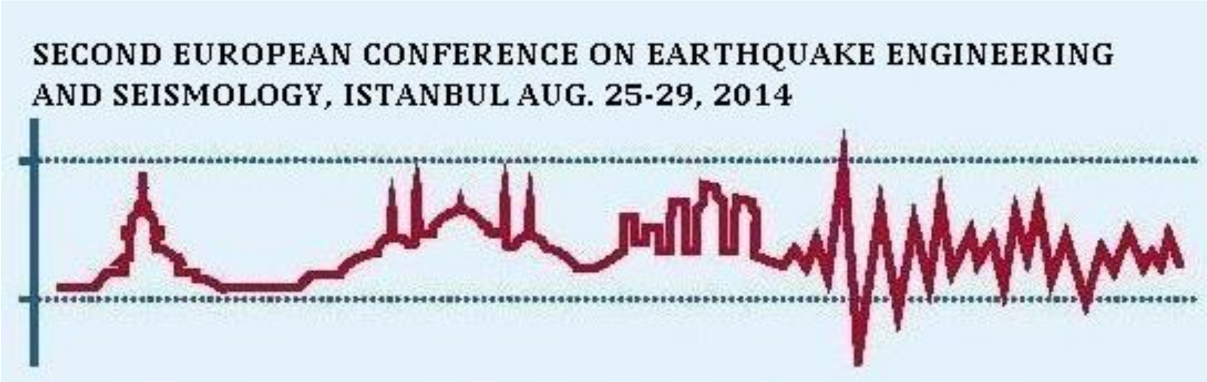

SEISMIC LOSS AND DOWNTIME ESTIMATES OF EXISTING TALL BUILDINGS AND STRATEGIES FOR INCREASED RESILIENCE

Carlos Molina Hutt[1], Ibrahim Almufti[2], Michael Willford[3] and Gregory Deierlein[4]

ABSTRACT

Tall buildings play an important role in the socio-economic activity of major metropolitan areas. The resilience of these structures is critical to ensure a successful recovery after major disasters. Events such as the Canterbury earthquake in 2011 have highlighted the impact of poor performing buildings on the business continuity of downtown districts, where tall buildings are typically clustered together. Following the 2011 earthquake, Christchurch's Central Business District (CBD) red zone covered a significant area of the city and more than 60% of the businesses were displaced (CERC 2012).

Until the introduction of Performance Based Seismic Design (PBSD) in the 1990s, buildings were designed using conventional building codes, which follow a prescriptive force-based approach based on the first mode translational response of the structure (FEMA 2006). Researchers and engineers have raised concerns that the prescriptive approach of building codes is not suitable for tall building design due to the significant contribution of higher mode effects (PEER 2010a). As a result of these shortcomings, several jurisdictions in areas of high seismicity throughout the Unites States (e.g. Los Angeles and San Francisco) have adopted a PBSD approach for the design of new tall buildings. While new designs follow a more adequate approach, little is known about the seismic performance of older existing tall buildings that were designed prior to the adoption of PBSD (Almufti et al. 2012).

This paper presents an assessment of the seismic performance of existing tall buildings in a case study city, San Francisco, where an archetype tall building is designed based on an inventory of the existing tall building stock. Non-Linear Response History Analysis (NLRHA) are conducted with ground motions representative of the design earthquake hazard level defined in current building codes, with explicit consideration of near-fault directivity effects. In order to influence decision making, performance is reported as the expected consequences in terms of direct economic losses and downtime. Once the performance of the archetype building is assessed, a range of structural and non-structural enhancements are explored for enhanced performance as well as mitigation measures for increased resilience. Expected direct economic losses for the archetype building are in the order of 34% of building cost and downtime estimates for functional recovery are 87 weeks. The strategies presented in this paper enable up to a 92% reduction in losses and minimize downtime for functional recovery to 1 day.

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INTRODUCTION

Seismic resilience – defined by the San Francisco Planning and Urban Research Association (SPUR) as the ability to respond to an earthquake emergency and to recover without lasting disruption– can be measured by the speed and completeness with which essential functions, and eventually routine operations, are restored. Earthquake resilience goes hand in hand with building performance (SPUR 2009). As tall buildings play a key role in the socio-economic activity of major business districts, the resilience of these structures is critical to ensure a successful recovery after major disasters. Furthermore, because much of the existing tall building stock was designed following guidelines that do not provide an understanding of seismic performance (FEMA 2006), quantifying the performance of these structures, not only in terms of direct economic losses, but also in terms of downtime, is critical in ensuring the resilience of major metropolitan areas.

Engineering seismologists, seismic, structural and geotechnical engineers have demonstrated that through Performance Based Earthquake Engineering (PBEE), detailed assessments of individual building performance can be conducted through complex NLRHA. These tools can be used for new design or assessment of existing buildings in order to ensure adequate performance under seismic events of a specified return period. Furthermore, estimates of the monetary losses associated to the performance of the building can be conducted through tools such as Performance Assessment Calculation Tool (PACT) (ATC 2012). Recent developments (Almufti and Willford 2013) have also been made in order to provide a direct measure of resilience through a downtime assessment methodology, which identifies the likely causes of downtime such that these can be mitigated to achieve a more resilient design.

