Toward functional recovery performance in the seismic design of modern tall buildings

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Abstract
Current building code requirements for seismic design are primarily intended to minimize life-safety risks due to structural damage under extreme earthquakes. While tall buildings designed under current standards are expected to achieve the life-safety goal, this study estimates that they may require up to 7.5 months of repair to return to functionality after a design-level earthquake (roughly equivalent to ground motion shaking with a 10% probability of exceedance in 50 years), and over 1 year after a risk-targeted maximum considered earthquake (roughly equivalent to ground motion shaking with a 2%–4% chance of exceedance in 50 years). These long downtimes, which correspond to median predictions, far exceed recovery goals for major employers and other recovery-critical uses and can have disproportionately harmful effects on businesses and residents. To address such extensive downtime risks, we evaluate the impact of recovery-based design guidelines for reducing recovery times through (1) more stringent drift limits under expected ground motions and (2) measures to mitigate externalities that impede recovery. The results suggest that by combining these strategies, expected recovery times following a design-level earthquake can be reduced to roughly 1 month, and to 2 months following a risk-targeted maximum considered earthquake. These findings are illustrated for an archetype 42-story reinforced concrete shear wall residential building and a 40-story steel buckling-restrained braced frame office building in San Francisco, CA.

Keywords
Tall buildings, performance-based seismic design, earthquake-induced repair costs, downtime and functional recovery

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Introduction

Currently enforced building code requirements for earthquakes are primarily intended to provide acceptable safety in extreme earthquakes. Non-prescriptive design procedures, such as the guidelines for performance-based seismic design of tall buildings, employ non-linear analysis to evaluate expected building response to large earthquakes and, thereby, provide greater assurance that buildings meet the performance targets intended in current building codes to minimize life-safety risks under extreme earthquakes. However, they are not intended to provide an enhanced level of performance, but rather are calibrated to provide equivalent performance to that of prescriptive designs (PEER, 2017). As a result, code conforming buildings, including non-prescriptive designs, may suffer significant damage from earthquakes and may not ensure performance beyond this life-safety objective, or preserve building functionality by limiting damage to structural and nonstructural building components.

In addition to direct financial losses due to repair costs, damaged buildings may incur indirect losses due to extensive 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 occupied for cleanup repairs or used for basic shelter, although functionality may not be restored. Functional recovery is achieved when the building repairs restore a significant measure of the building’s intended pre-earthquake use (EERI, 2019). Full recovery takes place when the building is fully restored to its pre-earthquake condition and use. This study focuses its evaluation on the functional recovery state.

Designing for functional recovery implies that functionality is a quantifiable state. Once quantified, communities can assess whether functionality is reached within an acceptable time frame (EERI, 2019). In an effort to improve its seismic resilience, the City and County of San Francisco has put forward a set of post-earthquake recovery goals for its building and infrastructure (Kornfield et al., 2011). For multi-family housing, the recovery target is to have 80%-90% of the buildings regain their pre-earthquake functionality within “weeks.” A similar recovery target is defined for major employers and hotels. Most of San Francisco’s tall buildings house one or more of these functions and pose a significant risk due to their high concentration in the City’s downtown district. Furthermore, due to their size and high occupant loads, significant damage to these buildings can have disproportionate impacts on the community.

Two recent studies of a hypothetical reinforced concrete shear wall (RCSW) residential building, which represents many of the recently constructed tall buildings in San Francisco, indicate that these structures 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). These values of economic losses are not uncommon in seismic performance assessments 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. These results, which correspond to median predictions of both loss and downtime, highlight how damage that requires only moderate repair costs could result in excessive downtimes, leading to displacement of residents with associated indirect costs. Motivated by this issue, this article further examines the expected seismic performance of modern tall buildings in terms of functional recovery time and how the performance relates to current building code requirements.
We examine the expected performance of a 42-story RCSW residential building and a 40-story steel buckling-restrained braced frame (BRBF) office building in San Francisco, CA. The buildings are designed following non-prescriptive procedures and their seismic performance is evaluated under code-level ground motions, namely ground motions consistent with the design-level and the risk-targeted maximum considered earthquakes. The results of the evaluation are benchmarked against the City’s recovery goals and are used to explore the potential costs and benefits of higher performance goals for new construction. A series of “recovery-based design guidelines” are put forward to bridge the gap between the expected performance of these buildings and current recovery goals. One such measure is to increase the strength and stiffness of new buildings. While greater strength and stiffness is not the only option to improve resilience, this approach offers the advantages that it could be implemented in practice by any structural designer without requiring additional technical expertise, software, or proprietary technology (Porter, 2020). Additionally, because designing for functional recovery means considering both safety and recovery time in design (EERI, 2019), we introduce planning strategies to supplement design strategies aimed at reducing physical damage in structural and nonstructural building components.

This study is the first of its kind to provide insights into how tall building seismic design criteria relates to functional recovery performance. This work supports ongoing efforts to characterize performance goals for the built environment in terms of post-earthquake reoccupancy and functional recovery time (42 U.S.C. § 7705b, 2018; Senate Bill 1768, 2018). The assessment methodology goes beyond the well-established FEMA P-58 (2012) framework for calculating earthquake-induced repair costs and the associated repair time by including elements of the REDi downtime framework (Almufti and Willford, 2013). This addition accounts for impeding factor delays between the earthquake event and the initiation of repairs, as well as realistic labor allocation and repair sequencing. The study highlights key challenges associated with loss and downtime assessment of tall buildings and proposes some modifications and special considerations in its implementation. The study also presents a survey of 35 recently completed tall building projects designed following a performance-based approach, which serves to benchmark the archetype buildings studied and permits understanding of how modifications to drift limits would affect engineering design practice. Finally, the availability and cost implications of the strategies proposed are explored to demonstrate their feasibility.

**Code objectives and seismic design requirements**

Based on national standards and the California state building code, San Francisco Building Code (SFBC, 2016) requirements for earthquake design are based on life-safety performance in extreme earthquakes. These requirements are intended to limit the risk of structural collapse, as well as damage to structural and nonstructural components that could pose life-safety risks under severe ground motions. Requirements to limit damage to components that could hinder building functionality or induce extensive recovery times are not currently required in design.

