for which costs and delays will have similar effects regardless of height.

To address these questions, San Francisco could study the recovery impacts of the lateral SSD trigger through neighborhood-level simulation. A more limited (less costly) study would estimate the recovery impacts on individual buildings of different types. Informed by the findings, the City could decide to waive the lateral SSD trigger for certain buildings when the damage is due to an event, like an earthquake, that affects thousands of buildings simultaneously.

If the policy adjustment is made after the event, the process would be able to adjust to facts on the ground and grant waivers only as needed. Although flexibility is valuable, however, making a general policy under emergency conditions is administratively and politically fraught. Doing so would take attention away from more pressing issues, would make preparations more difficult, could lead to confusion or inconsistency, and is likely to be subject to pressure from building owners, jeopardizing the trust of other stakeholders. For these reasons, advance policy making is recommended.

The policy-making process should also consider that the Federal Emergency Management Agency (FEMA) endorses all three current triggers, and FEMA policy requires compliance with the model code in order to be eligible for post-disaster assistance. This means that any relaxation of the lateral SSD trigger, which has been accepted in the CEBC since 2009, could jeopardize federal recovery funds. That said, FEMA assistance does not generally go to office buildings, so the waiver or code change might avoid this issue by applying only to certain occupancies, building sizes, or neighborhoods.

Reduced loads for triggered evaluation and retrofit are meant to mitigate some of the concerns about repair-triggered retrofit. However, by amending the criteria to include the building's original design loads, San Francisco negates that mitigating measure, especially for younger buildings. To preserve the allowance intended by the CEBC, San Francisco should amend the SFEBC to allow reduced loads in the event of repair-triggered retrofit. (The question of which loads to use is related to the currently unclear application of the SFEBC table of benchmark dates, discussed in Appendix A of this Part under SFEBC Section 301.2.1.)

Relaxing the current policy for repair-triggered retrofit of tall buildings might raise concerns about favoring recovery over safety. In fact, relaxing a repair trigger does not make any building less safe, but any perceived loss of safety can be made up through other programs. Indeed, there are more effective ways to address both safety and recovery than tinkering with the code's repair triggers. Pre-earthquake mandates and incentives, together with recovery planning, are likely to be more effective at risk reduction or risk transfer than changes to the evaluation rules that only get applied after losses have occurred. These alternatives are discussed in Part 6 and Part 7.

Recommendation. Study the benefits of relaxing SFEBC provisions regarding substantial structural damage to the lateral-force-resisting system.

- Consider the effects of lateral SSD-triggered evaluation and retrofit on post-earthquake recovery time, for individual tall and non-tall buildings.
- Consider the aggregate policy effects on neighborhoods with concentrations of tall buildings and buildings supporting recovery-critical functions.
- Consider waiving the current SFEBC requirement to consider "original loads" for lateral SSD-triggered seismic evaluation and retrofit.
- Consider allowing benchmarking per SFEBC Section 301.2.1 for lateral SSD-triggered evaluation and retrofit.

3.1.2 Routine Code Development

While San Francisco pioneered the concept of disproportionate damage and has for years enforced its own provisions for triggered retrofits, many of its local amendments are now out of coordination with the CEBC and IEBC. In many cases, as shown in Appendix A of this Part, because the CEBC has adopted some of San Francisco's ideas, the SFEBC is now duplicative or out of step. To improve the SFEBC, at least with respect to repair-triggered retrofit provisions, the City should review Appendix A of this Part and address the issues raised in its commentary. The easiest way to do this would be to make any necessary changes or corrections as the City updates its amendments for the forthcoming 2019 CEBC, since that process will already require a careful review and reorganization of current SFEBC Chapter 3 and Chapter 4.

Recommendation. Update the SFEBC.

- Correct section numbering errors for SFEBC amendments.
- To avoid duplication, omit the SFEBC definition and triggers for disproportionate damage. Use the CEBC content instead.
- Coordinate Sections 301.2 and 404.1.1 with Section 301.1.4.2.
    - Revise the text that incorrectly says Section 301.2.1 applies only when invoked by Section 403.
    - Clarify whether SFEBC benchmarking in Section 301.2.1 is allowed for repair projects.
    - Clarify whether SFEBC benchmarking is allowed when original design loads are also required per Section 404.1.1.
    - Since Section 301.2.1 applies only to cases where reduced forces are used (alterations and repairs), consider combining with Section 301.1.4.2 (to be Section 303.3.2 in the 2019 SFEBC).
    - Delete the unnecessary pointer from Section 301.2.3 to Section 301.1.4.2. Pointers to Section 301.2.3 should instead point directly to Section 301.1.4.2.
    - Move Section 404.1.1 into Section 301.2 (or Section 301.1.4.2) and clarify whether the requirement to consider original loads applies to gravity SSD cause by lateral load (Section 404.3.1) and to DD (Section 404.4.1).
    - Clarify whether Section 404.1.1 original loads requirement applies when AB-099 (and others) is applied, as those ABs are premised on reduced loads.

- Make revisions 2016 CEBC to correct errors that occurred while adopting the 2015 IEBC.
- Clarify or revise limits on upgrade triggers imposed by the Department of Housing and Community Development.
- Omit Section 404.4.2.

3.2 Administrative Bulletins and Reference Documents

3.2.1 AB-099

AB-099 will help implement SFEBC repair provisions after the next damaging earthquake. Even so, AB-099 has not been used since its adoption by DBI in 2012. Most important, AB-099 references and relies on FEMA 306, published in 1998 yet still unfamiliar to most engineers, plan checkers, and potential peer reviewers. Simply put, there have not been enough opportunities to apply and vet FEMA 306 with actual earthquake damage. In addition, parts of AB-099 and FEMA 306 have fallen out of coordination with the CEBC and SFEBC and with their reference standards. Therefore, an update of AB-099 and a review of FEMA 306 with respect to San Francisco tall building types is recommended.

Recommendation. Update AB-099 and clarify its application to tall concrete structural systems.

- Reference SFEBC Table 301.2.1 or remove the obsolete reference to the May 21, 1973, benchmark date.
- Reference current codes and standards, including the forthcoming 2019 CEBC and SFEBC and ASCE/SEI 41-17 (ASCE, 2017), replacing references to outdated documents, such as ASCE/SEI 31-03, ASCE/SEI 41-06, ASCE/SEI 41-13, FEMA 273, and FEMA 356. This should include revising the performance objective in AB-099 Step 3 to reference the BSE-1E hazard level.
- Make the alternative performance objective consistent with SFEBC Table 301.1.4.2, and with SFEBC Section 404.1.1 as needed.
- Update the definition of lateral SSD, now based on 33% (not 20%) lateral capacity loss. This will include revisions to the text, to Figure 1, to the FEMA 308 procedure, and possibly to the Table 1 thresholds.
- Clarify application to concrete elements in dual systems with steel frames.
- Review FEMA 306 to ensure that its references to FEMA 273 (or AB-099's references to ASCE/SEI 41-13) are consistent with ASCE/SEI 41-17.

- Review FEMA 306 applicability to tall buildings in light of the note in FEMA 306 Section 4.4.1 regarding applicability of the method's basic procedures to structures with significant response outside the fundamental mode.
- Coordinate post-earthquake safety inspection procedures for concrete buildings (see Part 6) with FEMA 306 guidelines for inspection and damage-finding.

3.2.2 New Administrative Bulletins

AB-099 offers procedures and interpretations that relate technical damage observations to SFEBC's policy prescriptions. Having procedures and interpretations defined in advance will greatly improve implementation of these rarely-used code provisions. The idea should be extended to other structure types. In particular, as discussed in Part 1, San Francisco's tall buildings are more likely to have steel than concrete seismic-force-resisting systems. A similar AB for steel moment frames and dual systems with steel moment frames, referencing FEMA 352, should be developed. After that, ABs for less common structural systems, including concentric and eccentric braced frames should be considered.

It is possible that some welded steel moment frames in San Francisco have damage from the 1989 Loma Prieta earthquake that has still not been repaired. An AB implementing FEMA 352 will be needed as the City develops a policy or program to find and repair, or rule out, that damage, as discussed in Part 6.

Recommendation. Develop a new AB for post-earthquake inspection and evaluation of welded steel moment frames.

- The AB should address both safety inspection and structural evaluation, as FEMA 352 covers both topics. The AB should reference FEMA 352 and set specific requirements where the document allows options. See also Part 6.
- Review FEMA 352 for application to representative San Francisco structures. In particular, review the findings of the SEAONC EBC Steel Frame Subcommittee (2004) showing that results are sensitive to the joint sampling method.
- The AB should apply FEMA 352 to interpret SFEBC repair-based retrofit triggers for pre-Northridge welded steel moment frames.

Appendix A

Annotated Building Code Provisions

This appendix presents the current building code provisions applicable to post-earthquake structural evaluation and repair-triggered seismic retrofit. The current, 2016, *San Francisco Existing Building Code* (SFEBC) amends the 2016 *California Existing Building Code* (CEBC), which itself adopts and amends the 2015 *International Existing Building Code* (IEBC). This appendix shows the relevant provisions from all three documents organized by chapters as they appear in the 2016 CEBC. In addition, it provides a commentary on the combined provisions and their application to existing tall buildings, and it offers notes on the forthcoming 2019 CEBC, which will adopt the 2018 IEBC.

This appendix focuses on earthquake evaluation and retrofit provisions that apply to post-earthquake assessment of tall buildings associated with repair projects. As such, it does not include all existing building provisions, all tall building provisions, or even all repair or retrofit provisions. In particular, the following specialty provisions might apply to certain tall buildings but are omitted from this appendix for brevity:

- *Historical Building Code* provisions
- Provisions for state-owned buildings in CEBC Sections 317-322; Section 317.1.1 allows these sections to be adopted by local jurisdictions for use on private buildings
- Regulations used by California's Office of Statewide Health Planning and Development (OSHPD) and Division of State Architect (DSA)
- San Francisco amendments regarding buildings "on a military base selected for closure" (SFEBC Section 326) and homeless shelters (SFEBC Section 401.7), which are not likely to affect tall buildings.

California amendments from the Department of Housing and Community Development (HCD) are included, since they are meant to cover apartment buildings, condominiums, and hotels that might be in tall buildings.

The text is formatted to distinguish between the various source documents using the following legend:

2015 IEBC. Model code. Plain text (except for section headings in bold).

2016 CEBC. California amendments, applicable in San Francisco. Italic plain text (except for section headings in bold).

2016 SFEBC. Additional San Francisco amendments. Bold italic text.

> *Commentary. Plain italic text, indented.*
>
> 2018 IEBC. Model code language already published and expected to be adopted into the 2019 CEBC and SFEBC. Plain text, shaded and indented.

CHAPTER 1
SCOPE AND ADMINISTRATION

1.8 Department of Housing and Community Development

1.8.10 Other Building Regulations

1.8.10.1 Existing structures. *Notwithstanding other provisions of law, the replacement, retention, and extension of original materials and the use of original methods of construction for any existing building or accessory structure, or portions thereof, shall be permitted in accordance with the provisions of this code as adopted by the Department of Housing and Community Development. For additional information, see California Health and Safety Code, Sections 17912, 17920.3, 17922 and 17958.8.*

> *Commentary: This and similar California amendments for HCD are generally interpreted to prohibit upgrade triggers for residential buildings. See the commentary at Section 401.2.1.*

CHAPTER 2
DEFINITIONS

DANGEROUS. Any building, structure or portion thereof that meets any of the conditions described below shall be deemed dangerous:

1. The building or structure has collapsed, has partially collapsed, has moved off its foundation, or lacks the necessary support of the ground.
2. There exists a significant risk of collapse, detachment or dislodgement of any portion, member, appurtenance or ornamentation of the building or structure under service loads.

> *Commentary: The definition of dangerous is qualitative in order to give discretion to the code official. In the context of post-earthquake evaluation, a damaged building might be deemed dangerous under either part of the definition. The identification of dangerous buildings, however, has nothing to do with the evaluation of seismic deficiencies or potential seismic damage under normal pre-earthquake conditions; "service loads" is understood to mean normal day-to-day conditions. In rare cases, the code official might designate an undamaged building dangerous in anticipation of aftershock damage.*

DISPROPORTIONATE DAMAGE. A condition of earthquake-related damage where:

1. The 0.3-second spectral acceleration at the building site as estimated by the United States Geological Survey for the earthquake in question is not more than 0.40g; and

2. In any story, the vertical elements of the lateral force-resisting system have suffered damage such that the lateral load-carrying capacity of the structure in any horizontal direction has been reduced by more than 10% from its pre-damage condition.

> *Commentary: The SFEBC originated the concept of disproportionate earthquake damage with the provision shown above. Neither the 2016 IEBC nor the 2016 CEBC uses this term, but the 2018 IEBC (and 2019 CEBC) will include a similar term with a similar triggering provision (see SFEBC Section 404.4.1, below).*
>
> *San Francisco ABs 098, 099, and 100 provide interpretations regarding calculation of capacity loss as needed to implement this definition. AB-099, for concrete buildings, will apply to some tall buildings.*

The 2018 IEBC definition follows here. Note the differences relative to the SFEBC:

- *Both the IEBC and SFEBC address only earthquake damage, but the IEBC term is more explicit about that.*
- *In the SFEBC, the bounding ground motion is 0.40g. In the 2018 IEBC, to cover the entire country, it is the more generic 0.4S_S.*
- *Relative to the SFEBC, the 2018 IEBC makes an editorial clarification to the second condition, with no change in intent.*

DISPROPORTIONATE EARTHQUAKE DAMAGE. A condition of earthquake-related damage where both of the following occur:
1. The 0.3-second spectral acceleration at the building site as estimated by the United States Geological Survey for the earthquake in question is less than 40% of the mapped acceleration parameter S_S.
2. The vertical elements of the lateral force-resisting system have suffered damage such that the lateral load-carrying capacity of any story in any horizontal direction has been reduced by more than 10% from its predamage condition.

SUBSTANTIAL STRUCTURAL DAMAGE. A condition where one or both of the following apply:
1. In any story, the vertical elements of the lateral force-resisting system have suffered damage such that the lateral load-carrying capacity of the structure in any horizontal direction has been reduced by more than 33% from its predamage condition.

2. The capacity of any vertical gravity load-carrying component, or any group of such components, that supports more than 30% of the total area of the structure's floor(s) and roof(s) has been reduced more than 20% from its predamage condition, and the remaining capacity of such affected elements, with respect to all dead and live loads, is less than 75% of that required by this code for new buildings of similar structure, purpose and location.

Commentary: San Francisco ABs 098, 099, and 100 provide interpretations regarding calculation of capacity loss as needed to implement this definition. AB-099, for concrete buildings, will apply to some tall buildings.

Item 1 of the definition will usually be of greatest interest to post-earthquake evaluation; the corresponding trigger is found in Section 404.2. The definition was changed with the 2012 IEBC to increase *the critical damage level from 20% to 33%. AB-099 still references the old definition.*

Item 2 of the definition is also applicable to post-earthquake evaluation, especially in older buildings in which the lateral force-resisting system is inadequate to protect the gravity system from damage; the corresponding trigger is found in Section 404.3.1.

The 2018 IEBC definition for substantial structural damage (SSD) has been edited for clarity, with no change in intent. It will also include a third category for damage caused by snow that will likely not apply to San Francisco tall buildings.

SUBSTANTIAL STRUCTURAL DAMAGE. A condition where any of the following apply:
1. The vertical elements of the lateral force-resisting system have suffered damage such that the lateral load-carrying capacity of any story in any horizontal direction has been reduced by more than 33% from its predamage condition.
2. The capacity of any vertical component carrying gravity load, or any group of such components, that has a tributary area more than 30% of the total area of the structure's floor(s) and roof(s) has been reduced more than 20% from its predamage condition, and the remaining capacity of such affected elements, with respect to all dead and live loads, is less than 75% of that required by the *International Building Code* for new buildings of similar structure, purpose and location.
3. The capacity of any structural component carrying snow load ....

CHAPTER 3
PROVISIONS FOR ALL COMPLIANCE METHODS

301.1.4 Seismic evaluation and design procedures. The seismic evaluation and design shall be based on the procedures specified in the *California Building Code* or ASCE 41. The procedures contained in Appendix A of this code shall be permitted to be used as specified in Section 301.1.4.2.

Commentary: In the 2018 IEBC, Section 301.1.4 has been edited for clarity and to coordinate with ASCE/SEI 41-17 instead of ASCE/SEI 41-13. The entire section has also been relocated and renumbered as Section 303.3.

The 2016 CEBC adopts only three of the 2015 IEBC's five Appendix A chapters. None of the three adopted chapters applies to tall buildings (or to steel or concrete structural systems of any size).

301.1.4.1 Compliance with International Building Code-level seismic forces. ...

> *Commentary: Section 301.1.4.1 gives the "full" earthquake design criteria applicable when seismic work is triggered by an addition or change of occupancy project; it does not apply to repairs.*

301.1.4.2 Compliance with reduced International Building Code-level seismic forces. Where seismic evaluation and design is permitted to meet reduced *California Building Code* seismic force levels, the criteria used shall be in accordance with one of the following:

1. The *California Building Code* using 75% of the prescribed forces. Values of R, Ω_0 and C_d used for analysis shall be as specified in Section 301.1.4.1 of this code.

2. Except where these requirements are triggered by Section 403.12, structures or portions of structures that comply with the requirements of the applicable chapter in Appendix A as specified in Items 2.1 through 2.5 and subject to the limitations of the respective Appendix A chapters shall be deemed to comply with this section.

...

2.5. Seismic evaluation and design of concrete buildings assigned to Risk Category I, II or III are permitted to be based on the procedures specified in Chapter A5.

3. ASCE 41, using the performance objective in Table 301.1.4.2 for the applicable risk category.

> *Commentary: Section 301.1.4.2 gives the "reduced" earthquake design criteria for use when seismic work is triggered by certain alteration or repair projects. See Section 404.*
>
> *The 2016 CEBC reference to Chapter A5 is inappropriate, as California does not adopt that chapter of the model code. Also, the 2018 IEBC (and presumably the 2019 CEBC) will no longer have Chapter A5; it was omitted because it is essentially identical to ASCE/SEI 41-17. As noted above, none of the other Appendix A chapters adopted by the CEBC is applicable to tall buildings or to concrete or steel structural systems of any size.*

Section 301.1.4.2 is referenced by SFEBC Section 301.2, which contains San Francisco's traditional reduced load criteria with benchmark dates. The CEBC allows three options where reduced loads are allowed. Presumably, all three options are acceptable within the intent of SFEBC Section 301.2.

- *Option 1 is the traditional "75%" approach long embraced by San Francisco.*
- *Option 2 allows five prescriptive approaches for specific building types. As noted above, however, California adopts only three of the five Appendix A chapters, none of which is applicable to tall buildings, and Chapter A5 (not adopted by the CEBC) has already been removed from the 2018 IEBC.*
- *Option 3 allows the use of ASCE/SEI 41-13 with performance objectives that vary by risk category: For RC I or II: Structural Life Safety with a BSE-1E hazard; for RC III: Structural Damage Control with a BSE-1E hazard. Tall buildings would not be expected to be assigned to Risk Category IV.*

301.2 Minimum Lateral Force for Existing Buildings

301.2.1 General. This section is applicable to existing buildings when invoked by SFEBC Section 403. This section may be used as a standard for voluntary upgrades.

Commentary: Section 301.2 gives additional provisions for seismic work triggered by alteration projects in Section 403. Despite the wording of Section 301.2.1, these provisions are also invoked for repair projects by Sections 404.1.1 and 404.2.1 (and by Section 404.4.1, which points back to 404.2). However, Section 404.1.1 requires consideration of two force levels—the reduced forces generally allowed by Section 301.2 and the original design forces, which for relatively recent buildings are likely to be larger than the reduced forces.

An existing building or structure which has been brought into compliance with the lateral force resistance requirements of the San Francisco Building Code in effect on or after the dates shown in Table 301.2.1 [Table 4-1]***, shall be deemed to comply with this section except when a vertical extension or other alterations are to be made which would increase the mass or reduce the seismic resistance capacity of the building or structure. Where multiple building types apply, the later***

applicable date shall be used. Where none of the building types apply, compliance shall be at the discretion of the Director. Building type definitions are given in ASCE 41-13, Table 3-1.

Commentary: Starting with the 2016 edition, this section includes benchmark dates for each structural system (as opposed to the prior provision, which gave a single date for all buildings). Where the benchmark table, as repeated in Table 4-1 below applies, it effectively eliminates the requirement for triggered seismic evaluation (and possible retrofit) in more recent buildings, some as old as 1995.

It is possible (but unclear in the SFEBC) that the benchmark table is not meant to apply when Section 301.2 is invoked as criteria for upgrades triggered by either substantial structural damage or disproportionate damage. As discussed below, the San Francisco amendment in Section 404.1.1 says that seismic evaluations and retrofits triggered by substantial structural damage must consider not only the reduced loads prescribed by Section 301.2 but also "the code under which the building ... was designed." By a plain reading, this changes nothing about whether the benchmark table applies to exempt relatively new buildings. Another interpretation, however, would recognize that the benchmark table (like the similar table in ASCE/SEI 41-17) is meant as a rough approximation of when traditional reduced criteria would be expected to be satisfied. Any requirement to use higher forces represents an attempt to be more conservative than the traditional reduced criteria, so the "original code" requirement in Section 404.1.1 means the benchmark table should not apply. By this interpretation, no building with triggering damage levels would be exempt from upgrade just because it is relatively new.

For example, consider a steel moment-resisting frame structure designed with the 1997 UBC in 1999. According to Table 301.2.1, the benchmark date for this system is 12/28/1995, so the building would be exempt from triggered seismic evaluation—unless the requirement to consider the original 1997 UBC design loads is understood to override the benchmark. Even with the additional requirement, the structure will not necessarily need to be retrofitted. Since it was designed with the 1997 UBC, any evaluation using its original design forces should find the structure adequate, so that it need only be repaired to its pre-damage condition. If the evaluation reveals an original design flaw, however—and especially if the triggering damage is thought to be related to it—then prohibiting the use of the *benchmark table will have been worthwhile. Thus, the point of prohibiting the benchmark table in cases of structural damage is not to force an upgrade but to ensure a thorough review of the original design. On the other hand, if the original design was well documented, it would seem that the engineer of record could simply show compliance by documentation, without the need for a costly and time-consuming full evaluation, so original errors, if there were any, might still not be found.*

Table 4-1 Dates Required to Demonstrate Building Compliance (reproduced from Table 301.2.1 in Section 301.2 of SFEBC)

Building Type	*Date of Compliance*	*Model Code (for reference)*
Wood Frame, wood shear panels (Types W1 & W2)	1/1/1984	UBC 1976
Wood Frame, wood shear panels (Type W1A) Floor areas greater than 3,000 ft2 per level	7/1/1999	UBC 1997
Steel moment-resisting frame (Types S1 & S1a)	12/28/1995	UBC 1994
Steel concentrically braced frame (Types S2 & S2a)	7/1/1999	UBC 1997
Steel eccentrically braced frame (Types S2 & S2a)	1/1/1990	UBC 1988
Buckling-restrained braced frame (Types S2 &S2a)	1/1/2008	IBC 2006
Light metal frame (Type S3)	1/1/2008	IBC 2006
Steel frame w/ concrete shear walls (Type S4)	12/28/1995	UBC 1994
Steel plate shear wall (Type S6)	1/1/2008	IBC 2006
Reinforced concrete moment-resisting frame (Type C1)	12/28/1995	UBC 1994
Reinforced concrete shear walls (Types C2 & C2a)	12/28/1995	UBC 1994
Tilt-up concrete (Types PC1 & PC1a)	7/1/1999	UBC 1997
Precast concrete frame (Types PC2 & PC2a)	1/1/2008	IBC 2006
Reinforced masonry (Type RM1) Flexible diaphragms	7/1/1999	UBC 1997
Reinforced masonry (Type RM2) Stiff diaphragms	12/28/1995	UBC 1994
Seismic isolation or passive dissipation	7/1/1992	UBC 1991

301.2.2 Wind forces. Buildings and structures shall be capable of resisting wind forces as prescribed in San Francisco Building Code Section 1609.

301.2.3 Seismic forces. Buildings and structures shall comply with the reduced International Building Code-level seismic forces, as defined in Section 301.1.4.2. The building separation limitations of Section ASCE 7-10 Section 12.12.3 need not be considered. ...

Commentary: Section 301.2.3 gives the “reduced” earthquake design criteria for use when a seismic upgrade is triggered by certain alteration or repair projects. The first part of this provision is not actually needed; as shown, it merely points to criteria already provided in CEBC Section 301.1.4.2. This sentence remains, however, as a vestige of prior SFEBC provisions that specified reduced forces directly, in coordination with San Francisco upgrade triggers for alteration projects (SFEBC Sections 403.12.1 and 403.12.2)—triggers the IEBC/CEBC does not have.

In addition to setting the force level, Section 301.2 provides benchmarking (Section 301.2.1) and waives building separation requirements (Section 301.2.3); these provisions are unique and traditional to San Francisco. As such, they are perhaps meant to apply only to work triggered by San Francisco amendments, but the way the code is written, they would apply to any project subject to a code section that invokes Section 301.2. As discussed above, however, in cases of substantial structural damage, Section 404.1.1 invokes Section 301.2 but also requires consideration of “the code under which the building or structure was designed.” So, for a building with substantial structural damage, if the original design code was more conservative than the reduced forces allowed by Section 301.2, then the triggered evaluation, as well as any subsequent upgrade, would have to use the larger forces. As discussed at Section 301.2.1, it is unclear whether the requirement to consider the original design code is meant to override or negate the benchmarking allowance.

CHAPTER 4
PRESCRIPTIVE COMPLIANCE METHOD

401.1.2 Existing state-owned structures. *[BSC] ... The provisions of Sections 317 through 322 may be adopted by a local jurisdiction for earthquake evaluation and design for retrofit of existing buildings.*

...

401.2.1 Existing materials. ... ***[HCD 1]*** *Local ordinances or regulations shall permit the replacement, retention and extension of original materials, and the use of original methods of construction [if the] structure complied with the building code provisions in effect at the time of original construction and ... does not become or continue to be a substandard building. ...*

Commentary: This and similar California amendments for HCD are generally interpreted to prohibit upgrade triggers for residential *buildings. However, the final phrase of the provision, regarding "substandard building," alters that general interpretation with respect to earthquake design. "Substandard building" is defined in Health and Safety Code Section 17920.3 to include any residential building with "inadequate structural resistance to horizontal forces."*

With an explicit statement of policy in AB-099, San Francisco defines any building with either disproportionate damage or substantial structural damage to the seismic-force-resisting system to be "substandard" for purposes of applying the HCD amendments to the CEBC. Thus, both of the repair-based seismic upgrade triggers in the SFEBC will apply even to HCD-regulated buildings.

This is a helpful policy clarification as well as a reasonable interpretation. The disproportionate damage trigger is explicitly about damage-prone, and thus inadequate, buildings. Substantial structural damage can occur even in otherwise adequate buildings, but it would not trigger upgrade unless the building would also fail an evaluation with reduced forces, which would indicate inadequacy.

401.3 Dangerous conditions. The building official shall have the authority to require the elimination of conditions deemed dangerous.

401.4 Dangerous conditions. [BSC] *Regardless of the extent of structural or nonstructural damage, the building official shall have the authority to require the elimination of conditions deemed dangerous.*

Commentary: CEBC Section 401.4 is an unnecessary restatement, slightly edited, of Section 401.3. The additional phrase regarding the extent of damage made sense when this provision was located in a different section. Now that the provision is at the top of Chapter 4, the leading phrase is no longer needed. In the 2018 IEBC, this provision has been moved to Chapter 3 where its broad application is even clearer.

SECTION 404
REPAIRS

404.1 General. Buildings and structures, and parts thereof, shall be repaired in compliance with Sections 401.2 and 404. Work on nondamaged components that is necessary for the required repair of damaged components shall be considered part of the repair and shall not be subject to the requirements for alterations in this chapter. Routine maintenance required by Section 401.2, ordinary repairs exempt from permit in accordance with Section 105.2, and abatement of wear due to normal service conditions shall not be subject to the requirements for repairs in this section.

404.1.1 Replacement, retention and extension of original materials. [HCD 1] *Local ordinances or regulations shall permit the replacement, retention and extension of original materials, and the use of original methods of construction [if the] structure complied with the building code provisions in effect at the time of original construction and ... does not become or continue to be a substandard building. ...*

> *Commentary: Section 401.2 allows existing materials to remain unless dangerous. The HCD 1 provision added as Section 404.1.1 is consistent with the amendment to 401.2.1. See the commentary there.*

404.1.1 Repairs. Unless otherwise approved by the Building Official, all structural damage shall be repaired.

Repairs to buildings or structures which have sustained substantial structural damage to lateral force resisting elements shall comply with the minimum lateral force design requirements of Section 301.2 or with the code under which the building or structure was designed, whichever is more restrictive.

Damage may be caused by events or a combination of events, including, but not limited to, fire, explosion, structural pest or wood-destroying organism attack, earthquake, wind storm, vehicular impact, ground subsidence or failure, or the collapse or dislodgement of any portion of any adjacent building or structure. The removal or alteration of structural elements as part of the work described in an approved building permit application shall not be considered to be "damage."

> *Commentary: This additional SFEBC section is misnumbered; it is not the intent of the SFEBC to replace CEBC Section 404.1.1. Further, the second sentence would be more logically located with Section 404.2, since it is specifically about the topic of that section: substantial structural damage to the lateral system.*
>
> *The second sentence invokes the reduced forces of Section 301.2 (which points to CEBC Section 301.1.4.2) but also requires consideration of "the code under which the building ... was designed."*

This means buildings with substantial structural damage to the lateral system (from any cause) must be evaluated with the full original design loads, not with reduced current loads. In addition, it might also mean that the benchmarking exemption in Section 301.2.1 does not apply to cases of substantial structural damage. See the commentary for Section 301.2.1.

404.2 Substantial structural damage to vertical elements of the lateral force-resisting system. A building that has sustained substantial structural damage to the vertical elements of its lateral force-resisting system shall be evaluated in accordance with the applicable provisions of Sections 404.2.1 through 404.2.3.

Exceptions:
1. Buildings assigned to Seismic Design Category A, B or C whose substantial structural damage was not caused by earthquake need not be evaluated or rehabilitated for load combinations that include earthquake effects.
2. One- and two-family dwellings ....

Commentary: Exception 1 does not apply in San Francisco, where high seismicity prevents any building from being assigned to SDC A, B, or C. In the 2018 IEBC, this provision is edited for clarity and renumbered:

405.2.3 Substantial structural damage to vertical elements of the lateral force-resisting system. A building that has sustained substantial structural damage to the vertical elements of its lateral force-resisting system shall be evaluated in accordance with Section 405.2.3.1, and either repaired in accordance with Section 405.2.3.2 or repaired and retrofitted in accordance with Section 405.2.3.3, depending on the results of the evaluation.

Exceptions:
1. Buildings assigned to Seismic Design Category A, B or C whose substantial structural damage was not caused by earthquake need not be evaluated or retrofitted for load combinations that include earthquake effects.
2. One- and two-family dwellings need not be evaluated or retrofitted for load combinations that include earthquake effects.

404.2.1 Evaluation. The building shall be evaluated by a registered design professional, and the evaluation findings shall be submitted to

the code official within 60 days of completion of the evaluation. The evaluation shall establish whether the damaged building, if repaired to its predamage state, would comply with the provisions of this code for wind and earthquake loads. Evaluation for earthquake loads shall be required if the substantial structural damage was caused by or related to earthquake effects or if the building is in Seismic Design Category C, D, E or F.

Wind loads for this evaluation shall be those prescribed in San Francisco Building Code Section 1609. Earthquake loads for this evaluation, if required, shall be according to section 301.2.

Commentary: SFEBC Section 404.2.1 replaces the similar provision in the CEBC. For reference, the corresponding wording from the CEBC and from the 2018 IEBC are given at the bottom of this commentary. Notes on the SFEBC wording in Section 404.2.1:

- *SFEBC includes a 60-day deadline for submittal of the evaluation.*
- *SFEBC makes an unnecessary (and potentially confusing) statement about when earthquake loads must be considered. It's unnecessary because this SFEBC wording merely duplicates Exception 1, already provided in Section 404.2. As indicated in the commentary there, this SFEBC wording is also moot, since every building in San Francisco would be assigned to SDC D, E, or F.*
- *SFEBC specifies reduced loads by referring to Section 301.2. Although not stated here, the additional requirement in Section 404.1.1 regarding the original design code also clearly applies.*

Following is the CEBC/IEBC wording that SFEBC replaces:
404.2.1 Evaluation. The building shall be evaluated by a registered design professional, and the evaluation findings shall be submitted to the building official. The evaluation shall establish whether the damaged building, if repaired to its predamage state, would comply with the provisions of the *California Building Code* for wind and earthquake loads. [Commentary: The balance of the section prescribes full wind loads per CBC Section 1609 but allows reduced seismic loads, including the ASCE 41 option in CEBC Section 301.1.4.2.]

Following is the 2018 IEBC wording, further clarified and renumbered:
405.2.3.1 Evaluation. The building shall be evaluated by a registered design professional, and the evaluation findings shall be submitted to

> the code official. The evaluation shall establish whether the damaged building, if repaired to its predamage state, would comply with the provisions of the *International Building Code* for load combinations that include wind or earthquake effects, except that the seismic forces shall be the reduced seismic forces.

404.2.2 Extent of repair for compliant buildings. If the evaluation establishes compliance of the predamage building in accordance with Section 404.2.1, then repairs shall be permitted that restore the building to its predamage state.

Commentary: In the 2018 IEBC, this provision is edited for clarity and renumbered:

> **405.2.3.2 Extent of repair for compliant buildings.** If the evaluation establishes that the building in its predamage condition complies with the provisions of Section 405.2.3.1, then the damaged elements shall be permitted to be restored to their predamage condition.