Previous studies have assessed the performance of existing buildings in the western United States (Muto and Krishnan 2011, Gupta and Krawinkler 1999), but these studies were limited to buildings with maximum heights of 20 storeys and focused on structural performance assessment alone. While structural performance provides valuable information for the structural engineering community, it fails to provide measures of risk associated with direct economic losses, which are typically used by commercial lenders and real estate investors for decision making. Other studies have assessed the performance of tall buildings up to 40 storeys (Jayaram and Shome 2012) and estimated economic losses associated with building performance (Shome et al. 2013), but employed simplified structural models that do not enable the study of detailed retrofit schemes for enhanced performance. Furthermore, while these studies provide a measure of direct economic losses, they do not quantify resilience.

This paper assesses the performance of existing tall buildings in a case study city, San Francisco, through the development of a database of the existing tall building stock, the design of a representative archetype building, performance assessment via NLRHA with ground motions representative of the design earthquake hazard level and estimation of associated direct economic losses and downtime. Once the performance of the archetype building is assessed, a number of strategies for increased resilience are proposed including structural retrofits, non-structural enhancements and mitigation measures. The work conducted to date provides a good understanding of the expected performance of an archetype tall building in the city of San Francisco under a design level earthquake. It also illustrates that a number of measures can be implemented to increase the resilience of these existing buildings.

METHODOLOGY

The first step of the methodology is to develop a database of the existing tall building stock for the city of interest. This database includes information about each building, such as address, year built, number of storeys, lateral resisting system, whether retrofit or upgrades have been conducted, building use, etc. When possible, existing building drawings are reviewed. However, such reviews are not always feasible due to the lack records or public access to these. The purpose of the database is to develop archetype building designs that are representative of the existing tall building stock. These archetype buildings are developed by disaggregating the existing tall building database in order to identify trends. Archetype buildings focus on the following main variables: year of construction (to identify relevant building codes for design and typical construction of the time), lateral resisting system type and number of storeys. These key variables influence building performance and are therefore critical in developing archetype buildings for design.

The second step is to visualize the geographical location of the existing tall building database in a Geographical Information System (GIS) tool in order to identify representative sites. The purpose of identifying representative sites is to conduct Probabilistic Seismic Hazard Analysis (PSHA). Soil data at the representative sites is of relevance as it influences the site specific hazard. This paper presents the results of an intensity based assessment (single intensity level) under a design level earthquake. An array of ground motions needs to be selected, scaled and modified at the selected intensity level. These motions are then utilized to evaluate the seismic demand of buildings through NLRHA simulations.

The third step is to develop a numerical model for the archetype building in order to conduct NLRHA. The analytical models are three-dimensional and represent all components and force and deformation characteristics that significantly affect the seismic demands. Element properties are based on expected values of strength (PEER 2010b) to capture anticipated behaviour.

The fourth step is to develop a building performance model - defined as a model to assess the probability of earthquake losses and downtime. The building performance model includes all structural and non-structural components that are susceptible to earthquake damage. It enables calculation of direct economic earthquake losses and downtime estimates.

Based on the structural performance, loss assessment and downtime estimates for each archetype building, strategies for increased resilience are developed. These strategies include structural retrofits, adoption of non-structural building components that are more resilient to earthquake damage or a combination of these. Analytical models for structural retrofit strategies are developed and re-analysed to quantify the reduction in seismic demands. Revised building performance models are developed including all enhanced non-structural components and the loss assessments are revisited to quantify the reduction in losses associated with these components. Similarly, downtime estimates are re-examined in order to quantify the impact of these strategies on the resilience of the building

The city of San Francisco is selected as a case study due to a number of factors. San Francisco is one of the most seismically vulnerable cities in the world due to its large number of older buildings and proximity to major active faults. The city has a large number of existing tall buildings that were designed from the 1960s to the 1990s following prescriptive code guidelines that do not provide an explicit understanding of building performance in earthquakes. Additionally, past earthquake events, such as the Northridge earthquake in 1994, highlighted deficiencies in the moment resisting connections of steel Moment Resisting Frames (MRF), which is a lateral resisting system adopted by many of the existing tall buildings in the city.

EXISTING TALL BUILDING DATABASE AND ARCHETYPE BUILDING

A database of the existing tall building stock in San Francisco was developed in collaboration with Arup and the Structural Engineers Association of Northern California (SEAONC) Committee on Performance Based Design of Tall Buildings. This database includes all buildings in San Francisco taller than 48.8 m (160 ft). This threshold height is selected because it is the limiting height above which only certain types of lateral resisting systems are permitted for building structures according to current building codes (ASCE 2010). The database tabulates building characteristics by location, height, number of storeys, year built and lateral system type. Approximately 240 buildings greater than 48.8 m (160 ft) in height are identified. A map of downtown San Francisco illustrating the location of the existing tall buildings in the database is shown in Figure 1.