Since 2008, any San Francisco building that is taller than ~73 m (240 ft) has been designed following the performance-based requirements of Administrative Bulletin 083 (AB-083, 2008), *Requirements and Guidelines for the Seismic Design of New Tall Buildings using Non-Prescriptive Seismic-Design Procedures*, and the *Guidelines for Performance-Based Seismic Design of Tall Buildings* (PEER, 2017). While these performance-based procedures help ensure reliability of building response under strong earthquakes, they are not
intended to provide an enhanced level of performance, but rather are calibrated to provide equivalent performance to that of prescriptive designs (PEER, 2017).

The archetype RCSW residential and BRBF office buildings assessed in this study are designed following non-prescriptive procedures to meet the current seismic design requirements of the SFBC (2016) and AB-083 (2008). The design process includes assessing the building performance under three levels of ground motion shaking: a risk-targeted maximum considered earthquake (MCE), a design earthquake (DE), and a service-level earthquake (SLE).

The MCE evaluation is used to ensure “collapse-prevention” under extreme ground motions and has a 2%–5% probability of exceedance in 50 years. The DE level is defined as two-thirds of MCE-shaking. The DE evaluation is intended to identify the exceptions taken to the prescriptive requirements of the SFBC (2016) and to ensure minimum strength and stiffness for earthquake resistance. The SLE performance assessment is required to ensure an acceptable seismic performance (essentially elastic) under earthquakes that are anticipated during the service life of the building. The SLE level has a return period of 43 years (50% probability of exceedance in 30 years). In contrast to linear analyses at SLE and DE, nonlinear response history analyses are employed in the MCE evaluation. Table 1 provides an overview of the design criteria adopted in the design of the archetype buildings. Detailed design requirements can be found in AB-083 (2008) and PEER (2017).

Archetype residential and commercial tall buildings

San Francisco tall buildings

Since the late 1950s, San Francisco’s skyline, like that of many large US cities, has changed dramatically with the construction of tall buildings in the downtown financial district. The past 15 years have added many high-rise residential and mixed-use buildings concentrated in the region south of Market Street. 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, among many other multi-faceted earthquake risks facing the city.

As a first step toward addressing these questions, San Francisco’s Tall Buildings Study (ATC, 2018) compiled an inventory of buildings taller than ~73 m (240 ft), roughly 18–24 stories above grade. The primary motivation of the inventory was to quantify the tall buildings in terms of height, age, usage, and structural system characteristics of the buildings. Although there are other earthquake risks, tall buildings are of special concern due to their size and large number of occupants, where earthquake damage to one tall building can have disproportionate effects on its occupants, its neighbors, and the community at large.

The inventory includes detailed information on 156 buildings over ~73 m (240 ft) tall, some of which do not yet exist but have been permitted for construction. Roughly 10% of the inventory is below 20 stories, 70% is in the 20- to 40-story range, and 20% is above 40 stories. With regard to occupancy, approximately 60% of the buildings are offices and just under 40% are residential or hotel. Over 55% of the tall building inventory was constructed between 1960 and 1990. Tall building construction slowed down in the 1990s, but has resurfaced since then, with almost 25% of the inventory constructed since 2000.
In terms of construction material and structural system, there are three noteworthy trends in the database: (1) the predominance of steel moment-resisting frame construction during the 1960s, 1970s, and 1980s, which make up approximately 50% of the inventory; (2) a transition from steel moment-resisting frames to steel braced frame dual systems in the 1990s; and (3) the emergence of RCSW and steel braced frame systems since 2000, most of which were designed following a performance-based seismic design procedure.

Archetype tall building design

The two tall building archetypes investigated in this study (see Figure 1) represent trends in modern tall building construction: one is a RCSW residential building and the other is a steel BRBF office building. The building designs are evaluated at two locations in San Francisco with distinct soil properties, classified as a function of the shear wave velocity in the top 30 m of soil ($V_{s30}$), a Site Class D ($V_{s30} = 180–360$ m/s or 600–1200 ft/s) and a Site Class B ($V_{s30} = 760–1500$ m/s or 2500–5000 ft/s) soil. Despite their distinct site class, the designs are essentially the same due to minimum base shear requirements at DE, which are identical at both locations, and other design constraints. The same design for two sites with different ground motion intensities highlights the variability in seismic performance that can occur across the city. Assessing the performance of the same design in two

| Table 1. Performance-based design criteria. |
|---------------------------------------------|
| **Service-level earthquake**<sup>a</sup> (SLE) | **Design earthquake**<sup>b</sup> (DE) | **Maximum considered earthquake**<sup>b</sup> (MCE<sub>R</sub>) |
| Objective | Limited damage | Code compliance (life safety) | Collapse prevention |
| Analysis method | Linear (response spectrum) | Linear (response spectrum) | Nonlinear response history |
| Response modification factor (R) | – | Code-prescribed R factor | – |
| Story drift limits | <0.5% | <2% | Mean transient < 3% |
| Component acceptance criteria | Demand < 1.5 Nominal Strength<sup>c</sup> | Demand < Design Strength | Mean residual < 1% |
| Guidance document | PEER TBI (PEER, 2017) | SFBC (2016), ASCE/SEI 7-10 (2012) | Force and deformation-controlled component checks |

**SFBC**: San Francisco Building Code.

<sup>a</sup>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.

<sup>b</sup>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 a deterministic limit defined as the 84th percentile hazard from a magnitude-8 earthquake on the San Andreas fault, corresponding to a return period on the order of 1200 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 2500 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).

<sup>c</sup>In (PEER, 2017): Demand < 1.5 Nominal Strength (i.e. no strength reduction factor is applied at the service-level check). In (PEER, 2010): Demand < 1.5 Design Strength (i.e. strength reduction factor is applied).

