

Distribution of Building Nonstructural Components in Height Subjected to Cost of Damage for Low-Rise Office Buildings

Haj Najafi, L.^{1*} and Tehranizadeh, M.²

¹ Ph.D., Department of Civil and Environmental Engineering, Amirkabir University of Technology, Tehran, Iran.

² Professor, Department of Civil and Environmental Engineering, Amirkabir University of Technology, Tehran, Iran.

Received: 31 May 2015;

Revised: 23 Sep. 2016;

Accepted: 04 Oct. 2016

ABSTRACT: Exceeded losses of nonstructural components from structural ones in most demolished buildings in previous earthquakes and its limitation on functionality of critical facilities and building serviceability after earthquakes should be got the point of view for accounting loss of building as a merit for building performance. This paper attempts to demonstrate the significant role of distribution pattern of building nonstructural components in height accounting for economic losses due to seismic excitation. To achieve this purpose a new modified distribution of nonstructural components in height is proposed for three typical steel moment frame models by office occupancy and comparative assessments between two competing distribution of nonstructural components are conducted. Dealing with discussions, it could be concluded that the economic losses could be reduced by more astutely situating building nonstructural components in height considering type of dominated demands in a specific story without requirement to any alternation in component's type or quantity.

Keywords: Damage Cost, Height of Building, Incorporation of Stories, Low-Rise Buildings, Nonstructural Components, Office Occupancy.

INTRODUCTION

Despite significant improvements in seismic design codes (e.g. better detailing requirements) translating in more predictable performance of modern buildings subjected to earthquakes compared to former ones, they still suffer from shortcomings. One of the inherent and underlying deficiencies with current structural design practice is that seismic performance could not explicitly quantified according to merits explicable to stakeholders (Haselton and Deierlein, 2007).

One approach for quantifying building performance proposed by recent researches is application of economic loss as a metric to gauge how suitable a specific building responds when subjected to seismic ground motion loads. (Krawinkler and Miranda 2004; Aslani and Miranda, 2005; Mitrani-Reiser and Beck, 2007). For this purpose, current performance seismic design practice attempts to control economic loss or specify an acceptable level of probability in which a structure maintains its functionality after an earthquake. Considering damage intensity on

* Corresponding author E-mail: lila_najafi@aut.ac.ir

structural and nonstructural components in former earthquakes such as Nisqually, Seattle earthquake occurred on February 28, 2001 (Filiatrault et al., 2001) and the Kona, Hawaii earthquake at 2006 (Chock et al., 2006; Gupta and McDonald, 2008), economic losses from damage to nonstructural components constitute far exceeded losses from structural damage in most affected buildings presenting an average factor ranging in magnitude from 5.7 to 8.4 for the nonstructural to structural damage costs. Moreover, nonstructural damage can severely limit the functionality of critical facilities such as hospitals' facilities, as demonstrated in the Northridge earthquake 1994 (McGavin and Patrucco, 1994), El Salvador earthquake 2001 (Boroschek and Retamales, 2001) and Kona, Hawaii earthquake 2006 (Chock et al., 2006). In view of development and implementation of performance-based earthquake engineering, harmonization of performance levels between structural and nonstructural components becomes vital as both of them contribute in economic losses as a merit for performance evaluation. Even if the structural components of a building

achieve a continuous or immediate operational performance level after a seismic event, failure of architectural, mechanical or electrical components can lead to inferior performance level of the entire building system. This reduction in performance caused by the vulnerability of nonstructural components has been observed during recent earthquakes worldwide.

The investment in nonstructural components and building contents is far greater than that of structural components and framing. Therefore, it is not surprising that in many past earthquakes, losses from damage to nonstructural components have exceeded losses from structural ones. Furthermore, the failure of nonstructural components can become a safety hazard or can more severely hamper safe movement of occupants evacuating buildings or of rescue workers entering buildings. According to Miranda and Taghavi (2003), nonstructural components make up approximately 82%, 87% and 92% of the total monetary investment in office, hotel and hospital buildings, respectively, in the United States (Figure 1).

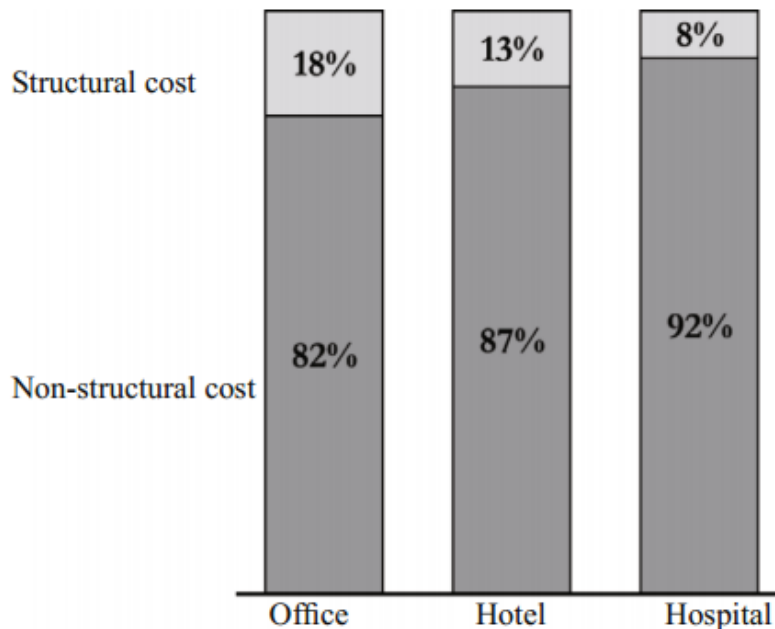


Fig. 1. Relative monetary investment in typical buildings (Miranda and Taghavi, 2003)

Furthermore, damage to nonstructural components occurs at seismic intensities much lower than those required to cause structural damage (Miranda and Taghavi, 2003). Therefore, it is not surprising that in many past earthquakes, losses from damage to nonstructural building components have exceeded losses from structural damage.

In comparison to structural components and systems, there is relatively limited information and specific guidance available on seismic design of nonstructural components for multiple-performance levels. Basic research work in this area has been sparse, and the available codes and guidelines are usually, for the most part, based on past experiences, engineering judgment and intuition, rather than on objective experimental and analytical results. Often, design engineers are forced to start almost from square one after each earthquake event to observe what went wrong and to try to avoid repetitions. This is a consequence of the empirical nature of current seismic regulations and guidelines for nonstructural components (Filiatrault and Sullivan, 2014). Summaries of many principal aspects in seismic assessments of nonstructural components behavior as well as the advancement of research and code efforts in the last 30 years can be found in (Soong, 1995; Filiatrault and Christopoulos, 2002).

A performance-based earthquake engineering methodology that does carefully treat nonstructural components could be quite valuable in risk-management decision-making, such as picking out design alternatives for new construction or judging the cost-effectiveness of a seismic retrofit, when decisions affect nonstructural components (Porter, 2005).

Picking out location of some nonstructural components is an inevitable feature like the decorating in lobbies or elevator equipment in the roof stories;

however, some of the others could be changed or modified; for example by well-done partition anchoring, they could be assumed as acceleration-dependent components rather than displacement-dependent ones and their cost distributions follow different type of demand.

The focused subject of this study is the pattern of distribution of building nonstructural components in height which could affect obtained damage cost amounts subjected to earthquake loading. This study could be very supportive for architectural planning phase of the project and could provide determining location of each component in a building in view of the cost of damage subjected to earthquake loading.

PERFORMANCE GROUPS, FRAGILITY FUNCTIONS AND NORMATIVE QUANTITIES

In order to provide further, comprehend understanding of nonstructural components, building codes generally classify nonstructural components into three broad categories: (1) architectural components; (2) mechanical and electrical equipment; and (3) building contents (FEMA P-58-1, 2012) and (ATC-58, 2011). Architectural components are built-in nonstructural components which form part of the building. They include interior partition walls, parapets, chimneys, penthouses, suspended ceilings, appendages and ornamentation, signs and billboards, egress stairways that are independent of the building, cladding systems, window systems and lighting systems. Mechanical and electrical components are built-in nonstructural components that form part of the building. They include heating, ventilation and air conditioning (HVAC) equipment, engines, turbines, pumps, compressors, pressure vessels, generators, batteries, motors, transformers, panel boards, switch gears, instrumentation cabinets,

communication equipment, computers, cooling towers, piping systems, ductwork and electrical conduits. Building contents are nonstructural components belonging to tenants or occupants of the building. They include filing cabinets, bookshelves and all pieces of furniture found inside buildings. Achieving cost amounts normalized by the story replacement value requires knowing the incorporated performance groups in each story and their normative quantities in addition to their fragility functions.

Different taxonomy (categorization system) for performance groups in building or story level have been proposed such as taxonomies developed by Antaki (2004), Porter (2005), HAZUS taxonomy for structural and nonstructural components, procurement and contacting requirements from NISTIR 6389 (NIST, 1999), classification of building elements based on UNIFORMAT II and the most commonly applied taxonomy is according to FEMA P-58-1 recommendation that is going to be utilized in this study too. Meeting objectives for taxonomic groups ensure meaningful fragility function creation; for more detail refer to (Porter, 2005). FEMA classification added some detailed subgroups to the NISTIR 6389 classification, for case in point, categorizing the structural components based on their ductility to special, intermediate or ordinary groups and dividing nonstructural components to anchored and non-anchored or based on their establishing requirements. The details for each fragility function have been achieved according to some resources:

- Actual demand data: specimens tested with slowly increasing (Engineering Demand Parameter) EDP to failure, EDP at failure is known.
- Bounding demand data: specimens observed in labor field, some failed, some not. Maximum EDPs are known.

- Capable demand data: specimens tested in lab, none failed. Maximum EDP for each is known.
- Derivation: estimate capacity with structural analysis.
- Expert opinion: capacity from engineering judgment (Porter, 2011).