In order to select a prototype building for this study, the data from the existing tall building database is disaggregated to identify trends based on the time of construction and the lateral resisting system type. More details on the existing tall building database can be found in Almufti et al. (2012). This data reveals that the steel MRF system is the most prevalent type in pre-1990s construction for buildings greater than 35 storeys in height.

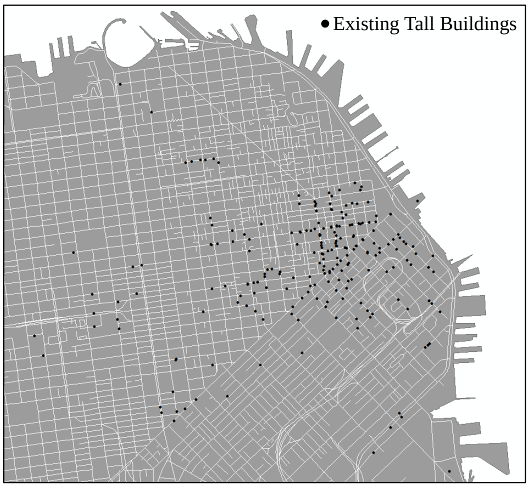

Figure 1. Map of downtown San Francisco illustrating the location of buildings included in the existing tall building database.

Based on the existing tall building database, a 40-storey steel MRF is selected as a representative archetype tall building. The prototype building attempts to represent the state of design and construction practice from the mid-1970s to the mid-1980s. The design of the prototype building follows the provisions of the Uniform Building Code of 1973 (UBC 1973) and the 1973 Structural Engineers Association of California (SEAOC) Blue Book (SEAOC 1973), which was commonly employed to supplement minimum design requirements. Detailed information on the design of the archetype building, including gravity and lateral design forces, deflection limits and section sizes can be found in Almufti et al. (2012). An illustration of the archetype building can be found in Figure 2.

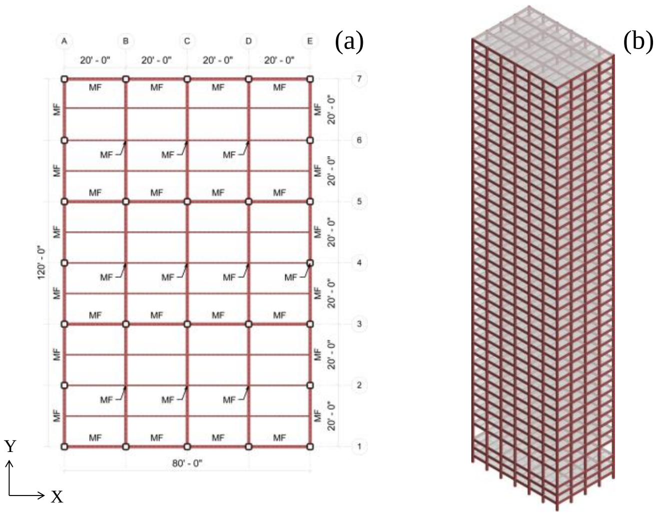

Figure 2. Prototype 40-storey office building plan (a) and isometric (b).
Source: Almufti et al. (2012).

ANALYTICAL MODEL

The analytical models must represent all force and deformation components characteristics that can significantly affect the seismic demands. To ensure such components are included in the analytical model, typical member sizes and connection details of the archetype building are verified against available existing building drawings. Consistent with these records, built-up box columns and wide flange beams are selected for the prototype building. Potential deficiencies of typical connection details are also assessed. The fracture prone pre-Northridge moment connections are commonly observed in these drawings. The switch in the weld process that led to welds with very low toughness, as evidenced by fractures observed in the 1994 Northridge earthquake, took place in the mid-1960s (FEMA 2000). Additionally, steel building designs in the 1970's did not include consideration of panel zone flexibility or strong column-weak beam principles. Krawinkler's panel zone model was not developed until 1978 (PEER 2010b), and strong column-weak beam requirements were not introduced in the UBC provisions until 1988 (Lee and Foutch 2000). Column splices are typically located 1.2 m (4 ft) above the floor level approximately every three floors. Observed typical splice connection details consist of partial joint penetration welds of half the thickness of the smaller section being connected. When subject to tensile forces, these splices can only carry half the capacity of the smallest section size being connected. Similarly, if subject to pure bending, these splices can only carry a fraction of the moment capacity of the smallest column. Experimental tests on heavy steel section welded splices have illustrated sudden failures with limited ductility (Bruneau and Mahin 1990). Based on this evidence, column splice failures are also considered in this assessment.

NLRHA is the best tool currently available for accurately predicting building response at varying levels of ground motion intensity. NLRHA aims to simulate all significant modes of deformation and deterioration in the structure from the onset of damage to collapse (PEER 2010b). Analytical component models utilized to represent the non-linear behaviour of the structure include non-linear columns, beams (including a fracture model in the connections), panel zones and splices. Concrete slabs are modelled as elastic cracked concrete 2D shell elements to represent the flexible floor diaphragm. A detailed discussion on the development of the different non-linear components of the analytical model for the archetype building can be found in Molina Hutt (2013).

