16th World Conference on Earthquake, 16WCEE 2017
Santiago Chile, January 9th to 13th 2017
Paper N° 2946
Registration Code: S-A1463177826

FRAGILITY ASSESSMENT OF BUILDINGS WITH RIGID WALLS/FLEXIBLE ROOF DIAPHRAGMS SUBJECTED TO EARTHQUAKE AND TORNADO

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Abstract

Buildings with rigid walls and flexible roof diaphragms (RWFD) are a common type of single-story construction in North and South America, Europe and New Zealand that incorporate rigid in-plane concrete or masonry walls and flexible in-plane wood or steel roof diaphragms.

In this study a fragility analysis methodology was developed to assess the response of RWFD buildings exposed to extreme seismic and wind (tornadoes) events. RWFD buildings incorporating concrete tilt-up wall panels and flexible steel roof diaphragms were considered. The performance goals and structural limit states considered for the fragility assessment of RWFD buildings were developed based on the observed performance of RWFD buildings during past tornadoes and earthquakes in the United States. Fragilities were developed for selected common building configuration and construction of the RWFD structures along the vertical and horizontal load paths for wind and seismic events, respectively. The results of this fragility assessment can be further considered for providing effective strategies to improve the structural safety and response of RWFD buildings as well as to mitigate socio-economic losses from competing natural hazards.

Keywords: Fragility Analysis; Earthquake Loads; Wind Loads; Performance Assessment

1. Introduction

Building with Rigid in-plane Walls and Flexible in-plane roof Diaphragms (RWFD) are widely used in single story light industrial construction in North and South America, Europe and New Zealand. These buildings are framed with exterior tilt-up concrete or masonry walls, interior columns and horizontal roof diaphragms. The roof diaphragms are constructed as wood or steel systems. Wood roof systems consist of a plywood or oriented strand board (OSB) deck fastened to the wood framing using common nails, while steel roof systems are framed with corrugated steel deck, bar joists, and joist girders.

RWFD buildings are highly susceptible to strong winds, such as tornadoes and have been significantly damaged during past tornadoes in the United States including the 2011 Joplin, 2013 Moore, 2013 St. Charles and 2015 Lancaster tornadoes [1]. Roof deck panel-to-joist failures, roof collapse in building corners nearest to the tornado path, roof deck failure, and open web joist failure leading to collapse of the exterior wall panels were the most common failure modes observed during such events. Furthermore, RWFD buildings have shown vulnerable response during historical earthquakes including the 1964 Alaska, 1989 Loma Prieta, 1994 Northridge, and 2010 and 2011 Christchurch earthquakes [2-4]. Damage due to insufficient roof-to-wall out-of-plane anchorage capacities was mainly observed in these events, while it was identified that the seismic response of RWFD buildings was highly dominated by the in-plane response of the roof diaphragm.

Damage to RWFD buildings due to tornado and earthquake loads has caused significant direct and indirect economic losses throughout the last few decades in the United States. Direct economic losses were associated with repair and replacement of the building stock as well as building contents and inventory, while indirect losses were related to the impact of the direct economic losses in the community and business evolution. Understanding and assessing the performance of these structures during these two competing hazards is essential for estimating and preventing such economic losses, while providing effective policies to improve structural safety.

Fragility analysis has been widely used in performance-based engineering (PBE) over the last decades to assess the response of infrastructure systems. In the context of PBE, fragility assessment of typical RWFD building archetypes subjected to tornado and earthquake hazards is conducted in this study. This study revealed that the building size (footprint) significantly affects the performance of RWFD buildings subjected to extreme wind loads, while the roof connector variability does not influence their response, given that the roof joist failure was observed prior to connector failure. On the other hand, the performance of RWFD buildings subjected to earthquake loads is highly related to the roof diaphragm connectors used in the design phase.

2. Description of Building Archetypes

Two typical RWFD building archetypes were considered in this study, which were originally designed for seismic loads, as described in [5] and [6]. The building archetypes were designed for US seismic design category C_{max} (S_{D1}=0.2 and S_{DS}=0.5) and Risk group II, according to current seismic codes and provisions including the 2012 IBC [7] and ACI 318-11 [8]. Both RWFD archetypes were framed with 9.14m tall, 7.62m wide and 184mm thick tilt-up wall panels as well as 22ga wide-rib steel roof deck and steel joists. The buildings' plan dimensions were 121.92m x 121.92m. A summary of the building archetypes' roof connector details is provided in Table 1.

