



## SAFETY OF TALL PRE-NORTHRIDGE STEEL FRAME BUILDINGS AND IMPLICATIONS ON CORDONING AND RECOVERY

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### Abstract

The downtown business districts of San Francisco, Los Angeles and other cities in the western United States have significant inventories of older (pre-2000) tall steel buildings that have elevated risk, compared to new code-conforming buildings, of damage and collapse from strong earthquakes. Many of these buildings have welded connections of the type that experienced sudden brittle fractures during the 1994 Northridge earthquake, and they were designed under building codes with less stringent requirements on seismic drifts, minimum column strengths, and seismic base shears. Compounding these deficiencies are new seismic hazard models which indicate that long-period spectral accelerations and durations of earthquake ground motions are larger than considered in older design codes. This study applies newly developed models to assess the response of older steel buildings from the onset of damage up to collapse. The analysis results are used to examine issues associated with post-earthquake inspection and safety, which have implications on building reoccupancy and recovery. The study considers the implication of structural damage on recovery of both the tall building itself, as well as implications of safety cordons around the tall building on the re-occupancy and recovery of neighboring buildings. A method is proposed for quantifying the post-earthquake safety of damaged tall buildings to inform criteria for establishing safety cordons. The building-specific performance-based assessment method of FEMA P58 is extended and applied to quantify the post-earthquake recovery time for neighboring buildings. In addition to providing more comprehensive estimates of community downtime, this framework can include metrics for whether certain buildings are likely to induce inordinately long downtimes in surrounding buildings or how a cordoning strategy will affect a community. This type of downtime assessment can inform a range of policy and planning decisions, such as requiring seismic retrofit of vulnerable tall buildings, evaluating emergency response strategies, and developing post-earthquake recovery plans.

**Keywords:** seismic assessment, tall buildings, steel frame, cordoning, collapse safety, recovery

## 1. Introduction

Tall buildings play an important role in the socio-economic activity of major metropolitan areas in the United States, and their resilience is vital to ensuring an effective recovery after major disasters. Damage to tall buildings has the potential to affect a large number of people and can have significant consequences on surrounding areas. Events such as the Canterbury earthquake in 2011 have highlighted the impact of poorly performing buildings on the business continuity of downtown districts, where tall buildings are clustered together. Following the Christchurch earthquake, the 26-story Hotel Grand Chancellor sustained significant damage and residual drifts, prompting authorities to cordon off an area with a radius that was roughly equal to its height [1]. Thus, in addition to the direct economic losses in the Hotel Grand Chancellor, there were significant indirect losses attributed to business disruption in surrounding buildings.

Since the 1906 San Francisco earthquake, structural engineers have generally regarded steel moment-resisting frame (MRF) systems as being among the most ductile and reliable seismic force-resisting systems for buildings [2]. The common expectation was that, when subjected to earthquake shaking, MRFs would experience only localized damage due to ductile yielding of members and connections. This expectation led to widespread construction of this system, particularly in the high seismic regions of the western US from the 1960s through the 1990s [2,3]. The 1994 Northridge earthquake dramatically changed perceptions of the performance of such frames, after which post-earthquake inspections revealed cracking in the beam-to-column joint welds in several dozens of low- and mid-rise, steel-frame buildings [2].

Until about 15 years ago, most tall buildings in high seismic regions were designed using conventional building code approaches, following a prescriptive force-based approach, without an explicit assessment of nonlinear dynamic performance during major earthquakes. More recently, several jurisdictions, including Los Angeles, San Francisco and Seattle, have adopted performance-based approaches for the design of new tall buildings, which employ nonlinear analyses of response. While these approaches generally provide more reliability in terms of building safety in modern buildings, little is known about the seismic performance of older existing tall buildings that were designed prior to the adoption of modern design approaches [4], many of which employ seismically vulnerable steel MRFs.

Recent studies have highlighted the vulnerability of these existing tall steel MRF buildings in terms of structural performance, direct economic losses and downtime. For example, Lai et al. [5] evaluated the performance of an existing 35-story steel building, under the design basis ground motions in San Francisco, and observed the tendency to form weak-story regions, connections failures of brittle pre-Northridge welded beam-to-column connection details, and a high probability of brittle failures of column splices. Molina Hutt et al. [4], conducted a seismic performance assessment of a similar vintage building under design basis motions and determined an expected direct economic loss of over 30% of building replacement cost and downtime estimates of more than 1-1/2 years. Under higher intensity maximum considered earthquake ground motions, the building had a significant collapse risk. These studies demonstrated the capabilities of state-of-art methods, such as FEMA P58 [6] and REdi [7], to perform detailed assessments of individual buildings. However, these existing frameworks only consider individual building performance and neglect the impact of damage to these buildings on neighboring buildings and overall community resilience.