In terms of construction material and structural system, there are three noteworthy trends in the database: (1) the predominance of steel moment-resisting frame construction during the 1960s, 1970s, and 1980s, which make up approximately 50% of the inventory; (2) a transition from steel moment-resisting frames to steel braced frame dual systems in the 1990s; and (3) the emergence of RCSW and steel braced frame systems since 2000, most of which were designed following a performance-based seismic design procedure.
locations provides insights into the performance of buildings that have proportionally larger strength and stiffness compared to the ground shaking hazard, that is, the buildings in Site Class D are regarded as a “baseline” design, whereas the buildings in Site Class B are regarded as an “enhanced” design. While this assumption is appropriate for the purpose this study, in reality, design checks at SLE and MCER would result in final designs that vary slightly across the two sites. In other words, the RCSW residential and the BRBF office buildings in Site Class B are slightly overdesigned. Nevertheless, the performance assessment for the buildings at the two sites (B and D) is considered 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, for example, smaller drift limits, the relative performance of

Figure 1. Typical superstructure plan view and isometric of the archetype buildings: (a) RCSW Residential and (b) BRBF Office.
the buildings across the two sites, one with smaller (baseline design) and one with larger (enhanced design) strength and stiffness compared to the ground shaking hazard, provides insights into how design criteria relates to building performance.

The RCSW residential building has 42 stories above grade and four basement levels. Story heights are typically 3 m (10 ft), resulting in an overall height, including the roof bulkhead, of 131 m (432 ft) above the ground. The building was initially designed by Magnusson Klemencic Associates (MKA) for a site in Los Angeles (Moehle et al., 2011), and later re-designed for a site in San Francisco (Tipler, 2014). The gravity system consists of post-tensioned 200 mm (8 in) thick slabs, measuring 32.9 m × 32.6 m (108 ft × 107 ft) in the superstructure plan, supported by RC columns. The seismic force resisting system is a 9.8 m × 14.6 m (32 ft × 48 ft) RC core consisting of coupled shear walls 810 mm (32 in) thick up to floor 13, and 610 mm (24 in) thick from floor 13 to the roof. The coupling beams are 760 mm (30 in) in depth in the superstructure and 860 mm (34 in) in the basement. Nominal compressive strength of concrete in the core is 55 MPa (8.0 ksi) and nominal yield strength of steel in the core and coupling beams are 410 MPa (60 ksi) and 520 MPa (75 ksi), respectively. Figure 1a shows an isometric and typical plan view of the RCSW residential building. The first two vibration periods of the building are summarized in Table 2. Further design information, including reinforcement detailing in coupling beams and core walls, can be found in Tipler (2014). The nonlinear analysis model of the archetype RCSW residential building can be found in the Electronic Supplement S1 (https://www.carlosmolinahutt.com/publications).

The RCSW dissipates energy by hinging at the base of the wall piers and at the ends of the coupling beams over the height of the building. This study uses a three-dimensional (3D) model of this structure, developed by Almufti et al. (2018) for the recently completed USGS HayWired study, to evaluate the performance of the building at DE and MCE_R levels by means of nonlinear response history analysis in LS-DYNA (LSTC, 2011). In this model, walls and coupling beams are modeled using distributed plasticity fiber beam-column elements and lumped plasticity elements, respectively. The hysteretic response of coupling beams is calibrated using the experimental data from testing at the University of California Los Angeles (Naish et al., 2009). Columns and slabs are modeled using linear elastic beam-column and shell elements, respectively. More details on the modeling approach can be found in Tipler (2014).

The steel BRBF office building has 40 stories above the grade and 4 basement levels. Story heights are typically 4.1 m (13.5 ft), resulting in an overall height of 166 m (545 ft) above the ground. The gravity system consists of a composite floor slab, 82 mm slab on 76 mm deck (3.25-inch slab on 3-inch deck), over 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 1b. The core area of the single-story BRBs of the interior frames ranges from 38.7 cm² (6 in²) to 77.4 cm² (12 in²), and the core area of the exterior mega-brace BRBs ranges from 58.1 mm² (9 in²) to 322.6 cm² (50 in²). The BRBF columns are box shaped with concrete fill. Nominal compressive strength of concrete fill is 69 MPa (10 ksi), and the steel for framing and braces are A992 and A36, with yield strengths of 345 MPa (50 ksi) and 248 MPa (36 ksi), respectively. The periods of vibration of the BRBF office building are summarized in Table 2.

The BRBF system dissipates energy primarily by brace yielding, while all other elements in the system are capacity designed to remain essentially elastic. Planar nonlinear models in each principal building direction were developed to evaluate the response of the structure at DE and MCER by means of nonlinear response history analysis 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 axial elements that 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. The nonlinear analysis models of the archetype BRBF office building can be found in the Electronic Supplement S1 (https://www.carlosmolinhutt.com/publications).

**Existing performance-based seismic design projects and past loss studies**

To demonstrate the archetype buildings’ compliance with current performance-based seismic design requirements, member force and deformation demands, as well story drift ratios, were evaluated at MCER shaking. Two ground motion suites of 11 horizontal acceleration records were selected and linearly scaled for each of the Site Classes, B and D. The records were selected to closely follow the target spectrum across the period range of interest from 1 to 8 s, as seen in Figure 2, as measured by the suite average maximum direction spectral acceleration, RotD100, 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. The resulting ground motion suites are summarized in the Electronic Supplement S2 (https://www.carlosmolinhutt.com/publications), including a unique ground motion identifier (per the PEER 2013 ground motion database; Bozorgnia et al., 2014), 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. The selected ground motions are input at top of foundation level, just beneath the basement, assuming a fixed base condition. The basement levels are explicitly modeled, where the basement floor slabs and perimeter walls are modeled as elastic and interaction with the surrounding soil is not considered.

Following ASCE/SEI 7-10 (2012), the SFBC (2016) defines DE shaking as two-thirds of MCE. To evaluate performance at DE shaking, the MCE target spectrum and associated ground motion suites were scaled accordingly by a factor of two-thirds. The MCE ground motions are used to ensure compliance with the PEER TBI seismic design guidelines, whereas both MCE and DE intensity ground motions are used in the performance assessments. The DE intensity is consistent with the “expected earthquake” level.
considered in the SPUR Resilient City (SPUR, 2009) study and other studies to assess community resilience.