Table 1- Summary of building archetypes and their characteristics

Archetype ID	Roof connectors detail
1	Framing: puddle welds - 19.1mm & 15.9 mm; Sidelap: screws
2	Framing: puddle welds - 19.1mm; Sidelap connectors: top seam welds

3. Fragility Modeling

Fragility analysis is a key concept to assess the response of RWFD buildings subjected to earthquake and tornado loads. Fragility curves presenting the conditional probability of exceeding a limit state (LS) of a specified engineering demand parameter (EDP) under given hazard were generated. The fragility curve is modeled by a lognormal cumulative distribution function (CDF) described as:

$$Fragility = \Phi\left(\frac{\ln(IM) - \mu}{\beta}\right) = P(EDP > LS \mid IM) \tag{1}$$

where, IM is the hazard intensity measure considered for each hazard, μ is the mean of the natural logarithm of the intensity/capacity based on the EDP considered, β is the standard deviation of the natural logarithm, $\Phi(\cdot)$ is the standard normal distribution function $P(\cdot)$ is the probability function and LS is the limit state values associated with the EDP considered.

The EDP considered is usually associated with the most critical structural damage observed during past events, while the IM represents the hazard investigated. The damage states (DS) for each hazard based on the EDPs considered should also be defined in terms of physical damage as well as their limit state (LS) values.

The 3-sec gust wind speed and the spectral acceleration at the fundamental period of the buildings ($S_a[T_1]$) were the IMs considered for the fragility curves under tornado and earthquake loading, respectively. The EDPs considered in this study for tornado and earthquake fragility assessment are associated with the performance of RWFD buildings during past events. More specifically, typical degrees of damage (DoDs) for tornado loads including: (i) loss of roof covering and siding (low DoDs), (ii) loss of the roof and exterior walls (moderate DoDs), and (iii) cleaning to the building slab, e.g. for wood-frame buildings (severe DoDs), were accounted in for defining the EDPs for tornado loading. The percentage failure of roof cover, doors, roof deck panels, steel joists, and integrity of tilt-up wall panels were also considered as tornadic EDPs. For seismic loads, the inter-story or residual drift is a commonly used EDP, however, these EDPs are not representative for RWFD buildings. Observations during past earthquakes as well as analytical/computational studies have shown that the global system response is dominated by the response of the roof diaphragm [9], thus the roof diaphragm drift ratio (*DDR*) was considered in this study as the representative seismic EDP. The *DDR*, which was used in studies of tilt-up structures as a representative EDP [5, 10], is defined as:

$$DDR(\%) = \frac{x_{mid,roof}}{\left(L_{roof}/2\right)} \times 100 \tag{2}$$

where, $x_{mid,roof}$ is the displacement at the center of the roof diaphragm, and L_{roof} is the horizontal span of the roof diaphragm.

Four damage states (DS) were considered for the fragility performance assessment following the HAZUS MH [11] nomenclature to allow consistency with existing studies. Details of the damage states (*Slight, Moderate, Extensive and Complete*) associated with each hazard are presented in Table 2.

Table 2- Summary of damage states and limit state values

Damage State ID	Damage State Description	Hazard Type*	Physical Damage Description	Limit States Description
1	Slight	EQ	Minor deformations in roof diaphragm connections or hairline cracks in a few welded roof connections.	0.0004<*DDR*<0.0015
		Torn.	Moderate roof cover loss that can be covered with a tarp to prevent additional water from entering the building.	Roof cover failure: 2%-15% Door failure: None Roof deck failure: None Roof joist failure: None Wall failure: None
2	Moderate	EQ	A few welded connections may exhibit major cracks through welds or a few bolted connections may exhibit broken bolts or enlarged bolt holes mainly towards the end roof regions.	0.0015<*DDR*<0.0035
		Torn.	Major roof cover damage with a maximum of two roof deck and one door failure.	Roof cover failure: >15% Door failure: 1 Roof deck failure: 1 or 2 Roof joist failure: None Wall failure: None
3	Extensive	EQ	Some connections may have exceeded their ultimate capacity exhibited by major permanent member rotations at connections and failed connections. Partial collapse of portions of the roof structure due to failed end connections.	0.0035<*DDR*<0.0080
		Torn.	Not able to be occupied, but repairable. Major loss of roof deck panels or joists as well as more than one broken door. Roof cover failure is certain and does not contribute in DS 3.	Roof cover failure: N/A Door failure: >1 Roof deck failure: 3 panels and ≤35% Roof joist failure: ≤15% Wall failure: None
4	Complete	EQ	Significant portion of the roof connections have exceeded their ultimate capacities and have failed resulting in dangerous permanent lateral displacement, partial collapse or collapse of the building resulting also from failed out-of-plane roof-to-wall anchorages.	*DDR*>0.0080
		Torn.	Not able to be occupied and not repairable. Extensive roof system failure and some tilt-up wall failure. Roof cover and door failures are certain and do not contribute in DS 4.	Roof cover failure: N/A Door failure: N/A Roof deck failure: >35% Roof joist failure: >15% Wall failure: Yes