SEISMIC HAZARD AND GROUND MOTION SELECTION

The majority of tall buildings in San Francisco are clustered in the downtown area, located approximately 14 km from the San Andreas Fault and 16 km from the Hayward Fault. A site specific Probabilistic Seismic Hazard Analysis (PSHA) is conducted at a representative site. The representative site is selected as the San Francisco Transbay Transit Center development due to its proximity to the majority of existing tall buildings and large amounts of relevant soil data available due to recent developments in the area. The representative site is shown in Figure 3. Based on shear wave velocity testing conducted at the representative site, the ground conditions are consistent with Site Class D as defined in ASCE 7-10 (ASCE 2010).

Figure 3. Representative site (red pin) near Transbay Transit Center Development in San Francisco, CA (Source: GoogleMaps).

Earthquake ground motions recorded at small site-to-source distances can have significantly different characteristics than those recorded at larger distances. Sites in the near-fault region may experience shaking described as forward-directivity (rupture towards the site) or backward-directivity (rupture away from the site), which induce intense pulse-type ground motions. These motions can have an adverse effect on the seismic performance of structures (NEHRP 2011). Baker (2007) developed a method for quantitatively identifying ground motions containing strong velocity pulses, such as those caused by near-fault directivity. Furthermore, Shahi and Baker (2011) and Almufti et al. (2013) have developed frameworks for incorporating velocity pulses in PSHA such that they can be accounted for in developing design ground motions for NLRHA. As described by Almufti et. al (2013), the methodology adds parameters to the traditional PHSA calculation to account for near-fault effects, which is referred to as a pulse-induced PHSA. The pulse-induced PSHA enables the development of a suite of target spectra to represent the contribution of pulse-type motions to the hazard. This methodology enables selection, scaling and matching of records consisting of pulse-type motions and conventional motions within the PSHA calculation.

Due to the site's close proximity to active faults, near-fault directivity effects are expected to significantly contribute to the hazard. Therefore, the methodology proposed by Almufti et al. (2013), is utilized to incorporate velocity pulses in the selection of the design level ground motions for this study. This methodology enables selection of an appropriate proportion of pulselike motions with characteristics (pulse amplitude and pulse period) representative of a desired hazard intensity level. A more detailed description of how this methodology is implemented in the PSHA and ground motion selection can be found in Molina Hutt et al (2013). The maximum and minimum demand surface response spectra for each suite of motions, one to characterize short period ground motions and one to characterize long period ground motions, are shown in Figure 4. Each suite of records consists of 11 bidirectional motions each. These ground motions are representative of the design level earthquake, defined as two thirds of the Maximum Considered Earthquake (MCE) (ASCE 2010). The spectra are consistent with the 475 year probabilistic estimate of the hazard (Molina Hutt et al. 2013).