This study applies newly developed models to assess the response of older steel buildings from the onset of damage up to collapse. The analysis results are used to examine issues associated with post-earthquake inspection and safety, which have implications on building reoccupancy and recovery. The study considers the implication of structural damage on recovery of both the tall building itself, as well as implications of safety cordons around the tall building on the re-occupancy and recovery of neighboring buildings. A method is proposed for quantifying the post-earthquake safety of damaged tall buildings to inform criteria for establishing safety cordons. In addition to providing more comprehensive estimates of community downtime, this framework can include metrics for whether certain buildings are likely to induce inordinately long downtimes in surrounding buildings or how a cordoning strategy will affect a community. This type of downtime

assessment can inform a range of policy and planning decisions, such as requiring seismic retrofit of vulnerable tall buildings, evaluating emergency response strategies, and developing post-earthquake recovery plans.

While only two moderate size cities [15,16] have introduced requirements for assessment and retrofit of existing steel MRF buildings, San Francisco, Los Angeles and other cities do have ordinances that require assessments and retrofit during substantial building renovation. Looking towards the future, San Francisco's Earthquake Safety Implementation Program [8] has plans to develop assessment and retrofit requirements for existing tall steel buildings, which would be modeled after ordinances that are currently being implemented for soft-story and non-ductile concrete buildings. Just as with current programs for concrete buildings, the development and implementation of an effective program for tall steel buildings will require guidelines for screening and assessment of steel MRFs, along with practical and cost-effective retrofit solutions.

## 2. Characterizing the Risk to the Urban Built Environment

### 2.1 Influence of Tall Buildings on the Resilience of San Francisco

Since the late 1950s, San Francisco's skyline has changed dramatically with the construction of tall buildings in the downtown area. Among the multi-faceted earthquake risks facing the city, the concentration of tall buildings and infrastructure in the densely populated downtown neighborhood raises questions about the risks to life, property, and recovery from large earthquakes. As a first step towards addressing these questions, San Francisco's Tall Buildings Study [3] developed an inventory of the city's buildings that are taller than 240 ft. The inventory classifies tall buildings in terms of height, age, use, and structural system characteristics. Although tall buildings are not the only structures at risk, they are of special concern due to their large size and occupancies, where earthquake damage to one tall building can have disproportionate effects on its occupants, its neighbors, and the community at large.

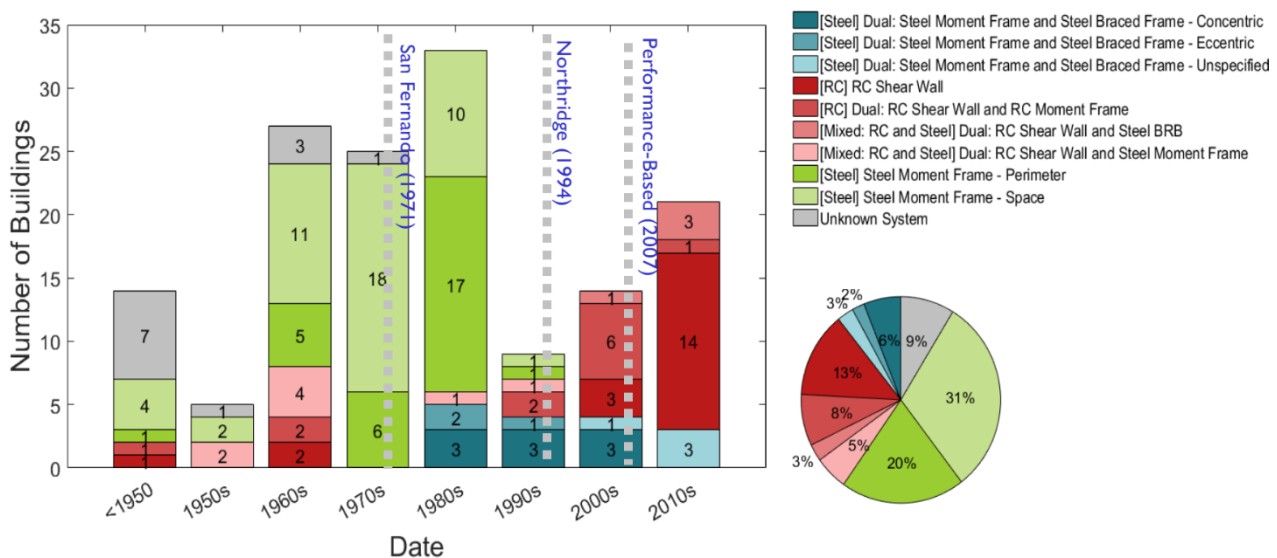


Figure 1 – Inventory of Tall Buildings in Downtown San Francisco [1]

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. In terms of 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. With regards to occupancy, approximately 60% of the buildings are commercial and just under 40% are residential or hotel. With regards to age of construction, 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. Illustrated in Figure 1 are the number of tall buildings that were constructed in San Francisco over the past sixty years, differentiated by structural materials (steel or concrete)

and structural system types. In terms of 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.

With regards to structural configuration, about 50% of the San Francisco tall building inventory is comprised of 65 steel MRF tall buildings constructed from the 1960s to the 1990s. These buildings are of interest due to: (1) their prominence as one of the most common structural system types in the tall building inventory; (2) their design, which followed an equivalent lateral force procedure based on the first-mode translation response, without capacity design principles that protect against story mechanisms, and lower base-shear strengths than those specified in modern building codes; and (3) concerns regarding the potential for fracture-prone welded connections, which came to light following the 1994 Northridge earthquake.