* EQ= earthquake; Torn.=tornado

4. Seismic Fragility Analysis of RWFD Buildings

A three-step numerical framework introduced by [12, 13] was used to conduct seismic fragility analysis. The three-step numerical framework is based on a sub-structuring approach, as illustrated in Fig. 1, and includes the following steps: (i) development of roof connector database and identification of the Wayne-Stewart [14] hysteretic parameters for each connector type, (ii) development of an analytical roof diaphragm model in MATLAB accounting for the shear deformation of the roof deck as well as the nonlinear response of the roof connectors (using Wayne-Stewart hysteretic springs) and (iii) development of a simplified building model incorporating roof diaphragm inelastic spring with properties identified in Step 2. A detailed description of the numerical framework is provided in [9, 12, 13].

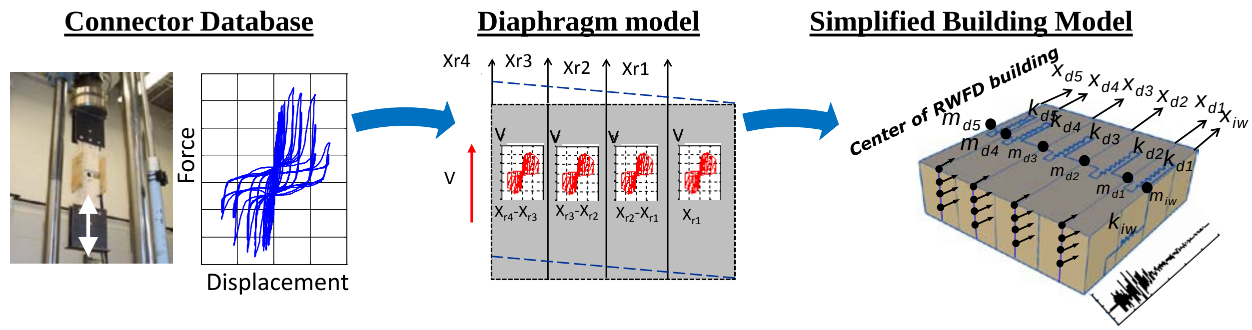
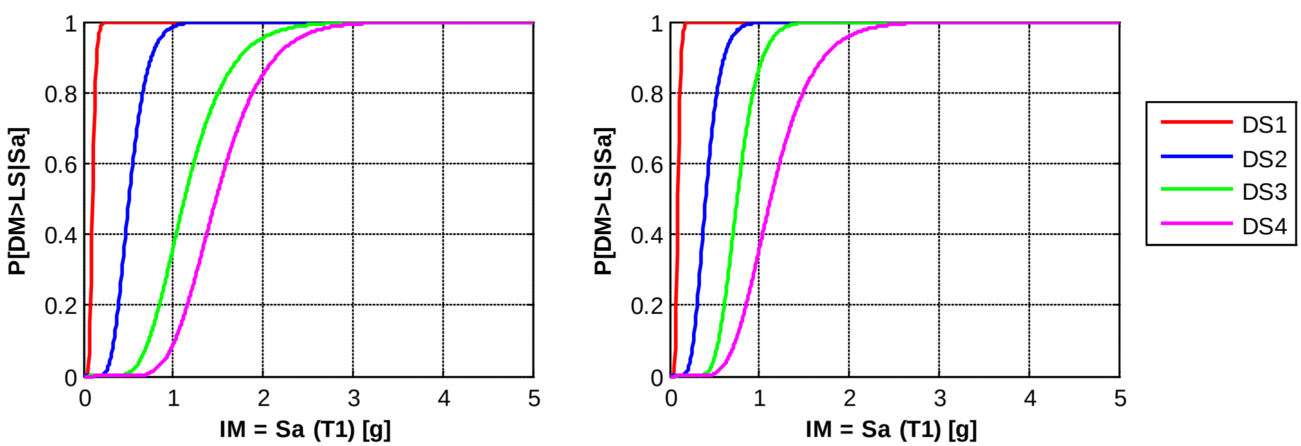