The incorporated attributes cause to reach the number of proposed performance groups to 700 where for all of them fragility functions and their corresponding cost and repair functions have been distinguished incorporating quantity of damaged materials, relative difficulties in accessing and manipulating repair and quality of finished materials. The given information for each defined performance groups could be classified in basic identifier information, fragility information and consequence information. All of these performance groups in company with their fragility functions are provided in Appendix-D of FEMA P-58-1 and Appendix-B of ATC-58 or in a file by the name of Fragility Database (Excel file) or Fragility Specifications (PDF file) acting as a supplementary file to the cost estimating program of PACT.

A central challenge in such an effort is that the source data can be highly detailed. These detailed elements must be aggregated systematically including all structural and nonstructural components. For this purpose, PACT code has been employed. This code is a computer open source software which computes cost of damage, casualties and time delays based on the full probabilistic procedure associated with the PEER approach and according to FEMA seismic performance assessment code (FEMA P-58-3, 2012). It could be freely downloaded from the site of Applied Technology Council (ATC-58, 2011). This code computes cost of damage (that is the governing concern of this paper) based on the component fragility curves which are by default available based on FEMA P-58-1 or could be handled based

on any predefined fragility curve by user. Lognormal probability distribution has been considered for both of the collapse and residual drift condition with statistical parameters defined by user in this code.

Utilizing FEMA specifications have some advantages. The first is dependence of the proposed fragility functions to large collection of information, reports of earthquakes and experimental data; the second is possibility of result modification chiefly based on Bayesian approach as well as the most important advantage is covering almost all fragility functions corresponding to structural and nonstructural components by gigantic details. However, the inadequacy of the FEMA's proposed group of fragility functions is the overestimation of damage cost deduced from overvaluing in the amounts of standard deviation of the fragility functions which affects severely the starting and finishing point of each fragility function. As in practice, limited number of fragility functions considered for a particular prototype building and the mentioned overestimating has been revoked by not covering all the performance groups in a building (FEMA P-58-1, 2012; Ramirez and Miranda, 2007).

After determining incorporated performance groups, the quantity for each of the groups should be determined. There are two approaches for determining normative quantities for each performance group; the first is deterministic approach working based on specific plan for each story of the model and the second is probabilistic approach operating based on the probability of observing specific amount of a performance group among buildings by the similar type of occupancy.

For selecting a plan for deterministic approach of determining normative quantities for each performance group, some attributes should be taken in mind like architectural characteristics; the location of

building, the city vastness and social characteristics related to architecture of the model. These items affect dominantly nonstructural components than structural ones. One of the studies in this field based on deterministic approach is (Ferench, 2011) which estimated damage cost of office type buildings subjected to six different earthquake scenarios for big cities of Los Angeles, Salt Lake and Shelby illustrating very great portion of nonstructural components (ceiling, piping and partition) in damage costs of buildings.

The other study which delivers very inclusive considerate about the significance of nonstructural components in damage cost of buildings subjected to earthquakes based on some preselected deterministic plans is a work conducted by Aslani and Miranda (Aslani and Miranda, 2005). In this study, the amount of partition-like components in a story assumed to be equal to 14%, 36% and 58% of the selected plans area for each story. This study illustrates noteworthy portion of nonstructural damages subjected to regular ground motions and shows that a slight amplification in quantity of this partition-like components could intensely amplify the amounts of damage costs.

Instead of the deterministic approach which is utilized generally in case-studies or benchmark studies, performance codes such as ATC or FEMA P-58-1 prefer to make use of probabilistic approach to provide normative quantities for performance groups. The proposed normative quantities of FEMA P-58-1 are based on studies on 3000 models with diverse occupancy types presented according to 10%, 50% and 90% probability of observing the proposed amount for a performance group subjected to certain occupancy of a model. Because of the large number of models, the proposed amounts for performance groups do not hold to a specific plan. The normative quantities

in this study are chosen based on the proposed quantities by FEMA P-58-1.

The point that is fine to be stated is that the location of components in a story is an important view for damage cost evaluation that is not going to be reflected in many studies and beyond the scope of this paper too; because of gigantic investigation about the possibilities and dependencies of performance groups. The focused point of this study is only dealt with the amount of each performance group in each of the stories.

Practically, the source chosen to establish cost estimation and distribution in a building is the RS Means Square Foot Costs codes (Balaboni, 2014). These codes provide cost distributions of the entire building components rather than the distributions at each story level for many different types of common building occupancies (e.g. residential high-rise, commercial low-rise, hospitals, etc.). Engineering judgment was used to translate this data into story cost distributions, while maintaining the overall building cost distribution.

Translating the building cost distribution to story distributions requires making assumptions about the variation of stories' values along building height. This will be highly dependent on how the building components are distributed amongst the different floors, which is typically a function of the occupancy of the building as well as it is the subject of this study.

Although different story cost distributions could be generated for every floor, the number of distributions served can be limited by making the following assumptions:

- The entire building will be used for office space (i.e. not a mixed-use facility)
- The value of the first floor has significant differences from the other floors because as the main entrance,

the layout, facades and finishes are typically different at this level.

- The value of the top floor, typically the roof of the building, has distinct differences from the other floors because typically this is where most of the buildings mechanical and electrical equipment is located (this floor comprises any equipment that may be located in a mechanical penthouse).
- The remaining intermediate floors are all dedicated to office occupancy. These floors will have the same story cost distribution.

Under these assumptions, it was decided that there would be three different types of story cost distributions: one for the first floor, one for the top floor, and one for the intermediate floors, which will be referred to as the typical floor.

Table 1 displays an example cost distribution for a 7-story commercial office building. The first column is the cost distribution for the entire building taken directly from RS Means (Balaboni, 2014). Based on this information, the cost distributions for the first floor, the typical floors and the top floor were approximated as shown respectively in the second, third and fourth columns of this table from the estimations by Ramirez and Miranda (2007). Most of the story cost distributions are similar to the overall building distributions with the exception of a couple of items that reflect the previously discussed features. For instance, the component group Exterior Enclosures has a higher contribution to the story cost in the first floor because it is common to have more expensive exterior elements around the building's main entrances. Conversely, component groups such as HVAC and Conveying have high cost contributions at the top floor because most of the equipment associated with these groups is typically located on the building's roof.

Table 1. Example building and story cost distribution for mid-rise office buildings (Balaboni, 2014; Ramirez and Miranda, 2007)

Component Group	Building Distribution	Story Distribution (% of story value)		
	(% of total building value)	1 st Floor	Typical Floor	Top Floor
A. SUBSTRUCTURE	2.3	0.0	0.0	0.0
B. SHELL				
B10 Superstructure	17.6	17.9	18.5	15.4
B20 Exterior Enclosure	16.3	18.8	16.2	16.9
B30 Roofing	0.6	0.0	0.0	4.5
C. INTERIORS	19.4	20.7	21.4	11.1
D. SERVICES				
D10 Conveying	9.5	9.1	9.4	11.8
D20 Plumbing	1.9	1.9	1.9	2.0
D30 HVAC	13.0	12.3	12.7	17.6
D40 Fire Protection	2.6	2.6	2.7	2.8
D50 Electrical	16.8	16.6	17.2	17.9
	100	100	100	100

By all discussions about cost of damage subjected to earthquake in a building and its distribution in height, this paper attempts to determine the portion of each story in the entire damage cost of a building and also to address the challenge of reducing total damage cost by redistributing building components in a proposed revised pattern.

DESCRIPTION OF STRUCTURAL SYSTEMS AND ANALYZING METHODS USED FOR EVALUATION

On account of the need for consideration of varieties in models' characterizations, the selected models should represent acceptably the picked out group of the buildings. Studied models in this study take only account of low-rise buildings due to dissimilar distribution of demands and also different cost distribution in mid-rise and high-rise buildings. For mid-rise and high-rise buildings other supplementary studies have to be conducted. In this study, three models with 3, 4 and 5 number of stories have been chosen to be representative of typical structures and analyzed through conducting nonlinear dynamic analysis. The

height of each story was assumed equal to 3.0 m. The plan of all stories was taken in to account similar with 4 spans in longitude side and 3 spans in the other side whereas the length of each span is equal to 4.0 m. The fundamental periods of the models are equal to 0.68 s, 0.85 s and 1.12 s for the models with 3, 4 and 5 number of stories respectively.

Loading has been accomplished based on ASCE7-05 by consideration of dead load equal to 620 kg/m² and live load equal to 200 kg/m². Design has been accomplished based on AISC 2005.

For modeling nonlinearity in structural responses, modified Ibarra-Krawinkler (MIK) model has been employed with bilinear hysteresis behavior (Lignos and Krawinkler, 2012) and (Lignos and Krawinkler, 2013). This model demonstrated very acceptable reconciliation between the gained results from analyses and experiments (Lignos et al., 2011). Modeling has been conducted by the means of the open system for earthquake engineering simulation through using concentrated plasticity in the end joints of each frame component (Opensees, 2009). Critical

damping ratios in the first and second modes of vibration are assumed equal to 0.03. Modification of stiffness and damping has been done by consideration of modification factor equal to 10 based on studies conducted by Medina and Zareyan (2010). Geometric nonlinearity has been taken in to account through consideration of P- Δ effects (Lignos, 2014). Panel zone modeling has been conducted based on nonlinear behavior suggested by Gupta and Krawinkler composed up three linear fragments (Gupta and Krawinkler, 1999).

Engineering Demand Parameters (EDPs) have been assessed in three subcategories; responses in near collapse, non-collapse and responses from residual drift situations. For getting responses near collapse situation, incremental dynamic analysis (IDA) has been utilized for determining median and dispersion of spectral acceleration of collapse fragilities based on procedure developed by detail in (Vamvatsikos and Cornell, 2002); whereas, structural responses in non-collapse and residual drift situations have been derived by the help of nonlinear dynamic in company with nonlinear static analyses. All nonlinear dynamic analyses were conducted as Direct Integration Transient time history analyses using direct integration in Hilber, Hughes and Taylor's method. Nonlinear static analyzing used to determine elastic displacement for each story is conducted based on first mode distribution pattern of lateral forces.