## 2.2 Benchmarking Older vs. Modern Steel Moment Resisting Frames

It is well established that changes in commonly used weld processes during the mid-1960s led to welds with low toughness, as evidenced by weld fractures observed in the 1994 Northridge earthquake [2]. Therefore, it is believed that fracture-prone pre-Northridge moment connections are present in many of the steel MRFs within the inventory. As outlined in [4], a comparison of pre-1990s Uniform Building Code seismic design requirements against modern design standards highlights a number of design considerations not present in older standards, including: (1) use of response spectrum analysis method as opposed to equivalent lateral force procedures; (2) consideration of lateral forces acting simultaneously in both building directions; (3) minimum base shear requirements; (4) consideration of p-delta effects; (5) consideration of accidental torsion and vertical and horizontal irregularities; (6) strong column weak beam requirements; (7) panel zone design checks; (8) capacity design principles; and (9) prequalified seismic connection details.

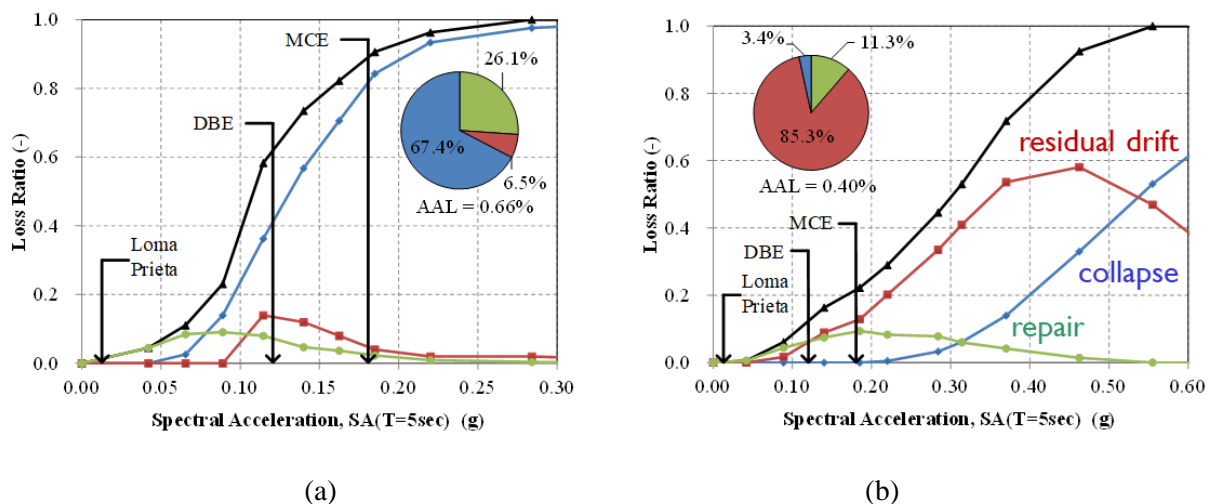


Figure 2 – Comparison of Loss Functions (a) 1973 Steel Moment Frame, (b) 2015 Steel Moment Frame [4]

While steel MRFs 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. [4] suggests that the older (pre-1994) steel MRFs with fracture prone connections could have a mean annual frequency of risk of collapse 28 times higher than new code conforming buildings, or approximately 13% versus 0.5% probability of collapse in 50 years. The study also found a 65% increase in average annual risk of economic loss, as illustrated in Figure 2, and a twofold increase in downtime to re-occupancy and functional recovery. The modest differences in annualized loss and downtime between older and modern steel MRFs are in rather stark contrast to the significant changes in collapse risk. The likelihood of observing collapse in older steel MRF buildings is comparable to the likelihood of demolition due to

excessive residual drifts in modern ones. While these outcomes have distinct impacts on life safety, they have similar implications on cost and downtime (i.e. total replacement cost and time).

Building performance evaluations at the design basis earthquake (DBE in Figure 2) and the maximum considered earthquake (MCE in Figure 2) shaking intensities further suggest that pre-1994 tall steel MRFs have much higher risks of collapse under extreme ground motions and of damage and building closure in moderate earthquakes. Furthermore, while modern building code requirements provide acceptable seismic collapse safety (negligible collapse risk), they do not necessarily ensure a level of damage control to assure a swift recovery after a damaging earthquake due to extensive downtime.

### 3. Nonlinear Response Simulation and Retrofit of Steel Buildings

#### 3.1 Simulation of Fracture Critical Beam-Column Connections

A key challenge in assessing the performance of pre-Northridge type steel frames is being able to accurately simulate the degrading response of fracture-critical welded connections. As illustrated in Figure 3, one approach for modeling the connections is through a fiber-hinge, where the fibers are defined to simulate (1) yielding and fracture in the welded beam-flange to column joint, and (2) yielding, bolt shear and/or tear out of the bolted shear-tab web connection. An overview of this approach is described in guidelines for nonlinear analysis of steel moment frames [9]. The equivalent fracture stress ( $\sigma_{cr}$ , ksi) in the welded flange connection (Figure 3) is described by the following fracture mechanics-based equation:

$$\sigma_{cr} = K_{IC} / (C + 2a_0) \quad (1)$$

where  $a_0$  (inches) is equal to the depth of the weld root flaw,  $C$  is equal to 1.2 or 0.5 for the lower and upper beam flange weld, respectively, and  $K_{IC}$  (ksi $\sqrt{\text{in}}$ ) is the fracture toughness, which can be determined from