Fig. 1 - Three step sub-structure modeling framework used for non-linear time history analyses

Nonlinear dynamic response analyses were conducted for the building archetypes using the FEMA P695 Far-Field Ground motion ensemble [15]. To estimate the probability of exceeding a certain limit state for the RWFD buildings, each ground motion was scaled to increasing earthquake intensities (Incremental Dynamic Analyses - IDA) [16]. Considering the IDA results, fragility curves were generated to represent the conditional probability of exceeding a specified limit state of the *DDR* as a function of the seismic intensity. Note that the median intensity and the standard deviation were computed from the IDAs, while. The median intensity value was defined as the median 2% damped spectral acceleration at the fundamental period of the building archetype for which 50% of the earthquake motions exceed a certain limit state. The fragility curves showing the probability of exceeding a specified limit state are plotted in Fig. 2, while the respective parameters of the lognormal distribution are presented in Table 3.

Table 3 - Summary of fragility distribution function parameters for RWFD buildings subjected to earthquake [17]

Archetype ID	**DS 1**		**DS 2**		**DS 3**		**DS 4**	
	μ	β	μ	β	μ	β	μ	β
1	-2.23	0.28	-0.67	0.31	0.12	0.34	0.39	0.29
2	-2.31	0.25	-0.90	0.32	-0.28	0.25	0.12	0.33



(a) (b)

Fig. 2 - Lognormal distribution fragility functions of RWFD archetypes subjected to earthquake hazard for archetypes IDs: (a) 1, (b) 2 [17]

Based on the results of this study, it was determined that RWFD buildings sharing the same footprint subjected to earthquake loading exhibit very similar response for damage state 1, while their responses diverge for higher damage states. This indicates that the roof connector variation accounted for in the design of the RWFD archetypes controls their response under extreme ground shaking. This is an expected observation since the roof diaphragm connectors represent the main source of nonlinearity in the roof diaphragm and play an important role in their failure mode. Therefore, it is recommended that the roof diaphragm connectors be explicitly accounted for in the modeling of RWFD structures when conducting fragility analysis.

5. Fragility Analysis of RWFD Buildings subjected to Tornado Loads

A simple methodology was considered in this study to model the tornado-induced load based on the ASCE 7 methodology [18, 19] for straight-line wind loads adopting modifications in order to represent the tornado wind loading conditions. The tornado wind pressure is computed [20] as follows:

$$p = q_h\left[T_e\left(GC_p\right) - T_i\left(GC_{pi}\right)\right](N/m^2) \tag{3}$$

where, q_h is the velocity pressure evaluated at mean roof height h, G is the gust-effect factor, GC_p is the external pressure coefficient, GC_{pi} is the internal pressure coefficient, T_e is the tornado external pressure adjustment, and T_i is the tornado internal pressure adjustment.

For the main wind force resisting system (MWFRS) and components and claddings (C&C), the velocity pressure is calculated as:

$$q_h = 0.00256 K_h K_{zt} V^2 (N/m^2) \tag{4}$$

where, K_h is the velocity pressure exposure coefficient at mean roof height h (based on exposure C), K_{zt} is the topographic factor which is set equal to 1.0 in this study, V is the 3-sec gust wind speed (m/s)

In order to account for the uncertainties related to the tornado external pressure adjustment, T_e, and tornado internal pressure adjustment, T_i, two approaches, introduced by [20], were considered in this study. The first approach (A) combines the ASCE 7-10 pressure coefficients with the tornado external pressure adjustment factors based on the work of [21], for computing the tornado load values, while approach B considered the ASCE 7-16 pressure coefficients along with a pressure adjustment factors equal to 1.0. A summary of the wind load resistance values is provided in Table 4 and Table 5.

The resistance statistics for each building component, considered in the probabilistic structural analysis of RWFD buildings subjected to tornado loads, are presented in Table 6. The dead load is described by a mean value of 0.48kPa and a coefficient of variation (COV) equal to 0.1 and is modeled by a normal distribution.