404.2.3 Extent of repair for noncompliant buildings. If the evaluation does not establish compliance of the predamage building in accordance with Section 404.2.1, then the building shall be rehabilitated to comply with the applicable provisions of the California Building Code for load combinations that include wind or seismic loads. [Commentary: The balance of this section requires full wind loads per CBC Section 1609 if the damage was caused by wind, but allows original wind loads otherwise. The section also allows reduced seismic loads.]

Commentary: In the 2018 IEBC, this provision is edited for clarity and renumbered:

> **405.2.3.3 Extent of repair for noncompliant buildings.** If the evaluation does not establish that the building in its predamage condition complies with the provisions of Section 405.2.3.1, then the building shall be retrofitted to comply with the provisions of this section. The wind loads for the repair and retrofit shall be those required by the building code in effect at the time of original construction, unless the damage was caused by wind, in which case the wind loads shall be in accordance with the *International Building Code*. The seismic loads for this retrofit design shall be those required by the building code in effect at the time of original construction, but not less than the reduced seismic forces.

404.3 Substantial structural damage to gravity load-carrying components. Gravity load-carrying components that have sustained substantial structural damage shall be rehabilitated to comply with the applicable provisions of the *California Building Code* for dead and live loads. [Commentary: The balance of this section accounts for snow loads and live loads and requires consideration of load path from any affected members.]

> *Commentary: This section is generally unrelated to earthquake, except for the subsection 404.3.1. Also, in the 2018 IEBC, edited for clarity and renumbered:*
>
> **405.2.4 Substantial structural damage to gravity load-carrying components.** Gravity load-carrying components that have sustained substantial structural damage shall be rehabilitated to comply with the applicable provisions for dead and live loads in the *International Building Code*. Snow loads shall be considered if the substantial structural damage was caused by or related to snow load effects. Undamaged gravity load-carrying components that receive dead, live or snow loads from rehabilitated components shall also be rehabilitated if required to comply with the design loads of the rehabilitation design.

404.3.1 Lateral force-resisting elements. Regardless of the level of damage to vertical elements of the lateral force-resisting system, if substantial structural damage to gravity load-carrying components was caused primarily by wind or seismic effects, then the building shall be evaluated in accordance with Section 404.2.1 and, if noncompliant, rehabilitated in accordance with Section 404.2.3.

Exceptions:
1. One- and two-family dwellings ....
2. Buildings assigned to Seismic Design Category A, B, or C whose substantial structural damage was not caused by earthquake need not be evaluated or rehabilitated for load combinations that include earthquake effects.

> *Commentary: The point of this section is to address archaic structures in which the gravity system elements are acting as a* de facto *lateral system, as evidenced by their damage in a wind or seismic event. The provision points back to Section 404.2.1, indicating that this case should be treated like a case of substantial structural damage (SSD) to the lateral system.*

There is one unclear aspect to this provision. For criteria, Section 404.2.1 points to Section 301.2, but Section 404.1.1 requires additional consideration of original design loads. Since Section 404.3.1 points back only to Section 404.2.1, but not to 404.1.1, so some might argue that the additional requirement there does not apply. This is almost certainly an oversight in the SFEBC, however, as the point of this section is to treat this case of SSD the same way one would treat SSD to the lateral system. If additional criteria apply to cases subject to Section 404.2.1, the same criteria should apply here.

In the 2018 IEBC, this provision is edited for clarity and renumbered:

405.2.4.1 Lateral force-resisting elements. Regardless of the level of damage to vertical elements of the lateral force-resisting system, if substantial structural damage to gravity load-carrying components was caused primarily by wind or seismic effects, then the building shall be evaluated in accordance with Section 405.2.3.1 and, if noncompliant, retrofitted in accordance with Section 405.2.3.3.

Exceptions:

1. Buildings assigned to Seismic Design Category A, B, or C whose substantial structural damage was not caused by earthquake need not be evaluated or retrofitted for load combinations that include earthquake effects.
2. One- and two-family dwellings need not be evaluated or retrofitted for load combinations that include earthquake effects.

404.4 Less than substantial structural damage. For damage less than substantial structural damage, repairs shall be allowed that restore the building to its predamage state. New structural members and connections used for this repair shall comply with the detailing provisions of the *California Building Code* for new buildings of similar structure, purpose and location.

Commentary: In the 2018 IEBC, this provision is edited for clarity and renumbered:

405.2.1 Repairs for less than substantial structural damage. Unless otherwise required by this section, for damage less than substantial structural damage, the damaged elements shall be permitted to be restored to their predamage condition.

404.4.1 Disproportionate Damage. Buildings with Disproportionate Damage shall be subject to the requirements of Section 404.2 for earthquake evaluation and rehabilitation as if they had substantial

structural damage to vertical elements of the lateral-force-resisting system.

Permit application for required rehabilitation work shall be submitted to the Department within 1 year after the earthquake, and the work shall be completed as specified in Table B of San Francisco Building Code Section 106A.4.4.

> *Commentary: SFEBC Section 404.4.1 is the San Francisco amendment that implements the definition of disproportionate damage given in Chapter 2. Since it is not necessarily true that disproportionate damage will always be less than substantial structural damage, and for general clarity, it would be better if future SFEBC editions would make this provision separate from Section 404.4. The 2018 IEBC organization and numbering are recommended.*
>
> *By pointing to SFEBC Section 404.2 for the triggered scope and criteria, this additional upgrade trigger invokes the reduced seismic loads and other criteria given in SFEBC Section 301.2. Presumably, by also referencing the requirements for substantial structural damage to the lateral system, this trigger also invokes the additional requirement in Section 404.1.1 to consider the original design loads as well. The SFEBC could be clearer, but it stands to reason that if additional criteria apply to cases subject to Section 404.2.1, the same criteria should apply here.*
>
> *Following is the similar triggering provision in the 2018 IEBC. The IEBC trigger applies only to SDC D, E, and F, but this limit is moot in San Francisco, where every building would be assigned to SDC D, E, or F.*
>
> **405.2.2 Disproportionate earthquake damage.** A building assigned to Seismic Design Category D, E or F that has sustained disproportionate earthquake damage shall be subject to the requirements for buildings with substantial structural damage to vertical elements of the lateral force-resisting system.

404.4.2. Other damage. For damage less than substantial structural damage that is not Disproportionate Damage, repairs shall be allowed that restore the building to its predamage state, based on material properties and design strengths applicable at the time of original construction. New structural members and connections used for this

repair shall comply with the detailing provisions of this code for new buildings of similar structure, purpose and location.

Commentary: This San Francisco amendment was perhaps necessary when the SFEBC introduced disproportionate damage, but it is no longer needed, as it merely duplicates the intent of CEBC/IEBC Section 404.4. For future SFEBC editions, the 2018 IEBC organization and numbering are recommended.

Appendix B

References

ASCE, 2017, *Seismic Evaluation and Retrofit of Existing Buildings*, ASCE/SEI 41-17, American Society of Civil Engineers Structural Engineering Institute, Reston, Virginia.

CEBC, 2016, *2016 California Existing Building Code, California Code of Regulations, Title 24, Part 10*, California Building Standards Commission, Sacramento, California.

CCSF, 2011, *CAPSS Earthquake Safety Implementation Program Workplan 2012-2042*, the City and County of San Francisco, California.

CCSF, 2012a, *Post-Earthquake Repair and Retrofit Requirements for Concrete Buildings*, Administrative Bulletin 099, Department of Building Inspections, San Francisco, California.

CCSF, 2012b, *Post-Earthquake Repair and Retrofit Requirements for Wood-Frame Residential Buildings with Three or More Dwelling Units*, Administrative Bulletin 098, Department of Building Inspections, San Francisco, California.

CCSF, 2012c, *Post-Earthquake Repair and Retrofit Requirements for One- and Two-Family Units*, Administrative Bulletin 100, Department of Building Inspections, San Francisco, California.

CCSF, 2016, *The San Francisco Existing Building Code*, the City and County of San Francisco, California.

CCSF, 2017, *Building Façade Inspection and Maintenance*, Administrative Bulletin 110, Department of Building Inspections, San Francisco, California.

FEMA, 1998, *Evaluation of Earthquake Damaged Concrete and Masonry Wall Buildings*, FEMA 306, prepared by the Applied Technology Council for the Federal Emergency Management Agency, Washington, D.C.

FEMA, 2000, *Recommended Postearthquake Evaluation and Repair Criteria for Welded Steel Moment-Frame Buildings,* FEMA 352, prepared by the SAC Joint Venture, a partnership of the Structural Engineers Association of California, the Applied Technology Council, and

California Universities for Research in Earthquake Engineering, for the Federal Emergency Management Agency, Washington, D.C.

ICC, 2018, *International Building Code, International Code Council*, Country Club Hills, Illinois.

SEAONC EBC Steel Frame Subcommittee, 2004, "Post-Earthquake Evaluation of Welded Steel Moment Frames: Case Studies Using FEMA-352," *Proceedings*, 2004 SEAOC Convention, Monterey, California.

PART 5:
Tall Building Effects on Post-earthquake Recovery

Chapter 1

Introduction

1.1 Background

In an effort to understand the impact that tall building performance may have on the recovery of the City of San Francisco after a major earthquake, this Part provides an overview of seismic risks that are disproportionately associated with tall buildings (i.e., tall building effects), with recommendations for new policy considerations and further research. This Part corresponds to Recommendations 2C, 2D, 3A, 3C, and 3G presented in the Summary Recommendations.

A characterization of building tallness in relevant building codes suggests that while no unique height threshold is valid to distinguish between “tall” versus “non-tall” buildings, the fact that different thresholds do exist for fire safety, seismic or planning amongst other considerations, is an indication that tall buildings present unique challenges.

This Part addresses aspects of the following tasks described in the *CAPSS Earthquake Safety Implementation Program Workplan 2012-2042* (CCSF, 2011):

- Task A.1.b: Provide information and assistance about insurance
- Task A.6.i: Study fire-related earthquake resilience topics
- Task A.4.f: Update post-earthquake inspection policies and procedures

1.2 Acknowledgements

The findings were developed by means of a literature review and consultation with a wide range of professionals including design engineers (structural and building services), building owners and managers, first responders, and the insurance sector among other stakeholders. ATC gratefully acknowledges the following parties for the insightful comments and discussion in the development of this report and its recommendations: Ibbi Almufti, Mary Comerio, Wayne Gaw, Kevin Geraghty, Prasad Gunturi, Akshay Gupta, Matt Hansen, Dennis Mulqueeney, John Osteraas, Robert Postel, Marcelo Ramirez, Nilesh Shome, members of the San Francisco Lifelines Council, and members of the San Francisco Building Owners and Managers Association (BOMA).

1.3 Organization

Chapter 2 reviews tall building effects and characterization of building tallness in current San Francisco policy for post-earthquake structural evaluation.

Chapter 3 discusses safety and emergency response.

Chapter 4 reviews consequences from damage.

Chapter 5 reviews programs and procedures to help accelerate recovery, including inspection and tagging, insurance, and concepts of re-occupancy and functional recovery.

Chapter 6 provides a summary of tall building effects identified, along with recommendations for new policy considerations and further research to mitigate these effects, and a brief discussion on their relevance to tall versus all buildings, and future versus existing buildings.

A list of references is provided at the end of this Part.

Chapter 2

Tall Building Effects

2.1 Overview

There are presently 156 buildings 240 ft or taller either constructed or permitted for construction in San Francisco (see Part 1). About 60% of these buildings house commercial (office) occupancies, and 40% are residential or hotel occupancies. With a large concentration of the tall buildings in the downtown districts of San Francisco, the tall buildings play a key role in the socio-economic activity of the City. Tall buildings house large concentrations of businesses and residents, which raises concerns that seismic damage to one or more tall buildings can have a significant impact on the building occupants and the surrounding neighborhood. Events such as the 2010-2011 Canterbury earthquake sequence in Christchurch, New Zealand highlight the impact of damaged tall buildings on the business continuity of districts where buildings are clustered close together. For example, following the 2011 Christchurch earthquake, the 26-story Hotel Grand Chancellor sustained severe damage, where the risk of collapse from aftershocks prompted authorities to set up a cordon around the building with a roughly 300-foot radius, roughly equal to the height of the damaged building (NZPA, 2011). Thus, there were significant indirect losses attributed to business disruption in surrounding buildings in addition to the direct economic loss associated with the damaged hotel building.

Roughly 75% of the existing tall buildings in San Francisco were constructed before 2000, following what are now considered outdated code prescriptive design requirements that do not necessarily provide consistent levels of safety from major earthquakes. Tall buildings designed since about 2008, when San Francisco introduced Administrative Bulletin (AB) 083 (CCSF, 2008), typically follow a performance-based seismic design approach where building's structural response under strong earthquakes is explicitly evaluated using advanced structural analysis. While the design approach is called "performance-based," the provisions of AB-083 are generally intended only to demonstrate compliance with minimum code requirements to ensure life safety under extreme earthquakes. Thus, while more recently designed buildings are more reliable in terms of life safety, both older existing and newly designed tall buildings are susceptible to damage from strong earthquakes, which could result in extensive downtime and potential need for demolition and replacement.

In an effort to understand the impact that tall building performance may have on the recovery of the City of San Francisco after a major earthquake, this Part provides an overview of seismic risk factors that are disproportionately present with tall buildings (in contrast with the entire building stock), with recommendations for new policy considerations and further research to better understand and control the risks.

2.2 Characterization of Building Tallness

From a structural and seismic perspective, tall buildings are generally associated with larger drifts, higher overturning forces, and more complex dynamic response. None of those complexities, however, necessarily makes a taller building riskier, in part because building codes have implemented provisions to account for such effects since at least the 1960s. In taller buildings, the structural design requirements are often governed as much or more by wind loading as compared to earthquake ground shaking. In addition, there are countless examples of short buildings behaving poorly in earthquakes, due to structural irregularities, falling hazards, and collapse-prone deficiencies.

To the extent that certain seismic deficiencies or potential risks might be associated with tallness, it is useful to review the precedents in existing building codes, standards, and policies that categorize buildings by height. This review includes the 2016 edition of the *San Francisco Building Code* (SFBC) and *San Francisco Existing Building Code* (SFEBC), hereinafter referred to as the "Current Code," which reference the 2016 *California Building Code* (CBC) and 2016 *California Existing Building Code* (CEBC). Two previous editions of the *Uniform Building Code* (UBC) adopted in San Francisco are also reviewed. Provisions in or similar to the 1964 UBC would have applied at the start of the tall building era in San Francisco. Provisions in or similar to the 1985 UBC would have applied at the peak of the downtown tall building boom of the 1970s and 1980s. Part 1 presents a detailed inventory of tall buildings in the City with regard to age.

The CBC defines a "high-rise" building, independent of its use, as one "having floors used for human occupancy located more than 75 feet above the lowest floor level having building access." The roof elevation of a high-rise building would therefore be about 85 feet above grade, but in this report, for simplicity, the "high-rise" critical height is defined as 75 feet. The CBC uses this "high-rise" definition only in provisions for fire safety and egress, so although a distinction is made based on height, this has little effect on earthquake design and makes no implication regarding the seismic performance of taller buildings.

Guidelines for Performance-Based Seismic Design of Tall Buildings (PEER, 2017), which is widely used for the seismic design of tall buildings in San Francisco, specifies its scope of application as applying to buildings having the unique seismic response characteristics of tall buildings including: (1) a fundamental translational period of vibration significantly in excess of 1 second; (2) high mass participation and lateral response in higher modes of vibration; and (3) a seismic-force-resisting system with a slender aspect ratio such that substantial portions of the lateral drift result from axial deformation of the walls and/or columns as compared to shearing deformation of the frames or walls. The following sections introduce and discuss structural response parameters that are considered meaningful measures of "tallness," and how past and current building codes have dealt with these parameters:

2.2.1 Building Period

The building period is a characteristic vibrational property of a building, corresponding to the time for the building to undergo one cycle of lateral sway motion. Assuming that a building period of 2 seconds qualifies as tall by the terms of the PEER *Guidelines*, i.e., "significantly in excess of 1 second," and applying the current building code's default formula for estimating the fundamental period, then the building heights that the PEER *Guidelines* might consider "tall" range from about 200 ft for moment frame systems to over 400 ft for stiffer wall and braced frame systems.

2.2.2 High Mode Effects

Higher mode effects cover building lateral drifts and accelerations and response properties associated with building vibrational modes with higher frequencies (shorter periods) than the fundamental vibration mode. Through the equivalent lateral force method, Current Code implicitly considers higher mode effects for every building. In past codes, higher mode effects were captured by modifying the vertical distribution of lateral design forces for certain buildings.

At the time of the downtown development boom of the 1970s and 80s, the UBC required consideration of high mode effects only where the building's fundamental period exceeded 0.7 seconds. Using1985 UBC's formulas for period, this corresponds to a structure height of about 7 stories (80 ft) for a ductile moment frame structure and up to about 12 stories (130 ft) for stiffer systems.

2.2.3 Aspect Ratio

This is the ratio of the building height to the plan view dimension at ground level. Because of its formula for estimating period, the critical heights listed in 1985 UBC for considering higher mode effects were related to the building's aspect ratio. In earlier codes from the 1970s, the higher mode effect was ignored for any building with an aspect ratio less than 3:1. In the 1964 UBC, higher mode effect was ignored for any building with an aspect ratio up to 5:1.

Some of San Francisco's oldest buildings still show a typical lot width of about 25 ft, suggesting a critical height of 125 ft in the 1960s or 75 ft in the 1970s. But the taller buildings from the 1960s and later do not have a typical width, thus the critical height by contemporary codes would have been significantly greater, but highly varied.

Aspect ratio was also considered as an indicator of higher floor accelerations in the design requirements for nonstructural components such as rooftop tanks, chimneys, and penthouses. The 1964 UBC increased the design force for these components by 50% when the building's aspect ratio exceeded 5:1. In the 1985 UBC, this requirement changed to a more general requirement to consider actual dynamic properties of the component and the building structure.

2.2.4 Story Drift Ratio

The story drift ratio is a measure of the lateral displacement of a building under earthquakes, calculated as the peak lateral drift occurring over one story, divided by the story height. At the peak of tall building construction, the 1985 UBC set a peak elastic story drift ratio limit of 0.005 (roughly equivalent to an inelastic drift ratio limit of 0.025), but it was independent of the structural system, dynamic characteristics, or height. At the start of the tall building era, the 1964 UBC required only that "lateral deflections or drift ... shall be considered in accordance with accepted engineering practice." The 2017 PEER *Guidelines* limit the inelastic story drift ratio under Maximum Considered Earthquake (MCE) ground motions to 0.03, which is roughly equivalent to the limit of 0.02 on design story drift ratio of the current CBC and ASCE/SEI 7-16, *Minimum Design Loads and Associated Criteria for Buildings and Other Structures* (ASCE, 2017).

2.2.5 System Ductility Requirements

These are building code requirements that limit the applicability of structural systems and materials, based on their ability to deform and absorb energy during a large earthquake. Aside from the criteria suggested by the PEER

Guidelines, the building code has long used height limits to guide the selection of seismic-force-resisting systems, as follows:

- 1964 UBC Section 2312(j): "Buildings more than 13 stories or one hundred and sixty feet (160') in height shall have a complete moment resisting space frame ... made of a ductile material or a ductile combination of materials."
- 1985 UBC Section 2312(j)1.B: "Buildings more than 160 feet in height shall have ductile moment-resisting space frames ...."

Instead of assigning systems to buildings of certain heights, current code provisions specify height limits to buildings with different systems (ASCE/SEI 7-16 Table 12.2-1):

- For typical office, hotel, or residential buildings in high seismic areas (Seismic Design Category D or E), ductile moment frames have no height limit, whereas ductile shear wall and braced frame systems are limited to 240 ft, or 160 ft if the building has an extreme torsional irregularity or an unbalanced configuration.
- For essential facilities (Risk Category IV and Seismic Design Category F), those same structural systems are limited further, to 160 or 100 ft, if the building does not have extreme torsional irregularity or an unbalance configuration.

2.2.6 Collapse-Prone Existing Buildings

San Francisco has three programs that mandate retrofit of existing buildings. SFEBC Chapters 4A and 4B cover the mandatory retrofit of unreinforced masonry bearing wall buildings, but their requirements are independent of building height. Similarly, SFEBC Section 329 mandates seismic evaluation of private schools, but it makes no distinctions based on building height.

SFEBC Chapter 4D covers the mandatory retrofit of woodframe residential "soft-story" buildings. Only buildings three stories tall (including two above grade stories and a basement) or taller are required to comply.

2.2.7 Importance or Risk Category

The building code acknowledges differences between building uses by assigning every new building to a risk category. Different Risk Categories are then subject to different design criteria, including the use of Importance Factors to amplify the Design Earthquake loads.

- 1964 UBC had no Importance Factors and no special earthquake design provisions even for what are now considered essential facilities.

- 1985 UBC increased design forces by 25% for assembly occupancies, and by 50% for essential facilities, but neither category was linked to building height.
- Current Code (2016 CBC Table 1604.5) requires a 25% increase in design forces for buildings assigned to Risk Category III: "Buildings ... that represent a substantial hazard to human life in the event of a failure," regardless of height.
- Current Code also requires a 50% increase, as well as consideration of post-earthquake functionality, for hospitals, fire stations, emergency shelters, and other "essential facilities" assigned to Risk Category IV, regardless of height.

In summary, the building code's conception of importance and risk is only loosely connected with building height. In concept, a collapse-prone building of almost any size could be said to represent "a substantial hazard to human life," but in practice, buildings are only assigned to Risk Category III if they match one of the examples given in the code. None of those examples is specifically related to building height, but the category does include "any other occupancy with an occupant load greater than 5,000." To establish a building's occupant load for design, the code presumes 100 net square feet per occupant in an office or similar area. Using that value, an occupant load of 5,000 would only occur in a building with at least 500,000 net square feet, or a gross square footage of at least 750,000 square feet.

Per *San Francisco Property Information Map* (CCSF, 2017a), only eight buildings within the inventory presented in Part 1 are office buildings with building areas of at least 750,000 square feet, and all of them are over 400 feet tall. Thus, it can be assumed that in San Francisco, the building code envisions office buildings taller than about 400 feet to be candidates for assignment to Risk Category III. Per Current Code, such a building would then be subject to the same additional requirements of other Risk Category III facilities, including restaurants or assembly halls with seating for 300. Due to lower occupant density, tall residential buildings would rarely, if ever, trigger the Risk Category III criteria.

2.2.8 Structural Integrity

Current Code (2016 CBC Section 1615) has special provisions for structural integrity meant to provide additional resistance to collapse, usually associated with blast or other unanticipated loads. They apply only to "high-rise buildings that are assigned to Risk Category III or IV."

The current CBC code defines “high-rise building” to include most buildings taller than 75 feet. As noted above, only a building approaching 400 ft tall would be assigned to Risk Category III on the basis of its height alone. Even so, the code’s association of special safety provisions with height does imply a systematic categorization of “tall” buildings.

2.2.9 Falling Hazards

The earthquake design requirements of building codes have provisions that apply to the design and installation of architectural components (façade and ceiling systems, partition walls), mechanical components (mechanical equipment) and other nonstructural components that can represent potential falling hazards. As with its drift limits, the building code’s nonstructural provisions are independent of the building’s height.

In 1969, San Francisco mandated bracing for unreinforced masonry parapets (2016 SFEBC Chapter 4C), but the requirements were independent of building height.

San Francisco has also implemented a mandatory façade inspection program (2016 SFEBC Chapter 4E) to identify damaged, deteriorated, or otherwise vulnerable cladding conditions that represent earthquake falling hazards. The program applies to buildings five stories and taller.

2.2.10 Fire Safety

As noted above, Current Code (2016 CBC Section 403) imposes special fire safety and egress requirements for high-rise buildings, defined as those taller than about 75 feet. Within Section 403, the code sets another line at 420 ft; buildings taller than that are limited in the types of fire-resistive construction they may use. The 1985 UBC (Sec 1807(a)) made a similar requirement for office and general use buildings (Group B, Division 2) and for hotels and apartment houses (Group R, Division 1).

Also related to fire resistance, but in the building code chapter on allowable building heights and areas, 2016 CBC Table 504.3 makes height distinctions for different types of construction with and without sprinkler systems. For most occupancies:

- 160 to 180 ft, for Type I.B construction, non-sprinklered and sprinklered, respectively
- 65 to 85 ft, for Type II, III, or IV construction, non-sprinklered and sprinklered, respectively
- 50 to 70 ft, for Type V construction (woodframe, generally), non-sprinklered and sprinklered, respectively

where construction Types I and II are those in which the building elements are of noncombustible materials; Type III is that in which the exterior walls are of noncombustible materials and the interior building elements are of any material permitted by the code; Type IV is that in which the exterior walls are of noncombustible materials and the interior building elements are of solid or laminated wood without concealed spaces; Type V is that in which the structural elements, exterior walls and interior walls are of any materials permitted by this code; Type A refers to fire-protected systems; and Type B refers to unprotected ones. See *International Building Code* (ICC, 2016) Section 602 for more detailed definitions.

1985 UBC Table 5-D sets similar limits for office buildings (Occupancy Group B) and hotels (Occupancy Group R-1):

- 160 ft or 12 stories, for Type II fire resistive construction
- 65 ft or 4 stories, for Type II, III, or IV one-hour construction
- 50 ft or 3 stories, for Type V one-hour construction

1964 UBC Table 5-D height limits for office buildings (Occupancy Group F-2) and hotels or apartment houses (Occupancy Group H) were slightly more restrictive:

- 95 ft or 6 stories (5 stories for H occupancy), for Type II construction
- 65 ft or 4 stories, for Type III or IV one-hour construction
- 50 ft or 3 stories, for Type V one-hour construction

2.2.11 Planning and Zoning

In addition to building code requirements, San Francisco sets height and bulk limits for purposes of planning and zoning. In the northeast portion of the city, which includes the downtown and adjacent residential neighborhoods, these are shown on San Francisco Zoning Map Sheet HT01 (CCSF, 2017b). The map includes special block-size zones for specific projects, and in most downtown neighborhoods the limits vary from block to block. Some neighborhood have consistent limits:

- 40 ft: Russian Hill, North Beach
- 50 ft: Chinatown
- 65 ft: Nob Hill
- 80 ft: Union Square
- 150 ft: Second Street

2.2.12 Summary Table

Table 5-1 summarizes the precedents discussed. Although each of the height values in the table has some meaning, the main point of the table is to illustrate there is no single height value that distinguishes “tall” buildings from “non-tall” buildings for all circumstances.

Table 5-1 Height Precedents in Past and Current Codes and Policies

Critical Height	Issue	Source and Description
420 ft	Fire Safety	Current Code (2016 CBC Section 403): Taller buildings must meet all fire-resistance requirements for Type I.A construction, as well as additional egress requirements
~400 ft	Risk Category	Current Code: Approximate height at which an office building would be assigned to Risk Category III based on occupant load of 5,000
240 ft	Risk Category, seismic, structural	Current Code: Height limit for shear wall and braced frame systems in normal occupancy without torsion-prone configuration
160 – 180 ft	Fire Safety	Current Code (2016 CBC Table 504.3): Maximum height for Type I.B construction*, non-sprinklered or sprinklered, respectively
160 ft	Seismic, structural	1985 UBC: Taller buildings must have a ductile moment-resisting frame
160 ft	Risk Category, seismic, structural	Current Code: Height limit for shear wall and braced frame systems in normal occupancy with torsion-prone configuration Current Code: Height limit for shear wall and braced frame systems in essential facilities* without torsion-prone configuration
13 stories	Seismic, structural	1964 UBC: Taller buildings must have a ductile moment-resisting frame
12 stories or 160 ft	Fire Safety	1985 UBC (Table 5-D): Maximum height for Type II construction
150 ft	Planning and Zoning	SF Zoning Map Sheet HT01 (CCSF, 2017b): Height limit in Second Street neighborhood
80 – 130 ft	Seismic, structural	1985 UBC: Structure subject to provision for higher mode effects
100 ft	Risk Category, seismic, structural	Current Code: Height limit for shear wall and braced frame systems in essential facilities* with torsion-prone configuration
95 ft or 5 – 6 stories	Fire Safety	1964 UBC (Table 5-D): Maximum height for Type II construction (6-story limit for office buildings; 5-story limit for hotels and apartment houses)
65 – 85 ft	Fire Safety	Current Code (2016 CBC Table 504.3): Maximum height for Type II, III, or IV construction, non-sprinklered or sprinklered, respectively
80 ft	Planning and Zoning	SF Zoning Map Sheet HT01 (CCSF, 2017b): Height limit in Union Square neighborhood
75 ft	Fire Safety	Current Code (2016 CBC Section 403): “High-rise buildings” must comply with special fire safety and egress provisions. 1985 UBC (Section 1807a): Taller office buildings, hotels, and apartment houses must comply with special fire safety and egress provisions
75 ft	Risk Category; Structural Integrity	Current Code: “High-rise buildings” assigned to Risk Category III or IV must comply with structural integrity provisions
7 stories	Seismic, structural	1985 UBC: Moment frame structure subject to provision for higher mode effects

Table 5-1 Height Precedents in Past and Current Codes and Policies (continued)

Critical Height	Issue	Source and Description
50 – 70 ft	Fire Safety	Current Code (2016 CBC Table 504.3): Maximum height for Type V construction, non-sprinklered or sprinklered, respectively
65 ft	Planning and Zoning	SF Zoning Map Sheet HT01 (CCSF, 2017b): Height limit for Nob Hill neighborhood
5 stories	Seismic, nonstructural	Current Code (2016 SFEBC Chapter 4E): Façade inspection required for buildings of this height or taller
4 stories or 65 ft	Fire Safety	1985 UBC (Table 5-D): Maximum height for Type II, III, or IV one-hour construction 1964 UBC (Table 5-D): Maximum height for Type III or IV one-hour construction
50 ft	Planning and Zoning	SF Zoning Map Sheet HT01 (CCSF, 2017b): Height limit in Chinatown neighborhood
40 ft	Planning and Zoning	SF Zoning Map Sheet HT01 (CCSF, 2017b): Height limit in Russian Hill and North Beach neighborhoods
3 stories or 50 ft	Fire Safety	1964 and 1985 UBC (Table 5-D): Maximum height for Type V one-hour construction
2 stories plus basement	Seismic, structural	Current Code (2016 SFEBC Chapter 4D): “Soft story” buildings of this height or taller must be retrofitted.

* Essential facilities corresponding to Risk Category IV (RC IV) and Seismic Design Category F (SDC F). See International Building Code (ICC, 2016) Table 1604.5 for RC and Section 1613 for SDC classifications.

Chapter 3

Safety and Emergency Response

3.1 Cordoning

Following moderate to severe earthquakes, there is risk of falling hazards due to structural and nonstructural building component damage. An earthquake damaged building with significant permanent deformations may pose a significant collapse risk under the threat of aftershocks. Similarly, nonstructural components, such as façades and building appendages, could be further damaged and pose a life-safety hazard on surrounding areas due to risk of falling.

These public safety reasons can lead to imposing block-long barricades, or cordons, around large areas. In addition to falling hazards and the collapse risk of one or more tall buildings, other reasons that may warrant such cordons include: shoring, stabilization, and demolition activities; extensive debris; damaged utilities; unstable geological features (e.g., landslides or liquefaction); and vandalism.

Recent earthquake disasters have highlighted gaps in safety and awareness for cordoning practices. For instance, in the Canterbury earthquake sequence in 2010-2011 in Christchurch, New Zealand, previously damaged buildings that had been partially stabilized or cordoned, subsequently collapsed during an aftershock, killing four pedestrians and eight people in a bus (CALBO, 2013).

3.1.1 Current Guidance

California Building Officials (CALBO) have developed interim guidelines for barricading, cordoning, emergency evaluation and stabilization of buildings with substantial damage in disasters (CALBO, 2013). This section provides a review of the guidance.

After damaging events, government personnel will deploy to districts within the jurisdiction that are reported to have experienced damage (or districts known to be vulnerable). Those personnel first to arrive at a scene of severe damage should take steps to initiate the evacuation of people out of harm’s way and prevent further access. As outlined in the CALBO (2013) guidelines, because first responders may be initially overwhelmed in the immediate aftermath of a damaging earthquake, Department of Building Inspection (DBI) personnel, must be trained to have appropriate levels of knowledge, skills and experience to:

- Identify structures that are obviously or suspected to be damaged
- Reduce the public's exposure to risks
- Set up temporary cordons at safe horizontal distances of falling or collapse hazards

Safe distances for initial barricades, as illustrated in Figure 5-1, are generally conservatively set at 1.5 times the height of falling hazards to allow for the possibility that falling items could bounce and shatter (NIOSH, 2009). Once the nature and extent of building damage is further investigated, shorter safe distances may be justified. While these safe distances for initial barricading have limited implication on surrounding areas when applied to low to mid-rise buildings, they could imply closing off entire city blocks when applied to tall buildings, as illustrated in Figure 5-2. Cordons around tall buildings are more likely to interfere with San Francisco's emergency priority routes. Furthermore, if cordons around multiple tall buildings are required, this could lead to large areas within the Financial District being closed off, where most of the city's tall buildings are clustered together.