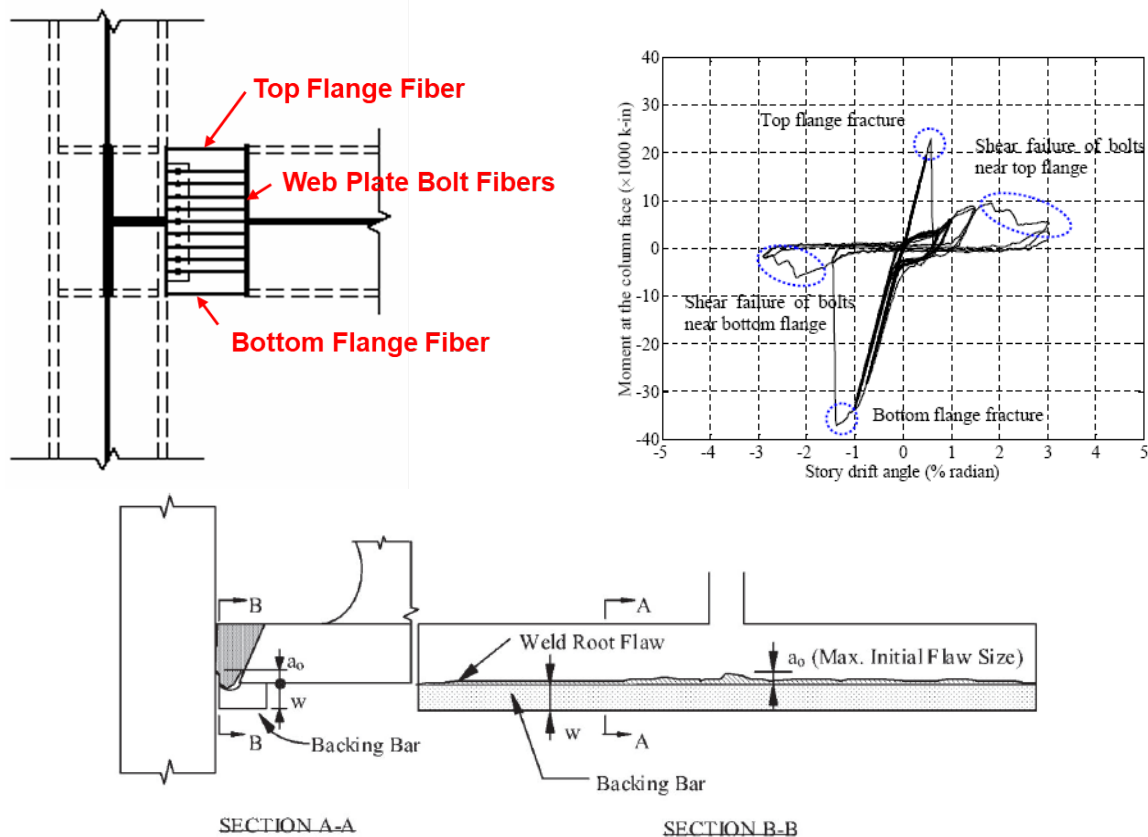


Figure 3 – Fiber Hinge Model of Steel Connection with Weld Fracture and Bolt Failure [9]



Charpy V-Notch (CVN) measurements of the weld metal. Equation 1 applies to welded flange joints, where the weld backing bars are left in place and the weld root flow  $a_o$  is limited to the lesser of one half of the flange thickness or 0.5 inch. For pre-Northridge connections, typical values of  $a_o$  are on the order of 0.1 to 0.2 inches and  $K_{IC}$  ranges from 50 to 100 ksi $\sqrt{\text{in}}$ .

To account for the cumulative effects of low-cycle fatigue behavior, fracture of the flange fiber is evaluated using the following damage index ( $DI$ ), which is an adaptation of Miner's rule [10]:

$$DI = \sum_i \left( \frac{\sigma_i}{\sigma_{cr}} \right)^{1/m} \quad (2)$$

where  $m$  is a parameter that is calibrated to a parametric equation that accounts for the material toughness (CVN) and flaw size ( $a_o$ ). For random loading cycles, the damage index is evaluated using rain-flow counting [11]. The cyclic fatigue-fracture model is implemented in OpenSees [12] in series with the Steel02 Menegotto-Pinto material model to simulate the inelastic response of beam flange welds. Shown in Figure 4 is an example of the resulting connection model, where the plot on the upper left shows the overall moment versus rotation response of the connection (superimposed over data from a connection test), and the plots on the right side show the equivalent uniaxial stress-strain response of the beam flanges, accounting for weld fracture in the upper flange.