The maximum story drift ratio (throughout the building height) of the average set of results (from the 11 ground motions) of the RCSW residential building under MCE_R ground motions in Site Class D were 1.18% and 1.14% in the x and y directions, respectively, and 0.58% and 0.49% in the x and y directions, respectively, in Site Class B. Similarly, the results of the BRBF office building under MCE_R ground motions in Site Class D were 1.82% and 1.28% in the x and y directions, respectively, and 0.58% and 0.75% in the x and y directions, respectively, in Site Class B. The maximum MCE_R story drift ratios, in both RCSW residential and BRBF office buildings, are significantly less than the 3% limit currently prescribed by PEER TBI guidelines (PEER, 2017) for performance-based seismic designs (see Table 1). To benchmark the archetype buildings against real building designs, data from over 30 recently constructed tall building projects in San Francisco, Los Angeles, Seattle, San Diego, and other locations were collected (Arup, Glotman Simpson and Magnusson Klemencic Associates, personal communication, August 2018). Figure 3 shows the MCE_R story drift demands (maximum story drift ratio—throughout the building height—of the average set of peak results—from the total number of ground motions used in each analysis—in each story) for evaluations in the principle (x, y) directions for the 35 buildings. These results serve to demonstrate conformance with PEER TBI’s (2017) requirement that “in each story, the mean of the absolute values of the peak transient story drift ratios from each suite or set of analyses shall not exceed 0.03.” The data suggest that despite the 3% drift limit at MCE_R, about two-thirds of the buildings have maximum drifts less than 2% and half of the buildings have drifts less than 1.5%. These data indicate that the Site Class D archetype buildings or baseline designs are fairly representative of the average stock of buildings, and further, that limiting MCE_R drifts to values less than the current 3% limit is not inconsistent with current design practice. In addition to benchmarking the design of the archetype buildings against the design of recently completed buildings, this information permits understanding of how modifications to drift limits would affect the engineering design practice.
A literature review also compared the estimated loss data from this study (described in more detail later in the article) against those of similar tall building research projects. Data were collected for buildings consistent with the steel BRBF office and RCSW residential buildings presented in this article (Almufti et al., 2018; Kourehpaz et al., 2021; Moehle et al., 2011; Tipler, 2014), as well as for dual systems including a RCSW and perimeter-reinforced concrete frames (Moehle et al., 2011), and steel moment resisting frames (SMRF) with pre-Northridge detailing (Almufti et al., 2018; Molina Hutt et al., 2016, 2019). Figure 4 shows expected losses, based on the FEMA P-58 methodology, against the maximum story drift ratio (throughout the building height) of the average set of peak results (from the total number of ground motions used in each analysis) in each structural system.
story. The data indicate that the loss estimates for the archetype buildings in the current study (shown in Figure 4 with a red border), while at the lower range, are generally consistent with those from past studies. The four data points for each archetype building in this study represent the different site classes (B or D) and hazard levels considered (DE and MCE\textsubscript{R}). The data also reveal 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%. Figure 4 provides overall trends in the relationship between story drift ratios and expected losses in tall building studies. However, certain subtleties are not captured, for example, racking story drift ratios, which significantly contribute to the losses in RC core-wall structures, are not included. Note that either the mean or median values are reported in Figure 4, depending on the study. Some of the data points, that is, Kourehpaz et al. (2021) and Molina Hutt et al. (2016, 2019), have contributions to the mean loss from residual drifts rendering the building irreparable (typically for realizations above the 90th percentile). While loss contributions from irreparable damage are generally considered as appropriate to include when calculating the expected loss, including these contributions can lead to mean loss ratios that significantly exceed the median predictions.

**Building performance modeling**

**Engineering demand parameters**

The overall loss and downtime assessment methodology adopted in this study is graphically illustrated in Figure 5. The input to the component damage and loss fragility functions, so-called engineering demand parameters (EDPs), are obtained from nonlinear response history 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). Damage and loss predictions are based on the
FEMA P-58 (2012) methodology. The total functional recovery time is calculated as the sum of the time to repair damaged components that hinder achieving a recovery state, and the longest sequence of impeding factor delays, which delay the initiation of repairs. The downtime assessment methodology outlined in REDi (Almufti and Willford, 2013) was used as the basis to evaluate the time to functional recovery in this study. However, a number of modifications (Modified REDi in Figure 5) were introduced where limitations to the method were identified in its application to tall buildings. The methodology introduces repair classes, based on the extent of damage and criticality of building components, repairs to which are assumed to be required to achieve the specified recovery state. Repair classes are assigned to each damage state for each building component and dictate whether the damage in the component hinders one or more of the recovery states. If so, the component needs to be repaired before each recovery state can be achieved. This study focuses specifically on the repairs required to achieve functional recovery, where it is assumed that damage to all structural and nonstructural components that pose a life-safety risk (e.g. moderate-to-severe damage to the seismic force resisting system) as well as nonstructural components that are essential to the basic building operation (e.g. damage to the elevator system) must be repaired.

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 is accounted for through a Monte Carlo simulation with thousands of realizations, where each realization represents one possible performance outcome for the building considering a single combination of possible values of each variable considered. Following this process, results are 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 this study focus exclusively on central tendency statistical measures, that is, mean and median results.

The following EDPs are used to evaluate performance in the RCSW residential building: transient story drift, residual drift, damageable wall drift, racking drift, coupling beam rotation, and floor acceleration. The following EDPs are used to evaluate performance in the BRBF office building: transient story drift, residual story drift, and floor acceleration. While racking deformations also exist around the core of the BRBF, the effects are much smaller than in the RCSW. All relevant EDPs are summarized in the Electronic Supplement S3 (https://www.carlosmolinhutt.com/publications).