In this paper, the selected EDPs are inter-story drift ratios (IDR) and peak floor acceleration (PFA) in each story of the building similar to other conducted studies on performance-based assessment of steel structures like (Tehranizadeh and Movahed, 2011) and many related studies in this field.

Selected Ground Motions

Statistical quantities (median and standard deviation) for structural responses

could be achieved regarding some ground motion records. Concerning the number of ground motions, typical practice in structural design is to use seven motions according to ASCE05-7 and eleven ground motions according to ATC, but the appropriate number of motions is still a topic of prospect researches. It is demonstrated that the ASCE/SEI 7-10 scaling procedure is conservative if less than seven ground motions are employed and in this condition the design values of EDPs are served as the maximum values of the EDPs where if at least seven ground motions are analyzed, the design values of engineering demand parameters (EDPs) are taken as the average of the EDPs determined by the analyses. Current ground motion selection and modification (GMSM) efforts are generally focused on predicting the median response of the engineering demand parameters (EDP) under prescribed seismic loads.

The procedure of this paper for the purpose of record selection is employment of random selection of records by consideration of minimizing deviations around the geometric mean of natural logarithmic spectral acceleration values to reduce the effects of record to record variations in structural responses. The efficiency of this record selection technique has been revealed in an accomplished study by the same authors (Haj Najafi and Tehranizadeh, 2015a).

For ground motion selection, a primarily set of records is required which the records are going to be picked out from it. Many researchers prefer to randomly set records in primarily list and some other recommends choosing records as a list comprises records with all groups of specification subjected to corresponding hazard possibilities.

In this paper, one of very frequently established primarily sets of records has been consumed. The 79 earthquake ground motions of this list have been carefully

selected by Medina and Krawinkler from the Pacific Earthquake Engineering Research center (PEER) strong motion database classified as far-filed records. This list of records has been employed in many prospect researches in PEER and SAC¹ centres and could be inputted for many studies in this field too (Aslani and Miranda, 2006). The earthquake magnitude in the selected primarily record set ranges in magnitude from 5.8 to 6.9 with the closest distance to rupture ranging from 13 km to 60 km.

Recorded motions could be derived from databases of PEER NGA database (PEER SGMD, 2015), COSMOS (COSMOS, 2015) or K-NET (K-Net, 2015). All ground motions were recorded on free-field sites classified as site class D according to National Earthquake Hazards Reduction Program (NEHRP) seismic provision (NIST, 2011). Most of the design codes like ASCE05-7 and seismic performance provisions like ATC-58-1 allow manipulation of this class of soil when the soil specification has not been studied and determined. The eleven selected records are presented in Table 2. It is fine to mention that any arbitrary list of records could be substituted.

The selected intensity measure for scaling procedure has been considered as spectral acceleration in the first period of the models. The efficiency of this intensity measure parameter for far-field records has been comparatively evaluated in a study by the authors (Haj Najafi and Tehranizadeh, 2015b).

The acceleration spectrum in the level of design earthquake (DE) representing 10% probability of earthquake occurrence by the

adopted intensity measure in 50 years is going to be acquired according to ASCE05-7 procedure for each earthquake. Through calculating geometric mean between the design earthquake spectrums for each station, the target design earthquake spectrum will be achieved.

This paper employs a frequently used method for record scaling based on the uniform hazard spectra (UHS) for the models by the fundamental periods of 0.68 s, 0.85 s, 1.12 s located in soil class D. Record scaling associated to a target value of elastic spectral acceleration, from a code-based design spectrum or (Probabilistic Seismic Hazard Analysis) PSHA-based uniform hazard spectrum at the fundamental vibration period of the structure, T_1 , provides improved results for structures whose response is dominated by their first-mode (Nau and Hall, 1984). Including a vibration property of the structure led to improved approaches of ground motion scaling; however, scaling only according to fundamental vibration period becomes less accurate and less efficient for structures responding significantly in their higher vibration modes or far into the inelastic range (Shome et al., 1998; Mehanny, 1999; Alavi and Krawinkler, 2000). As the evaluations in this study comprise only low-rise buildings, the procedure of scaling based on first fundamental period of the models seems to be sufficient for the purpose of higher mode consideration as well as the effects of proceeding in nonlinear behavioral zone. For more details about the scaling factors and target spectrum one could refer to (Haj Najafi and Tehranizadeh, 2015a).

¹ SAC center is a joint venture of the Structural Engineers Association of California, Applied Technology Council and Consortium of Universities for Research in Earthquake Engineering

Table 2. Eleven selected records

Number	Record ID	Event	Year	Station	Mw	R (km)	Mechanism	PGA (g)
1	IV79e13	Imperial Valley	1979	El Centro Array #13	6.53	21.90	Strike-slip	0.139
2	MH84g02	Morgan Hill	1984	Gilroy Array #2	6.20	15.10	Strike-slip	0.162
3	PM73phn	Point Mugu	1973	Port Hueneme	5.80	25.00	Reverse-slip	0.112
4	PS86psa	N. Palm Spring	1986	Palm Springs Airport	6.00	16.60	Strike-slip	0.187
5	WN87wat	Whittier Narrows	1987	Carson - Water St	6.00	24.50	Reverse	0.104
6	SF71pel	San Fernando	1971	LA - Hollywood Store Lot	6.60	21.20	Reverse-slip	0.174
7	SH87pls	Superstition Hill	1987	Plaster City	6.70	21.00	Strike-slip	0.186
8	BM68elc	Borrego Mountain	1968	El Centro Array #9	6.70	46.00	Strike-slip	0.057
9	LP89slc	Loma Prieta	1989	Palo Alto - SLAC Lab	6.90	36.30	Reverse-oblique	0.194
10	NR94del	Northridge	1994	Lakewood - Del Amo Blvd	6.70	59.30	Reverse-slip	0.137
11	CO83c05	Coalinga	1983	Parkfield - Cholame 5W	6.40	47.30	Reverse-oblique	0.131

Evaluation of Structural Responses

Mentioning all EDPs elongates the paper without any constructive view to its objectives. The evaluations of structural responses have been conducted in three subcategories; Collapse assessment, non-collapse assessment and residual drift condition.

Vamvatsikos and Cornell (2002) proposed a probabilistic framework on the basis of Incremental dynamic analysis (IDA) to relate ground motion intensity to structural response and performance. In this method, the displacement capacity and transition point of a structure are calculated using a set of ground motion records. The output curves indicate the cumulative

probability of structural collapse in terms of ground motion intensity. The developed framework of IDA has been commonly applied for damage detection in numerous studies, for instance in (Shahraki and Shabakhty, 2015). In this study, IDA analyses have been accomplished utilizing both N-S and W-E direction of ground motion sets in each of the x and y direction of the models for deriving collapse fragility curves. The diagrams of IDA analyses for the models and corresponding collapse fragility curves are presented in Figures 2 and 3; in addition, the obtained statistical parameters for consideration of collapse are exhibited in Table 3.

Table 3. Collapse fragility parameters

	3-St Model	4-St Model	5-St Model
Median of $S_{CT}(T_1)$ (g)	1.863	1.526	1.372
β	0.440	0.422	0.311

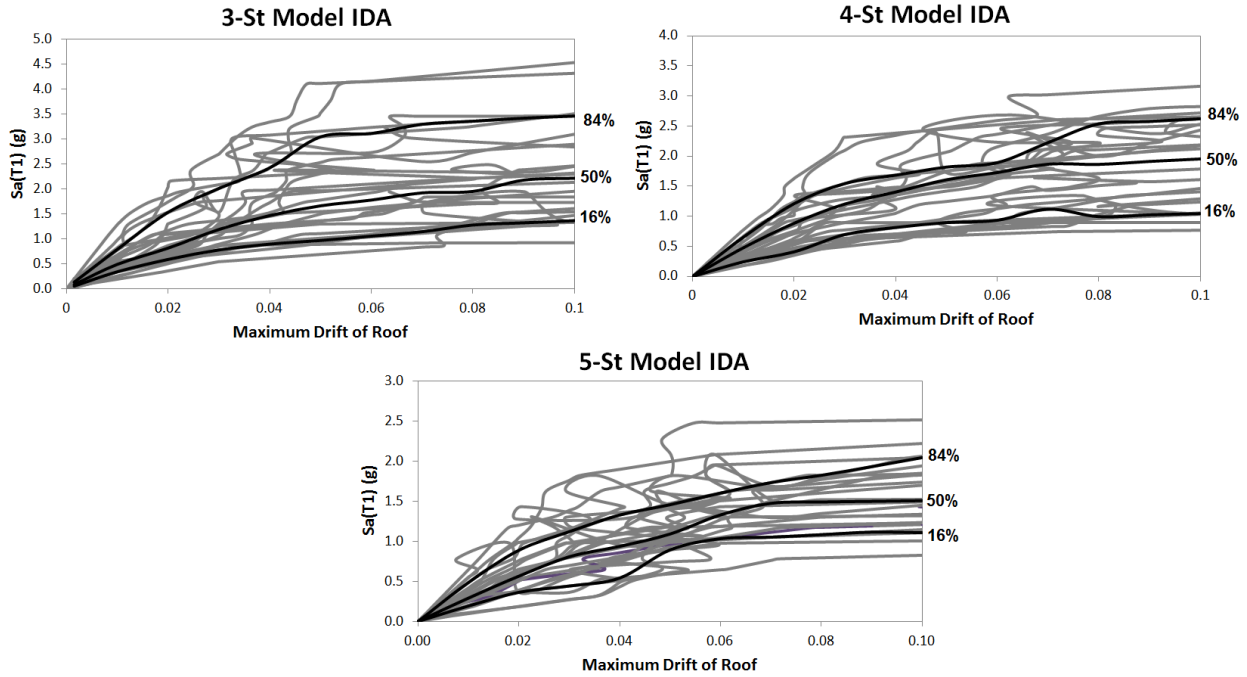


Fig. 2. Results of incremental dynamic analysis for different models, spectral acceleration amount according to maximum drift of the roof story