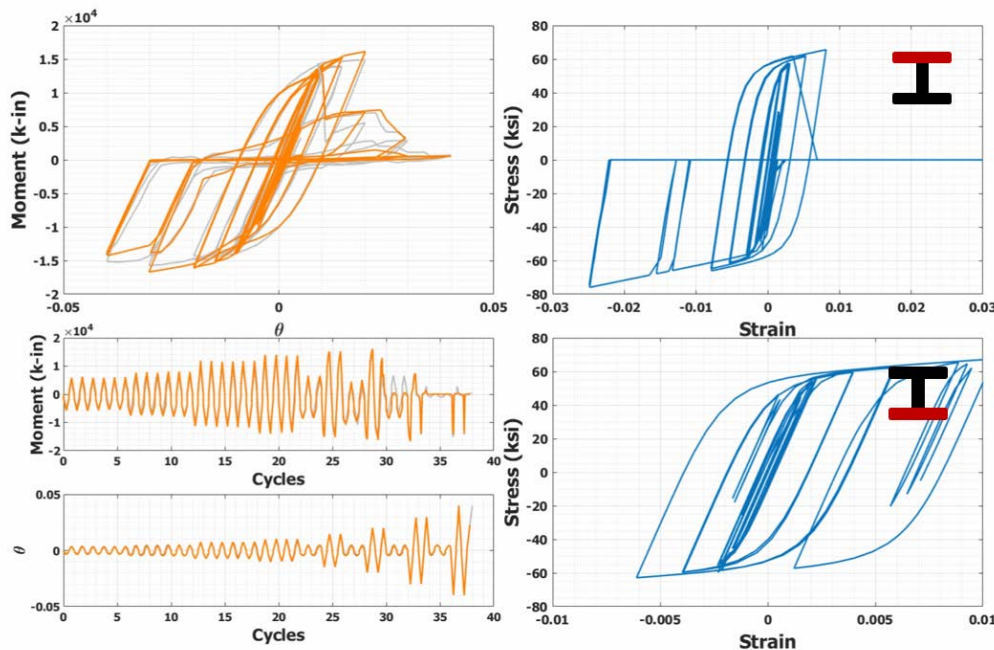


Figure 4 – Simulation of Steel Connection With Cyclic Damage Model

### 3.2 Case Study Building: Numerical Simulations vs Observations following Northridge Earthquake

The proposed connection model was incorporated in the analysis of a 17-story steel moment frame building that experienced connection fractures in the Northridge earthquake, to further evaluate and illustrate the model's capabilities. Details of the building and the observed connection fractures are described in a previous analysis study of the frame by Chi et al. [13]. In contrast to the analyses that Chi et al. conducted twenty years ago, which were limited in their ability to directly simulate weld fractures, the proposed connection model allows direct simulation of weld fractures, based on the assumed weld toughness and initial flaw sizes. Shown in Figure 5 is a comparison of the observed connection damage and the simulated connection fractures and story drift ratios, where the simulation was run using a ground motion that was recorded nearby to the building.

Overall, the results agree well, where the peak story drifts were consistent with previous frame studies and the locations of observed connection weld fractures agree well with the observed damage.

Among many questions that one can ask about the building performance, an important consideration is whether the building is safe to reoccupy prior to and during repairs. Whereas many of the steel moment frame buildings that were damaged in the Northridge earthquake remained occupied, the same decision may not be made today, considering (1) what we know about the widespread extent of the fractured connections, (2) greater appreciation about the risk of large aftershock motions, and (3) overall greater risk aversion. The FEMA 352 guidelines for post-earthquake evaluation and repair of welded steel moment frames [2] include a damage index for connections, which, when applied to the frame shown in Figure 5, would suggest that the frame is on the border of being safe to occupy during repairs. Whereas the FEMA 352 criteria for occupancy were based largely on expert judgment, it is now possible to better quantify the collapse safety, based on detailed nonlinear analyses coupled with rigorous earthquake hazard and risk assessment.

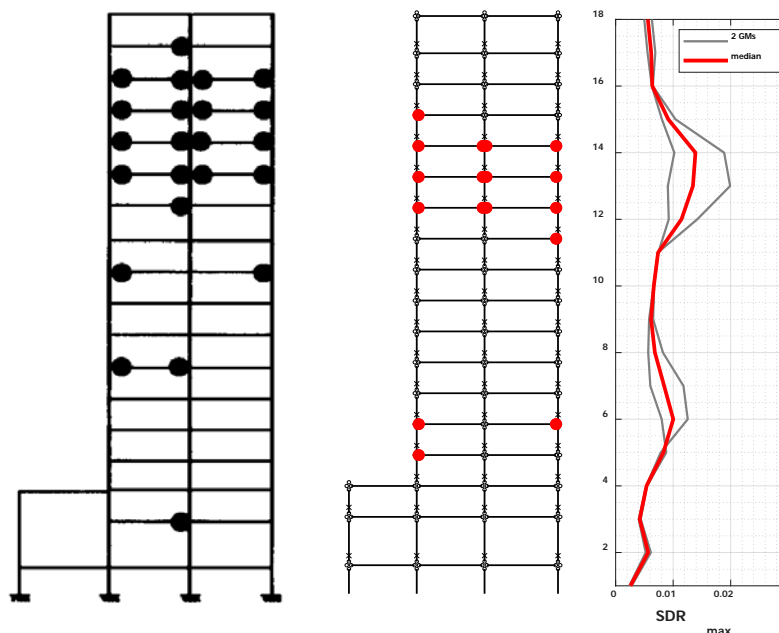


Figure 5 – Comparison of observed connection damage (left) and simulated response and damage (right) for a seventeen story steel moment frame