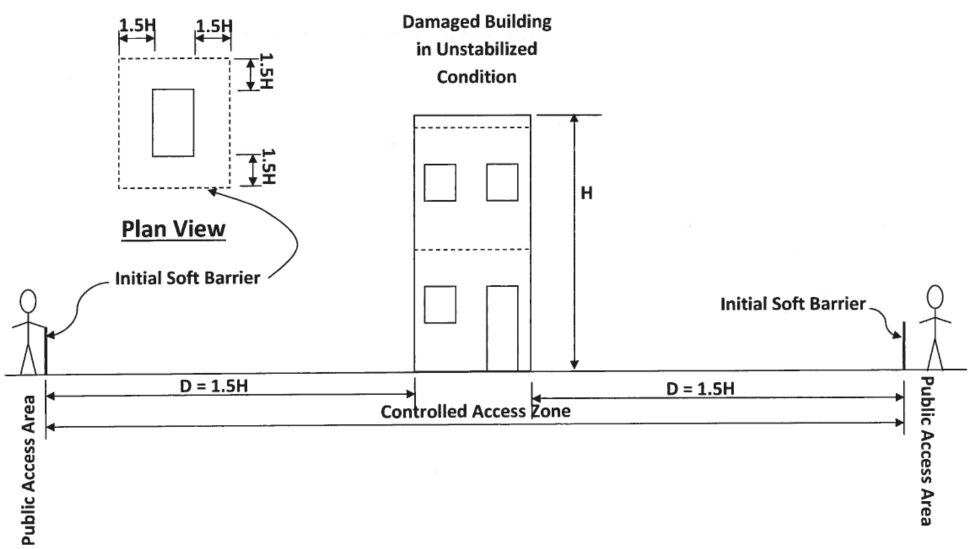


Figure 5-1 Initial cordoning safe distances (CALBO, 2013).

It is important that the City establish a lead agency, as well as clear roles and responsibilities associated with post-earthquake evaluations and the ensuing cordons around damaged buildings. Furthermore, guidelines for cordoning around tall buildings should be developed prior to the earthquake, in coordination with relevant groups, including building department, lifelines, fire, police, public works, and emergency management staff. The City should also carry out table-top exercises to familiarize staff with the challenges and procedures likely to follow disasters. Per CALBO (2013), the absence of a lead agency appointed by ordinance places the Building Official in the lead for these actions and requires emergency stabilization, debris removal, and barrier installation work to be initially paid by the jurisdiction and later recovered through legal action against building owners.

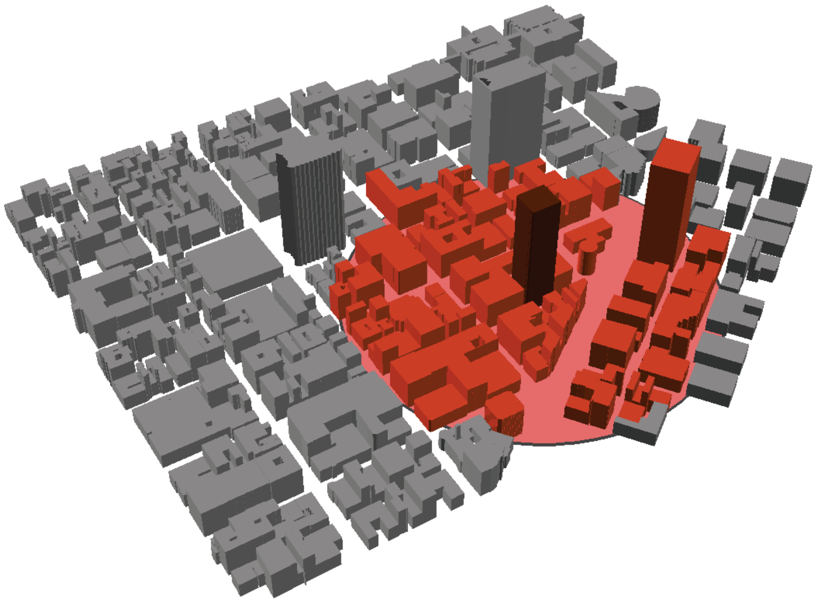


Figure 5-2 Illustration of a tall building cordon around a hypothetical 520-ft tall building with a cordon radius equal to 1.5 times the building height.

Following CALBO recommendations for complex buildings and critical facilities, the City should supplement available staff and expertise through mutual aid to speed emergency response and recovery by expediting barricading, emergency stabilization and falling hazard removal activities. In addition, establishment of a Tall Buildings Task Force to conduct the following work supplement other available resources as follows:

1. Assist in conducting rapid assessments of tall buildings.
2. Manage detailed investigations of damaged tall buildings, their potential fall zones, and how they might be prevented from further collapse.
3. Refine and confirm safe distances from tall building barricades and cordons.

Emergency access to archives of plans for major existing structures may be a critical source of intelligence even in the first hours after disasters (CALBO, 2013). This observation further highlights the relevance of maintaining and expanding the existing tall building database presented in Part 1 to include building drawings that may benefit emergency inspections and response.

Depending on the severity of the earthquake, cordons may remain in place for extended periods of time (potentially years). The expected duration of temporary stabilization may influence decisions about design criteria and permit processing to develop permanent repairs. Plans should be put in place to enable permit applications for demolitions, long term barricade locations, and stabilization works.

Tall building façade systems that suffer significant damage during an earthquake are an obvious life-threatening falling hazard. Stiff curtain wall panels attached to the exterior of a building may have insufficient lateral deformation capacity to accommodate the lateral interstory drift imposed on the building by the earthquake ground motion input (Filiatrault et al., 2002). This problem is mostly likely relevant to buildings with flexible seismic-force-resisting systems, such as steel moment-resisting frame buildings, which are the most prevalent system in San Francisco's tall building inventory.

Following a review of damage suffered by heavy cladding panels during the 1994 Northridge earthquake, Cohen (1995) indicated that efforts should be undertaken to provide improved engineering details for these elements, which have been previously ignored in design. Following the 1985 Mexico City earthquake, Goodno et al. (1989) found that in many cases, cladding systems increased the initial stiffness of the building before suffering extensive damage. While cladding systems in modern high-rise buildings are designed to be isolated from the structure, this may not be the case in older buildings. Where the cladding is not isolated, it may initially stiffen the building, which can affect (and potentially improve) the dynamic response. But, once the cladding strength is exceeded, the imposed forces and deformations can result in significant damage and falling hazards. In light of these observations, the City should consider utilizing inspections and reports resulting from San Francisco's façade ordinance implemented in 2016 *San Francisco Existing Building Code* to characterize the façade vulnerability and risks of falling hazards. A related effort should evaluate the impact of the associated debris on emergency priority routes (in coordination with the San Francisco Lifelines Council).

The decision to cordon parts of a jurisdiction is not one to be made lightly, as it can have disproportionate impacts on the urban community, and the broader socio-economic factors that affect resilience. While the overriding issue of the safety of the public must be sufficient to warrant the closure of a road or section of a city, these effects must be considered when developing cordoning guidelines to enable evaluation methods for progressive reduction of the cordoned area in order to expedite recovery.

3.2 Evacuation and Fire Following Earthquake

Most high-rise buildings have relatively small floor plan areas with travel distances to exit stairways that are well within the code requirements for evacuation time. However, emergency egress for high-rise buildings presents special challenges for vertical evacuation, including the cumulative occupant load leading to stairway congestion and the physical ability for people to descend many flights of stairs. Current design standards require buildings to be designed such that occupants are able to evacuate the building without outside assistance in a time not exceeding half of the required fire resistance time of the primary structural frame. This equates to evacuation times of 1 to 1.5 hours based on fire resistance ratings of 2 to 3 hours for Type IB or IA construction (Bukowski, 2009).

The Current Code requires local water storage in buildings that is sufficient to operate automatic suppression systems for 30 minutes. The rationale behind this is that the responding fire department is expected to be able to access a fire on any floor and begin suppression operations within 30 minutes of the transmission of the original fire alarm (Bukowski, 2009). The expected 30-minute fire department response time has an expectation of travel time to the building and building access through a fire service elevator that has emergency power backup. In the event of a fire following an earthquake in a high-rise building, there are reasons to question the assumed 30-minute response time, due to: (1) competing demands for fire department services in the City; (2) travel time delays due to road blockages; and (3) building damage that may affect access through the fire service elevator and stairwells. Therefore, to the extent that the automatic building fire suppression systems may become a primary line of fire defense following an earthquake, the minimum local water supply requirement to provide 30 minutes service should be re-evaluated to: (1) control fire spread; and (2) ensure safe building evacuation following a large earthquake.

A related topic concerns recent changes in building codes that permit the use of specially equipped passenger elevators for evacuation of tall building occupants for during a fire emergency (Kinateder et al., 2014). The seismic risk in San Francisco (and other west coast cities) raises questions as to whether elevators will function as intended after earthquakes. While ASME A17.1 (2014) provides earthquake design requirements for elevators, including seismic and counterweight displacement switches, and additional visual and audible information systems, without period testing, these systems may not perform as intended following a large earthquake (Kinateder et al., 2014). To the extent that elevators are relied upon for fire emergency evacuation of mobility impaired occupants (or building occupants in general), it is suggested to evaluate the risk that earthquake damage will cause elevators to become inoperable and implications on the safety of occupants in residential and office buildings.

3.3 Search and Rescue

Collapse prevention is the key underlying goal of earthquake design provisions in building codes. Over the past century, building code requirements for seismic safety have evolved based on observations of building performance in past earthquakes combined with laboratory tests of building components and consensus judgements of engineers, researchers, and building code officials. Historically, the collapse risk has not been explicitly quantified but instead is assumed to be sufficient for structures designed to standards specified by building codes. Modern design guidelines have recently become more explicit in the expected collapse safety of buildings. For example, ASCE/SEI 7-16 (ASCE, 2017) indicates that building structures designed and constructed in accordance with requirements for Risk Category II buildings (which applies to most new tall building designs), should have less than a 10% probability of collapse under the code-specified Maximum Considered Earthquake (MCE) ground motions.

While it is generally assumed that the actual collapse risk in tall buildings is less than the maximum permitted risk, there is very little information to validate the collapse risk of buildings. For example, studies such as Haselton and Deierlein (2007) have shown that buildings designed for the same site, using the same code, materials, and structural system can have different collapse risks due to differences in design decisions, such as structural layout or the distribution of strength and stiffness over the height. This means that without a detailed analysis, the collapse risk of a particular structure remains largely unknown (Eads, 2013).

Collapse risk studies of buildings designed per modern building codes indicate that collapse safety margins set by the building code are generally met. On the other hand, a comparison of older non-ductile concrete buildings (representative of California construction between 1950 and 1975) and modern buildings, Liel and Deierlein (2013) found that the average mean annual risk of collapse was about 40 times greater for older, non-ductile concrete buildings as compared to new concrete buildings with respect to modern collapse safety margins. Similar studies applied to tall steel moment-resisting frame buildings suggest that the mean annual frequency collapse risk of older (1970s era) buildings is 28 times greater than equivalent modern designs (Molina Hutt et al., 2018). Insofar as the tall building database indicates the building inventory to include many older steel moment-resisting frame buildings and several non-ductile concrete buildings, emergency personnel, including Urban Search and Rescue (USAR) teams, should be prepared to respond to building collapses under a strong earthquake.

Collapse is still the principal cause of death in the most large-scale earthquake disasters. Although the collapse of modern tall buildings can be generally regarded as a low probability-high consequence event, due to high occupant loads and because older design codes did not explicitly quantify protection against collapse, the collapse risk of older tall building cohorts identified in the Part 1, should be investigated in further detail. More importantly, emergency personnel, including Urban Search and Rescue (USAR) teams, should be prepared to respond to such event.

USAR personnel and equipment required for any collapsed building depends mainly on the construction and the damage type of the building, the building size, the degree of the collapse and the number of casualties (Schweier and Markus, 2004). One of the key factors in allocating search and rescue resources is knowledge about the location, the extent, and the damage characteristics of the totally or partially collapsed building. Different damage types require different resources, as inferred by the damage types illustrated in Figure 5-3. At large building collapses, the required number of rescue personnel is generally dependent on the probable number of trapped persons. Prior knowledge of expected occupancies of potentially vulnerable tall buildings could help an effective mobilization of USAR resources.

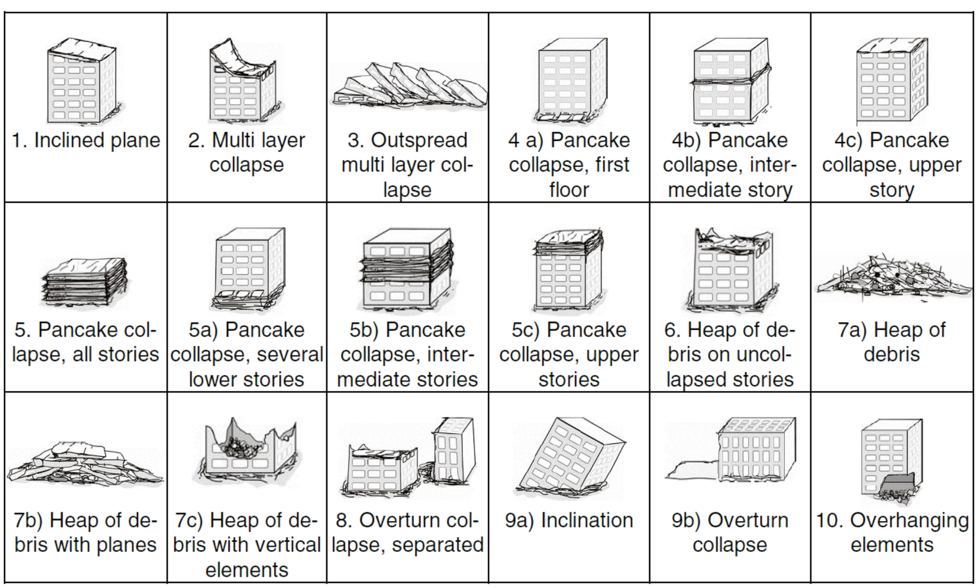


Figure 5-3 Damage types by Schweier and Markus (2004).

In carrying out search and rescue operations in a large collapsed building within a dense city center, USAR teams may also face challenges in carrying out the necessary structural stabilization tasks. FEMA USAR training guidelines (FEMA, 2009) generally specify lateral forces for shoring at a minimum of 2%, but ideally up to 10% of the weight to be stabilized. Depending on the collapsed condition of a tall building, these guidelines could result in prohibitively high lateral loads. It should also be noted that longer term stabilization structures should have more comprehensive design criteria than those established by search and rescue teams, since the latter may be inadequate for long term exposure by the public.

A related question is whether search and rescue resources are sufficient to cope with a large disaster in the dense urban downtown of San Francisco. Currently in California there are 8 USAR task forces: two in Los Angeles, one in Orange County, one in the City of Riverside, one in Menlo Park, one in Oakland, one in Sacramento, and one in San Diego. Although teams throughout the state will be able to assist in the event of a devastating earthquake, the City of San Francisco should study the benefits of establishing its own USAR task force as teams in closest proximity (Oakland and Menlo Park) may be needed in their own jurisdiction in the event of a major earthquake.

International guidelines for USAR activities are well established in INSARAG (OCHA, 2012). However, these guidelines include no explicit discussion on the recommended procedures in the event of tall building collapse. The City should establish recommended criteria and propose relevant training exercises that address these unique challenges, at least for USAR task force teams anticipated to respond within San Francisco.

Chapter 4

Consequences from Damage

4.1 Direct Costs to Owners and Tenants

Direct losses from earthquake damage include the financial costs of post-earthquake repair, demolition, and reconstruction work. Given that the primary objective of seismic building codes is to ensure life-safety under extreme earthquake events, significant damage to structural and nonstructural building components is likely to occur under moderate to extreme earthquakes. While new buildings are expected to perform better than older buildings, damage is likely to occur in both.

Structural components of a building include the structural framing (beams, columns, bracing, shear walls, and floor systems; typically consisting of steel or concrete) to resist gravity, earthquake, wind, and other types of loads. The structural system in a building is typically analyzed and designed by a structural engineer. Nonstructural building components include all of the architectural walls and interior finishes, cladding, along with the mechanical, electrical, and plumbing systems. Building contents, including furniture, moveable partitions, and equipment, are a third distinct component of buildings. The nonstructural components and contents are normally specified by architects, mechanical engineers, electrical engineers, or interior designers, but could also be purchased and installed directly by owners or building occupants. Many of the nonstructural building components and contents are vulnerable to earthquake damage (even if the structural system performs as intended) and they often account for a significant portion of direct financial losses under earthquakes. To put their relative costs in perspective, the structural framing and foundation systems typically comprise about one-quarter of the building construction cost, with the remainder being nonstructural components and contents. Thus, in evaluating the risk of damage and direct financial losses due to earthquakes, it is important to include the potential damage to nonstructural components and contents and ways that these can be reduced through measures to design or improve their ruggedness against earthquakes (FEMA, 2012b).

Following the 1971 San Fernando earthquake, a survey of 50 damaged high-rise buildings, which were located far away from the fault rupture and experienced only mild shaking, showed that whereas none had major structural damage, 43 of the buildings suffered damage to drywall or plaster partitions, 18 suffered damaged elevators, 15 had broken windows, and 8 incurred damage to their air-conditioning systems (Steinbrugge and Schader, 1973). These observations highlight that even under mild shaking, tall buildings may suffer considerable losses due to damage to nonstructural components.

Researchers have developed approaches to improve loss-estimating methods for individual buildings and the repair times associated with a given level of damage. The FEMA P-58 methodology and models provide the means for estimating direct economic losses and repair time as a result of damage to both structural and nonstructural components (FEMA, 2012a). Recent studies on the seismic performance evaluation of tall buildings indicate that older existing tall steel moment-resisting frame office buildings are expected to suffer losses in the order of 34% of building replacement cost under design earthquake ground motions (Molina Hutt et al., 2016) and 11% of building replacement cost under a magnitude-7.0 earthquake on the Hayward fault (USGS, 2018). Similarly, modern reinforced concrete shear wall residential buildings are expected to experience losses under a design earthquake ground motions on the order of 15% of building replacement cost (Tipler and Deierlein, 2014) and 5% of building replacement cost under a magnitude-7.0 earthquake on the Hayward fault (USGS, 2018).

Designing buildings to sustain less damage in earthquakes significantly decreases the uncertainty in the behavior of the building and increases the confidence that the building will perform as intended (Arup, 2013). The FEMA P-58 approach to estimate building losses enables separating losses into their constituent parts, making it possible to determine the contribution of each component to the overall loss, and permits comparison of different structural and nonstructural mitigation strategies. Performance-based seismic design, as currently implemented through San Francisco's AB-083 for the design of tall buildings requires minimum safety criteria for the structural system and building cladding, but it does not address the design and performance verification of nonstructural components. Discussions with engineers who design the building mechanical, electrical, and plumbing systems highlight the need to establish guidelines and criteria to address the effect that damage to nonstructural components on building occupancy and functionality. Where nonstructural components are designed for earthquake effects, typically the design criteria are limited to addressing the risk of falling hazards (e.g., by seismic anchorage or bracing), and the criteria do not address functionality of the components (e.g., whether they can remain operational or be repaired promptly after an earthquake). Part 3 addresses the need for greater emphasis on the design and installation of nonstructural components in tall buildings to ensure enhanced overall seismic performance. Part 3 also addresses some of the challenges in implementing existing methods such as the FEMA P-58 methodology to loss assessments in tall buildings.

4.2 Indirect Costs to Owners and Tenants

While seismic loss estimates associated with direct economic losses enable discussions with building owners and investors about how individual retrofit interventions can move buildings in the direction of becoming more resilient, they do not provide a quantitative measure of resilience. In addition to direct economic losses, there is great vulnerability to indirect economic losses due to downtime, defined as the time required to achieve a recovery state after an earthquake. Bonowitz (2011) defines three recovery states as follows: (1) re-occupancy of the building; (2) functional recovery; and (3) full recovery. Re-occupancy occurs when the building is deemed safe enough to be used for shelter, although functionality may not be restored. Functional recovery occurs when the building regains its primary function, i.e., it is operational but perhaps with some limitations or inconveniences. Full recovery occurs when the building is fully restored to its pre-earthquake condition.

Comerio (2006) identified rational and irrational components of downtime for buildings. Rational components include ones that can be more readily quantified, primarily repair construction costs and time. Irrational, situation-specific components take into account the time needed to plan and mobilize for repairs, and include financing, relocation of functions, workforce availability, regulatory changes, and economic uncertainty. Comerio also noted that delays due to owner decision making (indecision) is another important irrational component, which is very difficult to quantify ahead of time. To help quantify these factors, the pace of recovery can be linked to the scale of damage in a stock of buildings in the affected region.

The *Resilience-based Earthquake Design Initiative (REDi) for the Next Generation of Buildings* (Arup, 2013) guidelines provide a detailed downtime assessment methodology for individual buildings and identify the likely causes of downtime such that these can be mitigated to achieve a more resilient design. The methodology identifies the extent of damage and criticality of building components that may hinder achieving a recovery state. If the damage in any component hinders achieving a certain recovery state, the component needs to be repaired before such recovery state can be achieved. Once the components that need repairing to achieve a certain recovery state have been identified, the methodology includes delay estimates associated with impeding factors, defined as those factors which may impede the initiation of repairs. Impeding factors include post-earthquake inspection, engineering mobilization, contractor mobilization, financing, permitting, and long-lead-time components.

Recent studies on the seismic performance evaluation of tall buildings, which employ the methods discussed above, provide estimates of downtime to functional recovery. Analyses of a 1970s design of 40-story tall steel moment-resisting frame office building, modeled after existing buildings in San Francisco, estimate that it could experience downtime to functional recovery on the order of 87 weeks under a design earthquake (Molina Hutt et al., 2016) or 41 weeks due to ground motions from a magnitude-7.0 earthquake on the Hayward fault (USGS, 2018). A significant portion (about three-quarters) of the expected downtime is due to impeding factors, as opposed to the duration of repairs. Similarly, studies of a 42-story modern reinforced concrete shear wall residential building indicate that it could experience downtime to functional recovery on the order of 84 weeks under a design earthquake (Tipler and Deierlein, 2014) or 33 weeks under a magnitude 7.0 earthquake on the Hayward fault (USGS, 2018). In both of these studies, the ground motions experienced in San Francisco from the magnitude-7.0 Hayward earthquake are about half as strong as those of the design earthquake ground motion in San Francisco due to a larger magnitude earthquake on the San Andreas fault. Thus, what appeared to be moderate economic impacts due to direct losses, as presented in Section 4.1, could be potentially catastrophic to owners and tenants due to extensive downtime and the associated indirect costs. Part 3 addresses these disproportionate downtime estimates in tall buildings with possible policy recommendations to address them. Part 3 also addresses some of the challenges in implementing existing downtime assessment methods to tall buildings.

4.3 Consequences to City

Both direct and indirect losses, as identified in Sections 4.1 and 4.2 respectively, can lead to long-term impacts including loss of jobs, housing or human migration due to a re-alignment of economic activity. In addition to the loss of population and income, other evidence of long-term impact in the economic fortune of an affected region includes fluctuations in real estate values. These higher-order impacts can have long-term and far-reaching consequences, and therefore, they should be considered in post-disaster analysis and policy formulation (Nov and DuPont, 2016).

Earthquake resilience is the ability of an organization or community to quickly recover after a future large earthquake and the current building codes do not focus on it. The consequences of extended downtime and the inability of people to return to their homes, jobs, and schools are difficult to quantify: including the loss of culture, sense of community, and quality of life can impact communities for years and even decades after an earthquake (Arup, 2013). Because it is not possible to control these higher-order impacts in a post-earthquake disaster context, policy recommendations should address both direct and indirect economic losses to ensure business continuity and liveability of communities after the earthquake. Of particular relevance for tall buildings are cordoning (Section 3.1 of this Part) and the development of alternative habitability standards (as later described in Section 5.2 of this Part) as two critical considerations in the transition between emergency operations and recovery. The presence of extended cordons has implications for immediate (impacted) neighbors and the broader community, and alternative habitability standards are important for safely reoccupying tall buildings. While individuals should not be exposed to unnecessary risk, cordoning and reoccupancy posting decisions must carefully consider the implications on displaced individuals and businesses.

Chapter 5

Accelerating Recovery

5.1 Inspection and Tagging

Following damaging earthquakes, post-earthquake safety evaluations of buildings are typically carried out according to the procedures presented in ATC-20-1, *Field Manual: Postearthquake Safety Evaluation of Buildings*, (ATC, 2005), by which each inspected building is posted with one of three placards: Inspected (Green), Restricted Use (Yellow) or Unsafe (Red). San Francisco's application of this posting procedure for tall buildings is discussed in Part 6.

In addition to posting a building, it may be necessary to designate areas for which access is restricted. These hazardous areas may be inside or outside the building. According to ATC-20 procedures, areas outside the building and within a potential striking distance from a falling hazard must be barricaded to prevent entry and if necessary, the area can be formally posted with an Unsafe placard. The procedures suggest that barricades to establish a safety cordon, as previously discussed in Section 3.1 of this Part, may be prompted by post-earthquake safety evaluations, as seen in the third item of guidelines for barricading illustrated in Figure 5-4. The guidelines explicitly state that when a building is in danger of collapse, in danger from collapse of adjacent structures, or has falling hazards present, the building must be posted as Unsafe and barricades should be set up.

Even though ATC-20 guidance includes barricades within its scope, the evaluation may not be possible until a few days after the earthquake, depending on the extent of local damage and the number of available qualified inspectors and engineers. To prevent potential injuries that may occur due to collapse or other falling hazard conditions in tall buildings, which could occur unexpectedly or as a result of earthquake aftershocks, these evaluations can be expedited through the Building Occupancy Resumption Program (BORP), also discussed in Part 6.

BORP, which is administered by the City and County of San Francisco's Department of Building Inspection, allows San Francisco building owners to pre-certify post-earthquake inspection of their buildings by qualified private engineers and specialty contractors. The program requires the engineer to be familiar with the building's systems, have access to relevant drawings and that a copy of the completed emergency inspection plan be stored on site along with inspection supplies and pertinent construction drawings of the building's architectural, structural, and life-safety systems for use in future post-earthquake inspections. The minimum standard for this BORP assessment is the ATC-20 Detailed Evaluation. Some owners or engineers may choose a more complete engineering evaluation. Because the program spans beyond a structural evaluation and requires sign-off by the elevator firm and life-safety system maintenance personnel, its implementation could further expedite building reoccupancy and recovery.

Table 2-4. Guidelines for Barricading

1. Use caution tape or cones only initially. Chain link fences and wood or metal barricades make better long-term restraints.
2. Do not set barricades too close. Glass and brick walls can shatter on impact. Initially set the barricades wide. After a Detailed Evaluation or a period of stability (e.g., several days or a week or more), they often can be moved closed to the building.
3. When a downtown area (e.g., several city blocks) has extensive damage, recommend temporarily cordoning off the entire area. This avoids the need to barricade individual buildings and helps prevent theft.
4. Recommend the posting of guards at structures in imminent danger of collapse.
5. Recommend scaffolding and planking be installed over sidewalks and entrances to protect pedestrians when falling hazards are relatively small (e.g., a few bricks).
6. If safe to do so, try to keep one lane open for traffic when barricading street and sidewalk areas.

Figure 5-4 Guidelines for barricading (from ATC, 1995).

The liability in this type of evaluation is no different from that of regular building placarding. As outlined in the Limits of Liability section of the BORP Guidelines for Engineers (CCSF, 2017c), the structural engineer charged with performing the BORP inspection is acting as a City-authorized building inspector to assist building owners to obtain timely building inspection in the event of an emergency post-earthquake situation. As such, the structural engineer is intended to have the same exemptions from liability as is provided to the City building inspectors for emergency inspection.

Due to the complex nature of post-earthquake safety evaluationof tall buildings and the benefits provided by the voluntary BORP program, the City should consider expanding or mandating the program (or some variant of the program) for all tall buildings. Revisions to the program should be considered as outlined in Part 6, which covers both ATC-20 and BORP more extensively.

As an additional means to facilitate post-earthquake assessment, the City should evaluate the benefits of strong motion instrumentation of tall buildings for reoccupancy. With modern instrumentation technology, building response information is usually available immediately after an earthquake, which can provide the earthquake engineer with data to help quantify the condition of the building and, thereby, provide more assurance on the building safety to city officials along with building owners or tenants. Per Administrative Bulletin (AB) 058 (CCSF, 2008), adopted in 2008 all new buildings in San Francisco over 10 stories (~120 ft) in height (or 6 stories with floor area > 60,000 sf) should have strong motion instrumentation. Minimum instrumentation is 3 triaxial units, which should be located in the basement, mid-height, and near the top of the building. While BORP allows the use of recorded motions to guide building inspection, it currently does not include specific recommendations on how to use such data. Moreover, thought should be given to technologies for improving timely (real-time) access to and interpretation of strong motion data, e.g., such as the USGS program to facilitate evaluation of Veterans Administration hospitals (Kalkan et al., 2012). Thus, in addition to promoting (or requiring) more instrumentation in buildings, the City should consider an effort to facilitate consistent and effective use of the strong motion data. In addition to facilitating post-earthquake assessment, these data can also serve a longer term benefit of validating and improving models used to design and evaluate the performance of tall buildings.

5.2 Reoccupancy and Functional Recovery

Reoccupancy is allowed when the building is deemed safe enough to be used for shelter. According to the Cal OES Safety Assessment Program (SAP, see Part 6), reoccupancy can occur once a Green placard is posted following inspection by a qualified professional on the basis that any damage to structural and nonstructural components does not pose a threat to life safety and if egress paths are undamaged (ATC, 2005). A Green placard allows unrestricted access and reoccupancy to all portions of the building. Clean-up and/or minor repairs to some nonstructural components (such as fallen ceiling tiles) may be required so as not to impede egress in some areas of the building. If life-safety hazards to occupants are evident (which may include significant structural damage, exterior falling hazards due to damaged cladding and glazing, interior hazards from damaged components hung from the floor above or severely damaged partitions), these must be removed or repaired before a Green placard is awarded.

Since reoccupancy requires only the removal and repair of dangerous conditions, it can, and often does, occur before functionality is restored. Functional recovery represents the time required to regain capacity to serve the facility's primary function. For all occupancy types, this would require restoring power, water, fire sprinklers, lighting, and heating, ventilation, and air-conditioning (HVAC) systems while also ensuring that elevators are back in service. Back-up systems can help achieve functional recovery until the municipal utilities are restored and able to provide resources for full capacity. In residential buildings, functional recovery is related to regaining habitable conditions, which typically rely on the ready availability of power, water (potable or not), and heat.

Even so, SPUR (2012) recommended alternative habitability standards for houses to supersede regular code requirements (*California Health and Safety Code* and the *San Francisco Housing Code*) during a housing-emergency period declared by the City after a major earthquake. Such an emergency period might extend for days, weeks, or longer. Relaxing any habitability standard during some defined recovery period to facilitate reoccupancy and recovery implies additional risk. Tall buildings rely on sophisticated systems, such as HVAC, elevators, and fire suppression, for basic habitability and SPUR-recommended relaxation of habitability standards, developed for houses and small wood-frame apartment buildings to enable sheltering in place, are not applicable to tall buildings. Therefore, the City should consider developing alternative habitability standards for tall buildings after an earthquake, considering minimum requirements for fire suppression and safety systems, vertical transportation, water services, and electricity for pre-determined periods of time following an earthquake. Part 6 Section 2.2 presents furtehr discussion on this topic.

Another critical consideration in enabling building reoccupancy and functional recovery is monitoring and evaluating cordons, as discussed in detail in Section 3.1 of this Part, particularly in the event of aftershocks.

5.3 Insurance

Timely repair and recovery following earthquakes requires that tall building owners (and possibly tenants) have access to financial capital to pay for inspections, repairs, and other costs. Funds for repair and reconstruction can come from a variety of sources, including: bank accounts (either general accounts or reserve funds), insurance, or loans. The availability and access to funds for tall building repairs can vary significantly depending on the building ownership and stakeholders. In particular, the financial options are dramatically different for a single-owner commercial building versus a multiowner residential condominium building. Due to the number of buildings and diversity of owners, it is difficult to establish whether San Francisco's tall building inventory is financially well-prepared to recovery from a large earthquake. Available information suggests that some commercial buildings (especially those part of large real estate holdings) are financially well prepared, whereas others, including residential condominiums may not be.

The California Earthquake Authority (CEA) was established following the 1994 Northridge earthquake to manage and facilitate making earthquake insurance available from member insurance companies to residential homeowners, mobile home owners, condo unit owners, and renters. Despite efforts by the CEA to promote the value and benefits of earthquake insurance, earthquake insurance penetration in California is low. As of December 2017, there were just over 1 million earthquake insurance policies in the state compared to its over 12 million households (less than 10% market penetration). For tall residential condominium buildings, the earthquake insurance and repair financing situation is further complicated by shared responsibilities between homeowner associations (HOAs) and individual unit owners.

For commercial buildings, a recent NY Times article (Fuller, 2018) and other sources suggest that insurance penetration rates for commercial buildings is similarly low, on the order of 10%. This is contrasted by other anecdotal evidence from commercial tall building owners and insurance companies, which suggests that most tall building owners in San Francisco have some earthquake insurance. Based on these discussions, buildings owned by large developers as part of a larger building portfolio are more likely to be insured. For buildings with outstanding loans, the lending institution may require earthquake insurance, but even if required at the time when a loan is taken, the earthquake insurance may not remain over the life of the loan or building. Thus, without any formal procedure to report insurance information, the actual situation is difficult to verify.

Both for residential and commercial tall buildings, available evidence suggests that even when earthquake insurance is present, the insured amounts may be limited to a small fraction of the building asset value. Commercial owners may only insure for the expected loss under the design earthquake (probable maximum loss), and individual condominium owners may only insure for damage within their unit.

In summary, the available information raises doubts about the availability of capital to repair and recover from building damage after a strong to severe earthquake.

Earthquake insurance for commercial buildings is generally regarded as specialty insurance. Whether a building is insured purely for earthquake losses or through a multi-peril policy depends on how the contract with each insurer is structured. Retrofit or other measures taken to reduce the risk of earthquake damage can in concept be used to negotiate reduced insurance premiums, but the degree to which premiums are reduced depends on the market, the owner/risk manager, the broker and the insurance company. Due to lack of requirements, a direct and transparent link between retrofit or other seismic performance measures and insurance premiums does not exist.