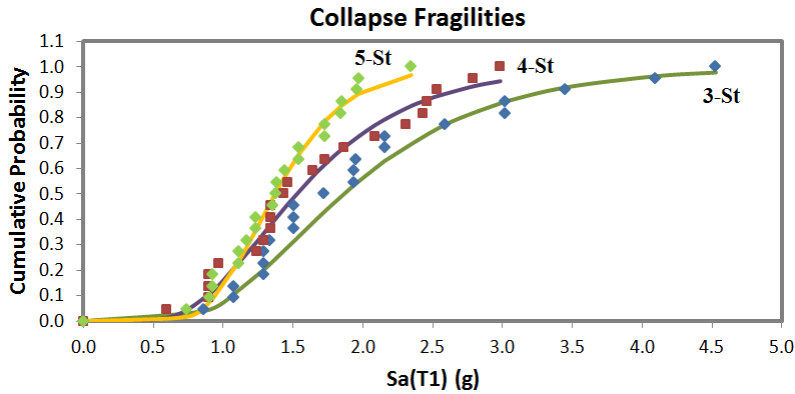


Fig. 3. Collapse fragilities of the models

For assessing non-collapse conditions, record scaling has been done according to five levels corresponding to design level earthquake (DLE) which was calculated based on first fundamental period and consideration of the design target spectrum. The level of scaling is selected as (0.5 DLE, 1.0 DLE, ..., 2.5 DLE). Nonlinear dynamic analyzing has been conducted according to the both N-S and E-W factors of the records; in this respect, the models have been analyzed subjected to 22 records in each

direction of x and y. Maximum level of scaling has been selected so that the models do not experience collapse at the most under half of the records. All EDPs are extracted for each story of the models with different number of stories subjected to multiple scaling levels for each ground motion record. As an example, EDPs of roof story in 3-story model have been presented in Figure 4 in scaling level of 1.5 DLE under San Fernando record.

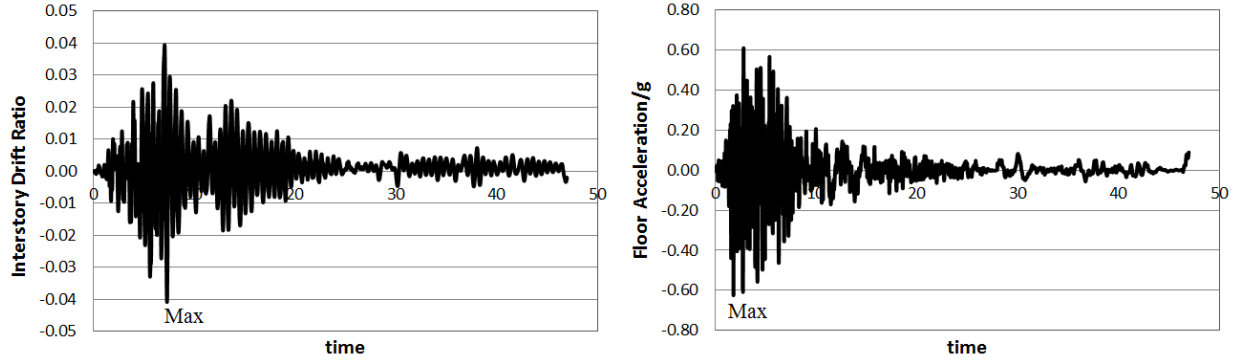


Fig. 4. Structural responses of roof story in 3-story model in scaling level of 1.5 DLE

The dispersion amounts of the EDPs are calculated by consideration of lognormal distribution of responses in each scaling level of response. The procedure for calculating dispersion factors (β) in each scaling levels are based on the recommendations in ATC-58 (ATC-58, 2011). The values of β could be gained based on the standard deviations of logarithmic amounts of EDPs. β values could be divided into three major parts, which contribute to β computation by the help of Eqs. (1) and (2).

$$\beta_m = \sqrt{\beta_c^2 + \beta_q^2} \quad (1)$$

$$\beta_{EDP} = \sqrt{\beta_m^2 + \beta_{aEDP}^2} \quad (2)$$

where β_m : is modeling dispersion, β_c : is dispersion associated with definition of the building, characteristics of the materials and available information of the building, β_q : dispersion associated with modeling

strategy, β_{aEDP} : is dispersion associated with structural responses calculated in two distinct groups of Maximum floor acceleration and inter-story drift ratio.

The ultimate calculated amounts of dispersion for three selected models are summarized in Tables 4 and 5.

Residual drift plays very important role in loss calculation. The effect of consideration of this factor in loss estimation in some cases especially systems with high ductility reach up to 50% of the loss amounts (Miranda and Ramirez, 2009). Diverse equations have been proposed by some researchers commonly based on ultimate nonlinear displacement and yield displacement of story. One of very frequently applied equations in researches (Ruiz-García and Miranda, 2006) and also codes (ATC-58, 2011) as well as this study is the mentioned Eq. (3).

Table 4. The ultimate calculated amounts of dispersion for peak floor acceleration (PFA)

T (s)	0.5 DL	1.0 DL	1.5 DL	2.0 DL	2.5 DL
0.68	0.28	0.29	0.31	0.31	0.33
0.85	0.28	0.29	0.31	0.32	0.32
1.12	0.28	0.29	0.32	0.32	0.33

Table 5. The ultimate calculated amounts of dispersion for interstory drift ratio (IDR)

T (s)	0.5 DL	1.0 DL	1.5 DL	2.0 DL	2.5 DL
0.68	0.31	0.38	0.38	0.40	0.44
0.85	0.30	0.35	0.38	0.41	0.43
1.12	0.32	0.39	0.41	0.41	0.45

$$\begin{aligned} \Delta_r &= 0 & \Delta &\leq \Delta_y \\ \Delta_r &= 0.3(\Delta - \Delta_y) & \Delta_y &< \Delta < 4\Delta_y \end{aligned} \quad (3)$$

$$\Delta_r = (\Delta - 3.1\Delta_y) \quad \Delta \geq 4\Delta_y$$

where Δ_r : is estimated median residual story drift in a story, Δ : is computed median ultimate drift in a story gained from nonlinear dynamic analysis and Δ_y : is median value for elastic drift of a story gained from static nonlinear analysis.

Static nonlinear analyzing has been performed based on ASCE/SEI 41-06 recommendation (ASCE/SEI 41-06, 2007) by utilizing gravitational load combination of (1.05 DL+0.25LL). Target drift has been considered equivalent to 0.1 and distribution of lateral load was based on first mode. The results of elastic drifts for each story of the models are presented in Table 6. Availability of nonlinear dynamic analyses results in company with elastic drifts and utilizing Eq. (4) contribute to residual drift ratios for models under each of the 22 records.

The dispersion values are calculated according to the Eqs. (1) and (2) by consideration of the calculated residual drifts values.

Fragility functions for non-collapse condition are derived from FEMA P-58-1 Specification. Selected fragility functions and associated performance groups and their normative quantities were discussed earlier in this paper. For collapse condition fragility functions for each model were derived from IDA analysis mentioned in Figure 3 and Table 3.

For consideration of residual drifts in loss calculations, repair fragility has been hired. The repair fragility is a lognormal distribution, typically having a median value of 1% of the residual drift and dispersion of 0.3 (ATC-58, 2011). In this study, the repair fragility function mentioned in ATC-58 has been used for consideration of residual drifts

in loss calculations similar to most of the studies which were considered this aspect like (Ruiz-García and Miranda, 2008).

DEFINITION OF THE COST MODEL IN PACT

The program PACT computes cost of damage (that is the governing concern of this paper) based on the component, collapse and repair fragility functions. Most of these fragilities are available by default in this program based on FEMA P-58-1 or ATC-58 or could be defined manually. PACT has been conducted on the foundation of Monte Carlo simulation in each try called in this program as a realization. The number of realizations in this study has been assumed equal to 200 which could influence the accuracy of the results as well as time of analyzing. There are so many non-structural components in a building that measuring them all is not feasible in terms of time and effort; however, the type and normative quantities of the applied performance groups in this study are developed based on FEMA P-58-1 recommendation which has been exclusively discussed formerly in this paper as well as referred in Appendix 1 of this paper in detail.

The estimated amounts of damage cost for the models have been available through conducting analyses according to the earlier mentioned clarifications about the EDPs and performance groups in this paper and have been presented in Tables 7 to 9 for the stories of the models in two subcategories; costs dependent on IDR and costs dependent on PFA.