### 3.3 Seismic Retrofit

The large population of pre-Northridge steel moment frames in San Francisco, Los Angeles, Seattle and other west coast cities, combined with (1) experiences of recent devastating earthquakes in New Zealand, Chile, and Japan and (2) community interest in earthquake resilience, has prompted discussions of whether these buildings should be retrofit [14]. The Cities of Santa Monica and West Hollywood have already adopted seismic retrofit ordinances for older steel moment frames [15,16], the City of San Francisco is evaluating development of an ordinance [8], and other cities are considering retrofit programs. Moreover, recognizing the significant investment in their buildings and valuing the safety of their employees or tenants, some building owners have already implemented voluntary seismic retrofits [14].

Seismic retrofit of older steel moment frame buildings can cost tens of millions of dollars in direct construction costs, in addition to the associated project costs due to disruption of operations, relocation of tenants, etc. As such, seismic retrofit should be undertaken carefully. Seismic retrofit solutions can vary significantly, depending on the specific configuration of the building, the goals of the seismic retrofit (basic life safety versus higher performance), and other factors, such as construction logistics and whether the building is undergoing

other major renovations or will be occupied during construction. In tall buildings, the seismic retrofit work may be staged over several years to minimize disruption to tenants by timing the work around lease renewals.

Design of seismic retrofits generally includes detailed nonlinear dynamic analyses to evaluate the seismic performance under the existing conditions and the effectiveness of the proposed retrofit. For pre-Northridge moment frames, areas of specific concern in the analyses include (a) modeling of the fracture-critical beam-to-column and column splice connections, (b) identification of weak stories or vertical irregularities, due to extra tall stories at ground floor lobbies or other locations, (c) in-plan irregularities introduced by large atria or other irregular floor plans, and (d) degradation due to non-compact shapes or members with large out-of-plane slenderness.

There are typically many possible retrofit strategies, the feasibility and effectiveness of which will depend on the specific circumstances. Some of the common strategies that have either been implemented on real projects or explored in research studies include:

- Adding viscous dampers or buckling restrained braces to stiffen the frame and reduce deformation demands
- Adding reinforced concrete shear walls or steel bracing (usually in the building core) to either supplement or replace the lateral stiffness of the moment frame
- Adding pivoting or rocking concrete walls or steel braced spines to regularize and reduce excessive story drifts
- Rewelding existing welded beam-column and/or column splice connections with toughness-rated materials to improve their fracture resistance
- Reinforcing or reconfiguring existing welded connections to improve their strength and fracture resistance

## 4. Risk and Policy Implications for Cities

### 4.1 Collapse Risk and Reoccupancy

Looking beyond the primary concern for adequate collapse safety of buildings in earthquakes, a further question that can affect retrofit policy decisions is whether buildings that are damaged by earthquakes are safe to reoccupy. Answering this question requires (1) evaluation of the collapse safety of damaged building to future earthquake ground shaking or other loading effects, and (2) criteria to judge the minimum acceptable level of required safety, recognizing that even an undamaged existing building is likely to have lower collapse safety than is generally deemed acceptable for new buildings. Thus, with respect to safety from aftershocks, one can approach the evaluation in either an absolute sense (i.e., what is the risk of collapse due to earthquake aftershocks) or in a relative sense (i.e., how much less safe is the damaged building to aftershocks as compared to the undamaged building).

Several prior studies to examine collapse safety of damaged buildings [17,18,19] have proposed to evaluate post-earthquake safety in terms of changes to the collapse fragility as a function of damage introduced into the structure from an earthquake. Following an approach proposed by Burton et al. [18], this study used nonlinear analyses of a 20-story reinforced concrete moment frame as a trial study to examine the effect of earthquake damage on collapse safety. These consisted of, first, subjecting the model to a damaging input ground motion (referred to herein as a ‘foreshock’), and then, performing incremental dynamic analyses to evaluate the collapse fragility. Shown in the left plot of Figure 6 are collapse fragility curves for the (1) undamaged frame, and (2) frames that had experienced foreshock ground motions that induced specific levels in peak story drifts. As expected, the collapse fragility curves (Figure 6a) shift to the left with increasing foreshock intensities, as measured by the story drift demands induced by the foreshocks. Note that the specific levels of peak foreshock story drift demands for each of the collapse fragilities in Figure 6a are identified by the corresponding points in Figure 6b, which relate the reduction in median values of collapse intensity to the foreshock story drift demands.



As shown in Figure 6, the median values of collapse capacity reduce to about 60% of the value of the intact frame due to damage induced by foreshocks with peak story drift demands of 5%. This shift in median translates to a large increase in the probability of collapse (reading vertical statistics from the collapse fragilities in Figure 6a). For this illustration, peak story drift is considered as a fairly robust metric of induced damage, which is easy to measure from the analyses. Ongoing efforts will look at other measures, such as residual drifts, fractured connections, etc. that would be physically observable damage measures. With this type of data, one could then make a determination regarding the minimum threshold of observed damage that would reduce the collapse safety below a minimum acceptable level. This analysis would provide a more quantitative basis to support existing criteria for building reoccupancy, which up to this point has been based largely on engineering judgement.