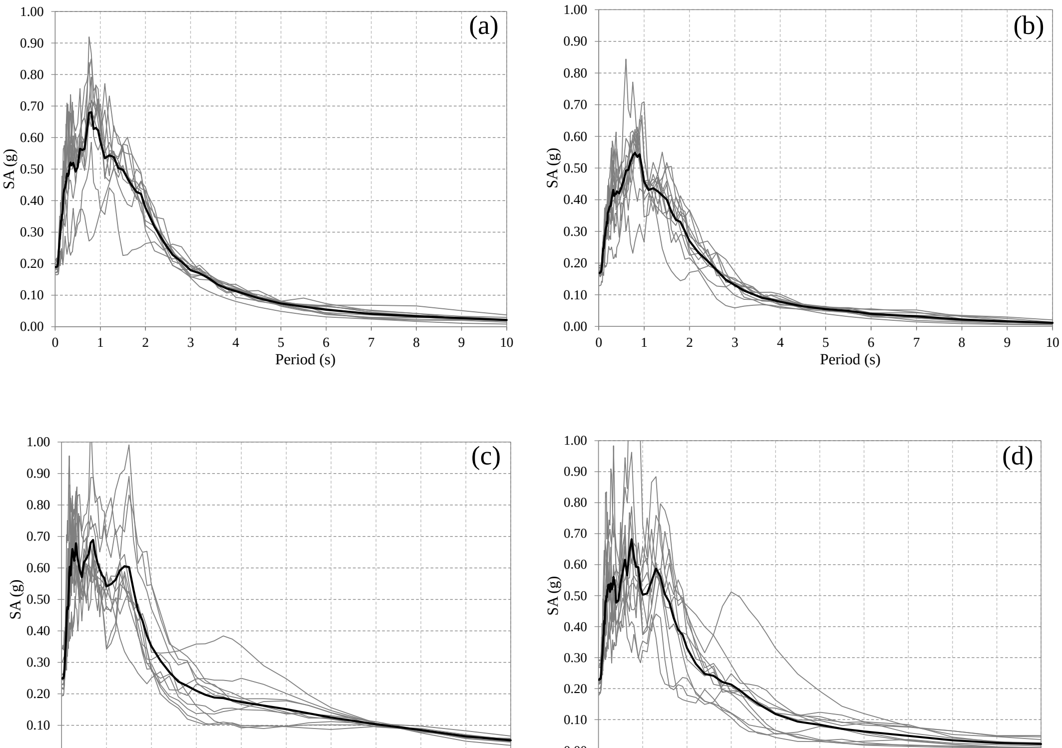

Figure 4. Maximum (a, c) and minimum (b, d) demand response spectra for short period suite of ground motions (a, b) and for long period suite of ground motions (c, d).
Source: Molina Hutt et al. (2013)

LOSS ASSESSMENT METHODOLOGY

Communicating performance in terms of the probable direct economic losses to repair earthquake damage can influence decision making. Financial institutions use quantitative statements of probable building repair cost expressed as a percentage of building replacement value. This metric is also used for this study, where costs are expressed in present dollars. Losses are expressed as the repair cost (i.e., the cost required to restore a building to its pre-earthquake condition), over total building cost (i.e., the cost required to rebuild with a new structure of similar construction).

Since there are many factors that can affect performance, such as intensity of ground shaking, building construction quality, building response or vulnerability of contents among others, there is significant uncertainty in the predicted performance of the building. This uncertainty can be accounted for by means of a Monte Carlo simulation, where each realization represents one possible performance outcome for the building considering a single combination of possible values of each variable considered. Therefore, losses can be expressed as a performance function, i.e. probability of losses of a specified amount or smaller incurred as a result of an earthquake. The Performance Assessment Calculation Tool (PACT) (ATC 2012) follows this methodology to conduct the loss estimates and is used for this study. Each performance assessment in PACT consists of 1000 realizations.

The building performance model includes all structural and non-structural components that are susceptible to earthquake damage. It includes component fragilities and consequence functions that can be used for predicting damage associated with the results obtained from the NLRHA as well as to translate such damage into repair or replacement costs. Structural quantities are based on the structural design of the archetype building. Non-structural quantities are estimated based on typical quantities found in buildings of similar occupancy by use of the Normative Quantity Estimation Tool (NQET) (ATC 2012). Normative quantities are an estimate of the quantity of components and contents likely to be present in a building of a specific occupancy based on gross square footage. In NQET, these quantities were developed based on a detailed analysis of approximately 2,000 buildings across typical occupancies (ATC 2012).

A component fragility function is a statistical distribution that indicates the conditional probability of incurring damage at a given value of demand. These typically are cumulative lognormal distributions, with a unique fragility function proposed for each damage state. The occurrence of damage states is predicted by individual demand parameters. Demand parameters include storey drift ratio, storey velocity and storey acceleration determined from the NLRHA of the archetype building. For each realization, fragility functions are used in conjunction with demand parameters to determine a damage state for each component. Consequence functions are then used to translate damage states into repair or replacement costs (ATC 2012). A more detailed description of the building performance model and loss assessment methodology can be found in Molina Hutt et al. (2013).

Residual drifts are an important consideration when estimating losses. As defined in ATC-58 (ATC 2012), typical building repair fragility as a function of residual drifts is a lognormal distribution with a median value of 1% residual drift ratio and a dispersion of 0.3. Residual drifts predicted by non-linear analysis are highly sensitive to component modelling assumptions (ATC 2012). Accurate statistical simulation of residual drift requires the use of advanced component models, careful attention to cyclic hysteretic response, and a large number of ground motion pairs. Therefore, residual drifts are estimated as a function of peak transient response of the structure and the median storey drift ratio calculated at yield based on ATC-58 (ATC 2012) recommendations. For each realization, PACT uses a random number generator to determine if the building is deemed irreparable as a function of residual drift. If irreparable, repair cost and repair time are taken as the building replacement values.

DOWNTIME ASSESSMENT METHODOLOGY

While seismic loss estimates associated with direct economic losses enable discussions with building owners and investors about how individual retrofit interventions can move buildings in the direction of becoming more resilient, they do not provide a quantitative measure of resilience. In addition to direct economic losses, there is great vulnerability to indirect economic losses due to downtime, defined as the time required to achieve a recovery state after an earthquake. The Structural Engineers Association of Northern California (SEAONC) defines three recovery states: re-occupancy of the building, pre-earthquake functionality and full recovery (Bonowitz 2011). Re-occupancy occurs when the building is deemed safe enough to be used for shelter, though functionality may not be restored. Functional recovery occurs when the building regains its primary function, i.e. it is operational. Lastly, full recovery occurs when the building is restored to its pre-earthquake condition, it follows from functional recovery once additional repairs for aesthetic purposes have been completed. In order to provide a more direct measure of resilience, the downtime to achieve building re-occupancy and functional recovery is assessed in this study. These estimates follow the REDi™ guidelines (Almufti and Willford 2013).