Table 6. Elastic drift for each story of the models in both x and y direction

	3-Story Model		4-Story Model		5-Story Model	
	x-dir	y-dir	x-dir	y-dir	x-dir	y-dir
Story 1	0.011	0.013	0.014	0.014	0.016	0.017
Story 2	0.013	0.012	0.015	0.016	0.017	0.017
Story 3	0.013	0.011	0.016	0.017	0.017	0.018

Table 7. The percentage of the amounts of damage cost for each story according to different intensity levels for the 3-story model with fundamental period of 0.68 s

Intensity Levels	Story Damage Cost to the Cost of Each Story (%)						Total Damage Cost to the Cost of Entire Building (%)
	First Story		Typical Story		Roof Story		
	IDR	PFA	IDR	PFA	IDR	PFA	
0.5 DLE	4.3	7.4	5.3	7.0	2.9	8.7	8.4
1.0 DLE	15.6	14.5	15.4	14.6	7.8	11.6	20.6
1.5 DLE	26.9	25.1	28.1	23.4	28.2	18.0	35.1
2.0 DLE	34.8	37.1	36.0	32.0	40.8	24.2	47.4
2.5 DLE	38.0	39.7	38.4	34.3	45.2	26.3	51.0

Table 8. The percentage of the amounts of damage cost for each story according to different intensity levels for the 4-Story model with fundamental period of 0.85 s

Intensity Levels	Story Damage Cost to the Cost of Each Story (%)						Total Damage Cost to the Cost of Entire Building (%)
	First Story		Typical Story (2 nd + 3 rd Stories)		Roof Story		
	IDR	PFA	IDR	PFA	IDR	PFA	
0.5 DLE	7.1	5.7	8.7+6.7	6.1+6.2	3.9	5.8	12.6
1.0 DLE	17.4	16.8	16.9+9.8	16.9+17.4	5.1	13.4	28.7
1.5 DLE	30.5	27.7	31.4+27.4	25.9+25.6	24.9	20.3	53.6
2.0 DLE	40.7	34.4	39.2+32.5	31.2+31.4	34.9	24.7	67.4
2.5 DLE	45.8	38.1	46.8+36.0	34.9+34.2	38.4	27.3	75.6

Table 9. The percentage of the amounts of damage cost for each story according to different intensity levels for the 5-Story model with fundamental period of 1.12 s

Intensity Levels	Story Damage Cost to the Cost of Each Story (%)						Total Damage Cost to the Cost of Entire Building (%)
	First Story		Typical Story (2 nd + 3 rd + 4 th Stories)		Roof Story		
	IDR	PFA	IDR	PFA	IDR	PFA	
0.5 DLE	1.5	4.6	4.2+5.6+4.4	4.5+4.5+5.1	2.0	3.9	8.1
1.0 DLE	7.3	11.7	22.6+21.2+16.0	12.0+11.8+12.2	8.7	9.5	27.0
1.5 DLE	21.8	19.0	27.9+26.9+21.5	18.2+17.9+17.9	16.2	14.2	40.7
2.0 DLE	34.7	26.5	40.3+40.7+32.0	21.1+20.9+19.9	32.5	19.7	57.8
2.5 DLE	42.4	32.0	46.8+46.7+34.4	29.5+29.4+29.4	36.0	23.2	70.4

The contribution portion for each story in the entire building's damage cost could be calculated due to the evaluated damage cost for all the models subjected to different scaling levels. The results, for different fundamental periods and different scaling levels are displayed in Table 10. For illustrating the impact of component distribution in height on total damage cost of building the ratio of DR has been defined as

the amount of total damage cost of building to the amount of total replacement cost of building. The amounts of this factor are also going to be presented in Table 10.

Table 10. Percentage of incorporation for each story in the total damage cost and the percentage of total damage cost to the total replacement cost of building (DR) according to different fundamental periods and different scaling levels

Periods Scaling Levels	3-Story (T = 0.68 s)				4-Story (T = 0.85 s)				5-Story (T = 1.12 s)			
	1 st St.	Type St.	Top St.	DR (%)	1 st St.	Type St.	Top St.	DR (%)	1 st St.	Type St.	Top St.	DR (%)
0.5 DLE	0.326	0.333	0.339	8.39	0.215	0.261	0.257	12.61	0.177	0.202	0.217	8.14
1.0 DLE	0.297	0.349	0.351	20.55	0.212	0.255	0.275	28.66	0.175	0.204	0.213	26.98
1.5 DLE	0.310	0.339	0.349	35.14	0.216	0.254	0.272	53.59	0.173	0.199	0.230	40.65
2.0 DLE	0.307	0.336	0.358	47.36	0.214	0.249	0.281	67.38	0.166	0.201	0.231	57.84
2.5 DLE	0.313	0.328	0.360	50.93	0.209	0.257	0.272	75.57	0.164	0.198	0.242	70.36

It could be observed that by increasing intensity level of spectral acceleration, the incorporation of displacement-dependent components has been decreased resulting in reduction of contribution portion for first story in the damage cost. However, the amplification of intensity level brings about intensification of contribution portion for acceleration-dependent components which are generally situated in top story. Also by increasing the number of stories and consequently the period of structures, the involvement of the typical stories increased bringing about reduction in portion of top and first floors where by increasing the intensity level, the impact of fundamental period of the models has been degraded.

Then, the strategies of situating displacement or acceleration-dependent components could be determined according to some indexes such as intensity level of earthquake, number of stories and period of the building.

In small earthquake intensities by transferring acceleration-dependent elements to the lower stories, the cost of damage could be reduced according to less gained accelerations in these stories and also according to further incorporation of lower stories in building damage costs. In the other hand, by increasing the scaling level of spectral acceleration intensity, if one could distribute displacement-dependent component in a way to afford larger portion of contribution to the upper stories, or by altering the type of components from displacement-dependent to acceleration-

dependent by well-done anchoring the components, the cost of damage has been decreased.

In some levels of intensity, the contribution portions of stories go across equality like the intensity of 0.5DLE for the 3-story model or 1.5DLE for the 4-story model. Although this situation is very ideal for investment, its occurrence would not guarantee the least amount of damage cost subjected to seismic excitation because of diverse structural responses and in addition dissimilar component distribution in different stories. However, it could be usually assure degrading entire building cost of damage in non-collapse situation of response and also profitability of investment in each of the building's stories. In the other word, it is the wisdom of designer to keep in mind the cost of damage subjected to earthquake as a decision making issue for determining the situation of nonstructural components.

Proposed Modified Pattern

Some procedures for modifying incorporations of stories could be followed which could be classified as modifications in structural responses and modifications in distribution of components.

By ductiling roof story and strengthening first story, the acceleration demands trends from roof story to the below ones and displacement demands trends from first story to the upper ones respectively, causing less participation of roof story and first story for acceleration-dependent and displacement-

dependent demands, respectively. There are lots of structural design approaches for applying these modifications which are not the concern view of this paper. The focused feature of this paper is on the modifications in distributions of components which derive in modifications in incorporation of stories in the total cost of damage for a typical low-rise building and conclude to less amounts of loss without any alternation in the presumed quantity for each performance groups.

For instance, some of the proposed modifications in this paper are as follow:

- Moving chiller equipment (D3031.011a) from roof story to the first story.
- Moving cooling tower (D3031.021a) from roof story to the first story.
- Moving motor control center (D5012.013a) from roof story to the

first story.

- Decreasing decorating expenses in first story by moving them to the 2nd story or completely anchoring the components; so, they could be removed from the first story.
- Moving electronic equipment from first story to the 2nd story.

So, the proposed component summary matrix by FEMA P-58-1 which has been stated in Appendix 1 of this paper could be adapted in some cases as mentioned in Table 11. The distributions of the other components are as formerly mentioned in Appendix 1 of this paper. For simplifying comparative referring purposes, Table 11 also lists equivalent performance group codes subjected to FEMA loss-estimation framework.

Table 11. Modified distributions of the presumed performance groups

Component Summary Matrix							
Occupancy	Fragility	Fragility Name	Assumed Quantity per Component within PACT	Quantity	Actual Quantity		Fragility Quantity Beta (Lognormal Direction)
Floor Name	Number			Non Directional	Value	Unit	
1 st	D3031.011a	Chiller-Capacity:<100 ton- Unanchored equipment that is not vibration isolated-Equipment fragility only	75 TN	1.0	3.3	TA	0.1
1 st	D3031.021a	Cooling Tower-Capacity:<100ton- Unanchored equipment that is not vibration isolated-Equipment fragility only	75 TN	1.0	3.3	TA	0.1
1 st	D5012.013a	Motor Control Center-Capacity: all- Unanchored equipment that is not vibration isolated-Equipment fragility only	1 EA	1.0	0.0	EA	0.5
2 nd	E2022.012	Fragility Contents on shelves in storage cabinets with latches.	15 EA	2.89	4.5	EA	0.4
2 nd	E2022.021	Electronic equipment on wall mount brackets	1 EA	1.0	1.0	EA	0.4

Evaluation of the Results

As the modifications are not as structural type, the structural responses of the models have not altered and the previously gained results could be utilized for new analyses too. The amounts of damage cost and the portion of contribution of each story in the damage cost of the models based on the modified distribution of components are presented in Table 12 for different models subjected to different intensity levels.

The results demonstrate prominent standing of the performance group's distribution in height of models. By application of these straightforward proposed modifications without any alternation in the amounts of performance groups or in structural characteristics of the models, the cost of damage in all of the models has been declined subjected to all assumed fundamental periods and all intensity levels. Although this reduction is not too considerable in the percentage of total damage cost of building in some cases; assuming the amount of building cost in dollars, this reduction could save great expenses particularly for the models with large areas and thus large initial costs. As for models with larger areas, both of the initial cost and normative quantities for performance groups are excessively greater than the presumed amounts for considered typical models of this study. To complement the results from the analyses, the percentages of total damage costs could be compared in different intensity levels before and after application of the proposed modifications for different models by the help of the diagrams in Figure 5.

The more significant impact of the proposed modifications is on the portion of incorporation of each story in the total damage cost of building. By conducting the recommended modifications, the portions of incorporation for the first and typical stories increase especially for low scaling levels of

intensities. This amplification is more intense in the first floor than the typical ones and further in 3-story building than the 4 or 5-story ones; because of fewer number of typical stories permits to the first floor to play more significant role in making cost of damage. In addition, new proposed distribution pattern of components indicates in almost near to equivalent portion of incorporation for the stories in the total damage cost of building.

For more evidently demonstrating the effects of the recommended modified distribution of components on the contribution portion of each story in different models, Figures 6 to 8 could be very supportive presenting intensification in participation part of lower stories and reduction in participation part of upper stories in the total damage cost especially for low intensity earthquakes.

It is noteworthy that uniformly distributed costs in height of building do not insure less total damage cost in all cases. For example, for a model near to collapse, the strategy for reduction of total damage cost and for preventing collapse is to decrease incorporations of lower stories in the damage procedure and also in damage cost; though the equivalent incorporation of stories in damage costs could insure the profitability of the built investment in each story; Noticing the fact that nonstructural components account for most of the total investment in a typical office building. While for the models of this study, the evaluations exhibit reduction in total amounts of damage costs in all situations just by employment of the proposed modifications in the pattern of component distribution in height without any alternation in specification or quantity of the incorporated components.