Corelogic (2018) estimates that under a magnitude-7.0 earthquake mainshock on the Hayward fault, residential earthquake insurance would enable recovering 9% of the losses incurred due to damage to residential buildings, whereas commercial earthquake insurance would enable recovering 20% of the losses incurred due to damage to commercial buildings. "The difference between the estimated property damage and the amount recovered by insurance is driven largely […] by the lack of insurance for most properties in California and, to a lesser degree, the effect of insurance deductibles and limits." This realistic earthquake scenario highlights the considerable insurance gap, both for commercial and residential properties, in the San Francisco Bay Area (RMS, 2017; CEA, 2011).

Another important consideration for building owners and tenants is business interruption insurance, which covers loss of income suffered by a business after a disaster, such as loss of rent. In the event of cordons, business interruption insurance could be triggered by a city-ordained or owner-initiated building closure, unless a specific clause excludes this specialty insurance. In the case of cordons caused by a single damaged tall building restricting access to surrounding buildings, this would be dealt with through general liability insurance, as opposed to earthquake or business disruption insurance.

Another situation unique to tall buildings is the disproportionate effect their damage could have on nearby neighbors and the City. For example, extended cordons around a damaged tall building would inflict losses on nearby neighbors and possibly require the City to step in with emergency measures to stabilize (or potentially demolish) a building. The extent to which tall building owners are responsible and able to compensate costs borne by neighbors and the City remains an important question.

Given the uncertainty regarding earthquake insurance coverage or other means to provide financial resources for repair and recovery, it is recommended that the City identify potential limitations to the availability of capital after a damaging earthquake and, if necessary, require minimum levels of (self-) insurance for tall building owners to ensure recoverability. Alternatively, other solutions, such as pooled recovery funds should be explored.

Chapter 6

Issues and Recommendations

A characterization of building tallness in relevant building codes suggests that while there is no single accepted height threshold is valid to distinguish between "tall" versus "non-tall" buildings, the fact that different thresholds do exist for fire safety, seismic, or planning (amongst other considerations), is an indication that tall buildings present unique challenges. This Part provides an overview of seismic risks that are disproportionately associated with tall buildings, referred to as tall building issues, as they relate to post-earthquake recovery. The following list provides a summary of tall building issues identified within this Part, recommendations for new policy considerations and further research to mitigate these effects, and a brief discussion on their relevance to tall versus all buildings, and future versus existing buildings. For each recommendation, references to relevant sections are provided.

1. **Issue(s).** Following a damaging earthquake, safe evacuation and inspection of tall buildings involve many considerations that are not addressed by existing post-earthquake inspection and response measures. In particular, where it becomes necessary to cordon around a damaged building, the standard safe cordoning distance of 1 to 1.5 times the building height (or the height of the falling hazard), could be extremely disruptive to the dense downtown neighborhoods of San Francisco. Review of inspection requirements and discussions with local authorities in the City of San Francisco reveal gaps in the understanding issues associated with inspection of tall buildings and tall building cordons.

   Recommendation(s). Define clear roles and responsibilities associated with post-earthquake inspection and evaluations of tall buildings as related to establishing cordon areas around damaged tall buildings and coordinating with the San Francisco priority routes.

   Study the benefits of establishing San Francisco's own Urban Search and Rescue team or develop plans to leverage regional, statewide, and national teams (e.g., Oakland, Menlo Park) for evacuating residents of damaged tall buildings and/or surrounding buildings that are at risk from falling or collapse hazards. Ensure that training exercises for teams anticipated to respond in San Francisco address the unique challenges associated with tall buildings.

Identify and anticipate liability issues related to cordons, including the following: (1) the entity that declares cordon, i.e., the City or others; (2) the owner of the cordoned building; and (3) neighbors of the cordoned building that are impacted by the cordon.

Utilize inspections and reports resulting from San Francisco's façade ordinance to facilitate a detailed study of risks of falling debris.

Relevance. The disproportionate effect on the urban community associated with adopting existing cordoning practices is a tall building effect. While this applies to future and existing buildings, the greatest concerns are with existing buildings, due to their greater risk of structural and cladding damage.

2. **Issue(s).** The complexity associated with barricading, emergency stabilization, and falling hazard removal activities for tall buildings may require supplementing City staff and expertise to speed emergency response and recovery after a damaging earthquake. Related to this are questions as to whether USAR and other regional and national search and rescue teams, which may be called in to assist the City, have sufficient guidelines and training for conditions that may be encountered with heavily damaged tall buildings.

   Recommendation(s). Establish a Tall Building Task Force to: (1) help effectively conduct rapid assessments of tall buildings; (2) manage detailed investigations of damaged tall buildings, their potential fall zones, and how they might be prevented from further collapse; and (3) help refine and confirm safe distances from barricades and cordons.

   Relevance. These complexities are relevant to tall buildings, but also applicable to critical facilities. They apply to future and existing buildings, but are of greater concern for existing buildings.

3. **Issue(s).** High-rises have a heavy reliance on the building's automatic fire-suppression systems to control the fire. The current building and fire codes include a requirement for 30 minutes of local water supply for automatic fire suppression systems, in the event that City water supply to the building is disrupted. The 30 minutes is generally based on the expected time for the fire department to arrive to combat the fire. This 30-minute requirement has been in place for several decades and may not be adequate in the event of a large earthquake where arrival times for the fire department are impacted by: (1) competing demands for emergency response; (2) disruptions to transportation routes; and (3) building damage to fire service elevators. A related concern is whether the 30-minutes is sufficient to allow for safe evacuation of the tall building (or

affected floors of the building) that has experienced stair or evacuation elevator damage during the earthquake.

Recommendation(s). The City should engage the fire department along with other agencies and experts to evaluate whether the local water storage requirement (30-minute supply) is sufficient following expected large earthquake in San Francisco. The study should also consider current practices and policies for evacuation of tall buildings, including the use of elevators for emergency evacuation. Based on this study, develop recommendations for changes to local water supplies or other measures to reduce the risk of post-earthquake fire spread and safe evacuation of tall buildings.

Relevance: The reliance on the building's fire suppression system and challenges in evacuation are unique to tall buildings. Reliance on water supply applies to both future and existing buildings. The use of elevators for evacuation applies to future buildings.

4. **Issue(s).** Following a damaging earthquake, even if a building is safe from structural failure and nonstructural hazards, occupancy is not normally allowed unless habitability standards (unrelated to seismic safety) are also met. Tall buildings rely on sophisticated systems, such as HVAC, elevators, or fire suppression, for basic habitability. Relaxation of habitability standards, such as those proposed by SPUR (2012) developed for houses and small wood-frame apartment buildings to enable sheltering in place, are not applicable to tall buildings.

   Recommendation(s). Develop alternative habitability standards for tall buildings after an earthquake, considering minimum requirements for fire barriers, suppression and safety systems, vertical transportation, water services, and electricity.

   Relevance: The need for alternative habitability standards applies to all buildings. However, the emphasis here is that these need to be different for tall buildings. These standards are applicable to both future and existing buildings, but should focus primarily on those with residential use to enable sheltering in place.

5. **Issue(s).** Tall buildings pose challenges to quickly inspect following a damaging earthquake. While programs such as Building Occupancy Resumption Program (BORP) exist to expedite post-earthquake inspection, these are voluntary. Similarly, while strong motion instrumentation of tall buildings is required, these are not well utilized for real-time post-earthquake evaluations of building safety.

Recommendation(s). Consider expanding or mandating BORP for all tall buildings. A detailed list of recommendations related to BORP can be found in Part 5.

Evaluate ways to expand and improve the use of strong motion instrumentation of tall buildings to facilitate rapid building evaluation for informing decisions about building tagging, re-occupancy, and recovery planning.

Relevance. The complexities of inspection are unique to tall buildings, but generally applicable to large buildings. This applies to future and existing buildings, but is of greater concern for existing buildings.

6. **Issue(s).** Insurance penetration for commercial and residential buildings is low, and the insured amounts are often limited to a small fraction of the building value. These raise questions about the availability of capital for repair and recovery operations after a damaging earthquake.

   Recommendation(s). Identify potential limitations to the availability of capital after a damaging earthquake and explore ways to facilitate and encourage minimum levels of (self-)insurance for tall building owners to ensure recoverability.

   Relevance. The large amount of capital required to recover from a damaging earthquake is generally applicable to all buildings but is of particular concern for tall buildings given: (1) the large number of building occupants; and (2) potential financial liability of tall buildings on their nearby neighbors and city services. This challenge applies to both future and existing buildings, but it is of greater concern for existing buildings that are at greater risk of damage.

Appendix A

References

Arup, 2013, *Resilience-based Earthquake Design Initiative (REDi) for the Next Generation of Buildings*, Arup, San Francisco, California.

ATC, 2005, *Field Manual: Postearthquake Safety Evaluation of Buildings*, *Second Edition,* ATC-20-1, Applied Technology Council, Redwood City, California.

ATC, 1995, *Addendum to the ATC-20 Procedures for Postearthquake Safety Evaluation of Buildings* ATC-20-2, Applied Technology Council, Redwood City, California.

ASCE, 2017, *Minimum Design Loads and Associated Criteria for Buildings and Other Structures,* ASCE/SEI 7-16, American Society of Civil Engineers Structural Engineering Institute, Reston, Virginia.

ASME, 2014, *Safety Code for Elevators and Escalators*, ASME A17.1, American Society of Mechanical Engineers, New York, New York.

Bonowitz, D., 2011, *Resilience Criteria for Seismic Evaluation of Existing Buildings*, 2008 Special Projects Initiative Report to Structural Engineers Association of Northern California, San Francisco, California.

Bukowski, R.W., 2009, *Emergency Egress from Buildings*, NIST Technical Note 1623, National Institute of Standards and Technology, Gaithersburg, Maryland.

CALBO, 2013, *Interim Guidance for Barricading, Cordoning, Emergency Evaluation and Stabilization of Building with Substantial Damage in Disasters*, Structural Safety Committee, California Building Officials, Sacramento, California.

CBSC, 2016a, *2016 California Building Code, California Code of Regulations, Title 24*, California Building Standards Commission, Sacramento, California.

CBSC, 2016b, *2016 California Existing Building Code, California Code of Regulations, Title 24*, *Part 10*, California Building Standards Commission, Sacramento, California.

CCSF, 2008, *Procedures for Seismic Instrumentation of New Buildings*, Administrative Bulletin 058, Department of Building Inspection, the City and County of San Francisco, California.

CCSF, 2016a, *The San Francisco Building Code,* the City and County of San Francisco, California.

CCSF, 2016b, *The San Francisco Existing Building Code,* the City and County of San Francisco, California.

CCSF, 2017a, *San Francisco Property Information Map*, Planning Department, the City and County of San Francisco, California. Available at: http://propertymap.sfplanning.org/, last accessed: August 2018.

CCSF, 2017b, *SF Zoning Map - Sheet HT01*, the City and County of San Francisco, California. Available at: http://sf-planning.org/zoning-maps, last accessed September 4, 2018.

CCSF, 2017c, *San Francisco Building Occupancy Resumption Program – Guidelines for Engineers*, Department of Building Inspection, the City and County of San Francisco, California. Available at: https://sfdbi.org/borp-guidelines-engineers, last accessed: September 4, 2018.

CEA, 2011, *Affordable & Available Earthquake Insurance: Underscoring the Need for Risk-Transfer Diversification*, California Earthquake Authority, Sacramento, California. Available at: https://www.earthquakeauthority.com/Press-Room/Media-Resources/Public-Affairs-and-Testimony/OECD-Affordable-Available-Earthquake-Insurance, last accessed October 30, 2018.

Cohen, J.M., 1995, "Seismic performance of cladding: Responsibility revisited," *Journal of Performance of Constructed Facilities*, Vol. 9, No. 4.

Comerio, M., 2006, "Estimating downtime in loss modeling," *Earthquake Spectra*, Vol. 22, No. 2, pp. 349-365.

Corelogic, 2018, *Financial Implications of the HayWired Scenario*. Available at: https://www.corelogic.com/downloadable-docs/corelogic_haywired-scenario_r5.pdf, last accessed October 30, 2018].

Eads, L., 2013, *Seismic Collapse Risk Assessment of Buildings: Effects of Intensity Measure Selection and Computational Approach*, Technical Report 184, John A. Blume Earthquake Engineering Center, Stanford University, California.

FEMA, 2009, *National Response System Shoring Basics Module 2a, Shoring Construction Module*, Federal Emergency Management Agency Washington, D.C.

FEMA, 2012a, *Seismic Performance Assessment of Buildings*, FEMA P-58, prepared by the Applied Technology Council for the Federal Emergency Management Agency, Washington, D.C.

FEMA, 2012b, *Reducing the Risks of Nonstructural Earthquake Damage: A Practical Guide*, FEMA E-74, prepared by the Applied Technology Council for Federal Emergency Management Agency, Washington, D.C.

Filiatrault A., Christopoulos C., and Stearns C., 2002, *Guidelines, Specifications, and Seismic Performance Characterization of Nonstructural Building Components and Equipment*, PEER Report No. 2002/05, Pacific Earthquake Engineering Research Center, University of California, Berkeley, California.

Fuller, T., 2018, "In quake-prone California, alarm at scant insurance coverage," *The New York Times*. Available at: https://www.nytimes.com/2018/08/31/us/california-earthquake-insurance.html, last accessed September 4, 2018.

Goodno, B.J., Craig, J.I., and Zeevaert Wolff, A., 1989, "Behavior of heavy cladding components in the Mexico earthquake," *Earthquake Spectra*, Vol. 5, No. 1, pp. 195- 222.

Haselton C., and Deierlein G.G., 2007, *Assessing Seismic Collapse Safety of Modern Reinforced Concrete Frame Buildings*, PEER Report 2007/08, Pacific Earthquake Engineering Research Center, University of California, Berkeley, California.

ICBO, 1964, 1985, *Uniform Building Code*, International Conference of Building Officials, Whittier, California.

ICC, 2016, *International Building Code*, International Code Council, Country Club Hills, Illinois.

Kalkan, E., Banga, K., Ulusoy, H.S., Fletcher, J.P.B., Leith, W.S., and Blair, J.L., 2012, *Helping Safeguard Veterans Affairs' Hospital Buildings by Advanced Earthquake Monitoring*, U.S. Geological Survey Fact Sheet 2012–3094. Available at: https://pubs.usgs.gov/fs/2012/3094/.

Kinateder, M.T., Omori H., and Kuligowski E.D., 2014, *The Use of Elevators for Evacuation in Fire Emergencies in International Buildings*, NIST

Technical Note 1825, National Institute of Standards and Technology, Gaithersburg, Maryland.

Liel A., and Deierlein G.G., 2013, "Cost-benefit evaluation of seismic mitigation alternatives for older reinforced concrete frame buildings," *Earthquake Spectra*, Vol. 29, No. 4, pp. 1391-1411.

Molina Hutt C., Almufti I., Willford M., and Deierlein G.G., 2016, "Seismic loss and downtime assessment of existing tall steel-framed buildings and strategies for increased resilience," *Journal of Structural Engineering*, Vol. 142, No. 8.

Molina Hutt C., Rossetto T., and Deierlein G.G., 2018, "Comparative risk-based seismic performance assessment of 1970s vs modern tall steel moment resisting frames," manuscript submitted for publication.

NIOSH, 2009, *Preventing Deaths and Injuries of Fire Fighters when Fighting Fires in Unoccupied Structures*, The National Institute of Occupational Safety and Health, Cincinnati, Ohio.

Noy, I., and duPont, W., 2016, "The long-term consequences of natural disasters," Working Papers from the School of Economics and Finance, Victoria University of Wellington, New Zealand.

NZPA, 2011, *Christchurch Earthquake: Cordon around Grand Chancellor Narrows*, New Zealand Press Association. Available at: www.stuff.co.nz/national/christchurch-earthquake/4716748/Christchurch-earthquake-Cordon-around-Grand-Chancellor-narrows, last accessed: September 2017.

OCHA, 2012, *INSARAG Preparedness-Response*, International Search and Rescue Advisory Group Guidelines and Methodology, United Nations Office for the Coordination of Humanitarian Affairs, New York, New York.

PEER, 2017, *Guidelines for Performance-based Seismic Design of Tall Buildings*, Version 2.03, Report No. 2017/06, Pacific Earthquake Engineering Research Center, University of California, Berkeley, California.

RMS, 2017, *25 Years After Loma Prieta, San Francisco Bay Area Faces Increased Earthquake Risk; Next "Big One" Could Cause $200 Billion in Losses*, Risk Management Solutions, Newark, California. Available at: https://www.rms.com/newsroom/press-releases/press-detail/2014-10-13/25-years-after-loma-prieta-san-francisco-bay-area-faces-increased-earthquake-risk-next-big-one-could-cause-200-billion-in-losses, last accessed October 30, 2018.

Schweier C., and Markus M., 2004, “Assessment of the search and rescue demand for individual buildings.” *Proceedings*, 13th World Conference on Earthquake Engineering, Vancouver, Canada.

SPUR, 2012, *Safe Enough to Stay*, San Francisco, California.

Steinbrugge, K.V., and Schader, E.E., 1973, *Earthquake Damage And Related Statistics in San Fernando, California, Earthquake of February 9, 1971*, edited by Leonard Murphy. Volume 1A, pp. 709-710 and 713, National Oceanic and Atmospheric Administration, Washington D.C.

Tipler, J., and Deierlein G.G., 2014, *Seismic Resilience of Tall Buildings – Benchmarking Performance and Quantifying Improvements*, Department of Civil and Environmental Engineering, Stanford, California.

USGS, 2018, *The HayWired Earthquake Scenario - Engineering Implications*, USGS Scientific Investigations Report 2017-5013-I-Q, US Geological Survey, Menlo Park, California.

PART 6:

Post-earthquake Occupancy and Safety Inspection

Chapter 1

Introduction

1.1 Background

This Part reviews and explains current San Francisco policy and practices for post-earthquake safety inspection of buildings, especially as they relate to the City's existing tall buildings. The recommendations presented correspond to Recommendations 3A, 3B, 3C, 3F, and 4B presented in Summary Recommendations.

This Part addresses aspects of the following Tasks described in the *CAPSS Earthquake Safety Implementation Program Workplan 2012-2042* (CCSF, 2011):

- Task A.4.a: Develop and adopt shelter-in-place policies and procedures
- Task A.4.d: Adopt disproportionate damage trigger
- Task A.4.f: Update post-earthquake inspection policies and procedures
- Task B.1.b: Develop non-structural upgrade program for businesses
- Task B.2.b: Mandatory evaluation of 5+ dwelling unit residential buildings and hotels/motels
- Task B.4.b: Develop post-earthquake repair and retrofit standards
- Tack C.2.b: Mandatory evaluation and retrofit of critical stores, suppliers, and service providers
- Task C.2.c: Mandatory evaluation and retrofit of larger assembly buildings

1.2 Organization

Chapter 2 presents the current San Francisco practice, as well as discussion and recommendations related to safety, re-occupancy, functional recovery, and resilience.

Chapter 3 presents discussion and recommendations related to the Safety Assesment Program (SAP) administered by the California Office of Emergency Services.

Chapter 4 presents discussion and recommendations related to San Francisco's Building Occupancy Resumption Program (BORP).

Chapter 5 presents discussion and recommendations related to post-eartqhuake safety evaluation of buildings.

Chapter 6 presents discussion and recommendations related to available guidance on evaluation and repair of welded steel moment-resisting frame buildings.

Chapter 7 presents a brief discussion on available additional techniques.

A list of references is provided at the end of the Part.

Chapter 2

Background

2.1 Current San Francisco Practice

Reoccupancy is the ability to re-enter a building after an earthquake and either resume normal operations or start on clean-up and repairs. For any building, safe reoccupancy is the first milestone on the path to functional recovery and eventually full recovery (Bonowitz, 2011). Since the Loma Prieta earthquake, safe reoccupancy in San Francisco—for a building of any height—has been assured by a process of safety inspections.

As part of this process, the City relies on the state-run Safety Assessment Program (SAP), which implements the evaluation procedures presented in ATC-20-1, *Field Manual: Procedures for Postearthquake Safety Evaluation* (CCSF, 2008, Section 6.5.1; ATC, 2005). For a tall building, this process can be complicated and delayed by the sheer size of the structure, to say nothing of its architectural complexity and variety of tenant spaces. To help address these challenges, San Francisco developed the Building Occupancy Resumption Program (BORP) as a way of outsourcing the inspection process to building owners themselves (CCSF, 2011). BORP also uses ATC-20 procedures, but for certain steel frame structures, it allows an alternative guideline developed after the 1994 Northridge earthquake, known as FEMA 352 (FEMA, 2000).

This Part discusses each of these four components of San Francisco's current practice—SAP, BORP, ATC 20, and FEMA 352—and recommends possible improvements.

2.2 Safety, Reoccupancy, Functional Recovery, and City Resilience

SAP and BORP are both focused on safety, in the service of reoccupancy and functional recovery. These three measures of building performance are related, as each is a function of building damage. In general, the lower the damage, the higher the safety, and the lower the damage, the faster the reoccupancy and functional recovery (Bonowitz, 2016). Yet they are not identical.

For example, a building can perform safely but still not be ready to reoccupy because of hazards posed by adjacent buildings or infrastructure, or because of restrictions set by local authorities (perhaps for public safety or debris removal). This was illustrated by the years-long closure of the entire central business district in Christchurch, New Zealand after a 2011 earthquake. Thus, while the poor performance of a tall building is likely to affect its neighbors, it is also possible that damage to San Francisco's downtown neighborhoods might delay the recovery of an otherwise safe tall building.

From a tenant's perspective, it is also possible to achieve a measure of organizational recovery and resilience even without a safe building. Many organizations use continuity-of-operations planning to limit downtime through remote data storage, online services, and even alternate or backup facilities. These are often intended as temporary measures, but in cases of severe building damage, permanent relocation is sometimes unavoidable; if anticipated and planned, it can also be an effective recovery strategy.

Setting aside relocation and continuity-of-operations, reoccupancy is also different from functional recovery (Bonowitz, 2011). Both require a safe structure, but functional recovery also requires working utilities and building services, as well as clean-up or repair of contents. In a tall building, this will almost always include elevators, mechanical and plumbing systems, life safety systems, electricity and telecommunications, and building security. SPUR (2012) recommends that in houses, and even in typical San Francisco apartment buildings and small commercial buildings, in order to speed reoccupancy and neighborhood recovery, the City should waive temporarily certain habitability standards that rely on these nonstructural systems. In tall buildings, however, these systems are more sophisticated and more essential than in small, older buildings. For example, few San Francisco houses and apartments have air conditioning, and for much of the year it is even possible to live temporarily without heat or electricity. But in a tall building (especially one without operable windows) the HVAC system is essential to any level of functionality. Section 5.2 of Part 5 presents additional discussion of post-earthquake habitability standards.

Typical post-earthquake safety evaluations using ATC-20 procedures must be quick and focused; they therefore do not look beyond safety. In Christchurch, however, officials eventually found it beneficial to modify their post-earthquake inspection forms to consider what they called "3S": safety, sanitation, and security (ATC, 2014). In San Francisco, habitability issues could be worth considering within the context of a more detailed or pre-planned inspection such as that contemplated by BORP. For a tall building, these issues might need to be considered for individual tenant spaces.

Functional recovery can also be affected by a building owner's rational choices. For example, an owner may choose to delay recovery in order to complete repairs with the building vacant. From a tenant's perspective, this indirect, externally-imposed delay has the same effect as a recovery delay stemming directly from damage. To the extent that San Francisco tall building tenants are more likely to be subject to such lease provisions, this might be considered a tall building issue. Similarly, while an owner or tenant might be willing to resume pre-earthquake functions in a space with repairs still incomplete, the same space might not be ready to serve as a shop, restaurant, bank, public parking, or public open space due to liability or security issues. This concern is not limited to tall buildings. Indeed, in most of San Francisco's tall buildings, public accommodations such as these are typically limited to the ground floor or basement levels.

Earthquake resilience is not an attribute of individual buildings, tall or otherwise, but an attribute of community functions and, in the aggregate, of the community as a whole (ATC, 2018). Because buildings, individually and in groups, provide shelter and vital resources for these resilience-critical functions, their timely reoccupancy and functional recovery makes an essential contribution to the City's resilience. To the extent that the City's tall buildings account disproportionately for its housing, or jobs, or revenue, then the tall buildings deserve special attention within the broader resilience policy. That said, while the City can and should set reoccupancy and recovery goals for buildings, San Francisco's earthquake resilience is ultimately measured by the return of functions and services, not buildings.

This study addresses the effect of tall building damage on the tall buildings themselves and, to a lesser extent, on the downtown neighborhoods where tall buildings are densely clustered and on the City overall. But it does not explicitly address the likely interactions between the tall buildings, the non-tall buildings that still comprise most of the Financial District, and the critical infrastructure that serves the neighborhood. Nor does it explicitly consider resource demands and capacities of the Financial District and adjacent neighborhoods' businesses, residents, workers, and other stakeholders. A separate recovery plan, drawing on the present study's findings, would bring these ideas together in a practical way to support a neighborhood and its functions, as opposed to just individual buildings with certain characteristics.

Recommendation: Develop a comprehensive recovery plan for the Financial District and adjacent neighborhoods. While the recovery plan is being developed, the City Administrator should develop an interim recovery plan that outlines at least the basic issues. By necessity, the interim plan will need to make many assumptions. The study recommended here would confirm or correct those assumptions and fill in the critical knowledge gaps.

- Supplement the newly developed inventory of tall buildings with additional inventory and analysis of specific building uses by industry or employment sector, in coordination with development of recovery goals (ATC, 2018) and with the recommendations presented in Part 1.
- Consider the combined effects of tall buildings, non-tall buildings, and infrastructure, including liquefaction effects.
- Develop the recovery curve for one or more scenario earthquakes. The recovery curve shows the level of immediate functional loss and the extent of reoccupancy and recovery over time. Such a study would provide a valuable basis for neighborhood recovery plans and for new programs aiming to improve the reoccupancy and functional recovery times of individual tall buildings.

2.3 Related Issues

The focus of this report is post-earthquake reoccupancy, specifically the safety inspection protocols used by SAP and BORP to assess whether shaking damage has rendered a tall building unsafe to reoccupy.

To be sure, earthquake effects other than shaking damage to an individual tall building (or its neighbors) can affect that building's reoccupancy. Although outside the scope of this report, many of these are addressed by *San Francisco's Emergency Response Plan* (CCSF, 2008, Section 6.5) as follows:

- Emergency response (CCSF, 2008, Section 6.1)
- Law enforcement (CCSF, 2008,Section 6.3)
- Large-scale evacuation (CCSF, 2008, Section 6.3.1)
- Perimeter control (CCSF, 2008, Section 6.3.2; see also cordoning discussion in Part 5)
- Traffic control and route recovery (CCSF, 2008, Section 6.4)
- Emergency shoring of adjacent properties (CCSF, 2008, Section 6.5.2)
- Debris removal (CCSF, 2008, Section 6.5.3)
- Fire or tsunami damage

Part 5 discusses some administrative and logistical issues related to post-earthquake reoccupancy, including owner-tenant relationships, the availability of financing and insurance, and the feasibility of expanding BORP.

Part 4 discusses the application of building code provisions to damaged buildings. The safety inspections that inform reoccupancy decisions are focused on the immediate safety of potential building occupants. Separate from that concern, the *San Francisco Existing Building Code* requires engineering evaluations to determine whether seismic retrofit will be required in addition to damage repair. These evaluations are outside the scope of the state's Safety Assessment Program.

Chapter 3

The Cal OES Safety Assessment Program

3.1 SAP as Applied in San Francisco

The Safety Assessment Program (SAP), administered by the California Office of Emergency Services (Cal OES), was created to supplement local government resources with mutual aid and volunteers from elsewhere in the state. All SAP volunteers must be registered Disaster Service Workers subject to California Code of Regulations Title 19. The program has been in continuous development since its creation in the late 1970s, with significant procedural improvements as recent as 2007 (Cal OES, 2015).

SAP responds to three accepted facts of post-earthquake recovery:

- Quick, efficient reoccupancy is essential to citywide recovery. Safety is the focus, but speed is fundamental to the effort. As SAP training materials note, “The primary goal of the Safety Assessment Program is to get as many people as possible back into their buildings as quickly and safely as possible” (SEAONC, 2013). Using a similar program, volunteer inspectors in Christchurch inspected 72,000 buildings in 10 days and 130,000 in three weeks (ATC, 2014).
- City staff and resources should be assigned first to inspect and assess essential infrastructure and facilities, especially those needed for public safety and to support emergency response (Cal OES, 2015).
- In a large, damaging earthquake, practically every affected jurisdiction will therefore need outside help to meet its inspection goals.

San Francisco’s priorities are consistent with SAP’s intent. The San Francisco *Emergency Response Plan* (CCSF, 2008) states in Section 6.5.1:

> *Assessments will be necessary immediately to determine whether public and private buildings can be safely entered or used and to develop priorities for implementing repairs. The primary goals of this process are to protect the public and to expedite resumption of residence, business, services, and community activity as soon as possible. ... The number of*

facilities and structures requiring immediate safety inspection is far greater than the number of trained inspectors. Supplemental resources will be required immediately.

The plan therefore calls for designated City representatives to "immediately request support from ... volunteer building Evaluators and Coordinators under the post-disaster SAP." Table 6-1 of the plan's *Earthquake Annex* states an objective of completing all necessary building safety inspections and achieving building reoccupancy within a week of a hypothetical earthquake (of unspecified intensity).

For technical criteria and on-site procedures, SAP relies on ATC-20 procedures (ATC, 2005), which offer a Rapid evaluation method suited to the objectives of SAP. ATC-20, discussed further in Chapter 4 of this Part, results in a public "posting" with a colored placard stating the building's eligibility for reoccupancy. The *San Francisco Building Code* (Section 101A.21) endorses ATC-20, customizes the placard text, and gives the building official discretion over other changes to placard design.

3.2 Recommendations Regarding SAP

Most of the following can apply to non-tall buildings as well as tall ones.

3.2.1 Activation

San Francisco activates SAP on the basis of instructions given in the *Earthquake Annex* to the City's *Emergency Response Plan* (CCSF, 2008). However, the instructions are incomplete, somewhat ambiguous, and in some respects, out of date. For example, current *Earthquake Annex* Section 6.5.1 charges the Department of Building Inspection (DBI) with implementing SAP for private buildings, but activation responsibility is given to the EOC Logistics Section, and Department of Public Works (DPW) is the lead agency for implementing that *Earthquake Annex* section overall.

Recommendation: Clarify and update roles and responsibilities associated with post-earthquake emergency response and safety inspection.

- The Department of Emergency Management (DEM) should clarify *Earthquake Annex* Section 6.5.1 regarding who acts on behalf of the City as the contact with Cal OES, making the request to activate SAP. The *Annex* should be coordinated with Cal OES requirements (Cal OES, 2015).
- DEM should undertake a general update of the *Earthquake Annex* to account for new technologies and recent construction (including major public projects), as well as substantial mitigation over the last decade.

How DEM prioritizes inspection will affect how DBI allocates its own staff and SAP volunteers.

3.2.2 Early Recovery for Critical Building Groups

The primary focus of SAP is safe reoccupancy, but SAP has a secondary goal regarding functional recovery, especially with respect to infrastructure (SEAONC, 2013, p.131): "Rapidly clearing for use vital services and infrastructure that will impact the public at large." To the extent that San Francisco's tall buildings (or any other particular building cohort) represent a recovery-critical function or service, their recovery should also be prioritized, or at least emphasized together with basic reoccupancy. Christchurch officials recognized the benefit of classifying and inspecting buildings based in part on their use and occupancy and in part on height, and they used the post-earthquake safety inspections also to consider habitability and security (ATC, 2014). BORP is a promising vehicle by which to introduce these ideas, but to be effective for an entire functional sector, they will need to be implemented more aggressively than through that voluntary program.

Recommendation: Plan to assign SAP volunteers to teams dedicated to recovery-critical facilities, including pre-selected tall buildings.

- DBI should plan to use specially-qualified SAP volunteers to inspect pre-selected groups of buildings.
- The building groups of interest should be related to the City's tentative recovery goals (ATC, 2018) but might also be associated with certain building characteristics (such as structural system or material) or particular occupancies (grocery stores, for example) that might have especially vulnerable mechanical systems or contents. In concept, tall buildings not covered by BORP could be one such group.

3.2.3 Indicator Buildings for Aftershock Tracking

The need to re-inspect buildings after significant aftershocks can challenge SAP implementation in terms of resource allocation and documentation. Officials in Christchurch identified a few buildings of specific structural types and used them as indicators of the need to re-inspect the larger population (ATC, 2014). Indicator buildings would be especially valuable for San Francisco's most common vulnerable building types (such as house-over-garage or Victorian "soft story" conditions) and perhaps for tall or large buildings where re-inspection would be especially demanding of scarce resources. Identifying indicator buildings, however, would require tracking early SAP inspections by structure type, and adding a documentation and database task for DBI.

Recommendation: DBI should plan to track common building types, perhaps including tall buildings with specific characteristics, in order to identify indicator buildings that will help manage re-inspection demands after significant aftershocks.

3.2.4 Consistency

Experience from the Northridge, Humboldt County, and Napa earthquakes shows inconsistent decision-making by SAP volunteers, leading to incomplete or unaccountable implementation. For example, different volunteer inspectors can often assess unfamiliar damage patterns in different ways, posting similar buildings differently.

Recommendation: Develop clear procedures and policies, and document them in San Francisco-specific training and guidance materials.

- DBI, as the agency responsible for implementing SAP for private buildings, should identify areas of potential inconsistency and establish procedures in coordination with DBI procedures and resources.
- DBI should encourage San Francisco-specific training of SAP volunteers addressing issues likely to arise from San Francisco’s unique building stock, some of which are related to the City’s tall buildings.

Chapter 4

Building Occupancy Resumption Program

4.1 Background

After the 1989 Loma Prieta earthquake, the San Francisco Department of Building Inspection (DBI) developed the Building Occupancy Resumption Program (BORP) to offer an optional “precertified emergency inspection program” that helps address this challenge (CCSF, 2011).