The REDi™ guidelines provide a detailed downtime assessment methodology for individual buildings and identify the likely causes of downtime such that these can be mitigated to achieve a more resilient design. The methodology identifies the extent of damage and criticality of building components that may hinder achieving a recovery state, it provides a logical approach for labour allocation and repair sequencing including structural, interior, exterior, mechanical, electrical, elevator and stair repairs on a floor per floor basis. Furthermore, the methodology includes delay estimates associated with impeding factors, defined as those factors which may impede the initiation of repairs such as post-earthquake inspection, engineering mobilization for review or re-design, financing, contractor mobilization, permitting and procurement of long lead items. Lastly, utility disruptions are also considered when estimating downtime for functional recovery.

SRATEGIES FOR INCREASED RESILIENCE

In order enhance the seismic performance of typical 1970s tall steel MRFs, a reduction in transient and residual deformations is required. This objective can be achieved by adding stiffness, damping or a combination of these to the structure. Two structural retrofit schemes are considered. The first scheme consists in the introduction of an elastic spine with steel bracing in the building core. The introduction of an elastic spine is intended to reduce transient and residual interstorey drifts up the building height. This concept has been implemented in a number of retrofit projects in Japan and has been explored in studies such as Günay et. al (2009) by means of introducing a rocking wall. A second retrofit scheme consists in the introduction of base isolation at ground level and is intended to significantly reduce the seismic demands to the structure. This technique has been implemented in a number of retrofit projects in Japan (Kani N. and Katsuta S. 2009). The impact of the two retrofit schemes considered in the results of the NLRHA is illustrated in Figure 5 in terms of interstorey drift ratios (IRD), floor velocities (V) and floor accelerations (A) in the principal axes of the building.

The cost of non-structural components in commercial buildings is typically considerably larger than the cost of the structure. Taghavi and Miranda (2003) found that non-structural components represent between 65% to 85% of the total initial construction cost. Therefore improving the seismic performance of non-structural components can lead to important reductions in the economic impact of earthquakes (Araya-Letelier and Miranda 2012). For example, based on a detailed evaluation of economic losses in 370 high rise buildings that were subjected to the 1971 San Fernando earthquake Whitman et al. (1973) found that the majority of the losses were the result of damage to non-structural components and concluded that improving the seismic performance of interior partitions would be one of the most effective ways to reduce seismic losses in buildings. In addition to the structural retrofit strategies, schemes for enhanced non-structural performance are adopted in this study. These are developed by employing non-structural components that are more resilient to earthquake damage. A building performance model with the same quantities as that of the archetype building is developed, but where applicable, employing more resilient non-structural components. These enhancements are adopted in the archetype building and in the two retrofit strategies considered. When standard non-structural components are used, these are referred to in the results discussion as the standard building performance model. When enhanced non-structural components are used in the archetype building and in the retrofit strategies considered, these are referred to in the results discussion as the enhanced building performance model.