In the RCSW residential building, the damageable wall and racking drifts are derived from the story drifts, taking into account the kinematics of tall core wall building response. Racking drift deformations occur due to the difference between the vertical deformation at the shear wall (due to flexural tensile strains) and the gravity columns. Figure 6 represents an idealized schematic of the deformed shape of the concrete core wall and perimeter gravity columns, with story heights of $H$ and span lengths of $L$. As shown in the figure, horizontal and vertical rotations of the top left gravity panel (i.e. $\theta_{h,AB}$ and $\theta_{v,AC}$, respectively) are in opposite directions, causing additional distortion of the panel, whereas the wall panels rotations (i.e. $\theta_{h,CD}$ and $\theta_{v,CE}$, respectively) are in the same direction and cause less panel distortion. Considering the rotations of all four edges of any panel ABCD (either a wall panel or a gravity panel), the damageable drift ratio (DDR) can be defined as the average shear strain of the rectangular panel as outlined in Figure 6, where $X_i$ and $Y_i$ denote the horizontal and vertical displacement, respectively, of node $i$. This generalized expression of DDR has been used in past loss assessment studies of RCSW structures.
The racking drift causes damage and losses associated with interior partition wall finishes and slab-to-column and slab-to-wall connections. The racking deformations and associated damage are largest in the upper floors of the building.

Figure 7 illustrates the distribution of maximum transient story drift ratios and floor accelerations up the building height under DE ground motions in Site Class D for both the RCSW residential and BRBF office buildings. The results illustrate a fairly uniform distribution of demands above grade, with mean story drifts below 1% and mean floor accelerations of 1 g. The bumps in the demand parameters of the BRBF office building are a direct result of the bracing configuration along the height. Table 3 summarizes the maximum value (throughout the building height) of the average set of results (from the 11 ground motions) in each story for each of the EDPs considered in the performance evaluation of the RCSW residential and the BRBF office buildings in Site Class B and D under DE and MCER. Residual story drift ratios were derived from peak transient story drifts as outlined in Appendix C of FEMA P-58 (2012). The residual drifts in Table 3 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 11 ground motions (a single residual drift parameter is input into the loss model for each ground motion in each building direction). In other words, in contrast to peak transient story drifts, which are calculated for each story, the average residual drift is calculated based on the maximum observed over the building height.

Loss assumptions

The expected damage and associated economic losses in the archetype buildings are evaluated based on the FEMA P-58 (2012) methodology (see Figure 5) as implemented in HBRisk’s Seismic Performance Prediction Program (SP3) (2019). The assembly-based
building performance models are defined by their structural and nonstructural building components that are susceptible to seismic damage. The structural component assignments are based on the structural design of each building, which was described previously. The nonstructural components are assigned using the Normative Quantity Estimation Tool of FEMA P-58. The resulting nonstructural component assignments were reviewed for consistency with previous studies on expected seismic performance of tall buildings similar to those under evaluation (Moehle et al., 2011; Tipler, 2014) and are summarized in the Electronic Supplement S4 (https://www.carlosmolinahutt.com/publications). Certain nonstructural components, such as air handling units, cooling towers, or chillers, required the calculation of parameters to characterize their seismic resistance. These were calculated automatically within SP3, which follows ASCE/SEI 7-10 (2012), Chapter 13: Seismic Design Requirements for Nonstructural Components.

In the FEMA P-58 methodology, performance estimates of the structural and nonstructural building components are determined using statistical fragility functions that define the conditional probability of incurring a damage state based on the imposed demand described by the associated EDP. 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 FEMA P-58 Normative Quantity Estimation Tool, the elevators, mega braces, and façades were included as user-defined components to more accurately characterize the damage.

### Table 3. Maximum of average engineering demand parameters (EDPs) for evaluation of the RCSW residential and the BRBF office buildings across site classes and ground motion intensity levels.

| EPDs      | Site class | Intensity | RCSW Residential | BRBF Office |
|-----------|------------|-----------|------------------|-------------|
|           |            | X Direction | Y Direction | X Direction | Y Direction | X Direction | Y Direction |
| SDR (%)   | D          | DE         | 0.82           | 0.77        | 0.90        | 1.01        |            |            |
|           |            | MCER       | 1.18           | 1.14        | 1.82        | 1.28        |            |            |
|           | B          | DE         | 0.42           | 0.34        | 0.45        | 0.62        |            |            |
|           |            | MCER       | 0.58           | 0.49        | 0.58        | 0.75        |            |            |
|           | D          | DE         | 0.09           | 0.08        | 0.22        | 0.22        |            |            |
|           |            | MCER       | 0.20           | 0.19        | 0.91        | 0.36        |            |            |
|           | B          | DE         | 0              | 0           | 0.11        | 0.09        |            |            |
|           |            | MCER       | 0.03           | 0.02        | 0.15        | 0.14        |            |            |
| Residual-SDR (%) | D | DE  | 1.60           | 1.68        | 0.71        | 1.12        |            |            |
| PFA (g)   | B          | DE         | 1.14           | 0.88        | 0.56        | 0.93        |            |            |
|           |            | MCER       | 1.62           | 1.31        | 0.78        | 1.33        |            |            |
| DWD (%)   | D          | DE         | 0.21           | 0.24        | –           | –           |            |            |
|           |            | MCER       | 0.64           | 0.66        | –           | –           |            |            |
|           | B          | DE         | 0.09           | 0.11        | –           | –           |            |            |
|           |            | MCER       | 0.14           | 0.17        | –           | –           |            |            |
| Racking-SDR (%) | D | DE  | 1.27           | 1.16        | –           | –           |            |            |
|           | B          | DE         | 0.78           | 0.64        | –           | –           |            |            |
|           |            | MCER       | 1.01           | 0.86        | –           | –           |            |            |
| CBR (rad) | D          | DE         | 0.012          | 0.012       | –           | –           |            |            |
|           |            | MCER       | 0.020          | 0.019       | –           | –           |            |            |
|           | B          | DE         | 0.006          | 0.005       | –           | –           |            |            |
|           |            | MCER       | 0.009          | 0.008       | –           | –           |            |            |

SDR: story drift ratio; Residual-SDR: residual story drift ratio; PFA: peak floor acceleration; DWD: damageable wall drift; Racking-SDR: racking story drift ratio; CBR: coupling beam rotation.
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). User-defined fragility functions were also 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 were developed for low- and mid-rise construction.