BORP allows an owner—of any building—to apply for pre-approval by DBI to use expert inspectors directly contracted with the owner to evaluate and post the building using ATC-20 procedures. Through the pre-approval process, the owner’s experts become familiar with the building’s structural and mechanical systems, compile documents, and develop plans for post-earthquake inspection. The experts, including structural engineers, are paid on retainer and the building owner applies for recertification every two years. Advance planning with BORP allows a more informed and detailed inspection in the same time as a typical SAP inspection, while freeing SAP volunteers to inspect other buildings. As an incentive, DBI charges the owner no fee to participate in BORP.

The primary purpose of BORP is the same as that of SAP, to expedite reoccupancy with safety inspections by experts other than City staff:

> *This private emergency inspection could facilitate rapid decisions regarding the closure or reoccupancy of building areas. Prearranged emergency inspection could reduce inspection delays, as City inspection personnel typically are dispatched first to areas of greatest damage or public hazard, which may not include the building in question.* (CCSF, 2011)

But BORP has a second goal beyond basic reoccupancy: to facilitate functional recovery, especially of commercial buildings.

> *[SAP’s priorities are] geared toward public safety rather than expeditious business resumption. Some building owners may wish to develop programs of private inspection for their buildings*

to permit rapid, individualized emergency response. (CCSF, 2011)

The recognition of business resumption as a benefit of BORP is consistent with the City's tentative recovery goals (ATC, 2018). In fact, the Department of Public Works (DPW) uses a BORP-like process to facilitate inspection, reoccupancy, and recovery of the City's own facilities. The implication that BORP can facilitate functional recovery suggests opportunities and rationales for extending the program. Generally, however, BORP, like SAP, is focused on safe reoccupancy.

4.2 Issues and Recommendations Regarding BORP

While BORP is currently used primarily by owners of large office buildings, it has benefits for all buildings where SAP procedures would be slow or inconclusive. Most of the following discussion apply to non-tall buildings as well as tall ones.

4.2.1 Training Simulation

In the decades since it was developed after the Loma Prieta earthquake, BORP has never been implemented after a significant damaging event. Each BORP plan is updated every two years, but the simultaneous execution of dozens of BORP plans has not been tested.

Recommendation: Test and improve current BORP procedures through training simulations with the following considerations:

- DBI should organize and implement regular training simulations with two purposes: (1) to identify building-specific logistical issues for the engineers; and (2) to test DBI procedures for receiving, approving, and tracking input from BORP engineers.
- The training should be simultaneous for at least a majority of enrolled buildings.
- Each enrolled building should be required to participate in a training simulation, perhaps once every four years, as a condition of continuing BORP approval.

4.2.2 Program Extension

For a tall building, an ATC-20 Rapid evaluation performed by a randomly assigned SAP volunteer is unlikely to be feasible, reliable, or conclusive. BORP, in concept, solves many of the anticipated challenges, but DBI could still face resource shortfalls if only a minority of the taller, larger, and more complicated buildings are enrolled.

Recommendation: DBI together with the Office of Resilience and Capital Planning (ORCP) should study the feasibility, benefits, and costs of extending BORP through legislative mandates, triggers, or funded incentive programs with the following considerations:

- Participation in BORP should be required for all new tall buildings. Further, DBI, together with the Building Inspection Commission (BIC), should consider requiring a version of BORP for all new buildings meeting certain conditions related to their size, complexity, use and occupancy, occupant load, or location within a recovery-critical neighborhood. This would impose a small additional cost on building developers for planning and documentation, but it would be marginal if produced as part of construction documentation by the design team or by building management during commissioning.
- Buildings identified as recovery-critical, or especially vulnerable, or located in a neighborhood (perhaps the Financial District) especially vulnerable to interaction and adjacency effects—that is, where many buildings will be unable to reoccupy or recover because of damage to other properties, should be subject to an extended BORP.
- For a mandatory or triggered program, a streamlined or "light" version of BORP would likely be beneficial, though this would need to be balanced with the recommendation to extend BORP beyond reoccupancy to functional recovery (see Section 4.2.10 of this Part).
- To encourage more voluntary participation in BORP, DBI should increase its outreach and marketing for the program and should consider additional incentives.
- DBI should extend its BORP outreach and marketing to major employers and tenants in buildings and neighborhoods where high BORP participation would be especially beneficial to the City.

4.2.3 Personnel and Financing

In general, BORP engineers are deputized by the City and are thus authorized to post ATC-20 placards (CCSF, 2011, Section III). However, they are not eligible for the liability protection afforded to SAP volunteers under Business and Professions Code Section 5536.27 because they are engaged and compensated by the building owners. It is unclear whether BORP engineers are eligible for state-funded workers compensation as SAP volunteers are. Administrative aspects of BORP are discussed further in Part 5.

4.2.4 BORP Report Organization

The BORP guidelines (CCSF, 2011) serve two purposes: (1) they describe what the final BORP plan, kept at the building with copies held by the retained engineers, should include; and (2) they describe what the BORP engineer must submit to the City for certification. The latter purpose tends to determine each BORP report's organization, and the result is that most reports look more like submittals to the City than practical tools to be used with urgency in the hours after a damaging earthquake. The following recommendations are based on a review of selected BORP reports made available by DBI during this study.

Recommendation: Modify the BORP instructions to ensure that the materials submitted for DBI approval are separate from the materials intended to be kept at the building and used to conduct the post-earthquake inspection with the following considerations:

- Inspector qualifications and certifications and submittal checklists should be put in a separate section for building staff reference, while technical materials needed to conduct the inspection should be in a separate front section for inspector convenience.
- DBI should clarify and enhance Sections IV.D.2 and IV.D.3 of its program guidelines (CCSF, 2011), which currently call for "detailed instructions" regarding the scope and conduct of the inspection. In addition to requiring compliance with ATC-20 Detailed evaluation requirements, BORP guidelines should call for inspection instructions to follow the outline of the two-page ATC-20 Detailed Evaluation Safety Assessment Form. The blank form in the BORP plan should be pre-filled with building-specific information—as should the DBI database that will hold the inspection findings. In addition, each line of the form should have a corresponding section in the BORP plan with information regarding the relevant locations in the building, their built condition, possible damage modes, and inspection procedures and tips.
- DBI should modify its instructions to require the listing of clear instructions for initiating the inspection process on the first page of each BORP plan. Each building's team may decide whether the building staff or the engineer will initiate the inspection, but the plan's first page should be clear and appropriate for the responsible party.
- DBI should recommend that each BORP plan should have separate pull-out sections for each expert inspector, e.g., structural, mechanical, life safety, or elevator. This would facilitate each inspection by avoiding

duplication and other material not needed to complete the inspector's assigned scope.

4.2.5 BORP Report Content

A review of selected BORP reports made available by DBI during this study show that content and completeness varies within the program. More uniform content requirements, together with the more pragmatic report organization recommended above, should improve reliability and could reduce costs by eliminating unnecessary or vague requirements. As recommended above, BORP report content related to specific building components or conditions should be organized to correspond to the ATC-20 Detailed Evaluation Safety Assessment Form.

Recommendations. Update and clarify BORP instructions to ensure that each BORP report contains all relevant material, considering current technologies and practices.

- BORP instructions should require information on potential pounding, plan irregularities, and vertical irregularities. This information should be linked to the ATC-20 form sections on building description and other structural hazards.
- DBI should update its program instructions to require information regarding the building's compliance with Chapter 4E, Building Façade Inspection and Maintenance, of the *San Francisco Existing Building Code* (CCSF, 2016) and Administrative Bulletin (AB) 110, *Building Façade Inspection and Maintenance* (CCSF, 2017a).
- DBI should update its program instructions to clarify requirements regarding photographs. "Before" photographs should not be necessary for typical conditions, but they can be useful to describe a damage-prone pre-earthquake condition or to document pre-earthquake damage; if used, they should be linked to the appropriate line in the ATC-20 form. "After" photos should be provided for cases where damage is perishable or to document the change relative to a "before" photo, but the current DBI instructions do not require them. "After" photos can also be useful to document the inspection process. Wherever photos are included, they must be detailed enough to convey the condition of interest, and accompanied by text captions as needed.
- DBI should update its program instructions regarding the required building information. As recommended above, the required information should be keyed to the ATC-20 form, and the form itself should be pre-filled wherever possible. In describing the building, the instructions should

distinguish the seismic-force-resisting system(s) from the gravity system(s); should distinguish the seismic-force-resisting system in each principal direction; and should specify combinations and changes in the lateral system(s) over the height of the building. DBI should consider recommending a tabular format to improve completeness and consistency.

- DBI should clarify its instruction (Section VII.F) to "take preventive measures regarding gas leaks, release of hazardous materials, or other life-safety mitigation." The intention of this instruction is not clear. The instruction should be coordinated with Section VII instructions regarding notification of DBI.
- DBI should clarify its instruction (Section IV.G): "At owner's and inspector's discretion, non-structural hazards may be mitigated per DBI procedures." The intention of this instruction is not clear. It should be clarified whether this instruction is regarding pre-earthquake mitigation or whether it is related to eliminating dangerous conditions related to earthquake damage to nonstructural components. Also, the specific DBI procedures referenced should be clarified.
- DBI should consider requiring use of the ATC-20 Fixed Equipment Checklist. This checklist is typically used only for essential facilities, but it could be a basis for expanding the BORP equipment list and for ensuring consistency and completeness. ASCE/SEI 41-17, *Seismic Evaluation and Retrofit of Existing Buildings*, (ASCE, 2017) and FEMA E-74, *Reducing the Risks of Nonstructural Earthquake Damage: A Practical Guide*, (FEMA, 2012) also provide checklists that should be considered.

4.2.6 Use of Instrumentation

BORP guidelines explicitly allow the use of ground motion records to assist damage finding and posting, and even to reduce the amount of inspection (CCSF, 2011, Section IV.D.5 and Appendix A). The guidelines do not, however, provide any technical procedures or criteria to follow. In particular, they do not require a pre-earthquake structural analysis to calibrate the instruments and establish baselines. See also the discussion of inspection techniques in Chapter 7 of this Part.

Recommendation: DBI, perhaps in collaboration with the Structural Engineers Association of Northern California (SEAONC) and the state's Strong Motion Instrumentation Program (SMIP), should establish criteria for the use of recorded motions for finding damage and assessing building safety. In the interim, if recorded motions are to be allowed for BORP, DBI should establish minimum requirements for documentation and approval of the findings and conclusions by BORP engineers.

4.2.7 Cordons and Barricades

BORP instructions (CCSF, 2011, Sections VII.B and VII.C) give unclear guidance on barricading, suggesting that the BORP team should “arrange for barricading of all unsafe areas” but should also “contact DBI immediately” so that City staff can erect barricades or establish cordons for the public areas. Meanwhile, ATC-20 procedures suggest that volunteer SAP inspectors can undertake this work in public areas as well.

Recommendation: Clarify and update both BORP instructions and any San Francisco-specific SAP procedures regarding the division of responsibility for establishing cordons and barricades.

- DBI, together with the Department of Public Works (DPW), should clarify and distinguish the roles of City staff and deputized inspectors.
- BORP instructions should also distinguish the roles of SAP volunteers and BORP inspectors, as the latter also have contractual responsibilities on behalf of their individual clients.
- The instructions, which currently discuss only the barricading of public areas, should also recommend or instruct appropriate practices for cases where an adjacent building threatens the building being inspected (both of which are private spaces), and where the building being inspected threatens its neighbors.

4.2.8 Use of FEMA 352

BORP instructions (CCSF, 2011, Section IV.D.1) allow and “recommend” the use of FEMA 352, *Recommended Postearthquake Evaluation and Repair Criteria for Welded Steel Moment-Frame Buildings* (FEMA, 2000), but “BORP does not require connection inspection.” Inspection of individual connections is fundamental to FEMA 352 and is the main difference between that system-specific document and the more generic ATC-20. Thus, BORP instructions appear to conflict with ATC-20, which references FEMA 352 for the Detailed evaluation that BORP anticipates, and with FEMA 352 itself. Further, Section IV.D.5 of BORP instructions allows instrument recordings to be used “as a means of reducing the percentage of joints required to be inspected after an earthquake,” and Appendix A makes a similar allowance. This instruction is confusing because if inspection of the welded connections is not required, there would be no need for instrument recordings to reduce the scope.

Recommendation: Develop BORP instructions for the use of FEMA 352 in coordination with a general FEMA 352 policy.

- Even if FEMA 352 is not required, DBI should improve its BORP instructions by adding specific provisions for acceptable voluntary use. Consistent use of FEMA 352 within BORP will require DBI instructions regarding the inspection triggers, minimum connection inspection scope (pre- and post-posting), and connection selection methodology, as well as administrative instructions regarding posting terminology, inspection documentation, and coordination with ATC-20.
- DBI should revise its program instructions (CCSF, 2011, Sections IV.D.1, IV.D.5, and Appendix A) for greater consistency with FEMA 352 principles. Even where FEMA 352 is not used specifically, DBI should require at least a specified number of connection inspections (pre- and post-posting).
- To facilitate connection inspections, DBI should revise its BORP instructions to require pre-selection of connections to be inspected post-earthquake, in order to address questions regarding access, fireproofing removal, or necessary tools. Pre-selection will also facilitate documentation and reporting.
- Since welded joints can include pre-earthquake flaws that become confusing or controversial when found after an earthquake, DBI should recommend pre-earthquake inspection and testing to set a baseline for comparison with post-earthquake findings.

4.2.9 Concrete Structures

BORP guidelines reference FEMA 352 for welded steel structures but do not reference similar technical documents for other structure types. For Engineering evaluations, however, ATC-20 does reference FEMA 306, *Evaluation of Earthquake Damaged Concrete and Masonry Wall Buildings* (FEMA, 1998). For BORP projects, where engineers have the opportunity to use recorded motions, review plans, perform destructive investigations, and run structural analyses, use of FEMA 306 would be appropriate.

Recommendation: DBI, with input from SEAONC, should consider supplementing BORP instructions with guidance on the use of FEMA 306 for concrete structures.

4.2.10 Functional Recovery

DBI recognizes BORP as a way to support “expeditious business resumption” in a way that SAP cannot (CCSF, 2011). With some enhancements, and at the owner's expense, BORP procedures might be enhanced to serve functional recovery goals in addition to safety and reoccupancy goals. The Department of Public Works already uses BORP-like procedures to facilitate recovery of City-owned facilities.

Recommendation: Require certain existing buildings to file recovery plans.

- DBI, together with ORCP, should consider enhancing the BORP guidelines with a module geared to functional recovery of the building and of individual tenant spaces. The module should be optional and designed not to interfere with the first priority of safe reoccupancy. For recovery-critical buildings, DBI and ORCP should consider requiring use of the functional recovery module as a condition of BORP certification.
- Beyond recovery-critical buildings, DBI and other relevant agencies should consider a requirement or incentive for selected buildings to produce and file a recovery plan. The required plan would include much of the information currently required by BORP, and would need regular updating, but it would omit the requirement to keep engineers and inspectors on retainer. Post-earthquake safety inspection would likely still be done by SAP volunteers, but the plan would include information that might assist them, as well as pre-compiled resources for owners and tenants to address recovery issues beyond the scope of SAP and ATC-20. The requirement to produce a recovery plan would nudge owners to think through the effects of potential damage and even to consider enrolling voluntarily in BORP.
- Alternatively, DBI and other relevant agencies should study a requirement for selected buildings to report building data for use by DBI in prioritizing post-earthquake inspections by City staff or by SAP volunteers. The data would include much of the structure and system information currently required in BORP plans but not normally available on short notice to SAP inspectors. The data would be used to support the typical ATC-20 process, enabling a more informed inspection and ideally resulting in a more reliable and permanent posting. Since document review is not part of the normal ATC-20 Rapid evaluation procedure, this system would be most effective if paired with a program of specialized inspectors assigned to specific building groups, as recommended for early recovery in Section 3.2.2 of this Part.

Chapter 5

ATC-20

5.1 ATC-20 as Criteria for SAP and BORP

ATC-20, *Procedures for Postearthquake Safety Evaluation of Buildings*, (ATC, 1989) was developed with California and Federal funding in the late 1980s and published just in time to be used after the 1989 Loma Prieta earthquake. The ATC-20 procedures and criteria have been used by SAP ever since.

The most current version of the ATC-20 procedures are documented in ATC-20-1, *Field Manual: Postearthquake Safety Evaluation of Buildings* (ATC, 2005), and are applicable to buildings of all heights and offer three evaluation methods:

- A Rapid Evaluation is expected to take only 10 to 30 minutes per building and is designed to be conducted without access to the building interior, though interior inspection is recommended if feasible. The Rapid Evaluation is most suitable where small buildings need to be inspected in high numbers, and it is most effective for identifying obviously unsafe conditions. As such, it is often inconclusive but still valuable as a triage measure.
- A Detailed Evaluation is expected to take one to four hours per building, including a full interior inspection and greater emphasis on details of the structural system. The Detailed Evaluation is used to resolve an inconclusive Rapid Evaluation (Cal OES, 2015).
- An Engineering Evaluation is expected to be used only when structural analysis or destructive investigation are deemed necessary to resolve an inconclusive Rapid or Detailed Evaluation.

Rapid and Detailed Evaluations both involve completion of a standard report form. The Engineering Evaluation is customized and expected to yield a more thorough report. SAP guidelines expect that Engineering evaluations will be performed at the owner’s expense but may be requested by the jurisdiction (Cal OES, 2015). When requested, a building is slated for Engineering evaluation based on the results of a preceding Rapid and/or Detailed evaluation, not based on its height or complexity alone.

While ATC-20 is focused on safe reoccupancy, the Rapid and Detailed evaluation forms do ask the inspector to estimate the observed damage as a percentage of the building's replacement cost. This is understood as a response to the jurisdiction's need to comply with Federal government requirements regarding disaster declarations (SEAONC, 2013).

SAP training and implementation are premised on the Rapid evaluation. Even so, an SAP inspector is expected to attempt to inspect the interior, including basements and "every floor, including roof and penthouse(s)," as well as nonstructural components and potential hazardous material spills (SEAONC, 2013).

For a tall building, such an evaluation would no doubt take more than the typical 20 minutes expected for high-volume Rapid evaluations. Even a Detailed evaluation of a relatively modern tall building might be inconclusive if the structural elements are hidden and the details are unknown in advance. San Francisco's Building Occupancy Resumption Program (BORP) has been motivated to resolve these issues. Even though BORP inspections are based on ATC-20 and are permitted to use the Rapid method, the advanced planning makes a BORP inspection more like the Detailed method by default. If analysis is performed in advance (and because the BORP engineer is selected and compensated by the building owner), BORP can even achieve some of the benefits of the Engineering method even if only a Rapid or Detailed evaluation form is submitted.

Regardless of which method is used, the ATC-20 procedure is intended to result in a "posting" indicating the building's safety and readiness for reoccupancy. Table 6-1 shows the three available postings with the ATC descriptions of their intended meaning.

The RESTRICTED USE placard has multiple uses. The Yellow placard is typically used for an inconclusive Rapid or Detailed evaluation, as might be expected for a tall building outside BORP. The same Yellow placard might be used to indicate a partial restriction, by time or location, to allow as much use as is deemed safe even while the further evaluation is being planned.

ATC-20 identifies six categories of damage that, if deemed "severe" with respect to the whole building, should be posted UNSAFE (ATC, 2005, Table 3-1):

- Collapse, partial collapse, or building off foundation
- Building or story significantly out of plumb
- Severe damage to primary structural members

- Chimney, parapet, or similar falling hazard
- Mass ground or slope movement
- Other: Hazardous material release, etc.

Table 6-1 ATC-20 Posting Classifications (ATC, 2005)

Posting Classification (Placard Color)	Descriptive Text on Placard	Description of Safety Assessment
INSPECTED (Green)	"Lawful Occupancy Permitted. This structure has been inspected (as indicated below) and no apparent structural hazard has been found."	"No apparent hazard is found, although repairs may be required. The original seismic resistance is not significantly decreased. No restriction on use or occupancy."
RESTRICTED USE (Yellow)	"This structure has been inspected and found to be damaged as described below. Entry, occupancy, and lawful use are restricted as indicated below."	"A hazardous condition exists (or is believed to exist) that requires restrictions on the occupancy or use of the structure. Entry and use are restricted as indicated on the placard."
UNSAFE (Red)	"Do not enter or occupy. (This placard is not a demolition order.) This structure has been inspected, found to be seriously damaged and is unsafe to occupy, as described below. Do not enter, except as specifically authorized in writing by jurisdiction. Entry may result in death or injury."	"Extreme structural or other hazard is present. There may be imminent risk of further damage or collapse from creep or aftershocks. Unsafe for occupancy or entry, except as authorized by the local building department."

Each of the six categories is possible in a tall building, and most are easily discerned. For a tall building, however, assessment of "severe damage to primary structural members" can require some expertise and ideally access to the building's structural plans (both of which BORP would provide). Even so, ATC-20 outlines the most common damage patterns for engineered concrete and steel structures. Table 6-2 summarizes those most applicable to tall buildings.

In addition to highlighting certain damage patterns, ATC-20 emphasizes inspection of building features that contribute to plan and vertical structural irregularities. Irregularities are common to San Francisco's tall buildings, in which the plan dimensions at upper levels often differ from those at grade. Although changes in the architectural floor plan do not always indicate shifts in the structure, tall building lateral systems do often feature step-backs toward the top and transfers or offsets toward the base. These structural irregularities are often hidden even from satellite imagery. This is another reason why review of the structural plans, perhaps through a BORP process, is helpful, if not essential, for post-earthquake safety evaluation of tall buildings.

Table 6-2 Concrete and Steel Damage Patterns Associated with an ATC-20 UNSAFE Posting

Structural element	Potential Damage Patterns Relevant to Tall Concrete Structural Systems (ATC, 2005, Chapter 8)	Potential Damage Patterns Relevant to Tall Steel Structural Systems (ATC, 2005, Chapter 9)
Foundation	Bowing of underground walls "Fractured" foundation	Bowing of underground walls "Fractured" foundation "Fractures" 1/2" wide in basement slab
Gravity system, and floor or roof framing	Flat slab punching shear cracking Failure or severe damage to gravity load-carrying element or connection	Failure or severe damage to gravity load-carrying element or connection Failure at shear connection between diaphragm and beam
Diaphragms	"Broken" floor-to-wall connection "Broken or seriously damaged" diaphragm	"Bowed, broken, or seriously damaged" diaphragm Damaged chord or collector Horizontal cracks 1/2" wide in concrete slab
Columns	Buckled or fractured column Spalling with exposure of column vertical reinforcing	Buckled or failed column Shear failure at column base connection
Moment frames	"Seriously degraded" moment frames Panel zone cracking	"Seriously degraded" moment frames "Significant" weld or other failure at moment joint Flange buckling near moment joint Column splices (by reference to FEMA 352) Panel zones (by reference to FEMA 352)
Other lateral force resisting systems	**Shear walls** "Failed shear wall" Diagonal cracks 1/8 wide extending between floors Failure or slippage of horizontal construction joints Spalling with exposure of vertical reinforcing at boundary elements Horizontal cracks 1/8 wide extending through boundary elements **Wall-pier systems** "Several failed piers at any one story" Failed spandrel beams	**Diagonal bracing** Buckled or stretched brace Broken brace or connection Failed chord

As alternatives to its general inspection guidance, ATC-20 also cites, but does not require, two FEMA documents for specific structure types: FEMA 306, *Evaluation of Earthquake Damaged Concrete and Masonry Wall Buildings*, (FEMA, 1998) and FEMA 352, *Recommended Post-earthquake Evaluation and Repair Criteria for Welded Steel Moment-Frame Buildings*, for Detailed Evaluation of welded steel moment frame structures (FEMA, 2000).

5.2 Issues and Recommendations Regarding ATC-20

ATC-20 has its own development and update process; an update is currently underway. Further, SAP trainings are produced for broad audiences and must cover more than San Francisco's particular conditions. Therefore, it might be difficult for the City to implement its own customization of ATC-20. Nevertheless, DBI should establish its own understanding of how it will interpret and implement ATC-20 guidance, especially through BORP, which is a San Francisco program. Thus, the following interpretations and suggested edits of ATC-20 materials might be developed for BORP guidelines.

Most of the following can apply to non-tall buildings as well as tall ones, and to implementation of ATC-20 through either SAP or BORP.

5.2.1 Limits on Exterior-Only Inspection

Exterior-only inspection, allowed by ATC-20's Rapid evaluation method, will be inconclusive for most tall buildings and for many non-tall buildings. BORP requires a Detailed evaluation to solve this problem, but any building not enrolled in BORP will be eligible to use the general ATC-20 procedures. That said, ATC-20 does acknowledge that certain structure types should only be posted after interior inspection. Unfortunately, this guidance is not as clear as it should be.

Recommendation: DBI should clearly list the structure and building types that may be posted INSPECTED only after an interior inspection.

- For these buildings, unless the building is posted UNSAFE, the posting must be RESTRICTED USE, and the placard must call for interior inspection. ATC-20 already recognizes that tilt-up structures are in this group and that steel braced frames might be.
- DBI should consider defining this group by structural system (with consideration for cases where the system type is unknown), height or size, use or occupancy that would be sensitive to interior nonstructural damage, or other attributes.

5.2.2 Limits on Rapid Evaluation

San Francisco developed BORP as an alternative to SAP in part because the proper application of ATC-20's Rapid evaluation method would be inconclusive for many large and relatively modern buildings in which the structural system is either not discernible or not observable from even an interior inspection. Similarly, FEMA 352 was developed as an alternative to ATC-20 in response to evidence that in welded steel moment frames, significant structural damage is not revealed by obvious story drift or by nonstructural damage. The same observation has been made in other structure types (ATC, 2005, p72), and the principle likely applies to many large steel and concrete structures, including most tall buildings. Further, even where some structural damage is found, ATC-20 notes that a posting of UNSAFE applies only where “severe” damage renders the entire structure dangerous (ATC, 2005, p53). In tall buildings, however, complicated multi-modal response can make the critical locations harder to identify without an advance review of the structural design. For these reasons, ATC-20, which is enormously valuable for most buildings, is less well suited to tall and complex structures.

Recommendation: DBI should clearly list the structure and building types that may be posted INSPECTED only after a Detailed evaluation.

- For these buildings, unless the building is posted UNSAFE, the posting must be RESTRICTED USE, and the placard must call for Detailed evaluation.
- DBI should consider defining this group by structural system (with consideration for cases where the system type is unknown), height or size, or other attributes.
- The Detailed evaluation should involve conditions throughout the building that are similar to any found damaged in an initial inspection.
- For tall and large buildings, DBI should consider supplementing ATC-20 guidance regarding its limits on UNSAFE postings for severe but local damage. Local damage might include partial collapse at one end or wing, floor or floor framing damage at only a small number of the building’s floors, or connection damage that changes the system characteristics but does not indicate a distinct collapse mode.

5.2.3 Damage Estimates

For both Rapid and Detailed evaluations, ATC-20 asks for an estimate of damage as a percentage of the building’s replacement cost. In its SAP training materials, SEAONC (2013, p66) has noted, “This is a matter of personal judgment; there is no set formula to calculate this information.” Cal OES (p3), advises each jurisdiction to have “a system of cost estimation developed prior to the disaster.”

Recommendation: Develop or update a protocol for damage estimation.

- DBI should ensure that its methodology for converting SAP and BORP inspectors’ damage estimates and building valuations into loss estimates is up to date.

- For all buildings, but especially for tall or other large buildings, to ensure rational and consistent damage estimates, DBI should consider developing guidelines for SAP and BORP inspectors regarding the characterization of observed damage in terms of percentage loss.

5.2.4 Placard Use and Tracking

The posting of San Francisco tall buildings by SAP volunteers is likely to involve considerable uncertainty, for which a RESTRICTED USE placard is appropriate. However, the RESTRICTED USE placard can be used also to indicate a firm conclusion applicable to only part of a building. Since the disposition of a tall building is likely to both affect and be affected by neighboring properties, these multiple meanings can lead to confusion and delays. In Christchurch, the ATC-20 forms were modified to include sub-categories to clarify the reasons for the selected posting. This made it easier for the jurisdiction to prioritize and track follow-up inspections (ATC, 2014). In particular, it is useful for DBI's status tracking to distinguish buildings where RESTRICTED USE is due to damage to the building itself or due to an unsafe condition in an adjacent (perhaps tall) building.

Recommendation: Develop or update a protocol for consistent placard use and tracking, as contemplated by SFBC Section 101A.21.

- Because San Francisco customizes and prints its own placards, DBI should consider adding check boxes to identify the particular basis for a RESTRICTED USE or UNSAFE posting. The ATC-20 Safety Assessment Forms should be modified to match.
- DBI should track the subcategories as well as the overall posting. In particular, if a building's posting status is linked to that of an adjacent building, the linkage should be tracked so that when the controlling building's issue is resolved, the dependent buildings' status is also updated.

5.2.5 Placard Text

The default placard text can be clearer and more consistent.

Recommendation: Because San Francisco customizes and prints its own placards, it should consider the following revised text:

- INSPECTED placard: "... no ~~apparent structural~~ <u>immediate safety</u> hazard has been found."
- RESTRICTED USE placard: "Entry, occupancy, and ~~lawful~~ use are restricted as indicated below."

Chapter 6

FEMA 352

6.1 Background

FEMA 352, *Recommended Post-earthquake Evaluation and Repair Criteria for Welded Steel Moment-Frame Buildings* (FEMA, 2000a), is one of a series of guideline and background documents produced in response to steel frame damage observed in the 1994 Northridge earthquake. FEMA 352 is applicable to welded steel moment-frame buildings of all heights.

FEMA 352 is premised on two Northridge lessons learned after the original publication of ATC-20 in 1989. First, pre-Northridge welded steel moment frames are prone to connection fracture, especially at welded joints. The 1989 version of ATC-20 had acknowledged the possibility of connection damage, particularly for “semirigid moment connections made with angles, tees, and plates in bolted or riveted joints,” which it characterized as “nonductile.” It called for the evaluator to “spot-check several beam-column joints,” but only those deemed “nonrigid.” For the welded connection common at the time, ATC-20 noted that “ductile moment frames are expected to sustain yielding in beams while columns remain elastic.”

Second, Northridge fractures were commonly found in buildings that showed few visible signs of structural damage. The ATC-20 Rapid evaluation method, in which “ordinarily, only the exterior of the building is inspected,” had been based on the experience that even when the structure is not exposed, serious structural damage will show as a visible lean or as corresponding nonstructural damage. By contrast, FEMA 352 notes, “often, damage to steel moment-frame buildings cannot be detected by rapid evaluations” (Section 1.5) and “The absence of significant observable damage ... in a preliminary evaluation ... should not be used as an indication that detailed evaluations are not required” (Section 3.3.4.3).

The recognition that exterior-only Rapid evaluation might be inadequate is not without precedent. ATC-20 already says that concrete tilt-up buildings should not be posted without an interior inspection of the roof-to-wall connections and cites examples of steel braced frame damage that was only observed after nonstructural finishes and fireproofing were removed (ATC, 2005). By referencing FEMA 352, ATC-20 is, if obliquely, saying the same about pre-Northridge welded steel moment frames.

FEMA 352 thus differs conceptually from ATC-20 in its fundamental attitude toward non-visible damage. This leads to two pragmatic differences: FEMA 352 uses different terminology for its inspection protocols and sets a higher minimum scope of inspection both before the building is posted and after it receives a green INSPECTED placard.

Table 6-3 compares the different evaluation methods of ATC-20 and FEMA 352. The key differences:

- Terminology: What ATC-20 calls a Detailed evaluation, FEMA 352 calls Preliminary evaluation Step 4, involving inspection of a specified number of beam-column connections. What ATC-20 calls an Engineering evaluation, FEMA 352 calls a Detailed evaluation. Thus, simplistic reference to "Detailed" evaluation can be confusing.
- Required scope prior to initial posting: ATC-20 allows a Rapid evaluation, even an exterior-only evaluation. In many cases the ATC-20 evaluator will recognize that the unknown conditions merit a Yellow RESTRICTED USE placard, but ATC-20 does not require it. ATC-20-2, *Addendum to the ATC-20 Postearthquake Building Safety Evaluation Procedures* (ATC, 1995, Section 4.4.1) acknowledged that "Rapid Evaluations can be inconclusive," but still allowed that "[Inspectors] may choose to post the structures UNSAFE or RESTRICTED USE until more detailed inspection is performed." By contrast, compliance with FEMA 352 requires actual connection inspection prior to any posting. Thus, by FEMA 352, an ATC-20 Rapid evaluation is always incomplete and should always yield a Yellow placard, unless a Red placard is already justified from the quick review (Bonowitz, 2013).
- Required scope after initial posting: If an ATC-20 Rapid evaluation results in a Green INSPECTED posting, the ATC-20 scope is complete. However, ATC-20-3 presents a case study that suggests, but does not require, connection inspection: "[S]teel frame buildings that have experienced severe seismic motions [should] be inspected for possible connection ... failures, even if the building is apparently not structurally damaged and has been posted INSPECTED." FEMA 352 would adopt this more thorough approach. Using FEMA 352, the inspection scope is not complete until the FEMA 352 Detailed evaluation assesses the significance of the inspection findings. Reoccupancy is allowed by inspector judgment after the initial Yellow placard is posted, but the FEMA 352 Detailed inspection must still be completed.

Table 6-3 Comparison of ATC-20 and FEMA 352 Terminology and Evaluation Methods

ATC-20 Evaluation Method	Equivalent FEMA 352 Evaluation Method
RAPID EVALUATION Required for posting Exterior-only inspection allowed	PRELIMINARY EVALUATION, Steps 1 – 3 Required for posting Interior and exterior inspection required Review of architectural and structural plans recommended
DETAILED EVALUATION Not required for posting Interior inspection typical Connection inspection recommended *	PRELIMINARY EVALUATION, Step 4 Required for posting Visual inspection of specified number of connections required *
ENGINEERING EVALUATION Not required for posting	DETAILED EVALUATION Required even if posted INSPECTED Based on connection inspections * May be prescriptive or analytical

* BORP does not require connection inspection even when FEMA 352 is used. See Section 4.2.9 of this Part.