Table 4 – Summary of tornado pressure adjustment values (internal and external) for both approaches A and B

<table>
<tr><th colspan="4">Parameters Description</th><th>Approach A</th><th>Approach B</th></tr>
<tr><td rowspan="8">Tornado Pressure Adjustment</td><td rowspan="4">Uplift Pressure</td><td rowspan="2">MWFRS</td><td>T_e</td><td>1.8 - 3.2</td><td rowspan="8">1.0</td></tr>
<tr><td>T_i</td><td>0.0</td></tr>
<tr><td rowspan="2">C&C</td><td>T_e</td><td>1.4 - 2.4</td></tr>
<tr><td>T_i</td><td>0.0</td></tr>
<tr><td rowspan="4">Lateral Pressure</td><td rowspan="2">MWFRS</td><td>T_e</td><td>1.0 - 1.5</td></tr>
<tr><td>T_i</td><td>1.0</td></tr>
<tr><td rowspan="2">C&C</td><td>T_e</td><td>1.2 - 2.0</td></tr>
<tr><td>T_i</td><td>0.0</td></tr>
</table>

Table 5 – Summary of wind load statistics

Wind Load Parameters	Parameters' Description	Statistical Parameters		
		Mean	COV	Distribution
K_h	0-49.2 m	0.82	0.14	Normal
	65.6 m	0.84		
	82.0 m	0.88		
	98.4 m	0.94		
	131.2 m	1.00		
Gust-effect factor (G)	-	0.82	0.10	Normal
Internal pressure coefficient (GC_{pi})	Enclosed Buildings	±0.15	0.33	Normal
	Partially Enclosed Buildings	±0.46		
External pressure coefficient (C_p)	Wall	0.69	0.15	Normal
	Roof	-0.81		
External pressure coefficients (GC_p) – Approach A	Door (Approach A& B)	-0.68	0.12	Normal
	Roof Cover- Zone 1	-0.95		
	Roof Cover- Zone 2	-1.71		
	Roof Cover- Zone 3	-2.66		
	Roof Deck- Zone 1	-0.90		
	Roof Deck - Zone 2	-1.24		
	Roof Deck - Zone 3	-1.52		
	Roof Joist- Zone 1	-0.86		
	Roof Joist - Zone 2	-1.05		
	Roof Joist - Zone 3	-1.05		
External pressure coefficients (GC_p) – Approach B	Roof Cover- Zone 1'	-0.86		
	Roof Cover- Zone 1	-1.62		
	Roof Cover- Zone 2	-2.19		
	Roof Cover- Zone 3	-3.04		
	Roof Deck- Zone 1'	-0.86		
	Roof Deck- Zone 1	-1.33		
	Roof Deck - Zone 2	-1.85		
	Roof Deck - Zone 3	-2.33		
	Roof Joist- Zone 1'	-0.62		
	Roof Joist- Zone 1	-1.05		
	Roof Joist - Zone 2	-1.43		
	Roof Joist - Zone 3	-1.52		

Table 6 – Summary of component resistance parameters for RWFD buildings subjected to tornado loads

Parameters Description		Mean	COV	Distribution
Roof Cover	Flashing Resistance	328.36 N/m	0.30	Normal
	Peeling Resistance	1.92 kPa	0.15	
	Bubbling Resistance	2.87 kPa	0.15	
Doors Resistance		2.39 kPa	0.20	
f_c (Concrete 28-day cylinder strengths)		45.85 MPa	0.13	
f_y (Yield strength of reinforcement bars)		475.74 MPa	0.07	
Joist Connection		111.43 kN	0.20	
19.1 mm. diameter puddle welds		1.96 kN	0.24	
15.9 mm. diameter puddle welds		1.60 kN	0.24	

Monte Carlo Simulation (MCS) for component analysis was conducted in order to develop tornado fragility curves for RWFD buildings following both approaches A and B [17]. A flowchart mapping the tornado fragility analysis is presented in Fig. 3. The building (system) fragility curves were derived for both building archetypes by accounting for the failure probability of each damage indicator (component fragility curves). Since, the damage indicators considered are not necessarily stochastically independent, their correlation was achieved through MCS. The fragility function parameters for both archetypes and all damage states are summarized in Table 7. It is worth mentioning that both building archetypes had nearly identical fragility curves following both approaches (A and B). This is justified since the roof joists fail before the roof connector failures. Therefore, the roof connector variability in the two designs does not affect their response and it is recommended for RWFD buildings of the same footprint to follow a unified fragility response. Fig. 4 and Fig. 5 presents the system as well as the component fragility curves for damage states 3 and 4 following approach A and B, repsectively. Details on the component fragility curves for RWFD buildings subjected to tornado loads is provided in [17].