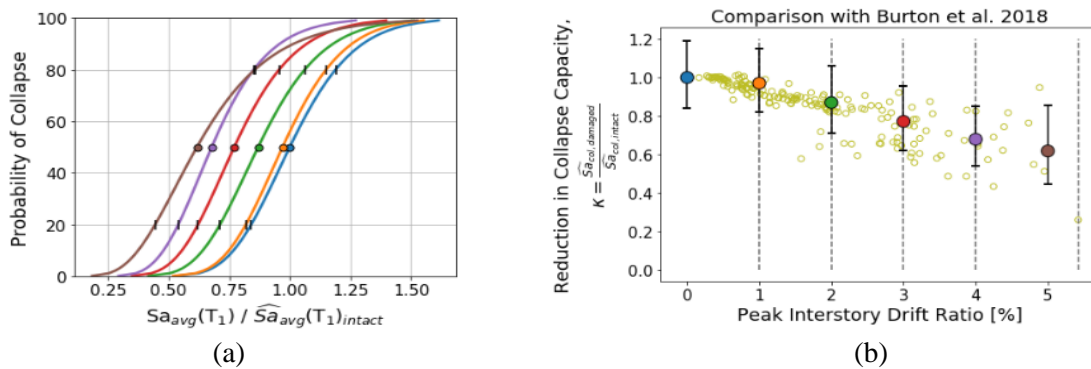


Figure 6 – Assessment of Post-Earthquake Building Collapse Safety (a) Normalized Collapse Fragilities and (b) Relationship Between Peak Foreshock Story Drifts and Reduction in Median Collapse Capacity

## 4.2 Safety Cordons

Along with reoccupancy of a building, an associated question is the risk that a damaged building poses to its neighbors. Where the risk from a damaged building is deemed too large, safety cordons are established to restrict access around the damaged building. While the cordons may be necessary for life-safety protection, they can significantly impede the recovery of neighboring buildings and the community in general. This was seen most dramatically in Christchurch, NZ following the 2011 Canterbury Earthquake, where entire regions of the downtown area were cordoned off for many months. As illustrated in Figure 7 for downtown San Francisco, the potential impact of cordons can be quite dramatic in dense urban downtown areas with many tall buildings.

This study developed a new community recovery assessment framework that includes the effect that safety cordons around damaged buildings may have on the accessibility of neighboring buildings. This concept is in contrast to current downtime models that consider communities as the sum of isolated buildings. The framework incorporates FEMA P-58's high resolution, building-level performance assessment methodology within a spatially distributed recovery model. Each building is evaluated at a range of shaking intensities, using FEMA P-58 to get the distribution of potential building consequences, including the level of damage and the time required to stabilize the building (in case it requires a cordon) and the time required for repairing it to a functional status. The regional hazard is based on earthquake scenarios (e.g. 7.2Mw on the San Andreas Fault), simulated through many possible ground motion realization maps to capture the uncertainty in the shaking intensity associated with a specific rupture magnitude and location. These hazard maps provide the shaking intensities at each building location, which are then used to sample from the distribution of building consequences for that shaking intensity. After sampling building damages consequences across the community, the framework uses REDI's [7] impeding factor model to consider the logistical delays prior to initiating each building's repairs. The cordon assessment is incorporated into this impeding factor model as follows. A cordon is established around any tall building (240ft+) that exceeds the cordon trigger criteria. (For the current example, the triggering threshold is set as 2% peak story drift demand). The duration of the cordon is equal to the total time required to stabilize the building. Just as all the buildings must wait until their logistical