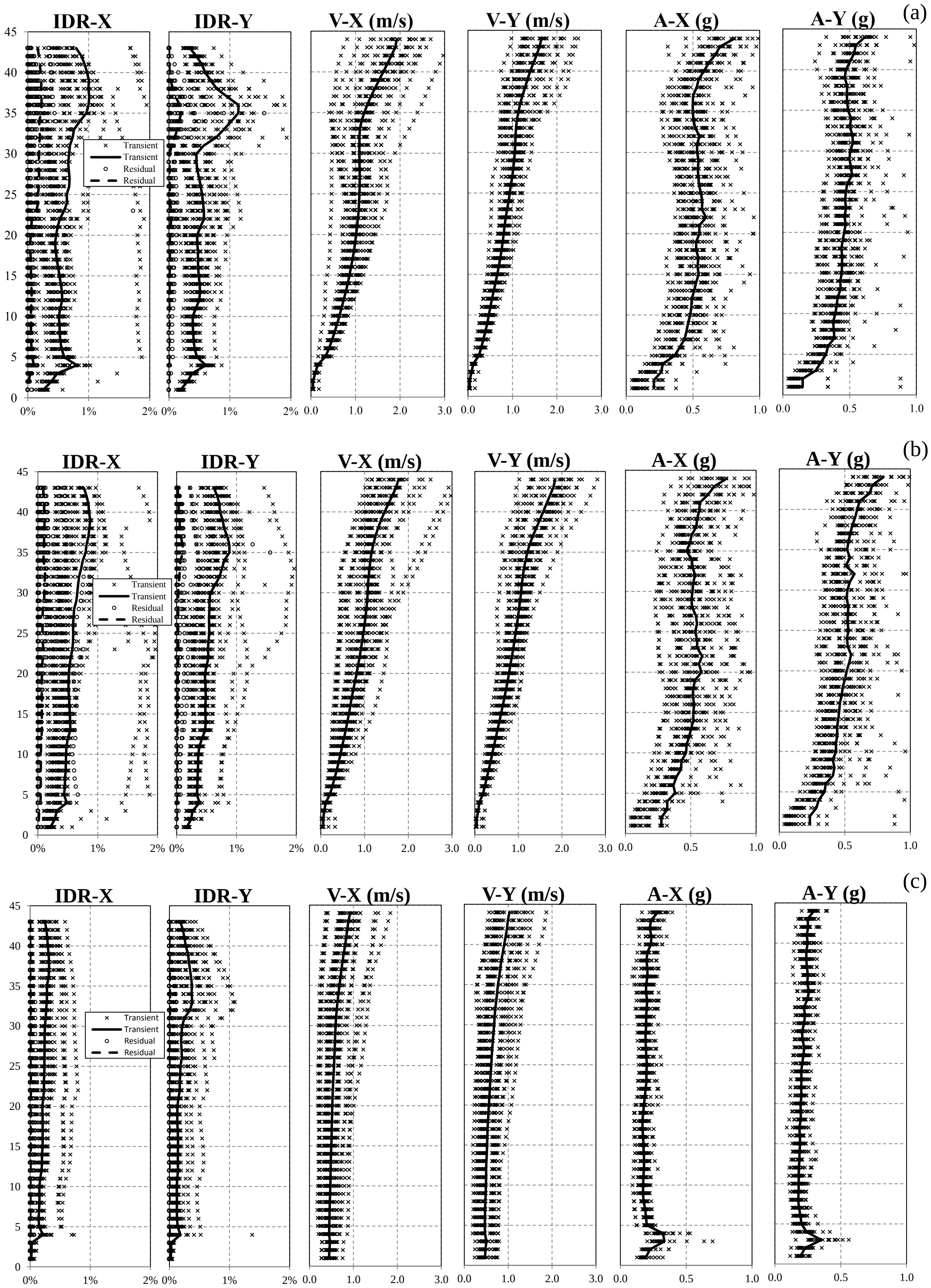

Figure 5. Results of NLRHA for archetype building (a), elastic spine retrofit scheme (b) and base isolated retrofit scheme (c). Source: Molina Hutt et al. (2013)

In order to increase the overall resilience of the building, a number of risk management strategies are explored to minimize downtime. While a reduction in damage to building components is achieved through the structural retrofits and non-structural enhancements, several risk management strategies can be adopted to minimize repair time and other impeding factors. These strategies are described in more detail in the REDi™ guidelines (Almufti and Willford 2013).

RESULTS

The performance functions for the archetype building and the two retrofit schemes presented with standard and enhanced non-structural components are shown in Figure 6. The results are visualized by fitting all 1,000 realizations in each performance assessment to a lognormal distribution. Expected losses for the archetype building and the strategies for increased resilience are summarized in Table 1 with and without consideration of residual drifts. Similarly, expected downtime for re-occupancy and functional recovery is summarized in Table 2.

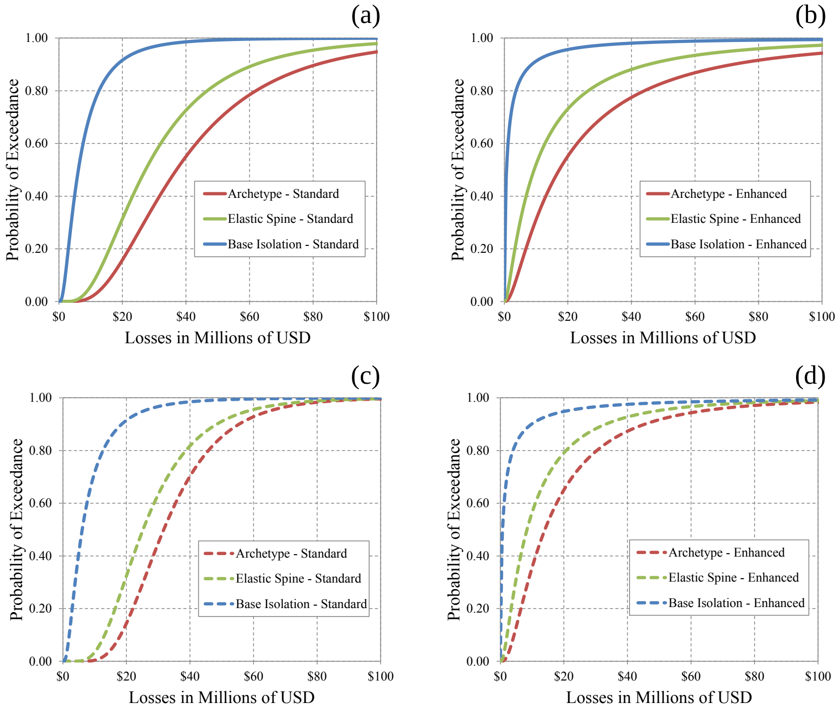

Figure 6. Loss estimates for archetype building, elastic spine and base isolation schemes with standard (a, c) and enhanced (b, d) non-structural components with (a, b) and without (c, d) consideration of residual drifts.