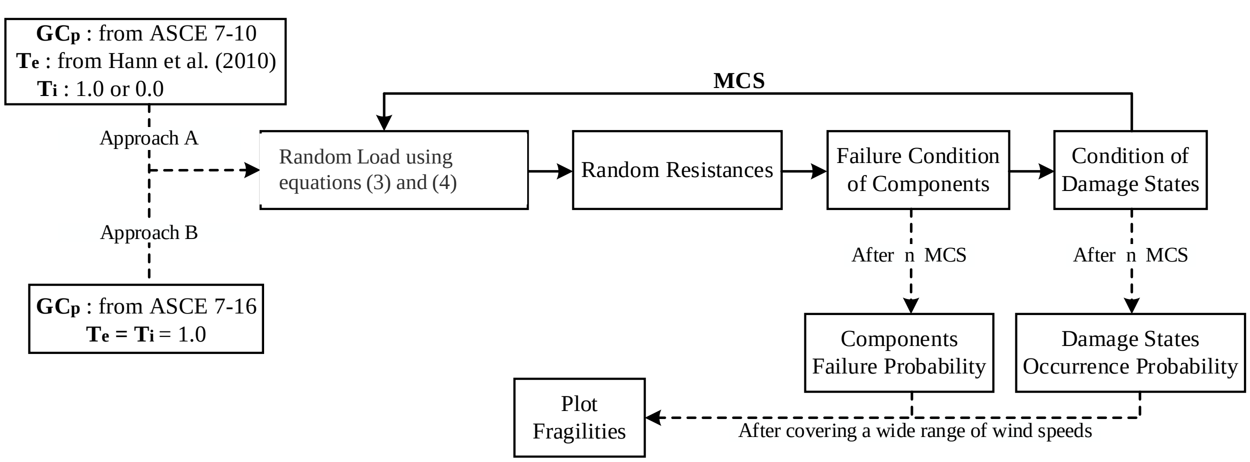
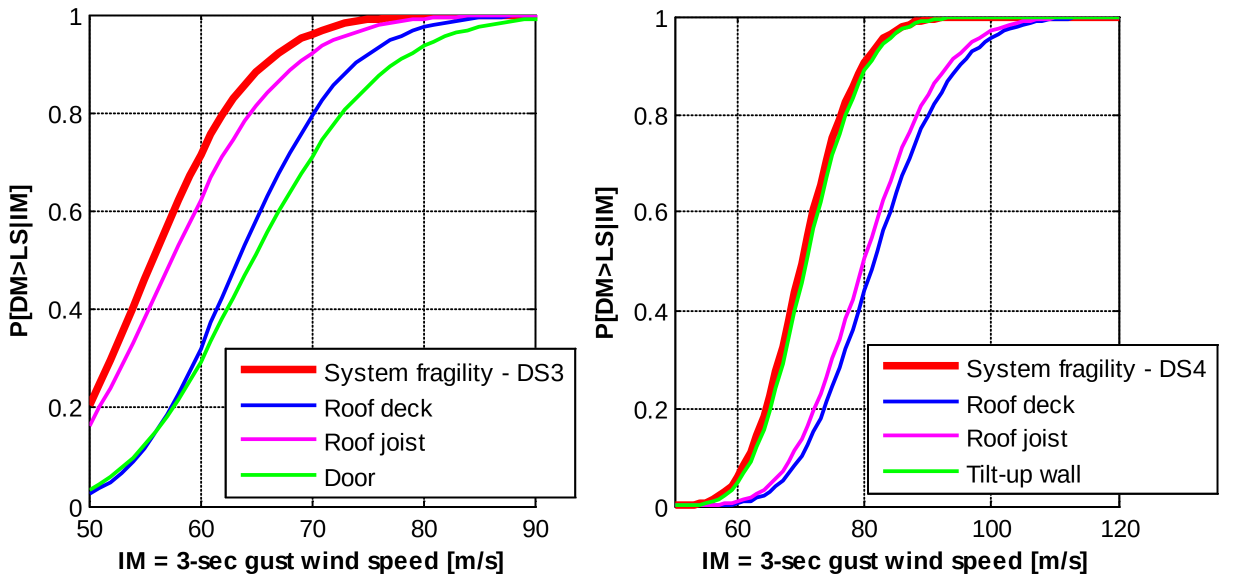