Table 12. Percentage of incorporation for each story in the total damage cost and the percentage of total damage cost to the total replacement cost of building (DR) according to different fundamental periods and different intensity levels

Periods Scaling Levels	3-Story (T = 0.68 s)				4-Story (T = 0.85 s)				5-Story (T = 1.12 s)			
	1 st St.	Type St.	Top St.	DR %	1 st St.	Type St.	Top St.	DR %	1 st St.	Type St.	Top St.	DR %
0.5 DLE	0.335	0.341	0.324	7.99	0.233	0.268	0.231	7.85	0.182	0.210	0.188	7.86
1.0 DLE	0.323	0.354	0.323	16.39	0.230	0.261	0.248	26.52	0.187	0.211	0.180	26.04
1.5 DLE	0.344	0.342	0.314	31.51	0.242	0.263	0.232	48.96	0.178	0.207	0.201	39.20
2.0 DLE	0.340	0.341	0.319	43.32	0.258	0.257	0.228	60.83	0.181	0.206	0.201	55.74
2.5 DLE	0.337	0.348	0.315	45.36	0.264	0.262	0.212	74.17	0.170	0.207	0.209	67.78

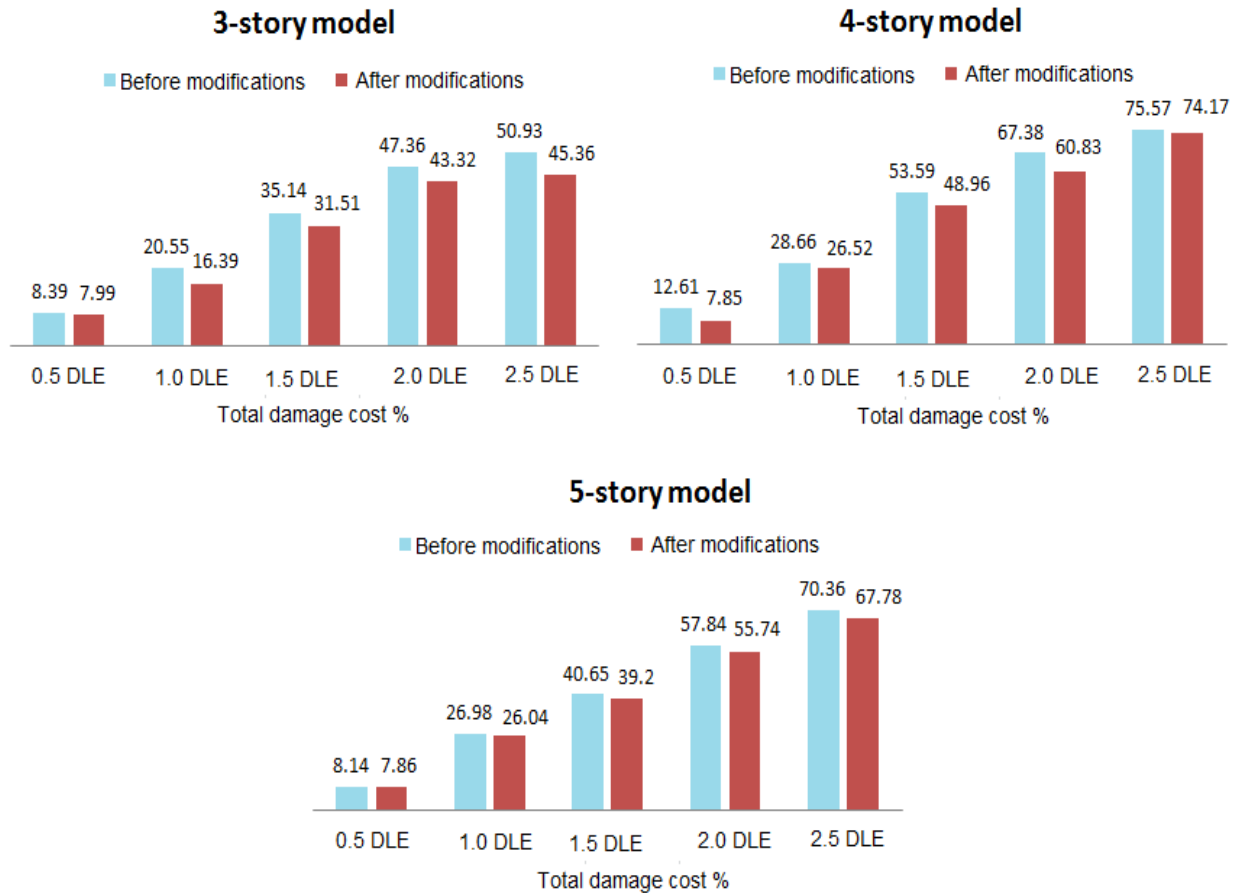


Fig. 5. Percentage of total damage cost for models with different number of stories before and after conducting the proposed modifications for components' distribution in height

CONCLUSIONS

This paper illustrates the significant role of distribution of nonstructural performance groups in height of models and its impact on stories incorporations and thus profitability of investments and also in the amounts of the total damage costs by assessing the cost of damage as a performance merit for two

competing nonstructural distributions (one commonly applied which has been recommended in associated codes and one modified distribution which has been proposed by the authors). Some of the obtained conclusions from the paper could be mentioned as:

- The strategies of situating displacement or acceleration-

dependent components could be determined according to some factors like intensity level of earthquake, number of stories and period of the building.

- In small earthquake intensities by conveying acceleration-dependent elements to the lower stories, the cost of damage could be reduced according to less gained accelerations in these stories and also according to further incorporation of lower stories in

building damage costs. In the other hand, by increasing the scaling level of spectral acceleration intensity, if one could distribute displacement-dependent component in a way to afford larger portion of contribution to the upper stories, or by altering the type of components from displacement-dependent to acceleration-dependent by well-done anchoring the components, the cost of damage has been decreased.

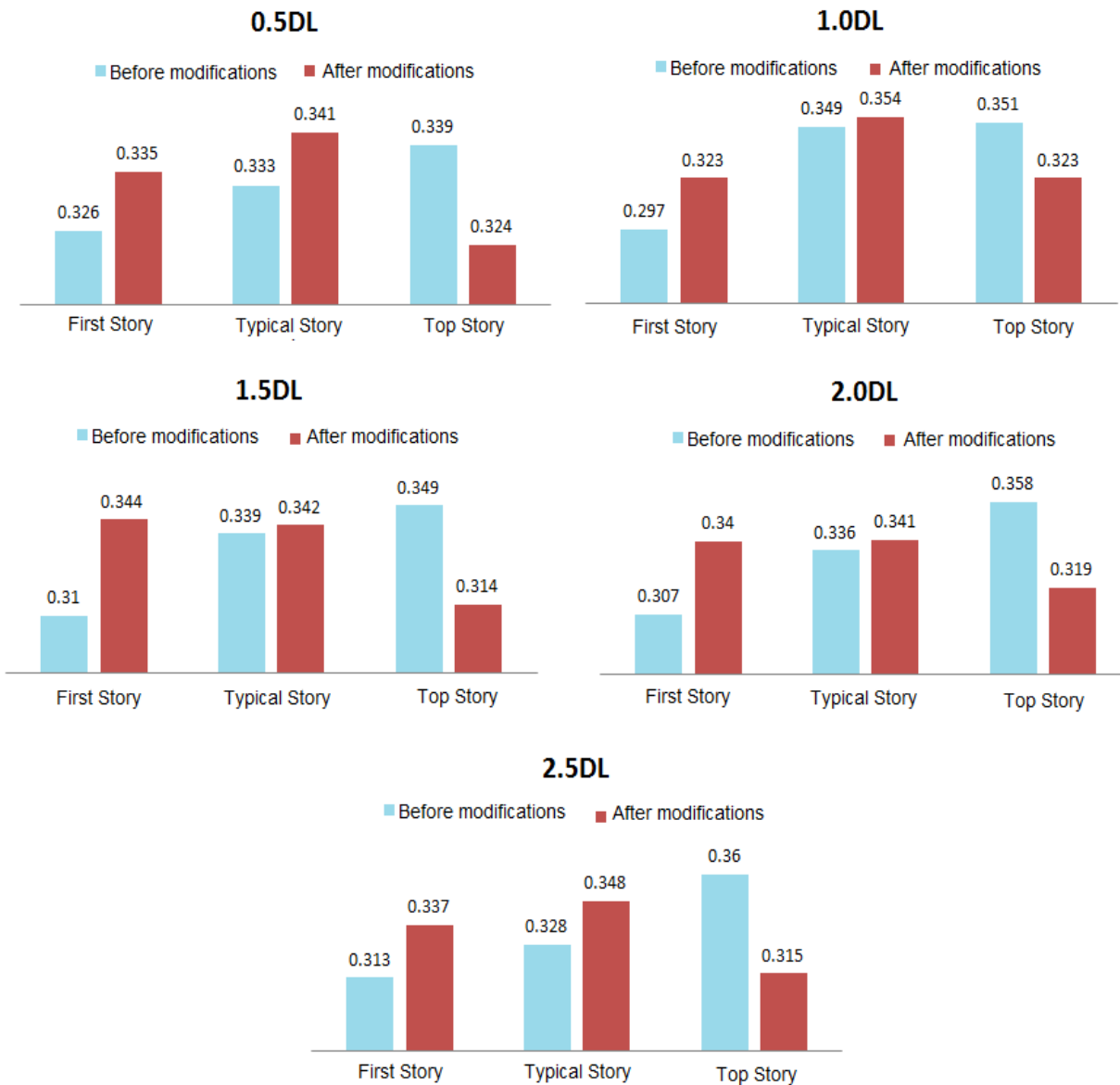


Fig. 6. Percentage of incorporation of stories in total damage cost for 3-Story model subjected to different scaling intensity levels for two competing nonstructural distributions