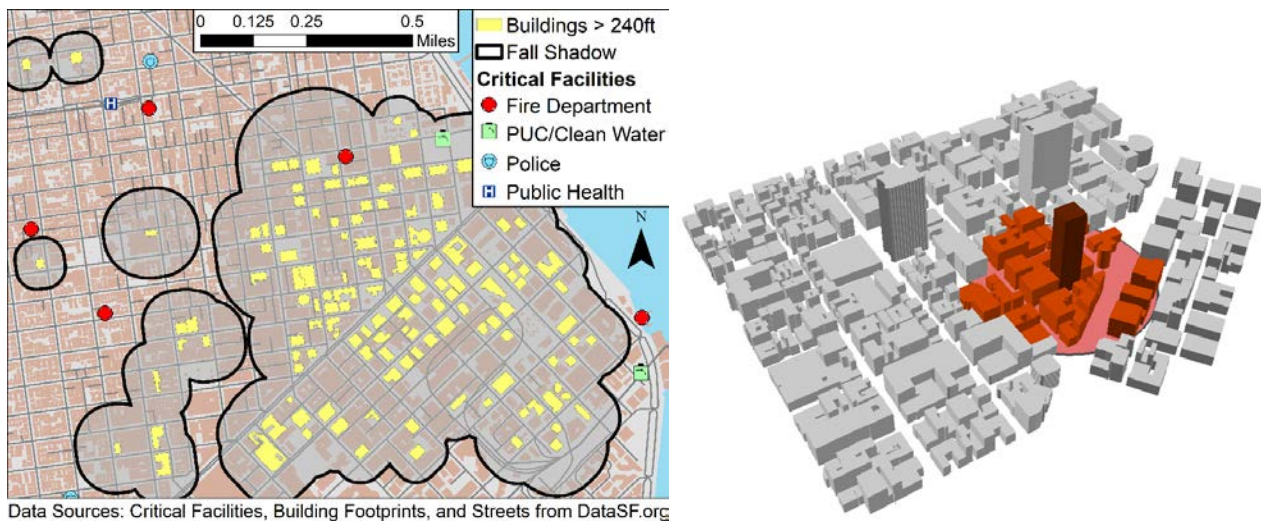


Figure 7 – Potential Extent of Cordoning Around Damaged Tall Buildings in Downtown San Francisco

impeding factors are resolved, any building that is located within a cordon must also wait until the cordon is removed before initiating repairs (or, in the case of an undamaged building, before the building can be reoccupied). Every building in the affected area is evaluated in this way to determine the time at which they are functional again. This recovery simulation is repeated for all realizations of the ground motion maps. Figure 8 shows the expected recovery over all the realizations in downtown San Francisco, where functionality measured as the percentage of available office space in the community. As illustrated in the figure, the presence of cordons (shaded area in orange) can have a significant effect on displacing long-time residents.

This type of assessment can provide a direct comparison to recovery targets, such as the time until a certain level of functionality is restored. There will often be a gap between the status quo and the recovery targets. In such cases, the recovery can be re-assessed under various policy options in order to evaluate the policies' efficacy in reducing the gap. The policies may include pre-earthquake planning for mitigating the logistical delays before the damaged tall buildings can be stabilized, accepting a higher level of risk when establishing the cordon criteria, or requiring retrofits to limit the potential for significant damage in tall buildings. The framework could also be used to in conjunction with a map of emergency routes to determine which routes are least likely to be inaccessible due to the location of the safety cordons (or which buildings need to be addressed in order to keep the emergency routes open).

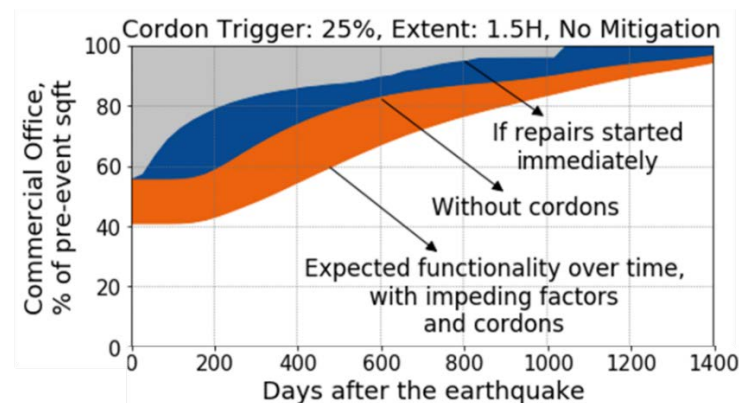


Figure 8 – Effect of Repairs, Impeding Factors and Cordons on Post-Earthquake Recovery of Commercial Office Function

## 4.2 Policy Interventions

As noted previously, two smaller cities in California have recently implemented policies for seismically retrofitting pre-Northridge steel moment frames to mitigate life safety risks from collapse, and San Francisco and other cities are continuing to evaluate policies to address the risk posed both to life safety and to recovery. Proposed policies for retrofit range from (1) mandatory requirements for screening and evaluation of all buildings, (2) mandatory requirements that are triggered by building code provisions related to major renovations, change in building use, and post-earthquake repairs, and (3) voluntary provisions with appropriate incentives (e.g., tax credits, zoning variances, etc.). Apart from structural retrofitting, there are other programs and interventions that could be employed to mitigate the effects of building damage on post-earthquake recovery. These include requirements for (1) building owners to develop earthquake recovery plans with the goal toward reducing impeding factors for inspection and repairs to stabilize and reoccupy the building, (2) installation of seismic instrumentation to facilitate post-earthquake evaluation of structural building damage.

## 5. Concluding Remarks

While the risks posed by older steel frame buildings are generally not as serious as those associated with unreinforced masonry or non-ductile concrete buildings, they nevertheless are a significant concern both for life-safety and post-earthquake recovery. The risks are particularly significant in cities with large numbers of tall pre-Northridge steel frame buildings, where damage to multiple buildings in close proximity can cause disproportionate disrupting effects, exacerbated by safety cordons and long recovery times. While the Northridge and Kobe earthquakes led to major building code changes to improve the reliability of new buildings, remarkably little has been done over the past quarter century to (1) quantify the risk posed by large inventories of existing steel frame buildings and (2) develop appropriate policy interventions to mitigate the risks. Recent initiatives to more proactively address disaster recovery and community resilience suggest that the situation may be changing. The research described in this paper is one effort to develop quantitative models and tools to help engineer effective policy interventions.

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