**Downtime assumptions**

The downtime assessment methodology assigns a repair class (RC) to each damage state (DS) of the structural and nonstructural building components. The repair classes identify
how critical each damage state is with regard to three successive recovery states (reoccupation, functional and full-recovery, which correspond to repair classes RC3, RC2 and RC1, respectively), requiring repairs for any relevant component damage in order to achieve that level of recovery. For example, local yielding in stairs (DS1) would only require cosmetic repairs that do not affect functionality (RC1), while the safety hazard associated more extensive damage (DS2 or DS3) would prevent reoccupation (RC3). The present study focuses on repairs required for functional recovery. The repair classes associated with each damage state for all building components are summarized in the Electronic Supplement S4 (https://www.carlosmolinahutt.com/publications).

The methodology also includes streamlining the sequence of repairs on a floor-per-floor basis, which is an important consideration for tall buildings. The necessary repair activities are grouped by trade, including structural, façade, egress (stairs and elevators), MEP (Mechanical, Electrical, and Plumbing), and 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. The resulting repair time estimates are distinct from those obtained from the FEMA P-58 methodology, which provides two estimates of building repair time: repair time in series (considering repairs in each floor are sequential) and repair time in parallel (considering repair in all floors simultaneously). Furthermore, FEMA P-58 only estimates the repair time required to achieve full recovery as opposed to the necessary repairs to achieve an intermediate recovery state, that is, functional recovery, as implemented in this study.

Referring back to Figure 5, 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, permitting, contractor mobilization, and financing. These impeding factors account for delays between the earthquake event and the initiation of repairs and are triggered when damage to any building component hinders achieving the desired recovery state, that is, functional recovery. Each impeding factor must be addressed before repairs can be initiated. The duration of each impeding factor is estimated by sampling the probability distributions summarized in Table 4, where in addition to the baseline case, a recovery-planning option is shown to assess the benefits of implementing measures to minimize different impeding factor delays.

In this study, the triggering of impeding factor delays is as follows. At the hazard levels considered, that is, DE and MCE_R, we assume post-earthquake inspection is always

| Impeding factor delays                  | Baseline case | Recovery planning |
|-----------------------------------------|---------------|-------------------|
| Inspection                              | 5 days, 0.54  | 1 day, 0.54       |
| Engineering mobilization weighted by    | [6/12] weeks, [0.40/0.40] | [2/4] weeks, [0.32/0.54] |
| [cosmetic/significant] damage           |               |                   |
| Contractor mobilization weighted by     | [28/40] weeks, [0.3/0.3] | [3/7] weeks, [0.66/0.35] |
| [cosmetic/significant] damage           |               |                   |
| Permitting for                          | [1/8] weeks, [0.86/0.32] | [0.5/4] weeks, [0.86/0.32] |
| [cosmetic/significant] damage           |               |                   |
| Financing                               | [15 weeks, 0.68] | [1 week, 0.54]    |
| [only triggered for > 10% costs]        |               |                   |
triggered. Financing is triggered as a function of the loss ratio as discussed later. Engineering mobilization and permitting are triggered if damage to any structural component hinders functional recovery, inferred as a function of repair class as described earlier. Similarly, contractor mobilization is triggered if damage to any structural or nonstructural component hinders functional recovery, also inferred as a function of repair class. The damage states in all structural and nonstructural components that would trigger these impeding factors are summarized in the Electronic Supplement S4 (https://www.carlosmolinahutt.com/publications).

These impeding factors are grouped into three delay sequences, the longest of which controls the impeding factor delays. All delay sequences, which are modeled in parallel, 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 and permitting. 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. There are no interactions between the delay sequences and the probability distributions that characterize different impeding factor delays are sampled independently.

The total functional recovery time is calculated as the sum of: (1) the time to repair damaged components that are required to achieve the functional 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, long-lead components, or demand surge are not evaluated in this study.

While the downtime assessment methodology outlined in REDi (Almufti and Willford, 2013) was used as the basis to evaluate the time to functional recovery in this study, modifications were introduced where limitations to the method were identified in its application to tall buildings. Key changes relate to financing and mobilization estimates. With regard to financing, 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, thereby avoiding delays associated with financing (on the order of 15 weeks as shown in Table 4).

Regarding 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, as illustrated in Table 4. This change is introduced to account for scenarios where severe damage to a single component would otherwise trigger unrealistic impeding factors. Almufti and Willford (2013) propose different impeding factor delay estimates associated with engineering mobilization as a function of the severity of damage to structural components, recognizing that minor structural damage might not require structural calculations, yet significant structural damage could trigger re-design. A similar rationale is applied to the contractor mobilization delay estimates considering damage levels (cosmetic vs significant) to both structural and nonstructural building components. While the Almufti and Willford approach recognizes the possibility for distinct impeding factor delays as a function of the severity of damage, lengthy delays would be triggered even if a single component experienced significant damage. For instance, if one coupling beam in the 42-story RCSW residential building experienced significant
damage, this would trigger ~12 weeks of impeding time for engineering mobilization and ~40 weeks for contractor mobilization. To address this issue, in this study we factor the expected delay estimates according to the damage levels of different components. For instance, if one realization resulted in 50% of structural components experiencing cosmetic damage and the remaining 50% experiencing severe damage, both distributions would be sampled (one associated with cosmetic damage and one associated with significant damage) and they would each be assigned a weight of 0.5. Note that sampling from untruncated lognormal distributions for the impeding factors can result in very long impeding factor durations or, for example, longer durations for cosmetic damage than for significant damage. However, this is mitigated by the total number of Monte Carlo realizations, such that the excessively long durations have negligible effect on the median results presented in this study.