Fig. 3 - Flowchart of fragility analysis of RWFD buildings subjected to tornado loads

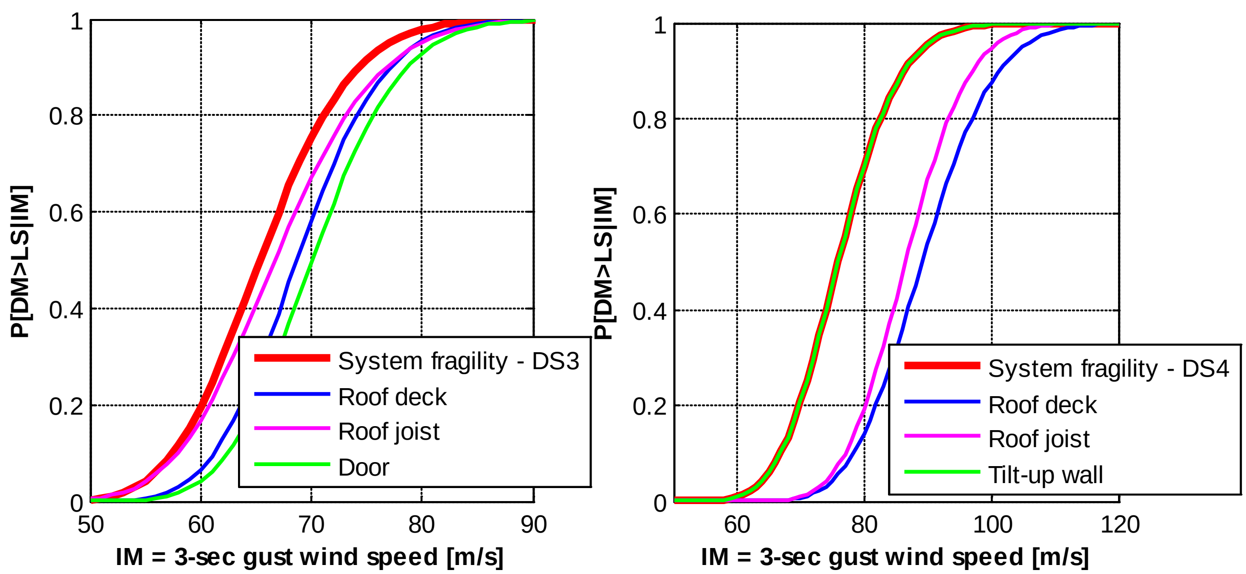
Table 7 – Summary of fragility distribution function parameters for RWFD buildings subjected to tornado loads

Approach	DS 1		DS 2		DS 3		DS 4	
	μ	***β***	***μ***	***β***	***μ***	***β***	***μ***	***β***
A	3.33	0.12	3.56	0.12	4.02	0.13	4.25	0.10
B	3.53	0.09	3.71	0.08	4.18	0.10	4.33	0.10



(a) (b)

Fig. 4 - Lognormal distribution fragility functions of RWFD buildings subjected to tornado loads (Approach A) for damage state: (a) DS3 and (b) DS4



(a) (b)

Fig. 5 - Lognormal distribution fragility functions of RWFD buildings subjected to tornado loads (Approach B) for damage state: (a) DS3 and (b) DS4

6. Summary and Conclusions

A fragility assessment study was presented in this paper for RWFD buildings subjected to two different, but historically damaging, hazards: earthquake and tornado. Based on the results of this study, it was observed that the building size significantly affects the response of RWFD building subjected to tornadic loading conditions, while the roof connector variability does not influence their performance. This is mainly attributed to the fact the roof joists are predicted to fail before the roof deck connectors. On the contrary, for RWFD buildings subjected to seismic loading, the variability or roof diaphragm connectors controls their response and the size of the building does not have a substantial influence.

The fragility curves developed in this study can be further considered to validate and assess the response of RWFD buildings, while mitigating subsequent direct and indirect economic losses and social disruptions due to the occurrence of either hazard. Finally, the system fragilities presented herein and the associated buildings may be able to serve as archetypes for community resilience studies since many businesses critical to the economic health of a community are housed in RWFD buildings.