Table 6-4 provides a further breakdown of differences between the minimum inspection scope each document requires prior to an initial posting. FEMA 352 expands the ATC-20 Rapid evaluation scope by requiring an interior inspection and at least visual inspection of a specified number of welded connections. FEMA 352 offers different methods for selecting the connections to be visually inspected in Preliminary Step 4, but they almost always require four locations per floor. For a tall building (and even for a short one), this is not feasible in the time normally budgeted for an ATC-20 Rapid evaluation.

Table 6-4 Comparison of Minimum Scope of ATC-20 Rapid and FEMA 352 Preliminary Methods

Evaluation Scope	ATC-20 Rapid Evaluation	FEMA 352 Preliminary Evaluation
Exterior visual inspection	Required	Required (Step 1)
Interior visual inspection	Not required	Required (Step 2) Hazardous materials review required Elevator inspection required
Permanent drift measurement	Required, visual only	Required (Step 3)
Connection visual inspection	Not required	Required (Step 4) Removal of fireproofing required only if damage is indicated Ultrasonic or other testing not required

FEMA 352 has not yet been implemented after an earthquake. A simulation by the SEAONC Existing Buildings Committee Steel Frame Subcommittee (2004) showed that FEMA 352's statistical sampling procedures can yield widely inconsistent results depending both on the method used and on choices left to the inspecting engineer. The study was based on real 5-story buildings damaged in the Northridge earthquake.

6.2 Issues and Recommendations Regarding FEMA 352

Because FEMA 352 is not a standard, DBI can develop its own implementation procedures. Specific recommendations for applying FEMA 352 within BORP are provided in Section 4.2.8 of this Part.

6.2.1 Implementation of FEMA 352

Although not an official standard, FEMA 352 is the only guideline specifically addressing inspection and evaluation of pre-Northridge welded steel moment frames. Nevertheless, it lacks a track record of actual implementation and has not been condensed into practical and enforceable provisions. Further, use of FEMA 352 as written requires some choices and discretion by the engineer. As discussed above, inspection of individual connections is fundamental to FEMA 352 and is the main difference between this system-specific document and the more generic ATC-20. Without the connection inspection protocol, FEMA 352 adds very little to ATC-20 except for modestly different inspection triggers. DBI, together with the Structural Subcommittee of the Code Advisory Committee (and perhaps with the SEAONC Existing Buildings Committee) should study the feasibility, benefits, and costs of requiring some form of FEMA 352 compliance for welded steel moment frame structures.

Recommendation: Develop a new Administrative Bulletin (AB) for post-earthquake inspection and evaluation of welded steel moment frames with the following considerations:

- The AB should address both safety inspection and structural evaluation, as FEMA 352 covers both topics. The AB should reference FEMA 352 and set specific requirements where the document allows options. See also Part 4.
- The AB should set San Francisco policy regarding connection inspection triggers, minimum connection inspection scope (pre- and post-posting), and connection selection methodology. It should provide administrative instructions regarding posting terminology, inspection documentation, and coordination with SAP and ATC-20. DBI should consider simply clarifying that given a certain shaking level or other inspection trigger, the building

must be posted RESTRICTED USE (indicating an incomplete inspection) until the minimum number of connection inspections is complete.

- Review FEMA 352 for application to representative San Francisco structures. In particular, review the findings of the SEAONC Existing Buildings Committee Steel Frame Subcommittee (2004) showing that results are sensitive to the joint sampling method.
- The AB should apply FEMA 352 to interpret generic SAP and ATC-20 guidance on appropriate posting of inspected buildings.
- DBI should develop tracking procedures to account for the multiple phases (FEMA 352 "steps") of a post-earthquake safety inspection and posting process based on FEMA 352. The procedures should ensure that required connection inspections are reported and completed, with postings that properly reflect the interim stages.

6.2.2 Application of FEMA 352 to Loma Prieta Damage

The 1994 Northridge earthquake revealed unexpected damage to dozens of welded steel moment frame structures throughout greater the Los Angeles area. Some of these were tall buildings; most were not. In most cases, connection damage did not reveal itself through obvious damage to finishes or noticeable changes in behavior under everyday use. Five years earlier, San Francisco's steel buildings were subjected to the Loma Prieta earthquake. Without the benefit of lessons that were later learned in Northridge, steel buildings in San Francisco were never systematically inspected for possible welded connection damage following the Loma Prieta earthquake.

About 30 Bay Area steel moment frame buildings are known to have been inspected for possible Loma Prieta damage in the few years following the Northridge earthquake. At least five buildings, all between 30 and 60 miles from the Loma Prieta epicenter, but none in San Francisco proper, were found to have weld fractures. Four of these damaged buildings were 12 stories or taller. Although details from these voluntary inspections are incomplete, the heaviest reported damage appears to have been in buildings on soft soil where available information on peak ground accelerations (PGA) exceeded 0.25g. For two of the damaged buildings, however, the closest recorded ground motion had a PGA of only 0.18g (FEMA 355E, 2000b, Section 5.9).

In 1996, the Structural Engineers Association of California, Applied Technology Council, and Consortium of Universities for Research in Earthquake Engineering (SAC) Joint Venture notified Bay Area building officials of these findings, but no Bay Area city is known to have adopted an official policy or implemented a formal inspection program in response. Since 1996, it is likley that additional buildings with welded steel moment frames have been inspected for Loma Prieta damage, either out of an owner's self-interest or in the course of typical pre-purchase investigations. Very likely, some of these have been in San Francisco, and some have probably been tall buildings, but the procedures for these voluntary inspections are not regulated by DBI, and thus, the findings are not recorded by the City.

Almost 30 years after Loma Prieta, should San Francisco implement a systematic inspection program for its pre-Northridge welded steel moment frames? Inspection can be costly and disruptive. If there is no reason to expect damage, or if any reasonably expected damage would have little effect on future performance, why expend the resources? A key issue involves the ground motion that should trigger a FEMA 352 inspection.

FEMA 352 itself addresses this question. FEMA 352 Section 3.2 recommends seven inspection triggers for a given building based on estimated ground motion at the site and known damage nearby. Specifically, for a city with high seismicity like San Francisco, FEMA 352 recommends a protocol of joint inspections where the PGA exceeds 0.25g, where nearby unreinforced masonry buildings show "prevalent" full or partial collapse, where nearby nonstructural damage is at "high levels," or where other nearby buildings show "considerable" damage.

Except for the PGA trigger, these conditions allow for ample judgment. Considering Loma Prieta building performance in or near downtown San Francisco, examples of FEMA 352 indicators of potential damage were observed in different parts of the city. Whether or not similar levels of damage would trigger FEMA 352 inspections for downtown buildings after a future earthquake would be a matter of building official judgment.

The PGA trigger in FEMA 352 is more quantitative, but it was based on sound (but still incomplete) data from just one event (Northridge) and on relatively sparse data, described above, from Loma Prieta. The majority of Northridge steel frame damage occurred in Los Angeles neighborhoods with PGA values of 0.50g or higher (USGS, 1994). In downtown Los Angeles, PGA values were under 0.30g, with some under 0.20g, and very little damage was found in buildings of any height. However, post-Northridge inspection there, especially among tall buildings, was relatively rare, and it effectively stopped when the scope of the city's mandatory inspection ordinance excluded most of downtown. "[I]t is possible that no damage was found in downtown structures because so little inspection had been done there early in 1994" (FEMA 355E, Section 6.3.2).

Loma Prieta ground motion records in San Francisco are limited. Available information includes data from recording stations and data inferred from Modified Mercalli Intensity (MMI) and Did You Feel It (DYFI) observations contained in Loma Prieta reconnaissance reports (USGS, 1989). Stations recorded PGA values between 0.06g and 0.16g, which are below FEMA 352 triggers for inspection, but observational data at several downtown locations suggest inferred PGA values of 0.24g, which is approaching the threshold value of 0.25g in FEMA 352 for areas of high seismicity. Additional observational data include 0.45g near the intersection of Market Street and Van Ness Avenue and more than 0.8g at locations in the Marina and near the Ferry Terminal. Although these data are outliers, they indicate a potential for more significant shaking in selected areas.

As a policy matter, the *San Francisco Existing Building Code* is clear: earthquake damage must be found and repaired. Beyond repair, seismic retrofit might even be triggered, as discussed in Part 4. How to implement these code provisions so long after the event raises interesting questions worthy of a coherent and official reponse. Had FEMA 352 been available as a guide in 1989, portions of San Francisco might have been considered to be at, or above, the triggering PGA value for inspection, and the presence of damage to other classes of buildings would have required the building official to make a judgment regarding the need for inspection. Considering the question again in 2018, the most useful information would come from voluntary inspections performed—but not reported to the City—over the past two decades. Information from voluntary inspections, in combination with recorded or inferred ground motion data, could be used to confirm the threshold indicators for possible damage in the Loma Prieta earthquake.

Recommendation: Apply the repair provisions of the *San Francisco Existing Building Code* with respect to possible Loma Prieta damage to welded steel moment frames.

- Criteria should be based on FEMA 352 and on the Administrative Bulletin recommended in Section 6.2.1 of this report. DBI should consider alternative criteria to accommodate voluntary inspection and repair work already performed in good faith.
- Application of FEMA 352 and the AB should consider available information and community consensus regarding appropriate ground motion triggers and damage indicators. In particular, the City should seek information from voluntary inspections performed since 1989 as a way of

gauging the likelihood of weld damage in San Francisco buildings of various heights.

- Enforcement should focus on finding and repairing damage. While the SFEBC also triggers seismic work in cases of substantial structural damage and disproportionate damage (see Part 4), the City might consider waiving these triggers, given the unusual circumstances of post-earthquake inspection so long after the damaging event.
- While enforcement is already supported by SFEBC provisions and should not require new legislation, the City should consider implementing the applicable code provisions through a special program to accommodate the unusual circumstances. The program might involve specific notifications and outreach to building owners and tenants, as well as liberal deadlines to allow phased or flexible compliance.

Chapter 7

Inspection Technologies

In addition to the policies and programs discussed in this part, the tools and methods of inspection, damage-finding, and documentation, while outside the scope of this report, are worthy of review as well. Some of these are new or emerging technologies, while others are merely common sense applications of existing ideas that might be required, funded, or encouraged by a building code, a program such as the City's Building Occupancy and Resumption Program (BORP) or a City initiative to develop standards and best practices. As City policy, some of these might benefit from (or be subject to) review by the City's Committee on Information Technology.

- Access openings built into the structure and architecture
- Building-specific social media data recorded or posted by tenants (who will not be present during the safety inspection)
- Satellite or GPS imagery, for overhead or birdseye views, especially for comparison of pre- and post-earthquake conditions
- Building instrumentation to record earthquake motions. This is more feasible than ever, and BORP guidelines already allow (but do not require) the use of motion records to assist damage-finding and posting. However, standards for processing, interpreting, and applying recorded motion data by non-experts do not yet exist. Also of interest are questions about ownership and publication of data, which might be private or privileged but have public benefit.
- Measurable building data, especially for comparison of post-earthquake values with pre-earthquake baseline values. These might include:
    - Three-dimensional laser scanning, as used to monitor cliff movements and selected buildings in Christchurch (ATC, 2014)
    - Laser measurements in elevator shafts, for gauging permanent lateral deformations
    - Ambient vibration testing to establish structural periods and mode shapes
    - Strain gauges on steel members

    - Movement gauges across existing joints, between floors, or between nonstructural components and the structure.

- Use of unmanned aerial vehicles (UAVs, also known as drones) and robots. Both were used in Christchurch (ATC, 2014). San Francisco has a drone policy that authorizes specific uses by different City departments (CCSF, 2017b), but neither DPW nor DBI—the two departments responsible for most post-earthquake building safety inspections—is authorized (DataSF, 2018).
- Computerized or online documentation and automated databasing to reduce data management demands and improve speed, consistency, and completeness of reporting.
- Barcoding of buildings to allow smartphone access to building information for City staff or deputized Cal OES Safety Assessment Program (SAP) volunteers performing post-earthquake safety inspections.
- Computer or smartphone apps to assist building owners and tenants with their own reoccupancy and recovery, to reduce the post-earthquake resource demand on the City.

Recommendation: Study new and emerging technologies and practices for damage finding and documentation. From the preceding list, the City should consider additional study and program development for the topics deemed most likely to improve citywide resilience.

Appendix A

References

ASCE, 2017, *Seismic Evaluation and Retrofit of Existing Buildings*, ASCE/SEI 41-17, American Society of Civil Engineers Structural Engineering Institute, Reston, Virginia.

ATC, 1989, *Procedures for Postearthquake Safety Evaluation of Buildings*, ATC-20, Applied Technology Council, Redwood City, California.

ATC, 1995, *Addendum to the ATC-20 Postearthquake Building Safety Evaluation Procedures*, ATC 20-2, Applied Technology Council.

ATC, 2005, *Field Manual: Postearthquake Safety Evaluation of Buildings, Second Edition*, ATC-20-1, Applied Technology Council, Redwood City, California.

ATC, 2014, *Building Safety Evaluation after the February 22, 2011 Christchurch, New Zealand Earthquake: Observations by the ATC Reconnaissance Team*, ATC-109, Applied Technology Council, Redwood City, California.

ATC, 2018, *Recommended Earthquake Performance Goals for San Francisco's Buildings (ESIP Task A.6.h)*, final report submitted to the City and County of San Francisco prepared by the Applied Technology Council, Redwood City, California.

Bonowitz, D., 2011, *Resilience Criteria for Seismic Evaluation of Existing Buildings*, Structural Engineers Association of Northern California, August 5.

Bonowitz, D., 2013, *ATC 20 Simulation*, presented as part of SEAONC ATC-20 Training, Santa Clara, June 22.

Bonowitz, D., 2016, *Earthquake Resilience: A New Context for Engineering Research*, presentation to the PEER-SEMM Seminar, Berkeley, March 14.

Cal OES, 2015, *Post-Disaster Safety Assessment Program: Guideline to the Activation and Utilization of Program Resources*, California Office of Emergency Management, Sacramento, California. Available at http://www.caloes.ca.gov/cal-oes-divisions/recovery/disaster-mitigation-technical-support/technical-assistance/safety-assessment-program, last accessed September 18, 2018.

CCSF, 2008, *City and County of San Francisco Emergency Response Plan: Earthquake Annex*, Department of Emergency Management, the City and County of San Francisco, California.

CCSF, 2011, *Building Occupancy Resumption Program*, Department of Building Inspection, the City and County of San Francisco, California.

CCSF, 2016, *The San Francisco Building Code*, the City and County of San Francisco, California.

CCSF, 2017a, *Building Façade Inspection and Maintenance*, Administrative Bulletin 110, Department of Building Inspection, the City and County of San Francisco, California.

CCSF, 2017b, *Citywide Employee Drone Policy, Committee on Information Technology*, the City and County of San Francisco, California.

DataSF, 2018, *Authorized Drone Uses*, March 18, https://data.sfgov.org/City-Management-and-Ethics/Authorized-Drone-Uses/5e3s-sj56

FEMA, 1998, *Evaluation of Earthquake Damaged Concrete and Masonry Wall Buildings, Basic Procedures Manual*, FEMA 306, prepared by the Applied Technology for the Federal Emergency Management Agency, Washington, D.C.

FEMA, 2000a, *Recommended Post-earthquake Evaluation and Repair Criteria for Welded Steel Moment-Frame Buildings*, FEMA 352, prepared by the Structural Engineers Association of California, Applied Technology Council, and Consortium of Universities for Research in Earthquake Engineering (SAC) Joint Venture for the Federal Emergency Management Agency, Washington, D.C.

FEMA, 2000b, *State of the Art Report on Past Performance of Steel Moment-Frame Buildings in Earthquakes*, FEMA 355E, prepared by the Structural Engineers Association of California, Applied Technology Council, and Consortium of Universities for Research in Earthquake Engineering (SAC) Joint Venture for the Federal Emergency Management Agency, Washington, D.C.

FEMA, 2012, *Reducing the Risks of Nonstructural Earthquake Damage: A Practical Guide*, FEMA E-74, prepared by the Applied Technology Council for the Federal Emergency Management Agency, Washington, D.C.

SEAONC, 2013, *Post-Disaster Safety Assessment Program (SAP) ATC-20 Training: Student Manual (Version 11)*, Structural Engineers Association of Northern California, San Francisco, California.

SEAONC, 2004, "Post-earthquake evaluation of welded steel moment frames: case studies using FEMA 352," in *SEAOC 2004 Convention Proceedings*, Structural Engineers Association of California, August.

SPUR, 2012, *Safe Enough to Stay*, San Francisco, California.

PART 7:
Pre-earthquake Evaluation

Chapter 1

Introduction

This Part reviews and explains current San Francisco policy for pre-earthquake seismic evaluation, especially as it relates to the City's existing tall buildings. It recommends modifications to the current policy and proposes additional study or development of new regulations to address identified issues. The recommendations presented correspond to Recommendations 2B and 3H presented in the Summary Recommendations.

This Part addresses aspects of the following Tasks described in the *CAPSS Earthquake Safety Implementation Program Workplan 2012-2042* (CCSF, 2011):

- Task C.1.a: Mandatory evaluation on sale or by deadline of building types not otherwise covered by the *Workplan*
- Task C.1.b: Evaluation of buildings retrofitted prior to 1994 or built to non-conforming performance standards
- Task C.2.a: Mandatory retrofit of older non-ductile concrete residential buildings
- Task C.2.d: Mandatory evaluation and retrofit of pre-1994 welded steel moment frame buildings
- Task C.2.e: Mandatory evaluation and retrofit of other low-performance buildings

1.1 Organization

Chapter 2 presents the current San Francisco practice.

Chapter 3 presents several alternative approaches.

Chapter 4 presents a summary of issues and recommended modifications to the current policy and proposes additional study or development of new regulations to address identified issues.

Appendix A presents an annotated set of current *San Francisco Existing Building Code* provisions related to this topic.

Appendix B presents a brief discussion on seismic retrofit.

A list of references is provided at the end of the Part.

Chapter 2

Current Policy for Pre-earthquake Evaluation

Current San Francisco policy requires evaluation, and possibly seismic retrofit, when a building undergoes a substantial alteration or a change of use, but these cases are rare, especially for tall buildings. Other jurisdictions, as well as some private sector stakeholders, have developed more proactive policies that San Francisco could adopt.

The *San Francisco Existing Building Code* (SFEBC) regulates most aspects of pre-earthquake structural evaluation and retrofit. SFEBC provisions combine the *California Existing Building Code* (CEBC, which itself is an amended version of the *International Existing Building Code* (IEBC), and San Francisco amendments.

The SFEBC is organized by project type, recognizing five categories: additions, alterations, repairs, changes of occupancy, and relocations. Of these, only alteration and change of occupancy provisions are at all likely to affect pre-earthquake evaluation of San Francisco tall buildings. In concept, a tall building could be made even taller with an addition, but even in that rare case, the current policy is not likely to change; the existing building would have to be retrofitted to make its seismic performance equivalent to that of a new building.

Appendix A of this Part presents the SFEBC provisions regarding seismic evaluation triggered by either alteration or change of occupancy projects.

2.1 2016 SFEBC Alteration Triggers

Modern building codes consider whether an intended alteration project—typically, a nonstructural renovation or tenant improvement—warrants unintended, or “triggered,” upgrades to the rest of the building. Seismic retrofit is one of the potential upgrades.

Codes used throughout California have veered back and forth on the issue. Until the mid-1970s, model codes used a cost trigger: if the value of the alteration exceeded 50% of the value of the existing building, a full-building upgrade was required. When it was recognized that the cost trigger discouraged renovation, especially in multi-family residential buildings, the provision was weakened to a “do no harm” rule, with essentially no upgrade trigger at all. A seismic retrofit trigger was restored to the California code when it adopted the 2006 *International Building Code*. This version measured structural effects with the intent of triggering at least a partial seismic upgrade when the intended alteration would increase loads or decrease member strength by 10% or more. Significant clarifications were made in the 2009 edition, so the current alteration provision is still less than ten years old (Bonowitz et al., 2014). The current CEBC provision is in Section 403.4 (CBSC, 2016) and annotated in Appendix A of this Part.

In practice, this provision, known to engineers as “the 10% rule,” rarely results in retrofit. Typical alteration projects do not touch the building’s structural system and rarely add enough mass to affect the expected seismic performance. Where the project *would* trigger seismic work, developers can reduce the project scope to avoid it. Even when retrofit is triggered, the current provision has the effect of upgrading only individual structural elements, without requiring a thorough review of the full earthquake-resisting structural system or any nonstructural elements. This provision focuses on the size and impact of the proposed alteration, not on the adequacy of the structure it is going into. Thus, it is common for alteration projects to be permitted in seismically deficient or even collapse-prone buildings.

Meanwhile, during the “do no harm” era, some California cities, including San Francisco, adopted their own alteration triggers or continued to enforce local interpretations of the earlier ones. While this preserved a policy of well-intended, “passive” seismic improvements, it led to inconsistent rules and enforcement across the state (Hoover, 1992). San Francisco retains its own triggers, developed in the mid-1970s, as amendments to the CEBC, in SFEBC Sections 403.12.1 and 403.12.2 (CCSF, 2016). They are summarized as follows:

- For nonstructural alterations, the SFEBC requires a full-structure seismic retrofit when the intended work would add, remove, or modify architectural elements in two-thirds or more of the building’s stories within any two-year period. Thus, a typical tenant improvement on a single floor would never trigger seismic upgrade in a multi-story building. In a tall building, a dozen or more floors would all have to undergo significant renovation within two years for the trigger to apply.
- For structural alterations, the SFEBC requires a full-structure seismic retrofit when a significant portion of the floor framing, columns, and bearing walls are altered by the intended project. This sort of structural

alteration is necessary when an owner wants, for example, to create a two-story space or an internal stair by removing part of an existing floor. In a tall building, however, the triggering alteration would almost certainly have to involve a column or bearing wall in the lower half of the structure. Such alterations are rare even without triggered seismic improvements. Especially at the ground level of a tall building where lobby or retail spaces are frequently renovated, the structure is already so heavy, expensive, and critical that designers are wise not to touch it.

Each of these San Francisco triggers is more aggressive than the CEBC, because the "10%" exception that tends to limit triggered work is not offered. For tall buildings, however, neither is more effective at prompting seismic improvements, since each trigger applies only in rare cases and can almost always be avoided by carefully scoping the intended project. Further, even where the provision might apply, it triggers only structural review, ignoring any nonstructural deficiencies.

On the other hand, the CEBC alteration trigger and the additional San Francisco alteration triggers do apply in some cases (particularly in smaller buildings), and they can be effective as part of a suite of policy tools that includes mandates and incentives (Bonowitz et al., 2014). However, it should be recognized that the purpose of the building code's alteration triggers is not to find seismically risky buildings and force their retrofit. Rather, the current purpose of the code's alteration triggers is to find a balance that encourages modernization and adaptive reuse of the building stock while leveraging reasonable opportunities to reduce risk. Further discussion is provided in Appendix B.

This balancing approach makes sense especially for tall buildings where retrofit can be extremely costly and disruptive. If the current policy is too lenient or too slow, the challenge is to adjust the code and to consider alternative risk reduction measures without necessarily mandating retrofit, as discussed in Chapter 3 of this Part.

San Francisco's amendments are due for update and coordination as the CEBC adopts two new provisions in the 2018 *International Existing Building Code* (IEBC, ICC, 2018). First, the 2018 IEBC will allow the use of reduced loads for alteration-triggered seismic work, much like the reduced seismic loads already allowed in the SFEBC. Second, the 2018 IEBC will have a new trigger for "substantial structural alteration," much like the one already in the SFEBC. These two changes will bring the CEBC closer to the SFEBC, but the new versions are not identical to the SFEBC versions, so some coordination to clarify and retain San Francisco's policy intent will be needed, as discussed in Appendix A to this Part.

2.2 2016 SFEBC Change of Occupancy Triggers

As with alteration projects, the SFEBC's provisions for change of occupancy projects include both the basic trigger of the CEBC, which rarely applies, and a more aggressive supplemental trigger unique to San Francisco. (Most change of occupancy projects also involve some alterations. In these typical cases, both the change of occupancy provisions discussed here and the alteration provisions discussed in the previous section apply, with the more demanding provision governing the combined project.)

The basic provision for change of occupancy is in CEBC Section 407.4: If a change of use or occupancy also changes the assigned risk category, then the structure is subject to a full seismic upgrade. San Francisco's new and existing tall buildings are typically assigned to Risk Category II, a broad group that includes nearly all residential and commercial buildings. Thus, even if a building's primary use were to change from office to mercantile or from multi-family residential to hotel, that would not change the Risk Category, so the retrofit trigger would not be pulled.

Risk Category is assigned by CEBC Table 1604.5. An occupant load greater than 5,000 puts a building in Risk Category III, but a change of use or occupancy that increases the occupant load from under 5,000 to over 5,000 would be quite rare. Public assembly facilities are also assigned to Risk Category III, but even if a theater or banquet hall were built into an existing tall building, that would not change the building's "primary occupancy," so the CEBC Section 407.4 trigger would still not apply.

If part of an existing tall building were to be converted to a residential care facility (Occupancy I-2) with 50 or more patients, to a K-12 school with more than 250 students and staff, or to post-secondary education with more than 500 occupants, such a change would move the building into Risk Category III. In concept, such a change is possible. In practice, the cost of the triggered seismic work would likely discourage the project.

San Francisco supplements this basic provision with a more likely trigger that does not require a change of Risk Category. If a change of use involves an increase in occupant load of more than 10% *and* more than 100 people, a retrofit using San Francisco's reduced seismic loads is triggered. With this two-part trigger, the creation of a public assembly space or the conversion from residential to office could actually lead to retrofit. It seems likely that for San Francisco's tall buildings, this supplemental change of occupancy trigger might be pulled even more often than any of the alteration triggers discussed in Section 2.1. Then again, the additional cost of a triggered retrofit might be enough to discourage the change, as it often does with alteration projects.

Chapter 3

Alternative Approaches

As discussed in Chapter 2 of this Part, current San Francisco policy almost never requires a seismic evaluation or retrofit of an existing tall building even as it undergoes significant changes and renovations over decades. Retrofits do occur, but these rare projects are typically done voluntarily, and the building is often vacant during the work. Few existing buildings present that kind of opportunity. In all likelihood, the vast majority of seismic work performed on San Francisco tall buildings is voluntary with respect to the building code. Therefore, if San Francisco adopts the performance goals presented in *Recommended Earthquake Performance Goals for San Francisco's Buildings* (ATC, 2018) and better performance is required from existing tall buildings (or any building cohort), alternative approaches should be considered.

3.1 Mandatory Retrofit Ordinances

One alternative is to set the building code and its triggers aside and work toward mandatory retrofit ordinances instead. After all, if a community is at risk from an identifiable group of buildings, that risk is not likely to go down by waiting for each building, one by one, to propose a substantial voluntary alteration that happens to pull an arbitrary trigger. To address persistent community-wide risk, it has always been more effective to mandate retrofit, whether the concern is public safety, as in the case of unreinforced masonry, or essential functions, as in the case of California hospitals. San Francisco has been a leader in developing mandatory programs for specific building groups that pose risks to the city, such as “soft-story” apartment buildings (*San Francisco Existing Building Code*, SFEBC Chapter 4D) and private schools not regulated by the state (SFEBC Section 329). Indeed, *CAPSS Earthquake Safety Implementation Program Workplan 2012-2042* (CCSF, 2011) already contemplates broad mandatory programs for most buildings designed before 1994, including non-ductile concrete and welded steel moment frame structures of any height.

If San Francisco’s tall buildings pose risks comparable to its 2,000 unreinforced masonry buildings or 5,000 soft-story wood-frame buildings, and if voluntary work is not achieving the City’s reasonable safety and recovery goals, then the case for a mandatory program should be easy to make—

provided the costs are bearable by the community. However, retrofit of a tall building while it remains occupied and functional can be enormously expensive and disruptive. Further, tall buildings do not comprise a uniform class in terms of risk (unlike unreinforced masonry or soft-story buildings) or function (unlike hospitals, schools, or multi-family residential), so a mandatory program based only on height will have scattered effects, with benefits that might be hard to demonstrate.

For these reasons, even if the *Workplan 2012-2042* contemplates programs for all structures of a certain age or type, subsets of tall (or otherwise large or complex) buildings merit special attention, perhaps even different prioritization or criteria. A study of non-ductile concrete buildings in Los Angeles reached this conclusion, recognizing that mandatory retrofit of all 1,500 buildings would not be the most effective policy; in particular, the study found that mandatory retrofit for only the tallest buildings (8 stories or taller) would be significantly less effective than retrofit of about the same number of smaller, older buildings with distinct deficiencies, largely because the retrofit cost for the tall buildings was so much greater (Anagnos et al., 2016).

Based on these considerations, the most effective and feasible response is not necessarily to mandate retrofits for dozens or hundreds of buildings. Instead, it will be more effective to accommodate the resource constraints of owners and tenants, to ensure that risks are properly valued, and to plan for effective repair and recovery through programs such as the Building Occupancy Resumption Program (BORP; discussed in detail in Part 6).

3.2 Aggressive Triggers with Focused Targets

Although San Francisco's alteration trigger, "2/3", is more aggressive than the *California Existing Building Code*'s (CEBC) basic provision, it is found that for a tall building it has practically no effect. Similarly, San Francisco's "10% and 100 persons" change of occupancy trigger is rarely pulled. This section presents the idea of triggering seismic work more frequently, but only for the most critical buildings.

Some simple changes to SFEBC Section 403.12.1 or Section 407.4.1 that might make the triggers apply in more cases include:

- Lowering the triggering value for alterations from "2/3" of a building's stories and supplementing it with an absolute number of altered stories. For example, the trigger could apply whenever any four stories are altered, so that a few substantial projects would trigger even in a tall building.

- Extending the two-year accumulation period to five or ten years so that the trigger cannot be as easily avoided by spacing projects over time.
- Lowering the triggering value for change of occupancy projects, perhaps from “10 percent [and] 100 persons” to “5 percent [and] 50 persons.”

That said, these triggers are judgmental and largely arbitrary, so any such alternative values merely reset the line for an owner seeking to avoid an upgrade. More important, the concern for any aggressive upgrade trigger is that it can backfire, discouraging otherwise beneficial renovations or even prompting work to be done without permits (Bonowitz et al., 2014). If necessary, the more aggressive triggers can be tempered by the following:

- Continuing to allow reduced seismic loads and age benchmarks, as the SFEBC currently does.
- Allowing a “10% rule” or similar exception to ensure that only substantial deficiencies need to be addressed.
- Revising the triggers to apply only to certain alterations or changes of use that truly extend the life of the building. Building-wide mechanical upgrades, information technology upgrades, and the addition of amenities, more so than basic improvements to a single tenant space, indicate a long-term investment that merits a seismic review. (Typical tenant improvement projects often come when a space is newly leased, so this alteration trigger could be replaced, or should at least be coordinated with, a potential acquisition trigger discussed in Section 2.3 of this Part.)
- Revising the triggers to apply only (or more aggressively) to problematic conditions identified in advance, such as vulnerable structure types or geologic conditions.
- Revising the triggers to apply only to certain uses or occupancies critical to city or neighborhood recovery. Of the uses typically found in tall buildings, ATC (2018) has identified multi-family housing, hotels, and major employers as deserving of “better than code” recovery goals.

The concept of “extended useful life” has precedents from other cities, from institutional policies, and even from the CEBC itself. In high seismic areas like San Francisco, the Federal Government requires seismic evaluation and potential retrofit when an alteration “significantly extends the building’s useful life through alterations ... that total more than 30 percent of the replacement value of the building” (BSSC, 2011). The wording of the provision is meant to give discretion to each Federal agency, but in practice, most agencies rely on the quantitative replacement cost trigger. California retains a similar cost trigger for State-owned buildings. CEBC Section 317.3.1 triggers seismic work when the alteration's "total construction cost, not including cost of furnishings, fixtures and equipment, or normal maintenance, for the building exceeds 25 percent of the construction cost for the replacement of the existing building."

Cost triggers were once common in California building codes (see Section 2.1 of this Part), but they are now mostly used where the authority having jurisdiction is also the owner, or at least has a financial stake, as is the case for public buildings. For private buildings regulated by a City, cost triggers are hard to implement, and they have been removed from model codes. Building officials prefer not to review financial calculations or replacement cost estimates. Owners dislike the uncertainty of cost triggers; a complete design and cost estimate is needed before one can know whether the trigger will be pulled.

The *Seattle Existing Building Code* (City of Seattle, 2015a) applies the concept of "extended useful life" with a subjective trigger for "substantial alteration" and defines the term to include "remodeling ... that substantially extends the useful physical or economic life of the building or a significant portion of the building, other than typical tenant remodeling" as well as "a significant increase in the occupant load of an unreinforced masonry building" (Section 304.1.1). With this provision, the City of Seattle has replaced a strict measure of cost or work area with ample building official judgment. In practice, the provision typically requires a conference with the building official, and the designation considers project cost, project scope, the vacancy rate, building age, and other building-specific conditions (City of Seattle, 2015b). What is unique about this Seattle provision is the desire to consider all of these factors together and in context and the emphasis on building official judgment. On one hand, this allows policy flexibility to accommodate special circumstances; on the other, it introduces subjectivity and uncertainty.

Interestingly, one example given in Seattle's guidance document (City of Seattle, 2015b) involves a hypothetical tall building: "A large new restaurant in a fully occupied high-rise building clearly is not a substantial alteration project. However, a similar project in an older, partially-occupied, three-story building is likely to be a substantial alteration." As noted in Section 2.2 of this Part, this very example of an assembly occupancy added in the ground floor of a tall building could pull San Francisco's current change of occupancy trigger.