**Toward recovery based-design guidelines**

*Impact of design schemes on loss, repair time, and downtime to functional recovery*

The methods previously discussed to estimate building losses and downtime were used to evaluate the expected performance of the RCSW residential and steel BRBF office buildings at two ground motion shaking intensities, DE and MCE_R across the two sites, Site Class D and B. As discussed above, the small drift demands for the Site Class B ground shaking can be interpreted as estimates of Site Class D performance for a “better than code” building, specifically one designed with tighter drift limits. Recovery planning measures can also reduce downtime by minimizing impeding factor delays. Table 5 provides a summary of the median losses, repair times (for components that inhibit functional recovery), and downtime to functional recovery (which include delays associated with different impeding factors) for all design variations (structural and recovery planning measures) considered in the RCSW residential and BRBF office buildings, respectively. 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 median results in Table 5 do not necessarily correspond

| Archetype  | Structural Design | Recovery Planning | Loss (%) | Repair Time (days) | Downtime to Functional Recovery (days) |
|------------|-------------------|-------------------|----------|--------------------|---------------------------------------|
| RCSW Residential | Baseline (Site Class D) | Baseline | 7.7 | 51 | 222 |
| | Enhanced (Site Class B) | Enhanced | 122 | 316 | 512 |
| | Baseline | Baseline | 12 | 433 | 122 |
| | Enhanced | Enhanced | 145 | 182 | 145 |
| | Baseline | Baseline | 67 | 182 | 67 |
| | Enhanced | Enhanced | 85 | 85 | 85 |
| BRBF Office | Baseline (Site Class D) | Baseline | 3.2 | 29 | 67 |
| | Enhanced (Site Class B) | Enhanced | 3.1 | 37 | 169 |
| | Baseline | Baseline | 1.7 | 14 | 98 |
| | Enhanced | Enhanced | 2.4 | 22 | 98 |
| | Baseline | Enhanced | 31 | 149 | 31 |
| | Enhanced | Enhanced | 58 | 58 | 58 |

Table 5. Summary of median loss results, repair times, and functional recovery times of the RCSW residential and BRBF office buildings at DE and MCE_R considering variations in structural design and recovery planning measures.
to the same realization, but rather represent the median value of each metric considered: loss, repair time, and downtime to functional recovery.

The maximum story drift values, shown in Table 3, help gage the relationship between drift demands and expected losses. For the RCSW residential building the peak DE drifts are on the order of 0.8% and 0.4%, and for the BRBF office building are on the order of 1% and 0.6% (in comparison to the code-permitted maximum DE story drifts of 2%—see Table 1). The loss results in Table 5 indicate that expected losses in the BRBF office building are considerably lower than in the RCSW residential building. For instance, at Site Class D under DE shaking, the expected losses in the RCSW residential building are 7.7% versus 3.1% in the BRBF office 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. RCSW losses are higher, even if the drift ratios are lower, due to (a) more structural damage to coupling beams and slab-wall/column connections, and (b) the effects of racking drift deformations on partition walls.

The loss methodology enables disaggregating losses into their constituent parts, which can help inform loss mitigation strategies. Figure 8 illustrates the average contribution of different building components to the overall expected loss under DE shaking. Losses in the RCSW residential building are dominated by damage to structural components and interior partitions, particularly those components that are sensitive to racking drifts, which are amplified by about 1.5 to 2 times over story drifts (see Figure 7a). As mentioned previously, 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-wall/column connections. Losses in the steel BRBF office building, which are about half of those for the RCSW residential building, are dominated by damage to interior finishes, plumbing, and HVAC (Heating, Ventilation, and Air-Conditioning). Note that while elevator damage is a relatively small contributor to damage in both buildings, it is a more significant contributor to downtime. Based on the nonlinear response history analysis results under the ground motion shaking intensities considered, and as expected based on

![Figure 8](image_url)  
**Figure 8.** Contribution of different building components to expected losses under Design Earthquake ground motions in Site Class B and Site Class D for the (a) RCSW residential and (b) BRBF office buildings.
the relatively small transient drift demands, the estimated residual drifts are negligible (essentially zero) for both buildings.

In this study, losses attributed to acceleration-sensitive components ("Plumbing and HVAC" and "Other Nonstructural" in Figure 8) are fairly uniform for both designs at the hazard levels considered. While current tall building seismic design requirements do not address the performance of acceleration sensitive components, the impact of stiffer designs, due to tighter drift limits, on acceleration-sensitive components should be evaluated to ensure that a reduction in damage to drift-sensitive components does not lead to an increase in damage to acceleration-sensitive components.

Beyond the direct losses associated with repair costs, displacement of building tenants and disruption to business is another significant source of losses. The time needed to make the repairs required to achieve functional recovery is a better measure of the impact of tall buildings on the recovery of a city. As evident from Table 5, despite moderate economic loss (repair cost) levels, repair and functional recovery times are extensive. For instance, at Site Class D, the median losses in the RCSW residential building of 7.7% correspond to repair times of over 1.5 months, with functional recovery extending to 7.5 months. Similarly, at Site Class D under DE shaking, median losses in the BRBF office building of 3.1% correspond to a repair time of over 1 month and a time to functional recovery of approximately 5.5 months. Consistent with the smaller drift demands for Site Class B ground shaking, the data in Table 5 indicate significantly shorter repair and recovery times at Site Class B than at Site Class D, that is, enhanced versus baseline structural design.

The benefits of designing with recovery planning in mind are also reflected in the results shown in Table 5. 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. Measures (1) and (2) have already been implemented to some extent through San Francisco’s Building Occupancy Resumption Program (BORP) (Lang et al., 2018) and through arrangements that building owners frequently make with engineering consultants to be on retainer for conducting building inspections and designing necessary repairs. Measures (3) and (4), while not routinely available, are evaluated to explore their impact on expediting recovery. To the extent that effective pre-earthquake plans can be implemented and carried out, they can significantly reduce building functional recovery times—potentially cutting downtime in half for many cases.

Although disaggregating the contributors to downtime is not as straightforward as disaggregating losses, it is possible to identify the major contributions to repair and functional recovery times. As described previously, depending on the component type, the influence of 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 residential building subjected to DE ground motions at Site Class D, the major contributors to functional repairs and downtime are elevators, structural repairs, and mechanical equipment. For the BRBF office building subjected to DE ground motions at Site Class D, the major contributors are elevators, curtain wall, and mechanical equipment. Figure 9 summarizes the different contributors to functional recovery time, including the impeding factor estimates previously discussed, such as post-earthquake inspection, engineering mobilization, contractor mobilization, permitting, and financing for the RCSW residential and BRBF office buildings. The figures also illustrate the different sequences of
delays, which are quantified in the recovery “paths,” termed financing, engineering, and contractor. The results in Figure 9 represent a single realization of functional recovery time corresponding to the median loss realization of each building (RCSW residential and BRBF office) and design strategy (baseline and recovery planning) considered. The loss ratio associated with each realization is also noted in the figure.