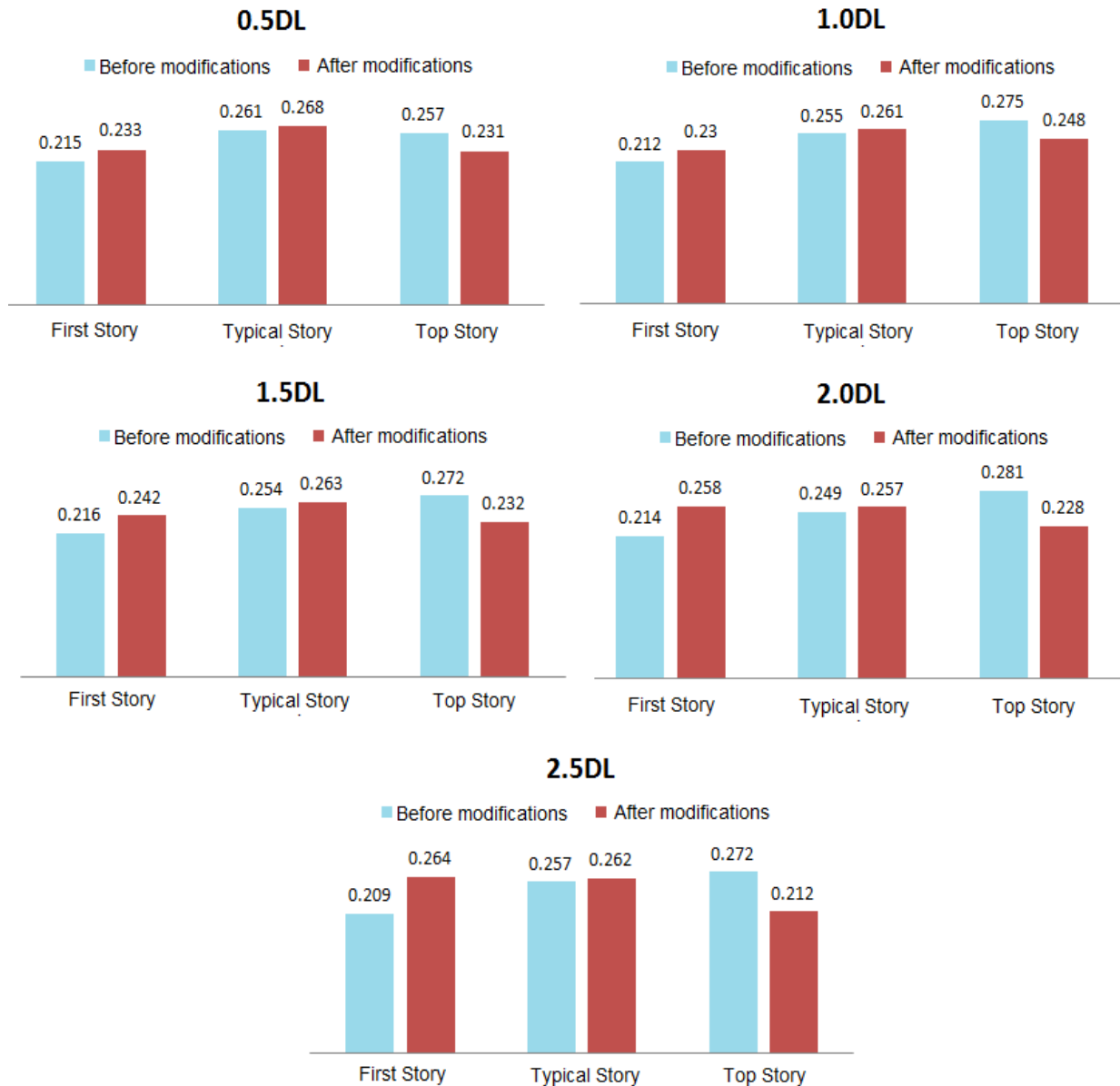


Fig. 7. Percentage of incorporation of stories in total damage cost for 4-Story model subjected to different scaling intensity levels for two competing nonstructural distributions

- By the straightforward proposed modifications without any alternation in the amounts of performance groups or in structural characteristics of the models, the cost of damage in all of the models has been declined subjected to all assumed fundamental periods and all intensity levels. This reduction could save great expenses particularly for the models with large areas and thus large initial costs.
- By conducting the proposed modifications, the portions of incorporation for the first and typical stories increase especially for low scaling levels of intensities. This amplification is more intense in the first floor than the typical ones and also in 3-story building than the 4 or 5-story ones; because of fewer number of typical stories permits to the first floor to play more superior role in

making cost of damage.

- New proposed distribution pattern of nonstructural components indicates in almost near to equivalent portion of incorporation for the stories in the total damage cost of building. Although uniformly distributed costs in height of building do not insure less total damage cost in all cases, the equivalent incorporation of stories in damage costs could insure the profitability of the built investment in each story; Noticing the fact that

nonstructural components account for most of the total investment in a typical office building. While for the models of this study, the evaluations exhibit reduction in total amounts of damage costs in all situations just by application of the proposed modifications in the pattern of component distribution in height without any alternation in specification or quantity of the incorporated components.

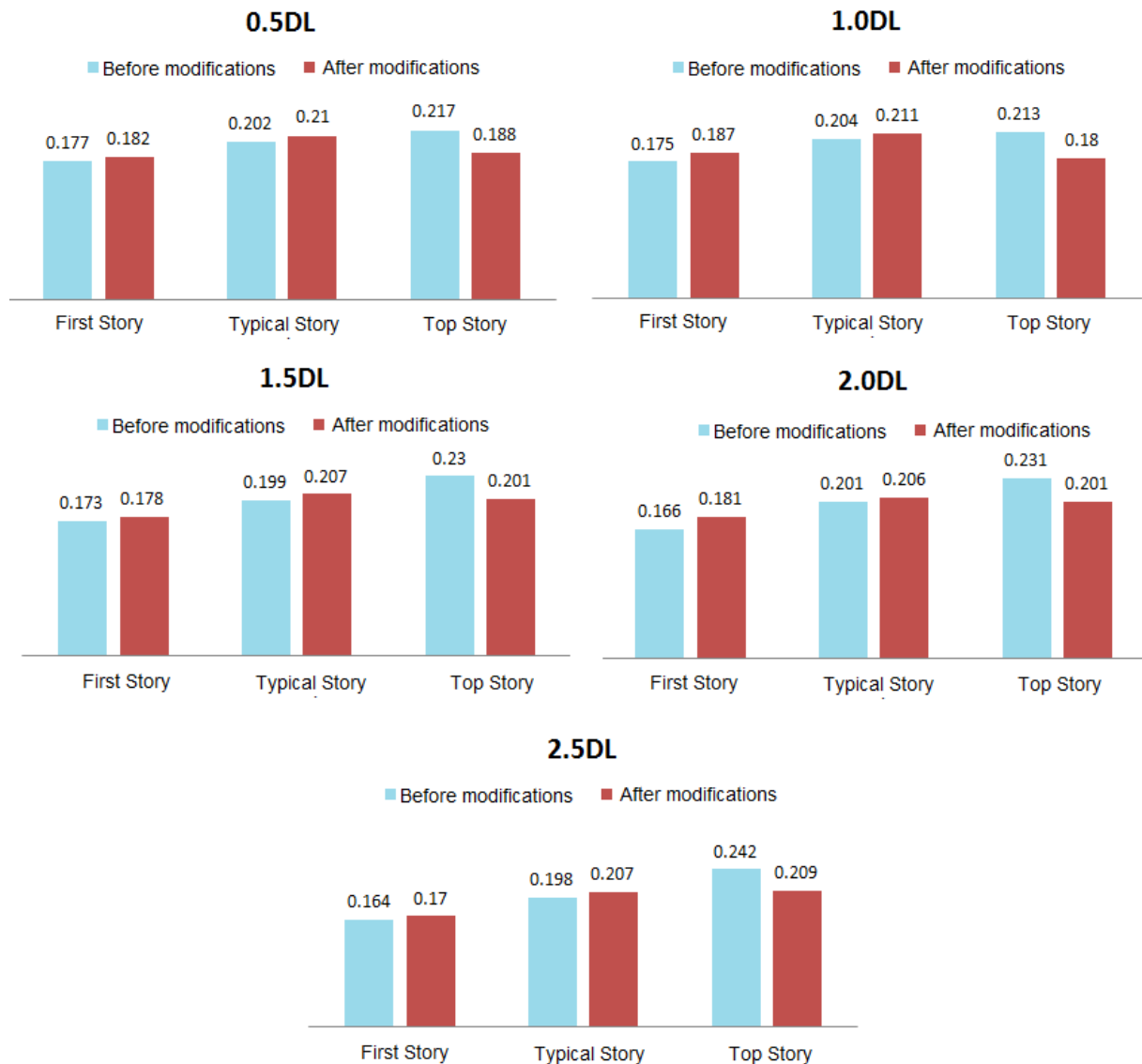


Fig. 8. Percentage of incorporation of stories in total damage cost for 5-Story model subjected to different scaling intensity levels for two competing nonstructural distributions