7. Acknowledgements

The first and third authors acknowledge the support of the cooperative agreement 70NANB15H044 between the National Institute of Standards and Technology (NIST) and Colorado State University. Financial support was provided to the second and third authors as part of the National Science Foundation (NSF) grant CMMI-1452725. The content expressed in this paper are the views of the authors and do not necessarily represent the opinions or views of NIST and/or NSF.

7. References

[1] Maldonado, D., 2013, "Analysis of tilt-up building design and industry standard practices in tornado-prone regions," THE FLORIDA STATE UNIVERSITY.

[2] SEAONC, 2001, "Guidelines for Seismic Evaluation and Rehabilitation of Tilt-Up Buildings and Other Rigid Wall/Flexible Diaphragm Structures," Structural Engineers Association of Northern California, San Francisco, CA.

[3] EQE, 1989, "The October 17, 1989 Loma Prieta Earthquake," EQE Engineering, San Francisco, CA.

[4] Henry, R., and Ingham, J., 2011, "Behaviour of tilt-up precast concrete buildings during the 2010/2011 Christchurch earthquakes," Structural Concrete, 12(4), pp. 234-240.

[5] Koliou, M., Filiatrault, A., Kelly, D. J., and Lawson, J., 2016, "Buildings with Rigid Walls and Flexible Roof Diaphragms. I: Evaluation of Current US Seismic Provisions," Journal of Structural Engineering, 142(3), p. 04015166.

[6] FEMA, 2015, "Seismic Design of Rigid Wall-Flexible Diaphragm Buildings: An Alternate Procedure " FEMA P1026, Federal Emergency Management Agency, Washington, DC.

[7] IBC, 2012, "International Building Code," International Building Council.

[8] ACI, 2011, "Building Code Requirements for Structural Concrete (ACI 318-11) and Commentary," American Concrete Institute.

[9] Koliou, M., 2014, "Seismic Analysis and Design of Rigid Wall - Flexible Roof Diaphragm Structures," Ph.D. , University at Buffalo - The State University of New York.

[10] Bai, J.-W., Hueste, M. B. D., and Gardoni, P., 2015, "Seismic vulnerability assessment of tilt-up concrete structures," Structure and Infrastructure Engineering, 11(9), pp. 1131-1146.

[11] HAZUS-MH, E. L. E. M., 2003, "Technical Manual," Prepared by the National Institute of Building Sciences for Federal Emergency Management Agency.

[12] Koliou, M., Filiatrault, A., Kelly, D. J., and Lawson, J., 2014, "Numerical Framework for Seismic Collapse Assessment of Rigid Wall-Flexible Diaphragm Structures," Proceedings of the 10th National Conference in Earthquake Engineering, Earthquake Engineering Research Institute, Anchorage, AK.

[13] Koliou, M., Filiatrault, A., Kelly, D. J., and Lawson, J., 2016, "Distributed yielding concept for improved seismic collapse performance of rigid wall-flexible diaphragm buildings," Journal of Structural Engineering, 142(2), pp. 1-16.

[14] Stewart, W. G., 1987, "The Seismic Design of Plywood Sheathed Shear Walls," Ph.D. Dissertation, University of Canterbury, Christchurch, New Zealand.

[15] FEMA, 2009, "Quantification of Building Seismic Performance Factors," Federal Emergency Management Agency, Washinghton, DC.

[16] Vamvatsikos, D., and Cornell, A. C., 2002, "Incremental Dynamic Analysis," Earthquake Engineering and Structural Dynamics, 31, pp. 491-514.

[17] Koliou, M., Masoomi, H., and van de Lindt, J. W., 2016, "Performance Assessment of Tilt-up Buildings subjected to Extreme Hazards: Tornadoes and Earthquakes " Journal of Performance of Constructed Facilities, under review.

[18] ASCE, 2010, "Minimum Design Loads for Buildings and Other Structures," American Society of Civil Engineers Reston, Virginia.

[19] ASCE, 2016, "Minimum Design Loads for Buildings and Other Structures," American Society of Civil Engineers, Reston,VA.

[20] Masoomi, H., and van de Lindt, J. W., 2015, "Tornado Fragility and Risk Assessment of an Archetype Masonry School Building.," Engineering Structures, under review.

[21] Haan Jr, F., Balaramudu, V. K., and Sarkar, P., 2010, "Tornado-induced wind loads on a low-rise building," Journal of structural engineering, 136(1), pp. 106-116.