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; and the financing path includes time to arrange financing of repairs. The total downtime estimate is the combination of the longest of these three recovery paths plus the repair time itself. For the baseline RCSW residential building, the longest sequence of delays comes on the contractor path. Comparing the left and right plots in Figure 9a, implementation of recovery-planning strategies can reduce median functional recovery

**Figure 9.** Downtime de-aggregation under Design Earthquake shaking with and without recovery planning in Site Class D corresponding to the median loss realization of the (a) RCSW residential and (b) BRBF office buildings.
time from approximately 6 months (177 days) to just over 3 months (94 days). Similar trends are seen in Figure 9b for the BRBF office building, where the longest sequence of delays comes on the engineering and permitting path, and where implementation of recovery-planning strategies can reduce functional recovery time from approximately 4.5 months (140 days) to just over 2 months (69 days).

**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. 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). Moehle et al. (2011) evaluated the costs associated with performance-based versus code-based design of tall buildings (consistent with the RCSW residential and BRBF office 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 NEHRP (2013) showed that the benefits associated with improved seismic design can be significant and cost beneficial (though that study was about areas of moderate seismicity and not specifically about tall buildings). 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 $5000 to $10,000 per year. These estimates were derived in consultation with design firms that provide such services (Arup and Magnusson Klemencic Associates, personal communication, August 2018).

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 feasibility and costs for having a contractor on retainer could vary significantly depending on the nature of the repair work (e.g. light partition wall damage vs severe structural damage). Similarly, while earthquake insurance is available to building owners, developing transparent estimates of earthquake insurance premiums for major buildings is difficult because (1) earthquake insurance is often lumped with insurance for other perils, (2) owners do not necessarily insure their entire asset value, (3) premiums are derived based on portfolio-level assessments (not individual buildings), and (4) earthquake insurance coverage varies considerably depending on the building ownership and management (e.g. multi-owner condominium buildings vs commercial office buildings owned by large real estate companies). 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 pre-
sented 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 move-
  ment 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 sys-
  tem 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.

**Conclusion**

Current design requirements for tall buildings are primarily intended to protect life safety
by minimizing the risk of collapse or significant structural or façade damage under extreme
(i.e. MCE\(_R\)) ground motions. While the requirements also include serviceability require-
ments 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), with much
lower intensities (by a factor of \(\sim 5\) at the sites considered) than that of the DE ground
motions (with roughly a 10% chance of exceedance in 50 years), which are typically used
to define recovery targets.

One of the key design criteria that controls the level of building damage under earth-
quake ground motions is the story drift ratio. Current building requirements for tall build-
ings limit the maximum story drift ratios to 3% under MCE\(_R\) 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 considera-
tions. A survey of recently constructed buildings in San Francisco, Los Angeles, Seattle,
and San Diego reveals their MCE\(_R\)-level drift ratios to range from 1% to 3%, with typical
values between 1.5% and 2%. For comparison, assuming that the DE drift demands
would typically be about two-thirds of the MCE\(_R\) level drifts, the surveyed MCE\(_R\) drift
ratios of 1%–3% would relate to DE drift ratios of 0.7%–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 indi-
cates 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 7.5 months, including time for
inspections, mobilization of engineering and contractor services, financing, and permitting. By employing BORP (Lang et al., 2018) 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 tentative recovery goal of “weeks” for multi-family residential buildings. 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 flexural wall elongation amplifies the racking drifts to about 1.3%. The calculated repair costs of about 7.7% of the building replacement value are not out of line with expectations for standard repair cost loss analyses. Damage and downtime analyses of a 40-story steel braced-frame office building yield slightly better performance, with an estimated repair time of roughly 1 month and functional recovery times of approximately 2.5–5.5 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.

Analyses 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 1 month and 2 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 half a month and 1 month, respectively, by limiting the building story drifts to 0.6% (about 30% of the code limit). These estimates approach 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).

The results of this study, which focus exclusively on central tendency statistical measures, that is, mean and median results, suggest that tall buildings designed per current minimum building code requirements may experience earthquake damage that will result in functional recovery times that significantly exceed 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, which is beyond the scope of the current study.

Based on the results of the buildings evaluated, the target of 1 month to functional recovery may be achievable by limiting story drift ratios to about 1% under the DE ground motions, roughly half the amount permitted by current building codes. Assuming that DE drift demands are roughly two-thirds of MCE_R drift demands, 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 elevators and other critical nonstructural components, (2) implementation of building recovery plans and other measures to mitigate impeding factors for recovery, and (3) provisions for possible disruption to utility service to the building. While this study explicitly studies the impact of reduced drift limits and recovery planning measures to minimize downtime, the impacts of extended utility disruptions are not considered. Furthermore, recommendations for enhanced nonstructural component design are informed by the observation that key contributors to functional repairs
and downtime in the buildings studied included elevators and mechanical equipment, as well as observations from other recently completed projects (e.g. Almufti et al., 2016) designed to achieve immediate occupancy.

In the absence of extensive empirical evidence from past earthquakes, this study uses performance-based engineering simulations to evaluate functional recovery times in modern tall buildings. Systematic earthquake reconnaissance data collection procedures for future events are needed to validate and refine the input assumptions of both the loss (FEMA P-58) and the downtime assessment (Modified REDi) results presented. While the downtime assessment methodology follows a logical sequence of component repairs and considers impeding factors that must be addressed before repairs can be initiated, future work should explore the use of more nuanced agent-based models which can more accurately account for region-specific constraints as well as other factors such as demand surge, which are only implicitly considered in this study. In addition, to the extent that the estimated functional recovery times are predicated on the buildings being unfit to function until they are repaired, research is needed to assess the safety required for reoccupancy and functionality of damaged buildings. Consensus-based models and criteria for post-earthquake building recovery simulation are needed to supplement FEMA P-58’s performance assessment calculation tool for damage and loss assessment.

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Supplemental material
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