Recent proposals to amend the *International Existing Building Code* (IEBC) further extended the idea of a customizable, targeted alteration trigger (ICC, 2016, proposal EB25-16 and similarly, 2012 proposal G213-12). The 2016 proposal, made by the National Council of Structural Engineers Associations (NCSEA), offered a tabular format with which a jurisdiction would list the specific building groups that would be subject to a trigger, slightly modified and filled out in a table format. NCSEA described its proposal (ICC, 2016) as beneficial to local resilience planning as follows:

> *The table format allows a jurisdiction to identify the buildings of greatest interest to local mitigation and resilience plans. We find that in some jurisdictions the concern is about a particularly vulnerable structure type (like URM, or non-ductile concrete), in some it is about school safety and recovery, in some it is about protecting senior or low-income housing, for some it is about revitalizing a commercial district. A uniform, one-size-fits-all approach no longer suits the needs of communities thinking about natural disaster recovery and resilience.*

Table 7-1 Tabular Format for Identifying Mitigation Priorities Triggered by Alterations*

Priority Type	Occupancy	Risk Cat.	Seismic Design Cat.	Size	Location	Structural Attribute	Original Permit Date	Other
Emergency	—	IV	F	—	—	—	—	—
Soft story	R	—	—	2+ st, 3+ units	—	Wood frame target story	pre-1978	—
Private school	E	—	—	—	—	—	—	26+ students
Welded steel moment frame	—	—	—	3+ st	—	WSMF	pre-1996	—
Non-ductile concrete	R	—	—	3+ st	—	Concrete	pre-1980	—
Central business district (downtown)	—	—	—	8+ st	CBD	—	pre-1996	—
Tall	B	—	—	12+ st	—	—	pre-1996	Recovery-critical

* (Based on ICC, 2016. See text for explanation.)

In its simplest form, the code change would be nothing but a change in format, from code text to a table. In Table 7-1, for example, the first row, priority type "Emergency," reflects a requirement already in the IEBC, CEBC, and SFEBC: Section 403.4.2 already requires seismic retrofit when a near-fault Risk Category IV facility undergoes a major alteration. By the NCSEA proposal, a jurisdiction could keep that row and leave the rest of the table blank.

Or, a jurisdiction could amend the model code by adding rows to the table to identify mitigation priorities in terms of use or occupancy, risk category, building height or size, age, structural characteristics, soil profile type, or even neighborhood. For example, the “Soft story” and “Private school” rows in Table 7-1 would complement two of San Francisco’s current mandatory programs by adding a related alteration trigger. The “welded steel moment frame” and “non-ductile concrete” rows show how the City might use this format to address other structure types already identified in *Workplan 2012-2042*. The final two rows of Table 7-1 show how the City might target certain tall buildings (shown, arbitrarily, as 8 stories or 12 stories and taller) based on concerns discussed elsewhere in this report, such as buildings clustered in the dense central business district (downtown) or those supporting business and commerce functions deemed recovery-critical.

None of the completed rows in Table 7-1 represents a specific policy recommendation of this report. Rather, it is the format of the table that appears useful as a way to balance more aggressive triggers with more focused intent. The tabular format by itself would not change the triggers, or the triggered scope (structural only), or the criteria currently in the SFEBC. But if the targets of retrofit triggers can be more thoughtfully identified and justified, then the triggers can be adjusted so that they will apply to more projects and be more effective as risk reduction tools.

This approach can still face feasibility hurdles. Even in a seismically deficient and recovery-critical building, if the triggered work is simply unaffordable, the owner can always choose to scale back or cancel the intended project, and in the end, the City gets neither risk reduction nor building renovation.

3.3 Aggressive Triggers with Reduced Scope

Another alternative is to make the current triggers more aggressive, so they apply to a wide range of intended projects, but limit the triggered scope to work that is less costly and less disruptive. Three ways to limit the triggered retrofit scope are:

- Set the structural retrofit criteria low, so that all but the riskiest structures or most egregious deficiencies are exempt. This approach makes sense for building groups that would justify a mandatory program if they existed in large numbers, like unreinforced masonry buildings. For tall buildings, or commercial buildings generally, the criteria could be based on a Collapse Prevention performance level or a hazard level lower than the code’s already reduced seismic loads, or both.

- Set the structural retrofit criteria to allow partial retrofits of collapse-prone deficiencies only. This approach makes sense for buildings with known and easily identifiable weak links, such as “soft-story” apartments, cripple wall houses, or obsolete tilt-up structures. For tall buildings, this approach might be useful for concrete or steel braced frame structures with discontinuous systems or for pre-Northridge welded steel moment frames.
- Set the retrofit criteria to cover only critical nonstructural components. This approach would not reduce collapse risk, but in many commercial buildings, including tall buildings, the collapse risk in a design earthquake is already relatively low compared with other vulnerable structure types. Since the current SFEBC triggers focus exclusively on structural retrofit, this approach offers an opportunity to shift focus to the nonstructural systems that affect functional recovery and that tend to be damaged even in smaller events. New triggers could address critical nonstructural systems, perhaps including elevators, fire suppression systems, and exterior cladding, leaving the current code provisions to cover structural issues (in the rare cases in which those current triggers apply).

Another way to reduce the triggered scope is to focus on evaluation and disclosure, as opposed to actual retrofit. City of Portland, for example, supplements the Oregon building code with an alteration cost provision that triggers only an evaluation *(City of Portland, 2015, Section 24.85.060)* as follows :

> *When an alteration for which a building permit is required has a value (not including costs of mechanical, electrical, plumbing, permanent equipment, painting, fire extinguishing systems, site improvements, eco-roofs and finish works) of more than $175,000, an ASCE 41 evaluation is required. This value of $175,000 shall be modified each year after 2004 by the percent change in the R.S Means Construction Index for Portland on file with the Director. A letter of intent to have an ASCE 41 evaluation performed may be submitted along with the permit application. The evaluation must be completed before any future permits will be issued.*

The current Los Angeles program for non-ductile concrete structures offers a related precedent. The program mandates retrofit, but the retrofit deadline is 25 years after notification by the building department. In the near term, only evaluation is required, so the program functions as a mandatory evaluation only (City of Los Angeles, 2015, Chapter IX, Division 95).

In both Portland and Los Angeles, the intent is that evaluation findings will prompt owners to plan and execute their own voluntary retrofits, or at least take steps to reduce their risks. This is also the logic behind *Workplan 2012-2042* tasks that call for mandatory evaluation followed a few years later by mandatory retrofit. Many believe that the production of a formal evaluation report should motivate an owner by confirming liability for damage if a retrofit is not performed (Bonowitz, 2018).

The evaluation reports produced by these mandates and triggers are presumably public documents available to potential buyers, tenants, and the general public. If San Francisco were to develop an evaluation trigger, it could include a provision requiring disclosure to current and prospective tenants. Public disclosure is the purpose of placarding programs used by some jurisdictions for unreinforced masonry buildings and was a feature of a pioneering risk reduction program in Palo Alto (City of Palo Alto, 1986, Section 16.42.070).

Having and sharing more information regarding expected earthquake performance is beneficial to stakeholders and to the City in the long run, but the disclosure process between owners and tenants can be fraught, especially if one or both parties is unfamiliar with the technical issues. To help address these concerns, government agencies (including the State of California), institutions, and private organizations have developed various "rating systems" over the years. San Francisco has its own, a four-level Seismic Hazard Rating used by the Department of Public Works (Lui, 2018). For communicating with non-expert stakeholders, however, San Francisco would benefit from a rating system that uses a more pragmatic, non-technical terminology; separates safety, repair cost, and recovery time into separate ratings; accommodates multiple evaluation methodologies commonly used by engineers; and is available for free public use, allowing the City to specify it without a Request for Proposal (RFP) process (SEAONC BRC, 2015).

Other recommendations presented in this report can also work with codified triggers. San Francisco can write an alteration or change of use trigger to require development of a recovery plan or participation in the City's Building Occupancy Resumption Program (BORP,discussed further in Part 6). Either of these triggers would represent a recovery-focused extension of the triggered seismic evaluation. At the very least, the City can use a code trigger to require the building owner to file basic information to enhance the building database described in Part 1 and to inform the City's own response and recovery plans as discussed in Part 6.

Well-intentioned retrofit triggers can lead to unintended consequences when owners decide they would rather cancel a project than undertake the additional triggered work. This is especially likely in tall buildings where the triggered retrofit might be disproportionately costly and disruptive. A reduced scope can represent a balance of the City's and the owner's interests.

3.4 Acquisition Triggers

Codified retrofit triggers are meant to take advantage of the opportunity provided when an owner invests in a major alteration or change of occupancy. The intended project proves the building is a viable investment, and the work often exposes the structure while the building is at least partially vacant. As discussed above, however, the current SFEBC triggers simply do not catch many tall buildings. More buildings might be triggered for seismic work if San Francisco could also take advantage of the opportunity presented when a building is bought or when a major tenant starts or renews a lease. Task C.1.a of *Workplan 2012-2042*, "mandatory evaluation on sale or by deadline for all building types not otherwise covered," already contemplates a program of "mandatory evaluation on sale" for broad groups of San Francisco buildings.

Acquisition (by lease or sale) is not a project type within the CEBC or SFEBC for private buildings. Many government agencies and institutions, however, have internal policies for "seismic due diligence" that could be adopted to supplement the codes. Private lenders and insurers also rely on seismic evaluations at the time of sale, though their interests are different from those of the City as a regulator and recovery planner.

As with cost-based alteration triggers, acquisition triggers are most feasible when the regulating agency also has a stake in the building as the owner or lessee. Federal government agencies require a seismic evaluation when a building is "added to the Federal inventory through purchase or donation" (NIST, 2011, Section 2.1) and when an agency starts or renews a lease in a non-Federally owned building (NIST, 2011, Section 1.3.2). The University of California and California State University both require a demonstration of acceptable performance (using the State's seven-level rating system) for most leased facilities and for any building acquired by purchase or title transfer (University of California, 2017; CSU, 2016).

For private property, California law requires the disclosure of "earthquake fault zones" and mapped landslide and liquefaction hazards when real property is transferred (Civil Code Section 1103). These disclosures say nothing about expected performance of any structures on the property, however, and can thus be misleading. For the buildings themselves, California also requires the seller of a house built before 1960 (wood-frame construction, up to four units) to provide the buyer with a bare bones seismic evaluation in the form of a checklist (Government Code Section 8897.1). The program is widely seen as ineffective, however, since sellers are allowed (even encouraged) to check the "Don't Know" option, the report is not required for multi-family buildings, and owners are not required to share the information with tenants. In San Francisco, title transfer through condominium conversion is specifically exempt from seismic review or disclosure, even while a general building inspection and other corrective work is required.

For commercial buildings, California law addresses the transfer of concrete "tilt-up" and similar reinforced masonry buildings (typically only one or two stories tall), requiring the seller only to provide a copy of a Seismic Safety Commission document. No building-specific information is required, no retrofit is required, and the statue applies only to pre-1975 buildings, even though tilt-up design provisions continued to improve even after the 1994 Northridge earthquake. The law does provide an interesting negative incentive, however: any buyer who fails to retrofit within three years of the purchase "shall not receive payment from any state assistance program for earthquake repairs resulting from damage during an earthquake until all other applicants have been paid" (Government Code Section 8893).

For the residential and commercial properties addressed by California law, compliance is the responsibility of the seller or the seller's agent. Presumably, the compliance status could be noted on a title report, but enforcement by the building department is not expected. Thus, precedents for effective acquisition triggers exist, but they are more in institutional policy than in enforceable legislation or building code provisions.

If San Francisco were to create an acquisition trigger for private buildings, the process would probably require legislation, as opposed to just an administrative bulletin or building code amendment. In addition, implementation would require new coordination between the Department of Building Inspection, which would set criteria and approve the triggered evaluation reports, and the Office of Assessor-Controller, which would have to initiate the process based on recorded title transfers. It is not clear which City department would be responsible for triggering evaluations based on new leases. Possibly, as with the California statutes for residential and tilt-up transfers, compliance would be left to the private stakeholders themselves, and would be initiated by their realtors and attorneys without much City oversight.

The legislation would need to develop consensus on a number of questions. The legislative process will need to address the following issues, among others:

- Which buildings should be subject to the trigger? This study is about tall buildings, but if the City goes through the effort to create an acquisition trigger, it might consider a broader application. If a broad scope is politically infeasible, some of the scope narrowing and target focusing ideas in previous sections of this report might be helpful. Should relatively new, or recently retrofitted buildings be exempted or benchmarked?
- How should the sale trigger be set? For a residential building, should the conversion to condominiums count? Should the sale of a single condominium pull the trigger? Partial sales of large office buildings have become more common throughout the country, largely to avoid transfer taxes and, in California, property tax reassessment that applies when at least half the building is sold (Li, 2017; Morris, 2018). In 2016, San Francisco raised transfer taxes on sales over five million dollars, further encouraging partial sales, including several in well-known tall buildings (Littman, 2018; Li, 2018). An effective acquisition trigger would therefore need to be set below 49 percent, but should also be adjustable to account for future trends in the real estate market and likely adjustments to current tax law.
- How should the lease trigger be set? Leases are very common, so an effective acquisition trigger should apply to at least some of them as well. That said, many new leases come with tenant improvements and alterations, so this acquisition trigger will overlap to a degree with any revised alteration triggers in the SFEBC.
- What scope and criteria should be used for the triggered evaluation? Should it include nonstructural components and systems? Should it consider only safety, or should it consider prospects for reoccupancy and recovery?
- Who will be allowed to produce a triggered evaluation? Since there will be no design or construction in an acquisition-triggered seismic evaluation, will a licensed engineer be required? Should this depend on the scope and criteria selected? California's Board for Professional Engineers, Land Surveyors, and Geologists has generally not considered the production of seismic evaluations as within the practice of civil or structural engineering. For a tall building, even the design requires only a Civil Engineer, not a Structural Engineer, though SEAOC has proposed

legislation to require an SE for the design of buildings taller than just 45 feet (Schinske, 2015).

- What role should the Department of Building Inspection play? Should acquisition-triggered evaluations be subject to approval by DBI? If the requirement is in statue, but not in the building code, DBI's regulatory authority is less clear. A City approval will probably improve quality and compliance rates, but it will also increase costs. Owners will probably prefer the approach of the Government Code sections cited above, which leave compliance largely to the parties and their selected consultants.
- Should the triggered evaluations be public? As the fruit of a City policy, the reports would presumably be filed with the City and available for public review. Alternatively, if the ultimate purpose is only to raise the awareness of owners and tenants, leaving any actual risk reduction as voluntary, an argument might be made for requiring disclosure to current and prospective tenants but restricting access to the general public, as is the case for reports required by the Government Code sections cited above. See also the discussion of disclosure and communication issues in Section 2.2 of this Part.

3.4.1 The PML Market as an Alternative to Public Policy

Another alternative is to leverage an existing practice from the private sector. When a building is sold, the standard practice of most lenders is to call for an analysis of its "probable maximum loss," or PML. If the PML is too high, the lender will either decline the loan or insist that the buyer purchase earthquake insurance as a hedge against the lender's risk. PML analyses are common for commercial buildings but have been provided for some multi-family residential buildings as well. Any San Francisco tall building being sold in the near future would be expected to have a PML analysis performed as part of the transaction.

If the City could compel building owners to submit or make public their PML reports, or perhaps just to disclose them to tenants, that might meet some of the goals of a legislated acquisition trigger. Owners might even prefer this approach if it would mean they can preserve some confidentiality of the findings and keep the transaction process free of City approvals.

But there are also reasons why current PML practices might not serve the City's interests, at least not as a replacement for broader public policy. These include:

- Two ASTM standards (2007a; 2007b) define terminology and procedures but the practice is largely unregulated. Because PML analyses are produced to support private transactions, they do not receive peer review or building department approval. Even if the parties agree to abide by them, they do not provide quality assurance. Since no engineering design is involved, California licensure rules do not apply in a reliable way. As a result, PML evaluation scopes, methods, and conclusions are frequently inconsistent between evaluators (Searer et al., 2009).

- PML findings are used primarily to support financial decisions, which differ in their nature and intent from policy decisions focused on public safety, functional recovery, or community resilience. Where the financial decision is tied to a fixed PML value, reported results tend to cluster around that value, essentially saying "above" or "below" without providing more useful detail. Indeed, typical PML practice has become so rote that many do not even consider it especially valuable for its own stated purpose of properly valuing the risk (Porter et al., 2004).
- PML reports are traditionally considered private and confidential, so much so that owners have been known to commission the evaluations through their lawyers to preserve attorney client privilege and avoid disclosing the findings.
- PML analyses are not provided for lease transactions.

Thus, even if San Francisco would want to take advantage of the analyses already being performed in the PML market, it would have to establish its own standards that might interfere with current private sector practices.

3.5 Incentives for Voluntary Evaluation or Retrofit

Seismic risk reduction can be classified as mandatory, triggered, or voluntary. Practically all current work on San Francisco's tall buildings falls into the latter category. Voluntary work can sometimes be incentivized, but experience has shown that substantial benefits need to be provided to spur actual seismic retrofit. Some incentives that already exist—a waiver on property tax assessment (Revenue and Taxation Code Section 74.5), the 2017 CalCAP loan guarantee program (CPCFA, 2017), and even the growing availability of PACE financing—have had very little impact, especially on tall buildings. Thus, feasible incentives for seismic improvements to San Francisco tall building safety or recovery are likely to be less effective than focused mandates or triggers.

That said, dozens of San Francisco tall buildings already participate in the City's post-earthquake reoccupancy program, BORP, discussed in detail in Part 6. BORP can be considered an incentivized voluntary program that benefits both the City and the building's owner and tenants. In exchange for

City approval of a privatized and expedited safety inspection, a building owner agrees to develop a reoccupancy plan and often commissions a seismic evaluation as part of the process.

Chapter 4

Issues and Recommendations

4.1 Revisions and Amendments to the SFEBC

San Francisco supplements the CEBC with its own provisions that trigger seismic upgrade when a major alteration or a change of occupancy is proposed. Because of their size and typical uses, however, tall buildings are almost never affected by these San Francisco amendments. Therefore, even the most collapse-prone tall buildings only rarely receive the scrutiny intended by the code. That said, since the retrofit of an occupied tall building is especially expensive and disruptive, a more aggressive trigger provision could discourage modernization or tenant improvement. Thus, the *San Francisco Existing Building Code*'s generic provisions are problematic for tall and similarly large or complex facilities. Still, these provisions can be effective if they are made more aggressive, so that they apply to more alteration and change of occupancy projects, but if they are also made more focused, so they address the conditions of greatest concern in ways that building owners can afford.

Recommendation: Amend the *San Francisco Existing Building Code* provisions for alteration and change of occupancy projects.

- Lower the trigger levels, while also exempting buildings that do not present well-recognized safety or recovery risks. Identify these subject buildings in advance as particular combinations of occupancy or use, building size, structural attributes, building age, and other critical conditions.
- Lower the trigger levels, but for tall buildings, consider limiting the scope of triggered work to allow partial retrofits, to focus on nonstructural systems only, to require evaluation and disclosure only, or to require participation in BORP or the filing of a recovery plan.
- Amend the evaluation criteria in SFEBC Section 301.1.4.2, and develop an Administrative Bulletin as needed, to facilitate the use of FEMA 351, *Recommended Seismic Evaluation and Upgrade Criteria for Existing Welded Steel Moment-Frame Buildings* (FEMA, 2000), for eligible structures. The findings of the SEAONC EBC Steel Frame Subcommittee (2004) might be valuable in reviewing the options provided by FEMA 351.

- Update and amend the SFEBC to coordinate San Francisco amendments with existing and forthcoming CEBC provisions.
  - Revise the text that incorrectly says Section 301.2.1 applies only when invoked by Section 403.
  - Since Section 301.2.1 applies only to cases where reduced forces are used (alterations and repairs), consider combining with Section 301.1.4.2 (to be Section 303.3.2 in the 2019 SFEBC).
  - Delete the unnecessary pointer from Section 301.2.3 to Section 301.1.4.2. Pointers to Section 301.2.3 should instead point directly to Section 301.1.4.2.
  - Correct the reference in the first sentence of Section 403.4. It should refer to Section 403.9 regarding voluntary seismic improvements, not Section 403.5.
  - Correct the numbering of Sections 403.12.1 and 403.12.2.
  - Coordinate Sections 403.12.1 and 403.12.2 with new provisions on similar topics coming in the 2019 CEBC. In particular, confirm the intended use of the term "substantial structural alteration," which will have a different meaning in the 2019 CEBC.
- Clarify or revise limits on upgrade triggers imposed by the Department of Housing and Community Development (HCD).

4.2 New Legislation

To supplement the SFEBC's rarely applied alteration and change of occupancy triggers, the City could take advantage of opportunities that arise when a tall building is sold or a sizable portion of it is leased to a new tenant, as contemplated by the *CAPSS Earthquake Safety Implementation Program Workplan 2012-2042* (CCSF, 2011). Even if only a private evaluation and disclosure is triggered, that would at least ensure that buildings are properly valued with respect to the risks they pose to owners and tenants. Because acquisition is not a project type covered by the CEBC or SFEBC, and because only evaluation and disclosure would be triggered, an acquisition-based program would likely require legislation, and implementation would likely require coordination with the Office of Assessor-Recorder.

Recommendation: Develop a program for evaluation and disclosure triggered by sale or lease.

- With a working group or advisory panel, develop recommendations for applicability and triggering conditions considering changing trends in real estate practice and tax law.
- Develop recommendations for the technical scope and criteria for the triggered evaluation. For purposes of resilience and recovery planning, the triggered evaluation might be required to include an estimate of recovery time as well as safety.
- Develop recommendations regarding City approval, disclosure, and public posting of evaluation findings.

Appendix A

Annotated Building Code Provisions

This appendix presents the current building code provisions applicable to triggered pre-earthquake seismic evaluation and retrofit. The current, 2016, *San Francisco Existing Building Code* (SFEBC) amends the 2016 *California Existing Building Code* (CEBC), which itself adopts and amends the 2015 *International Existing Building Code* (IEBC). This appendix shows the relevant provisions from all three documents. In addition, it provides a commentary on the combined provisions and their application to existing tall buildings, and it offers notes on the forthcoming 2019 CEBC, which will adopt the 2018 IEBC.

This appendix focuses on earthquake evaluation and retrofit provisions that apply to pre-earthquake assessment of tall buildings associated with alteration or change of occupancy projects. As such, it does not include all existing building provisions or all tall building provisions. In particular, the following specialty provisions might apply to certain tall buildings but are omitted from this appendix for brevity:

- *Historical Building Code* provisions
- Provisions for state-owned buildings in CEBC Sections 317-322; Section 317.1.1 allows these sections to be adopted by local jurisdictions for use on private buildings
- Regulations used by OSHPD and DSA
- San Francisco amendments regarding buildings “on a military base selected for closure” (SFEBC Section 326) and homeless shelters (SFEBC Section 401.7), which are not likely to affect tall buildings

California amendments from the Department of Housing and Community Development (HCD) are included, since they are meant to cover apartment buildings, condominiums, and hotels that might be in tall buildings.

The text is formatted to distinguish between the various source documents using the following legend:

2015 IEBC. Model code. Plain text (except for section headings in bold).
2016 CEBC. California amendments, applicable in San Francisco. Italic plain text (except for section headings in bold).
2016 SFEBC. Additional San Francisco amendments. Bold italic text.

> *ATC Commentary. Plain italic text, indented.*
>
> 2018 IEBC. Model code language already published and expected to be adopted into the 2019 CEBC and SFEBC. Plain text, shaded and indented.

CHAPTER 1
SCOPE AND ADMINISTRATION

1.8 Department of Housing and Community Development

1.8.10 Other Building Regulations

1.8.10.1 Existing structures. *Notwithstanding other provisions of law, the replacement, retention, and extension of original materials and the use of original methods of construction for any existing building or accessory structure, or portions thereof, shall be permitted in accordance with the provisions of this code as adopted by the Department of Housing and Community Development. For additional information, see California Health and Safety Code, Sections 17912, 17920.3, 17922 and 17958.8.*

> *Commentary: This and similar California amendments for HCD are generally interpreted to prohibit upgrade triggers for residential buildings. See the commentary at Section 401.2.1.*

CHAPTER 2
DEFINITIONS

ADDITION. An extension or increase in floor area, number of stories, or height of a building or structure.

ALTERATION. Any construction or renovation to an existing structure other than a repair or addition. Alterations are classified as Level 1, Level 2 and Level 3.

Commentary: The 2018 IEBC will change this definition by removing the second sentence about alteration levels. This change has no substantive effect.

The 2018 IEBC (and presumably the 2019 CEBC) will also have a new definition of Substantial Structural Alteration, a term that was previously used only in the IEBC's Work Area method but will, in the new edition, be used also in the Prescriptive method adopted by California. See the discussion of 2018 IEBC Section 503.11 in the commentary to 2016 SFEBC Section 403.12.2. The new definition will read as follows:

SUBSTANTIAL STRUCTURAL ALTERATION. An alteration in which the gravity load-carrying structural elements altered within a 5-year period support more than 30 percent of the total floor and roof area of the building or structure. The areas to be counted toward the 30 percent shall include mezzanines, penthouses, and in-filled courts and shafts tributary to the altered structural elements.

CHANGE OF OCCUPANCY. A change in the use of the building or a portion of a building. A change of occupancy shall include any change of occupancy classification, any change from one group to another group within an occupancy classification or any change in use within a group for a specific occupancy classification.

Commentary: The 2018 IEBC will revise this definition, but the change should have no substantive effect.

SEISMIC REHABILITATION. Work conducted to improve the seismic lateral force resistance of an existing building.

Commentary: 2018 IEBC will remove this definition, which is neither used nor needed in the model code.

WORK AREA. That portion or portions of a building consisting of all reconfigured spaces as indicated on the construction documents. Work area excludes other portions of the building where incidental work entailed by the intended work must be performed and portions of the building where work not initially intended by the owner is specifically required by this code.

Commentary: This definition is not used by the 2016 CEBC. It will be used by the 2018 IEBC (and presumably the 2019 CEBC). See the *discussion of 2018 IEBC Section 503.11 in the commentary to 2016 SFEBC Section 403.12.2.*

CHAPTER 3
PROVISIONS FOR ALL COMPLIANCE METHODS

301.1 General. The repair, alteration, change of occupancy, addition or relocation of all existing buildings shall comply with one of the methods listed in Sections 301.1.1 through 301.1.3 as selected by the applicant. Sections 301.1.1 through 301.3.3 shall not be applied in combination with each other. Where this code requires consideration of the seismic force-resisting system of an existing building subject to repair, alteration, change of occupancy, addition or relocation of existing buildings, the seismic evaluation or design shall be based on Section 301.1.4 regardless of which compliance method is used.

Exception: Subject to the approval of the code official, alterations complying with the laws in existence at the time the building or the affected portion of the building was built shall be considered in compliance with the provisions of this code unless the building is undergoing more than a limited structural alteration as defined in Section 907.4.4. New structural members added as part of the alteration shall comply with the *California Building Code*. Alterations of existing buildings in floor hazard areas shall comply with Section 701.3.

Commentary: The 2016 CEBC references to different "methods" and to Sections 701 and 907, though unchanged relative to the 2015 IEBC model code, are inappropriate, as the CEBC uses only the Prescriptive Method of 2016 CEBC Chapter 4.

The 2018 IEBC will change this general scoping provision to account for new chapter organization and section numbering. It will also replace the exception's reference to "limited structural alteration" with the following:

This exception shall not apply to the structural provisions of Chapter 5 or to the structural provisions of Sections 706, 806 and 906.

With this change, the 2018 IEBC (and presumably, the 2019 CEBC and SFEBC) remove a discretionary waiver that might have applied to typical alteration projects. The code official will retain discretion to waive alteration-triggered requirements related to heights and areas, *egress, etc., but will no longer be able to waive the normal structural triggers.*

301.1.4 Seismic evaluation and design procedures. The seismic evaluation and design shall be based on the procedures specified in the *California Building Code* or ASCE 41. The procedures contained in Appendix A of this code shall be permitted to be used as specified in Section 301.1.4.2.

Commentary: In the 2018 IEBC, Section 301.1.4 has been edited for clarity and to coordinate with ASCE 41-17 instead of ASCE 41-13. The entire section has also been relocated and renumbered as Section 303.3.

The 2016 CEBC adopts only three of the 2015 IEBC's five Appendix A chapters. None of the three adopted chapters applies to tall buildings (or to steel or concrete structural systems of any size).

301.1.4.1 Compliance with International Building Code-level seismic forces. Where compliance with the seismic design provisions of the *California Building Code* is required, the criteria shall be in accordance with one of the following:

1. One-hundred percent of the values in the *California Building Code*. Where the existing seismic force-resisting system is a type that can be designated as "Ordinary," values of R, Ω_0 and C_d used for analysis in accordance with Chapter 16 of the *California Building Code* shall be those specified for structural systems classified as "Ordinary" in accordance with Table 12.2-1 of ASCE 7, unless it can be demonstrated that the structural system will provide performance equivalent to that of a "Detailed," "Intermediate" or "Special" system.

2. ASCE 41, using a Tier 3 procedure and the two-level performance objective in Table 301.1.4.1 for the applicable risk category.

Commentary: Section 301.1.4.1 gives the "full" earthquake design criteria applicable when seismic work is triggered by a change of occupancy (See Section 407). The criteria vary by risk category: For RC I or II: Structural Life Safety with a BSE-1N hazard and Collapse Prevention with a BSE-2N hazard; for RC III: Structural Damage Control with a BSE-1N hazard and Structural Limited Safety with a BSE-2N hazard. Tall buildings would not be expected to be assigned to Risk Category IV.

301.1.4.2 Compliance with reduced International Building Code-level seismic forces. Where seismic evaluation and design is permitted to meet reduced *California Building Code* seismic force levels, the criteria used shall be in accordance with one of the following:

1. The *California Building Code* using 75 percent of the prescribed forces. Values of R, Ω_0, and C_d used for analysis shall be as specified in Section 301.1.4.1 of this code.

2. Except where these requirements are triggered by Section 403.12, structures or portions of structures that comply with the requirements of the applicable chapter in Appendix A as specified in Items 2.1 through 2.5 and subject to the limitations of the respective Appendix A chapters shall be deemed to comply with this section.

...

2.5. Seismic evaluation and design of concrete buildings assigned to Risk Category I, II or III are permitted to be based on the procedures specified in Chapter A5.

3. ASCE 41, using the performance objective in Table 301.1.4.2 for the applicable risk category.

> *Commentary: Section 301.1.4.2 gives the "reduced" earthquake design criteria applicable when seismic work is triggered by certain alteration projects. See Section 403.*
>
> *The 2016 CEBC reference to Chapter A5 is inappropriate, as California does not adopt that chapter of the model code. Also, the 2018 IEBC (and presumably the 2019 CEBC) will no longer have Chapter A5; it was omitted because it is essentially identical to ASCE 41. As noted above, none of the other Appendix A chapters adopted by the CEBC is applicable to tall buildings or to concrete or steel structural systems of any size.*
>
> *Section 301.1.4.2 is referenced by SFEBC Section 301.2, which contains San Francisco's traditional reduced load criteria with benchmark dates. The CEBC allows three options where reduced loads are allowed. Presumably, all three options are acceptable within the intent of SFEBC Section 301.2.*
>
> - *Option 1 is the traditional "75 percent" approach long embraced by San Francisco.*

- *Option 2 allows five prescriptive approaches for specific building types. As noted above, however, California adopts only three of the five Appendix A chapters, none of which is applicable to tall buildings, and Chapter A5 (not adopted by the CEBC) has already been removed from the 2018 IEBC. Note also that SFEBC modifies Option 2 with an introductory phrase prohibiting its use on retrofits triggered by major alterations (SFEBC Section 403.12).*
- *Option 3 allows the use of ASCE 41-13 with performance objectives that vary by risk category: For RC I or II: Structural Life Safety with a BSE-1E hazard; for RC III: Structural Damage Control with a BSE-1E hazard. Tall buildings would not be expected to be assigned to Risk Category IV.*

In addition to ASCE 41, San Francisco should consider allowing the use of FEMA 351, Recommended Seismic Evaluation and Upgrade Criteria for Existing Welded Steel Moment-Frame Buildings, for eligible structures (FEMA, 2000). An Administrative Bulletin to guide its use will probably be needed.

301.2 Minimum Lateral Force for Existing Buildings

301.2.1 General. This section is applicable to existing buildings when invoked by SFEBC Section 403. This section may be used as a standard for voluntary upgrades.

Commentary: Section 301.2 gives additional criteria for seismic work triggered by alteration projects in Section 403. Despite the wording of Section 301.2.1, these provisions are also invoked for change of occupancy projects by SFEBC Section 407.4.1.

An existing building or structure which has been brought into compliance with the lateral force resistance requirements of the San Francisco Building Code in effect on or after the dates shown in Table 301.2.1 [Table 7-2 below]***, shall be deemed to comply with this section except when a vertical extension or other alterations are to be made which would increase the mass or reduce the seismic resistance capacity of the building or structure. Where multiple building types apply, the later applicable date shall be used. Where none of the building types apply, compliance shall be at the discretion of the Director. Building type definitions are given in ASCE 41-13, Table 3-1.***

Commentary: Starting with the 2016 SFEBC, this section includes benchmark dates for each structural system (as opposed to the prior provision, which gave a single date for all buildings). Where the benchmark table reprinted in Table 7-2 applies, it effectively eliminates the requirement for alteration-triggered seismic evaluation (and possible retrofit) in more recent buildings, some as old as 1995.

However, the waiver for recent structures does not apply to alterations where the intended work "would increase the mass or reduce the seismic resistance capacity." This exception distinguishes seismic work triggered by structurally significant alterations (already covered by IEBC/CEBC Section 403.4 and SFEBC Section 403.12.2) from architectural alterations covered by San Francisco's own trigger in SFEBC Section 403.12.1.