REFERENCES

- AISC. (2005). *Manual of steel construction, AISC 2005*. American Institute of Steel Construction, Chicago, Illinois, United States.
- Alavi, B. and Krawinkler, H. (2000). "Consideration of near-fault ground motion effects in seismic design", *Proceeding of the 12th World Conference on Earthquake Engineering*, Auckland, New Zealand.
- ASCE. (2007). *Seismic rehabilitation of existing buildings, ASCE/SEI 41-06*. American Society of Civil Engineers, Reston, United States.
- ATC-58. (2011). *Guidelines for seismic performance assessment of buildings, ATC-58*. Applied Technology Council, Washington D.C. Retrieved October 13, 2014, from <https://www.atccouncil.org/pdfs/ATC-58-50percentDraft.pdf>
- ASCE. (2010). *Minimum design loads for buildings and other structures, ASCE/SEI 7-10*. American Society of Civil Engineers, Reston, Virginia, United States.
- Aslani, H. and Miranda, E. (2005). "Probabilistic earthquake loss estimation and loss disaggregation in buildings", Report 157, Ph.D. Dissertation, John A. Blume Earthquake Engineering Center, Stanford University, United State.
- Balaboni, B. (2014). *RSmeans square foot costs 2014*, RSMMeans Engineering Department, 35th ed. New York, United States.
- Boroschek, R. and Retamales, R. (2001). "Damage observed in El Salvador's public hospital system during the January 13, 2001 earthquake", WHO/PAHO Collaborating Center for Disaster Mitigation in Health Facilities, University of Chile, Santiago, Chile.
- Chock, G., Robertson, I., Nicholson, P., Brandes, H., Medley, E., Okubo, P., Hirshorn, B., Sumada, J., Kindred, T., Linurna, G., Sarwar, A., Dal Pino, J. and Holmes, W. (2006). "Compilation of observations of the October 15, 2006, Kiholo Bay (M_w 6.7) and Mahukona (M_w 6.0) earthquakes, Hawaii", Report 31, Earthquake Engineering Research Institute, Oakland, United State.
- COSMOS Ground Motions Databases, Retrieved May 05, 2016, from <http://db.cosmos-eq.org/scripts/default.plx>
- FEMA. (2009). *Quantification of building seismic performance factors, FEMA P695*, Building Seismic Safety Council for the Federal Emergency Management Agency, Federal Emergency Management Agencies, Washington D.C.
- FEMA. (2012). *Seismic performance assessment of buildings, Volume 1 – Methodology, FEMA P-58-1*, Building seismic safety council for the Federal Emergency Management Agency, Federal Emergency Management Agencies, Washington D.C.
- FEMA. (2012), *Seismic performance assessment of buildings, Volume 3 - Supporting Electronic Materials and Background Documentation: 3.1 Performance Assessment Calculation Tool (PACT), Version 2.9.65, FEMA P-58-3*, Building Seismic Safety Council for the Federal Emergency Management Agency, Federal Emergency Management Agencies, Washington, D.C.
- Filiatrault, A. and Christopoulos, C. (2002). "Guidelines, specifications, and seismic performance characterization of nonstructural building components and equipment", Report PEER 2002/05, Pacific Earthquake Engineering Research Center, Berkeley, CA.
- Filiatrault, A., Kuan, S. and Tremblay, R. (2004). "Shake table testing of bookcase – partition wall systems", *Canadian Journal of Civil Engineering*, 31(4), 664-676.
- Filiatrault, A. and Sullivan, T. (2014). "Performance-based seismic design of nonstructural building components: The next frontier of earthquake engineering", *Earthquake Engineering and Engineering Vibration*, 13(1), 17-46.
- French, S.P. (2012). "Modeling nonstructural damage for metropolitan building stocks", *Proceedings of the 15th World Conference on Earthquake Engineering*, Lisbon, Portugal.
- Gupta, A. and Krawinkler, H. (1999). "Seismic demand for performance evaluation of steel moment resisting frames structures", Report 132, Ph.D. Dissertation, John A. Blume Earthquake Engineering Center, Stanford University, United State.
- Gupta, A. and McDonald, B. M. (2008). "Performance of building structures during the October 15, 2006 Hawaii earthquake", *Proceedings of the 14th World Conference on Earthquake Engineering*, Beijing, China.
- Haj Najafi, L. and Tehranizadeh, M. (2015a). "Ground motion selection and scaling in practice", *Peryodica Polytechnica - Civil Engineering*, 59(2), 233-248.
- Haj Najafi, L. and Tehranizadeh, M. (2015b). "Selecting appropriate intensity measure in view of efficiency", *Civil Engineering Infrastructures Journal*, 48(2), 269-287.
- Haselton, C. B. and Deierlein, G. G. (2007). "Assessing seismic collapse safety of modern reinforced concrete frame buildings", Technical

- Report 156, Ph.D. Dissertation, John A. Blume Earthquake Engineering Center, Stanford University, United States.
- Haselton, C.B., Goulet, C.A., Mitrani-Reiser, J., Beck, J.L., Deierlein, G.G., Porter, K.A., Stewart, J.P. and Taciroglu, E. (2008). “An assessment to benchmark the seismic performance of a code-conforming reinforced concrete moment-frame building”, PEER Report 2007/12, Pacific Earthquake Engineering Research Center, College of Engineering, University of California, Berkeley, United States.
- Ibarra, L. F., Medina, R. A. and Krawinkler, H. (2005). “Hysteretic models that incorporate strength and stiffness deterioration”, *Earthquake Engineering and Structural Dynamic*, 34(12), 1489-1511.
- K-Net. (2015). “Strong-motions seismograph networks”, Retrieved May 05, 2016, from <http://www.k-net.bosai.go.jp>
- Krawinkler, H. and Medina, R. (2004). “Seismic demands for nondeteriorating frame structures and their dependence on ground motions”, Report EER 2003/15, Pacific Earthquake Engineering Research Center, University of California at Berkeley, Berkeley, CA.
- Lignos D.G., Krawinkler H. and Whittaker, A.S. (2011). “Prediction and validation of sideway collapse of two scale models of a 4-story steel moment frame”, *Earthquake Engineering and Structural Dynamics*, 40(7), 807-825.
- Lignos, D.G. and Krawinkler, H. (2012). “Sideway collapse of deteriorating structural system under seismic excitations”, Report 177, Ph.D. Dissertation, John A. Blume Earthquake Engineering Center, Stanford University, United State.
- Lignos, D.G. and Krawinkler, H. (2013). “Development and utilization of structural component databases for performance-based earthquake engineering”, *Journal of Structural Engineering*, 139(8), 1382-1394.
- Lignos, D.G. (2014). “Modeling steel moment resisting frames with OpenSees”, *OpenSees Workshop*, University of California, Berkeley, United States.
- McGavin, G. and Patrucco, H. (1994). “Survey of nonstructural damage to healthcare facilities in the January 17, 1994 Northridge earthquake”, Report prepared for HMC Group, Ontario, Canada.
- Mehanny, S. S. F. (1999). “Modeling and assessment of seismic performance of composite frames with reinforced concrete columns and steel beams”, Report 135, Ph.D. Dissertation, John A. Blume Earthquake Engineering Center, Stanford University, United State.
- Miranda, E. and Ramirez, C.M. (2009). “Building-specific loss estimation methods and tools for simplified performance-based earthquake engineering”, Report 171, Ph.D. Dissertation, John A. Blume Earthquake Engineering Center, Stanford University, United State.
- Nau, J. and Hall, W. (1984). “Scaling methods for earthquake response spectra”, *Journal of Structural Engineering, ASCE*, 110(2), 91-109.
- NIST. (1999). *UNIFORMAT II elemental classification for building specifications, cost estimating, and cost analysis, NISTIR 6389*, National Institute of Standards and Technology, Washington, D.C., Retrieved May 05, 2016, from <http://www.bfrl.nist.gov/oaepublications/nistir/6389.pdf>
- NIST. (2011). *Soil-structure interaction for building structures, NIST/GCR 11-917-14*, NEHRP Consultants Joint Venture for the National Institute of Standards and Technology, Gaithersburg, Maryland.
- Pacific Earthquake Engineering Research Center, Retrieved May 05, 2016, from www.peertestbeds.net
- PEER Strong Ground Motion Database, Retrieved May 05, 2016, from http://peer.berkeley.edu/peer_ground_motion_database.
- Porter, K.A. (2005). “A taxonomy of building components for performance-based earthquake engineering”, Department of Civil Engineering and Applied Mechanics California Institute of Technology, PEER Award number EEC-9701568, Report PEER 2005/03, Pacific Earthquake Engineering Research Center, University of California at Berkeley, Berkeley, CA.
- Porter, K.A., Johnson, G. Sheppard R. and Bachman, R. (2011). “Response to discussions of fragility of mechanical, electrical and plumbing equipment”, *Earthquake Spectra*, 27(1), 229-233.
- Ramirez, C. and Miranda, E. (2009). “Building-specific loss estimation methods and tools for simplified performance-based earthquake engineering”, Report 171, Ph.D. Dissertation, John A. Blume Earthquake Engineering Center, Stanford University, United States.
- Ruiz-García, J. and Miranda, E. (2006). “Residual displacement ratios for assessment of existing structures”, *Earthquake Engineering and Structural Dynamics*, 35(3), 315-336.
- Ruiz-García, J. and Miranda, E. (2008). “Probabilistic seismic assessment of residual drift demands in existing buildings”, *Proceeding of the 14th World Conference on Earthquake Engineering* Beijing, China.
- Shahraki, H. and Shabakhty, N. (2015). “Seismic performance reliability of RC structures:

- Application of response surface method and systemic approach”, *Civil Engineering Infrastructures Journal*, 48(1), 47-68.
- Shome, N., Cornell, C.A., Bazzurro, P., and Carballo, J.E. (1998). “Earthquakes, records and nonlinear responses”, *Earthquake Spectra*, 14(3), 469-500.
- Soong, T.T. (1995). “Seismic behavior of nonstructural elements-state-of-the-art-report”, *Proceedings of the 10th European Conference on Earthquake Engineering*, Vienna, Austria.
- Taghavi, S., and Miranda, E. (2003). “Response of nonstructural building elements”, Report PEER 2003/05, Pacific Earthquake Engineering Research Center, University of California at Berkeley, Berkeley, CA.
- Tehranizadeh, M. and Movahed, H. (2011). “Evaluation of steel moment-resisting frames performance in tall buildings in near fault areas”, *Civil Engineering Infrastructures Journal*, 44(5), 621-633.
- Vamvatsikos, D. and Cornell C.A. (2002). “Incremental dynamic analysis”, *Earthquake Engineering and Structural Dynamics*, 31(3), 491-514.
- Zareian, F. and Medina, R.A. (2010), “A practical method for proper modeling of structural damping in inelastic plane Structural systems”, *Computers and Structures*, 88(1-2), 45-53.