Table 1. Expected Loss estimates for the archetype building and enhanced performance schemes with and without consideration of residual drifts in absolute terms and as a percentage of building replacement cost.

		Non-structural Components:			
		Standard		Enhanced	
Residual Drift:		Considered	Neglected	Considered	Neglected
Structural Scheme:	Archetype (Baseline)	$46M (34%)	$35M (25%)	$31M (23%)	$19M (14%)
	Elastic Spine	$34M (25%)	$29M (21%)	$20M (15%)	$13M (10%)
	Base Isolation	$9M (7%)	$9M (7%)	$4M (3%)	$4M (3%)

Table 2. Downtime estimates for the archetype building and enhanced performance schemes for re-occupancy and functional recovery.

		Non-structural Components			
		Standard		Enhanced	
Downtime Estimate		Re-occupancy	Functional Recovery	Re-occupancy	Functional Recovery
Structural Scheme:	Archetype (Baseline)	72 weeks	87 weeks	14 weeks	32 weeks
	Elastic Spine	72 weeks	72 weeks	14 weeks	20 weeks
	Base Isolation	53 weeks	59 weeks	1 day	1 day

The results in Table 2 illustrate that, while structural retrofits may enable significant reductions in direct economic losses, these measures alone do not ensure a building is resilient. Downtime for re-occupancy for all the structural schemes with standard non-structural components is largely driven by delays associated with building inspection, contractor mobilization and long leads components that require replacement.

Figure 7 illustrates the contribution of different building components to the total expected losses. Building components are grouped into five main categories: structure, façade, MEP, office fitouts and egress (stairs and elevators). There are similarities in the distribution of building components contributing to the losses between the archetype building and the elastic spine structural retrofit scheme with either standard or resilient structural components. This can be attributed to the similarity in the demand parameter distribution throughout the height for both schemes, as shown in Figures 5a and 5b. The distribution of building components contributing to the losses for the base isolated scheme is distinct due to the unique distribution in demand parameters throughout the building height when compared to the other structural schemes, as seen in Figure 5c.

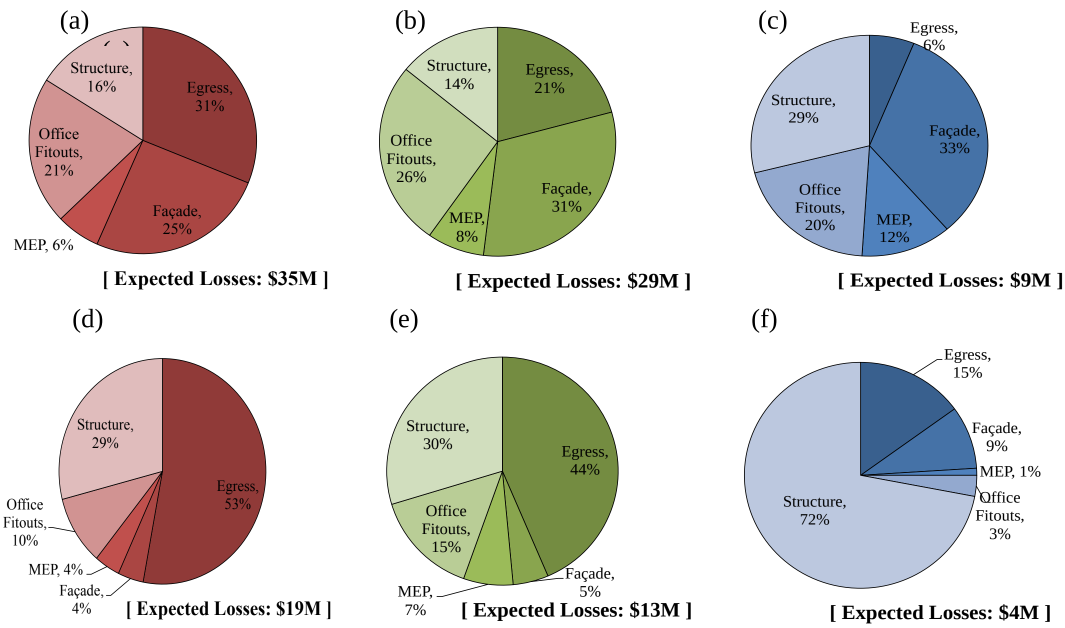

Figure 7.Contribution to losses of building components for archetype building (a, d), elastic spine retrofit (b, e) and base isolated retrofit (c, f) with standard (a, b, c) and enhanced (d, e, f) non-structural components.

CONCLUSIONS

An assessment of the seismic performance of existing tall buildings in a case study city, San Francisco, is presented. The assessment is conducted through the development of a database of the existing tall building stock, the design of a representative archetype building, performance assessment via NLRHA with ground motions representative of the design earthquake hazard level and estimation of associated direct economic losses and downtime. Expected losses are in the order of 34% building replacement cost. Expected downtime for functional recovery is in the order of 87 weeks. A number of strategies for increased resilience have been presented to enable a reduction in losses to 3% building replacement cost and downtime for functional recovery of 1 day.

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