Table 7-2 Dates Required to Demonstrate Building Compliance (reproduced from Table 301.2.1 in Section 301.2 of SFEBC)

Building Type	Date of Compliance	Model Code (for reference)
Wood frame, wood shear panels (Types W1 & W2)	1/1/1984	UBC 1976
Wood frame, wood shear panels (Type W1A) Floor areas greater than 3,000 ft2 per level	7/1/1999	UBC 1997
Steel moment-resisting frame (Types S1 & S1a)	12/28/1995	UBC 1994
Steel concentrically braced frame (Types S2 & S2a)	7/1/1999	UBC 1997
Steel eccentrically braced frame (Types S2 & S2a)	1/1/1990	UBC 1988
Buckling-restrained braced frame (Types S2 &S2a)	1/1/2008	IBC 2006
Light metal frame (Type S3)	1/1/2008	IBC 2006
Steel frame w/ concrete shear walls (Type S4)	12/28/1995	UBC 1994
Steel plate shear wall (Type S6)	1/1/2008	IBC 2006
Reinforced concrete moment-resisting frame (Type C1)	12/28/1995	UBC 1994
Reinforced concrete shear walls (Types C2 & C2a)	12/28/1995	UBC 1994
Tilt-up concrete (Types PC1 & PC1a)	7/1/1999	UBC 1997
Precast concrete frame (Types PC2 & PC2a)	1/1/2008	IBC 2006
Reinforced masonry (Type RM1) Flexible diaphragms	7/1/1999	UBC 1997
Reinforced masonry (Type RM2) Stiff diaphragms	12/28/1995	UBC 1994
Seismic isolation or passive dissipation	7/1/1992	UBC 1991

301.2.2 Wind forces. Buildings and structures shall be capable of resisting wind forces as prescribed in San Francisco Building Code Section 1609.

301.2.3 Seismic forces. Buildings and structures shall comply with the reduced International Building Code-level seismic forces, as defined in Section 301.1.4.2. The building separation limitations of Section ASCE 7-10 Section 12.12.3 need not be considered.

When upper floors are exempted from compliance by Section 403.12.2, the lateral forces generated by their masses shall be included in the analysis and design of the lateral force resisting systems for the strengthened floor. Such forces may be applied to the floor level immediately above the topmost strengthened floor and distributed in that floor in a manner consistent with the construction and layout of the exempted floor.

> *Commentary: Section 301.2.3 gives the "reduced" earthquake design criteria for use when a seismic upgrade is triggered by certain alteration or repair projects. The first part of this provision is not actually needed; as shown, it merely points to criteria already provided in CEBC Section 301.1.4.2. This sentence remains, however, as a vestige of prior SFEBC provisions for San Francisco's own alteration triggers (SFEBC Sections 403.12.1 and 403.12.2) – triggers the IEBC/CEBC does not have.*
>
> *In addition to setting the force level, Section 301.2 provides benchmarking (Section 301.2.1) and waives building separation requirements (Section 301.2.3); these provisions are unique and traditional to San Francisco. As such, they are perhaps meant to apply only to work triggered by San Francisco amendments, but the way the code is written, they would apply to any project subject to a code section that invokes Section 301.2.*

CHAPTER 4
PRESCRIPTIVE COMPLIANCE METHOD

401.1.2 Existing state-owned structures. *[BSC] ... The provisions of Sections 317 through 322 may be adopted by a local jurisdiction for earthquake evaluation and design for retrofit of existing buildings.*

...

401.2.1 Existing materials. ... ***[HCD 1]*** *Local ordinances or regulations shall permit the replacement, retention and extension of original materials, and the use of original methods of construction [if the] structure complied with the building code provisions in effect at the time of original construction and ... does not become or continue to be a substandard building. ...*

Commentary: This and similar California amendments for HCD are generally interpreted to prohibit upgrade triggers for residential buildings. However, the final phrase of the provision, regarding "substandard building," alters that general interpretation with respect to earthquake design. "Substandard building" is defined in Health and Safety Code Section 17920.3 to include any residential building with "inadequate structural resistance to horizontal forces."

Model "soft story" provisions proactively classify targeted buildings as substandard to avoid conflict over the HCD exception. SFEBC could do the same for other buildings, including tall residential buildings, that fail a triggered evaluation. This would be a helpful policy clarification as well as a reasonable interpretation.

SECTION 403
ALTERATIONS

403.1 General. Except as provided by Section 401.2 or this section, alterations to any building or structure shall comply with the requirements of the *California Building Code* or *California Residential Code*, as applicable, for new construction. Alterations shall be such that the existing building or structure is no less conforming to the provisions of the *California Building Code* or *California Residential Code*, as applicable, than the existing building or structure was prior to the alteration. ...

403.1.1 Replacement, retention and extension of original materials. [HCD 1] *Local ordinances or regulations shall permit the replacement, retention and extension of original materials, and the use of original methods of construction [if the] structure complied with the building code provisions in effect at the time of original construction and ... does not become or continue to be a substandard building. ...*

Commentary: Section 401.2 allows existing materials to remain unless dangerous. The HCD 1 provision added as Section 403.1.1 is consistent with the amendment to 401.2.1. See the commentary there.

403.4 Existing structural elements carrying lateral load. Except as permitted by Section 403.5, where the alteration increases design lateral loads in accordance with Section 1609 or 1613 of the *California Building Code*, or where the alteration results in a prohibited structural irregularity as defined in ASCE 7, or where the alteration decreases the capacity of any existing lateral load-carrying structural element, the structure of the altered building or structure shall be shown to meet the requirements of Sections 1609 and 1613 of the *California Building Code*. For purposes of this section, compliance with ASCE 41, using a Tier 3 procedure and the two-level performance objective in Table 301.1.4.1 for the applicable risk category, shall be deemed to meet the requirements of Section 1613 of the *California Building Code*.

Exception: Any existing lateral load-carrying structural element whose demand-capacity ratio with the alteration considered is no more than 10 percent greater than its demand-capacity ratio with the alteration ignored shall be permitted to remain unaltered. For purposes of calculating demand-capacity ratios, the demand shall consider applicable load combinations with design lateral loads or forces in accordance with Sections 1609 and 1613 of the *California Building Code*. For purposes of this exception, comparisons of demand-capacity ratios and calculation of design lateral loads, forces and capacities shall account for the cumulative effects of additions and alterations since original construction.

Commentary: This is the key provision for alteration-triggered seismic evaluation and upgrade. It sets three triggers by which the intended alteration is to be measured

- *Increase in lateral load, perhaps due to added mass,*
- *Creation of a prohibited irregularity, perhaps due to added mass or alteration of diaphragms, and*
- *Decrease in lateral capacity, perhaps due to removal of columns or wall elements.*

If a trigger is pulled, the triggered scope involves the entire structure, considering both wind and seismic loads. The exception typically reduces the triggered scope; if the intended alteration (a tenant improvement or mechanical upgrade, for example) does not touch the structure and adds little mass, the scope will likely be reduced to the point where no seismic work is required at all. In these common cases, it can be quickly shown that the change in demand-capacity ratio for any element of the lateral force-resisting system will not be close to 10 percent. Thus, the addition of mass triggers a review of the entire structure, but the exception then exempts every element, one by one.

The intent of this trigger, and its exception, is not to identify seismically deficient buildings and force them to be retrofitted. Rather, it is to identify alteration projects (or the accumulation of *alteration projects, as noted in the last sentence of the exception) that make a significant change to the building's expected seismic performance. By focusing on the effect of the intended alteration – not on the adequacy of the pre-alteration structure – the provision attempts to strike a balance between encouraging modernization and adaptive reuse of existing buildings and reduction of unacceptable risks. The result, well understood by the code writers, is that a highly deficient or even a collapse prone building would not trigger a seismic evaluation or retrofit if the intended alteration is small.*

This basic trigger is supplemented in the SFEBC by a San Francisco provision for explicitly structural alterations to which the "10 percent" exception does not apply. See Section 403.12.2.

For any part of the structure not exempt by the exception, the provision sets the criteria by reference to Table 301.1.4.1, requiring "full" seismic loads. SFEBC Section 301.2, which allows reduced loads and benchmarking waivers, does not apply to work triggered by Section 403.4 because of the exception in Section 301.2 for triggers based on added mass or reduced lateral capacity. However, the San Francisco building official should still be able to allow the use of reduced seismic loads by discretion, referencing any of three alternative standards:

- *In the 2018 IEBC, the section corresponding to 2016 CEBC Section 403.4 has been changed to allow reduced seismic loads for alteration-triggered seismic work.*
- *The IEBC model code on which the CEBC is based has three methods. For simplicity and consistency with past practice, California adopts only the Prescriptive method, but the IEBC's Work Area method allows the use of reduced seismic loads for alteration-triggered seismic work. The previous edition of the CEBC included a direct reference to the IEBC as an approved alternative, effectively allowing the use of reduced seismic loads. The 2016 CEBC does not have that explicit allowance of the Work Area method, but the Building Standards Commission has stated that in shifting from CBC Chapter 34 to the CEBC there was no intent to change the substance of any requirement or allowance. Therefore, the criteria of the IEBC Work Area method can be reasonably cited as acceptable alternative criteria here.*
- *CEBC/SFEBC Section 401.1.2 allows the jurisdiction to adopt the provisions for state-owned buildings in CEBC Sections 317 – 322.*

Those provisions allow the use of reduced seismic loads for alteration-triggered seismic work.

The first sentence of CEBC Section 403.4 contains an error. It should refer to Section 403.9, which covers voluntary seismic improvements, not Section 403.5.

2016 CEBC Sections 403.4.2, 403.5, 403.6, and 403.7 trigger seismic evaluation or retrofit for Risk Category IV facilities, unreinforced masonry parapets, and unreinforced masonry walls in cases of certain alterations. These provisions are not shown here because they are unlikely to apply to tall buildings.

***403.4.1 Existing structural elements carrying lateral load. [HCD]** ...*

Commentary: Section 403.4.1, for residential buildings regulated by HCD, is the same provision and exception as Section 403.4, but without the reference to Section 403.5 (perhaps because HCD did not recognize the section numbering erratum) and without the allowance to use ASCE 41.

403.9 Voluntary seismic improvements. Alterations to existing structural elements or additions of new structural elements that are not otherwise required by this chapter and are initiated for the purpose of improving the performance of the seismic force-resisting system of an existing structure or the performance of seismic bracing or anchorage of existing nonstructural elements shall be permitted, provided that an engineering analysis is submitted demonstrating the following:

1. The altered structure and the altered nonstructural elements are no less conforming to the provisions of the *California Building Code* with respect to earthquake design than they were prior to the alteration.

2. New structural elements are detailed as required for new construction.
3. New or relocated nonstructural elements are detailed and connected to existing or new structural elements as required for new construction.

4. The alterations do not create a structural irregularity as defined in ASCE 7 or make an existing structural irregularity more severe.

Commentary: This provision has been significantly revised for the 2018 IEBC, but its basic intent is unchanged. As noted in the first sentence of Section 403.4 (which should reference Section 403.9, not

Section 403.5), the normal triggers and the "10 percent" exception do not apply to voluntary work done exclusively to improve seismic performance.

403.12.1 Non-structural alterations. Whenever alteration work in a building or structure involves substantial changes to elements such as walls, partitions or ceilings, on 2/3 or more of the number of stories excluding basements, the building or structure as a whole shall comply with Section 301.2. The term "substantial change" includes the addition, removal, repair or modification of such elements. All such work included in alteration permits issued within two years of the date of a permit application shall be included in the determination of whether the application is proposing substantial change to the building or structure. ...

Commentary: This San Francisco provision supplements the basic alteration trigger of Section 403.4. (The SFEBC section numbering is inappropriate, because CEBC Section 403.12 addresses an unrelated topic, but this does not affect the substance of the provision.)

Unlike that basic provision, Section 403.12.1 has no "10 percent" exception; once the trigger is pulled based on the extent of non-structural work done over any two-year period, a seismic evaluation and a retrofit of any elements found deficient must be performed. The evaluation and retrofit criteria are the San Francisco criteria in Section 301.2, including the waivers for benchmarked buildings.

In practice, there has always been some code official judgment involved in the counting of altered spaces toward the "2/3" trigger. More significant for tall buildings, however, is the recognition that in an occupied building, two-thirds of the stories will almost never be altered by typical upgrades and tenant improvements within the course of two years.

403.12.2 Structural alterations. When more than 30 percent of the floor and roof areas of the building or structure have been or are proposed to be involved in substantial structural alteration, the building or structure shall comply with Section 301. The areas to be counted towards the 30 percent shall be those areas tributary to the vertical load carrying components (joists, beams, columns, walls and other structural components) that have been or will be removed, added or altered, as well as areas such as mezzanines, penthouses, roof structures and infilled courts and shafts.

Exceptions:
1. When such alterations involve only the lowest story of a wood frame building ...
2. When such alterations involve the lowest story of a Type V building or structure of R3 occupancy ...

Commentary: Like Section 403.12.1, this San Francisco provision supplements the basic alteration trigger of Section 403.4. (And as with Section 403.12.1, the SFEBC section numbering is inappropriate, but has no substantive effect.) Two exceptions to Section 403.12.2 apply only to wood-frame structures.

Like Section 403.12.1, Section 403.12.2 has no "10 percent" exception; once the trigger is pulled based on the extent of structural work intended by any one project, a seismic evaluation and a retrofit of any deficient elements must be performed. The evaluation and retrofit criteria are the San Francisco criteria in Section 301.2, including the waivers for benchmarked buildings. That said, it seems clear that this provision is not intended to apply to voluntary seismic work covered by Section 403.9; as such, this provision would be clearer if it included a "but for" clause at the top, as Section 403.4 does.

Even though the intent of the provision is to trigger lateral system evaluation and retrofit, the trigger is based on intended alteration to the gravity load-carrying elements, not to the existing lateral system elements. As in Section 403.4, the trigger is meant to measure the extent of the intended work, not the adequacy of the existing lateral system.

Section 403.12.2 uses the term "substantial structural alteration," which is not defined in the 2016 CEBC. Rather, the provision appears to be self-referential. Its context suggests that a "substantial" structural alteration is simply one where the area tributary to the altered structural elements exceeds the 30 percent threshold. In fact, this understanding is now confirmed by a definition in the 2018 IEBC, as described in the commentary to the definition of Alteration, above. The 2018 IEBC definition is close to the wording of this SFEBC provision, but it adds a five-year timeframe to require consideration of cumulative effects. Thus, while SFEBC Section 403.12.2 would apply only to a single project or permit application, the 2018 IEBC would consider any work done in the previous five years. The intent is to *prevent a project from avoiding the trigger by splitting the work into multiple, but near simultaneous, permits.*

The 2018 IEBC defines substantial structural alteration in order to use it in a new trigger provision similar to SFEBC Section 403.12.2. 2018 IEBC Section 503.11 will read as follows:

503.11 Substantial structural alteration. Where the work area exceeds 50 percent of the building area and where work involves a substantial structural alteration, the lateral load-resisting system of the altered building shall satisfy the requirements of Sections 1609 and 1613 of the International Building Code. Reduced seismic forces shall be permitted.

Exceptions:

1. Buildings of Group R occupancy with not more than five dwelling or sleeping units used solely for residential purposes that are altered based on the conventional light-frame construction methods of the International Building Code or in compliance with the provisions of the International Residential Code.
2. Where the intended alteration involves only the lowest story of a building, only the lateral load-resisting components in and below that story need comply with this section.

This coming model code provision is quite close to San Francisco's own current provision. The intent is to expand the basic alteration trigger in current CEBC Section 403.4 to acknowledge projects with intended structural work. Section 403.4 suggests the code's intent that when the intended project would not affect the structure, a wind or seismic upgrade is not justified. This additional provision says that when the intended work already involves substantial structural work, that is the time to look at the lateral system as well. That said, there are some differences between the SFEBC provision and the coming IEBC/CEBC provision that will need to be worked out as San Francisco adopts its next code:

- *The exceptions are different. In particular, the second IEBC exception would apply to all buildings with ground floor-only alterations, not only to woodframe structures.*
- *By citing the new definition of substantial structural alteration, the IEBC provision brings in a five-year timeframe for tracking cumulative effects that is not in the original SFEBC provision. Interestingly, however, because the SFEBC provision also uses the term "substantial structural alteration," it too could inadvertently be invoking the new definition.*

- *Most important, the IEBC provision applies only "where the work area exceeds 50 percent of the building area," while the SFEBC has no such limitation. Thus, the IEBC version of this structural trigger still only applies where the building is undergoing an extensive (and typically non-structural) alteration project – one that would be unlikely in an occupied tall building. The IEBC's work area wording reflects the intent of the new provision to match an existing provision in the IEBC's Work Area method.*

SECTION 407
CHANGE OF OCCUPANCY

407.1 Conformance. No change shall be made in the use or occupancy of any building unless such building is made to comply with the requirements of the *California Building Code* for the use or occupancy. ...

Exception: The building need not be made to comply with the seismic requirements for a new structure unless required by Section 407.4.

407.4 Structural. When a change of occupancy results in a structure being reclassified to a higher risk category, the structure shall conform to the seismic requirements for a new structure of the higher risk category. For purposes of this section, compliance with ASCE 41, using a Tier 3 procedure and the two-level performance objective in Table 301.1.4.1 for the applicable risk category, shall be deemed to meet the requirements of Section 1613 of the *California Building Code*.

Exceptions:
1. Specific seismic detailing requirements of Section 1613 of the *California Building Code* for a new structure shall not be required to be met where the seismic performance is shown to be equivalent to that of a new structure. A demonstration of equivalence shall consider the regularity, overstrength, redundancy and ductility of the structure.

2. Where a change of use results in a structure being reclassified from Risk Category I or II to Risk Category III and the structure is located where the seismic coefficient, SDS [sic], is less than 0.33, compliance with the seismic requirements of Section 1613 of the *California Building Code* is not required.

3. [BSC] For state-owned buildings ...

Commentary: This is the key provision for change of occupancy-triggered seismic evaluation and upgrade. The trigger is straightforward: Any change of occupancy that involves an upward change of Risk Category triggers a full-building seismic retrofit as needed to match the requirements for a new structure of the higher risk category. Essentially all existing tall buildings in San Francisco are assigned to Risk Category II, and an existing tall building is highly unlikely to undergo a change of use or occupancy that would move the building to Risk Category III or IV.

Unlike the alteration trigger in Section 403.4, there is no "10 percent exception," and "reduced" loads are not allowed. Most changes of use or occupancy that would pull this trigger would be accompanied by architectural changes that must be treated as a simultaneous alteration project. Thus, even if the simultaneous alteration does not trigger seismic work, the change of occupancy or occupant load might.

Exception 1 makes an allowance for obsolete detailing. However, the provision uses undefined terms and is thus largely unenforceable. For this reason, Exception 1 has been removed from the 2018 IEBC. Also, ASCE 41 explicitly accounts for obsolete detailing, so Exception 1 is not necessary in any case as long as ASCE 41 is allowed. Exception 2 does not apply in San Francisco. Exception 3 does not apply to San Francisco buildings regulated by the SFEBC.

The 2018 IEBC will add an exception for changes of occupancy that affect only small areas of a building. For San Francisco tall buildings, this new exception might apply in the rare cases of a school or residential care facility becoming a new tenant in an existing tall building. For example, if two floors of a 25-story residential building are converted to a care facility (Occupancy I-2), the building would be reassigned to Risk Category III but would still be exempt from seismic retrofit. 2018 IEBC Section 506.4.3, Exception 1, will read as follows:

Where the area of the new occupancy is less than 10 percent of the building area and the new occupancy is not assigned to Risk Category IV, compliance with this section is not required. The cumulative effect of occupancy changes over time shall be considered.

407.4.1 Structural. [HCD] *...*

> *Commentary: Section 407.4.1, for residential buildings regulated by HCD, is the same provision and exceptions as Section 407.4, but without the allowance to use ASCE 41 and without Exception 3.*

407.4.1 Change of occupancy. In addition to the other requirements of this code, the term "comply with the requirements of this code for such division or group of occupancy," as used in this section, shall also mean compliance with the lateral force provisions of Section 301.2 when the change results in an increase of more than 10 percent in the occupant load of the entire building or structure, and which also increases the occupant load by more than 100 persons as compared to the occupant load of the existing legal use or the use for which the building was originally designed. A building changing occupancy to an E occupancy, and is otherwise subject to Section 329, shall comply with Section 329.

Exceptions:

1. When a change of occupancy or use involves only one story of a building or structure, only the lateral force resisting elements in that story and all lateral force resisting elements below need comply with Section 301.2.

2. A change from a Group R, Division 3 to a Group R, Division 1 or Division 2 Occupancy *...*

> *Commentary: This San Francisco provision supplements the basic change of occupancy trigger of Section 407.4. (The SFEBC section numbering is inappropriate, because the CEBC already has a Section 407.4.1, as shown, but this does not affect the substance of the provision.)*
>
> *The first sentence of this San Francisco provision appears to reference the first sentence of CEBC Section 407.1, though the wording does not match exactly. The intent is to trigger seismic evaluation (and possibly retrofit) with a significant increase in occupant load even if the risk category does not change. Indeed, depending on how one reads the term "the change results," one could read the provision to apply even if there is no change of use or occupancy, but merely a higher occupant load. In any case, this*

provision represents a more conservative and more likely trigger than the basic provision in CEBC Section 407.4.

If the two-part trigger is pulled, the provision allows the reduced loads and benchmark dates of SFEBC Section 301.2. Thus, even if this provision is more conservative in intent than the basic provision in Section 407.4, it is less conservative and less onerous in application. This San Francisco provision also has two exceptions that do not apply to the CEBC provision, but neither is likely to apply to a triggered tall building.

Appendix B

Seismic Retrofit as an Alteration

Seismic retrofit is a type of alteration project, but it is not the intent of the code that any retrofit should trigger a different or additional retrofit. Nevertheless, a recent survey by the Structural Engineers Association of California has revealed that the provisions of the *California Existing Building Code* (CEBC; CBSC, 2016) are still commonly misunderstood and misapplied in ways that affect retrofit projects (Zepeda et al., 2018). Partly the misunderstanding is due to a typo in the CEBC (see the commentary to Section 403.4 in Appendix A of this Part), but partly it is due to the history of inconsistent local practices around the state. In any case, it is useful to clarify the code's intent as part of this review of current San Francisco policy.

Seismic retrofits can be classified as triggered, voluntary, or mandatory. Triggered retrofits should be straightforward. Even if there are questions about how to apply the "10% rule" or San Francisco's own "2/3 rule," it should be clear that any retrofit triggered by the code is a self-contained structural project. A triggered retrofit might in turn trigger other work (such as accessibility or energy conservation improvements), and of course it cannot be allowed to render another part of the structure noncompliant. But it is not the intent of the code that a local retrofit in one part of the structure (for example, one or two elements that miss the "10%" exception) should then be considered a structural alteration that leads to a full-structure retrofit.

Might a retrofit triggered by addition, repair, or change of occupancy be considered an alteration that triggers additional retrofit? In concept, yes. But in practice, a retrofit triggered by one of the other project types would already exceed what the alteration provisions might ever require, so the question becomes moot.

Voluntary seismic retrofit is also an alteration, and many voluntary retrofits use reduced criteria or address only certain critical deficiencies. But the code explicitly prevents these alterations from triggering additional seismic work; the first sentence of CEBC Section 403.4 says plainly that it does not apply to voluntary seismic improvements, which are regulated only by Section 403.9. (The current CEBC references the wrong section number, but this error should be corrected in the 2019 edition.) Thus, neither the triggers in Section

403.4 nor the “10% rule,” which is an exception to Section 403.4, apply to voluntary retrofit.

Mandatory retrofit is an alteration project as well. Like voluntary retrofits, many mandatory retrofits are partial and could, in concept, pull the code’s alteration trigger. Unlike voluntary retrofits, however, there is no code provision that explicitly protects the intended project from additional triggered work. Therefore, the clearest way to avoid confusion and scope creep is for the mandating ordinance to include a provision that explicitly waives CEBC Section 403.4.

Appendix C

References

Anagnos, T., Comerio, M., and Stewart, J., 2016, "Earthquake loss estimates and policy implications for nonductile concrete buildings in Los Angeles," *Earthquake Spectra*, Vol. 32, No. 4.

ASCE, 2017, *Seismic Evaluation and Retrofit of Existing Buildings*, ASCE/SEI 41-17, American Society of Civil Engineers Structural Engineering Institute, Reston, Virginia.

ASTM, 2007a, *Standard Guide for Seismic Risk Assessment of Buildings,* ASTM E2026-07, ASTM International, West Conshohocken, Pennsylvania.

ASTM, 2007b, *Standard Practice for Probable Maximum Loss (PML) Evaluations for Earthquake Due-Diligence Assessments*, ASTM E2557-07, ASTM International, West Conshohocken, Pennsylvania.

ATC, 2018, *Recommended Earthquake Performance Goals for San Francisco's Buildings (ESIP Task A.6.h)*, final report submitted to the City and County of San Francisco prepared by the Applied Technology Council, Redwood City, California.

Bonowitz, D., 2018, "Theories of owner liability for earthquake losses: A debate by lawyers for non-lawyers," *Eleventh U.S. National Conference on Earthquake Engineering: Integrating Science, Engineering & Policy*, June 25-29, Los Angeles, California.

Bonowitz, D., McCormick, D., Somers, P., 2014. "Alterations and seismic upgrade: The building code as mitigation policy," Earthquake Engineering Research Institute, November 19, Oakland, California.

CBSC, 2016, *2016 California Existing Building Code*, California Code of Regulations, Title 24, Part 10, California Building Standards Commission, Sacramento, California.

CCSF, 2011, *CAPSS Earthquake Safety Implementation Program Workplan 2012-2042*, the City and County of San Francisco, California.

CCSF, 2016, *The San Francisco Existing Building Code*, the City and County of San Francisco, California.

City of Los Angeles, 2015, *City of Los Angeles Municipal Code*, Los Angeles, California.

City of Palo Alto, 1986, *Palo Alto Municipal Code*, Palo Alto, California.

City of Portland, 2015, *Portland City Code and Charter*, Portland, Oregon.

City of Seattle, 2015a, *2015 Seattle Existing Building Code*, Seattle Department of Construction and Inspections, Seattle, Washington.

City of Seattle, 2015b, *Seattle Building Code Requirements for Existing Buildings that Undergo Substantial Alterations [Tip 314]*, Seattle Department of Construction and Inspections, March 23, Seattle, Washington.

CPCFA, 2017, *California Capital Access Program (CalCAP) Seismic Safety Financing Program,* Office of the State Treasurer, California Pollution Control Financing Authority, Sacramento, California.

CSU, 2016, *CSU Seismic Requirements*, California State University, Office of the Chancellor, November 1, Long Beach, California.

FEMA, 2000, *Recommended Seismic Evaluation and Upgrade Criteria for Existing Welded Steel Moment-Frame Buildings*, FEMA 351, prepared by the Structural Engineers Association of California, Applied Technology Council, and Consortium of Universities for Research in Earthquake Engineering (SAC) Joint Venture for the Federal Emergency Management Agency, Washington, D.C.

Hoover, C., 1992, *Seismic Retrofit Policies: An Evaluation of Local Practices in Zone 4 and their Application To Zone 3*, Earthquake Engineering Research Institute, Oakland, California.

ICC, 2016, *2016 Group B Committee Action Hearings*, International Code Council, April, Louisville, Kentucky.

ICC, 2018, *International Existing Building Code,* International Code Council, Country Club Hills, Illinois.

Li, R., 2017, “By feasting on S.F. ‘partial stakes,’ big real estate investors are saving millions in taxes,” *San Francisco Business Times*, October 26, San Francisco, California.

Li, R., 2018, “SF tower leased to Facebook could set price record in sale.” *San Francisco Chronicle*, September 10, San Francisco, California.

Littman, J., 2018, “Partial stake in transamerica pyramid for sale,” Bisnow.com, January 24.

Lui, R., 2018, *SHR and City-BORP Update*, January 22, San Francisco, California.

Morris, K., 2018, "Why sell a billion-dollar building when you can unload a piece of it instead?" *The Wall Street Journal*, March 11, New York, New York.

NIST, 2011, *Standards of Seismic Safety for Existing Federally Owned and Leased Buildings: ICSSC Recommended Practice 8 (RP 8)*, NIST GCR 11-917-12 report, prepared by the Building Seismic Safety Commission for the National Institute of Standards and Technology, Gaithersburg, Maryland.

Porter, K., Beck, J., and Shaikhutdinov, R., 2004. "Simplified estimation of economic seismic risk for buildings," *Proceedings of the 13th World Conference on Earthquake Engineering*, August 1-6, Vancouver, British Columbia, Canada.

Schinske, D., 2015, "SEAOC Stumps for Significant Structures," seaoc.org/news, March 24.

SEAONC BRC, 2015, *Earthquake Performance Rating System User's Guide*, Structural Engineers Association of Northern California, Building Ratings Committee, San Francisco, California.

SEAONC EBC Steel Frame Subcommittee, 2004, "Pre-earthquake evaluation of welded steel moment frames: Case studies using FEMA 351," in *SEAOC 2004 Convention Proceedings*, Structural Engineers Association of California, August 25-28, Monterey, California.

Searer, G., Paret, T., Kehoe, B., and Hachem, M., 2009, "The problems with PMLs," *Proceedings of the 2009 ATC & SEI Conference on Improving the Seismic Performance of Existing Buildings and Other Structures*, December 9-11, San Francisco, California.

University of California, 2017, *Seismic Safety Policy*, University of California, Office of Capital Asset Strategies and Finance, May 22.

Zepeda, D., Hagen, G., O'Connell, K., McLellan, R., Buckalew, J., and Sumer, A., 2018, "Existing buildings and the '10% rule': Are we in agreement?" *Proceedings of the 2018 SEAOC Convention*, Structural Engineers Association of California, Sacramento, California.

Project Participants

City and County of San Francisco Office of Resilience and Capital Planning

Brian Strong
Chief Resilience Officer and Director
Office Resilience and Capital Planning
1 Dr. Carlton B. Goodlett Place
City Hall, Room 347
San Francisco, California 94102

Danielle Mieler (Project Manager)
Principal Resilience Analyst
Office Resilience and Capital Planning
1 Dr. Carlton B. Goodlett Place
City Hall, Room 347
San Francisco, California 94102

Applied Technology Council

Ayse Hortacsu
Applied Technology Council
201 Redwood Shores Parkway, Suite 240
Redwood City, California 94065

Justin Moresco
Applied Technology Council
201 Redwood Shores Parkway, Suite 240
Redwood City, California 94065

Project Technical Committee

John D. Hooper (Task Leader)
Magnusson Klemencic Associates
1301 Fifth Avenue, Suite 3200
Seattle, Washington 98101

David Bonowitz
605A Baker Street
San Francisco, California 94117

Gregory Deierlein
Department of Civil and Environmental Engineering
Blume Earthquake Engineering Center
Stanford University
Stanford, California 94305

Shahriar Vahdani
Applied GeoDynamics, Inc.
1205 Contra Costa Drive
El Cerrito, California 94530

Project Working Groups

Carlos Molina Hutt
Department of Civil Engineering
The University of British Columbia
Vancouver, British Columbia
V6T 1Z4 Canada

Preetish Kakoty
Department of Civil Engineering
The University of British Columbia
Vancouver, British Columbia
V6T 1Z4 Canada

Anne McLeod Hulsey
Department of Civil and Environmental Engineering
Stanford University
Stanford, California 94305

Alireza Eksir Monfared
Department of Civil Engineering
The University of British Columbia
Vancouver, British Columbia
V6T 1Z4 Canada

Max Rattie
Magnusson Klemencic Associates
1301 Fifth Avenue, Suite 3200
Seattle, Washington 98101

Wen-Yi Yen
Department of Civil and Environmental Engineering
Stanford University
Stanford, California 94305

Task Review Panel

Mark X. Haley (reviewer for Part 2)
Haley & Aldrich, Inc.
465 Medford Street, Suite 2200
Boston, Massachusetts 02129

William Walton (reviewer for Part 2)
GEI Consultants
120 West Madison
Suite 1305
Chicago, Illinois 60602

City and County of San Francisco Tall Buildings Executive Panel

Naomi Kelly (Chair)
City Administrator
Office of the City Administrator
1 Dr. Carlton B. Goodlett Place
City Hall, Room 362
San Francisco, California 94102

Mary Ellen Carroll
Executive Director
Department of Emergency Management
1011 Turk Street
City Hall, Room 362
San Francisco, California 94102

Kathryn How
Chief Engineer, Assistant General Manager of Infrastructure
Public Utilities Commission
525 Golden Gate Avenue
San Francisco, California 94102

Tom Hui
Director
Department of Building Inspection
1660 Mission Street
San Francisco, California 94102

Brian Strong
Chief Resilience Officer and Director
Office Resilience and Capital Planning
1 Dr. Carlton B. Goodlett Place
City Hall, Room 347
San Francisco, California 94102

City and County of San Francisco Tall Buildings Stakeholders

Board of Supervisors
- District 3
- District 6

Developers
- San Francisco Apartment Owners Association
- Tishman Speyer
- Boston Properties

Private Engineers and Architects
- DBI Code Advisory Committee
- Heller Manus Architects
- Building Inspection Commission
- HOK
- Maffei Structural Engineering
- BXP
- SEAONC AB 82/83 Code Advisory Committee
- Fennie+Mehl Architects

Business
San Francisco Chamber of Commerce

Real Estate
Real Estate Advisory Services

Community and Non-profit Organizations
SPUR
BOMA
The East Cut Community Benefit District

City and County of San Francisco Staff
Department of Building Inspection
Real Estate Division
Department of Emergency Management
Planning Department
Fire Department
SFPUC
Public Works
Office of Resilience and Capital Planning