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EFFECT OF INADEQUATE LAP SPLICE LENGTH ON THE SEISMIC FRAGILITY OF BARE RC FRAMES

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Abstract

Many researchers have investigated the seismic performance of low-ductile reinforced concrete (RC) frames during past decades. A literature review shows that the effects of inadequate lap splice length on the seismic vulnerability of RC frames have not been well addressed. However, it has been shown that columns with inadequate lap splice length exhibit significantly smaller rotational capacities. Therefore, this study investigated the effects of inadequate lap splice length on the seismic fragility of bare RC frames. For this purpose, 3, 6, and 9-story RC frames were designed, and their fragility curves were derived, assuming an inadequate lap splice length for their columns' reinforcement. In the derivation of fragility curves, 15 far-field natural earthquake records were used. Incremental dynamic analysis was employed to determine the drift capacities and demands. The obtained results indicated that shorter structures had larger inter-story drift capacities. Besides, frames with inadequate lap splice length exhibited a significantly larger probability of exceeding damage states than frames with adequate lap splice length.

Keywords: Capacity and demand, Far-field natural earthquake, Fragility curves, Inadequate lap splice length, Low-ductile RC frame.

1. Introduction

Peninsular Malaysia is located in a low seismicity region. However, there is a growing trend in the country's seismotectonic studies due to the 2015 earthquake in Ranau, Sabah [1]. It has been mentioned that earthquakes originated from Sumatra, particularly from the Sumatra subduction zone and the Sumatra transform zone, can reactivate the ancient inactive faults within Peninsular Malaysia [2]. Loi et al. [3] also investigated the seismic hazard of Peninsular Malaysia using deterministic and probabilistic approaches and concluded that the central-western cities of Peninsular Malaysia were most susceptible to high peak ground acceleration because of their proximity to active Sumatran sources such as the Sumatran fault zone and Sumatran subduction zone. In another study, Wang et al. [4] mentioned that the Ranau earthquake resulted from the rupture of a northwest-dipping normal fault capable of generating magnitude seven earthquakes.

The Ranau earthquake in 2015 with a moment magnitude of 6.0 that lasted for 30 seconds showed that many buildings designed according to the existing code for gravity and wind loads were not safe even under a moderate earthquake. Since then, many studies were conducted to assess the vulnerability of existing buildings in Malaysia. Vafaei et al. [5] investigated the seismic response of low-ductile moment-resisting frames in Malaysia when subjected to far-field earthquakes and concluded that seismic retrofitting was necessary for ground soft-story buildings. In another study, Aisyah et al. [6] worked on the seismic fragility of tall concrete wall buildings in Malaysia and showed that the probability of exceeding minor and severe damage states in Kuala Lumpur city were respectively, 55% and 11%.

Seismic fragility curves were developed for concrete box girder bridges in Malaysia by Ghazali et al. [7]. It was shown that the bridge with a pier height of 20 m had the largest probability of exceeding light to severe damage. Amiera et al. [8] worked on the seismic fragility of fully infilled low ductile reinforced concrete buildings in Malaysia. They showed that as the height of the building was increased, the probability of exceeding damage states was increased. In another research, Fazilan et al. [9] developed seismic fragility of bare low ductile reinforced concrete buildings in Malaysia and concluded that the probability of collapse of 3- and 9-story buildings located in Sabah is larger than 50%.

In addition to buildings and bridges, seismic fragility curves have also been developed for an industrial building in Malaysia. by Ahmadi et al. [10]. However, most of the previous studies did not consider the effect of inadequate lap splice length on the seismic vulnerability of the existing reinforced concrete (RC) buildings in Malaysia. It should be mentioned that the reconnaissance reports after the Ranau earthquake showed that many of existing RC buildings in Malaysia do not comply with the seismic codes' requirements for the lap splice length in columns [1]. On the other hand, it has been demonstrated that RC columns with inadequate lap splice length have a lower strength, stiffness, and ductility [11-14]. Besides, Koon et al. [15] showed that when the lap splice length was inadequate in columns, the collapse probability of the ground soft-story buildings was significantly increased. Barkhordary et al. [16, 17] also showed that RC frames with inadequate lap splice length had a larger probability of collapse.

A review of the literature shows the significant effect of inadequate lap splice length on the seismic vulnerability of RC buildings. Therefore, this study investigated the effect of inadequate lap splice length on the seismic vulnerability of bare RC frames in Malaysia through the derivation of fragility curves. The selected frames and their finite element simulations have been explained in the next section. Results and discussions have been presented in subsequent sections.

2. Selected frames and their finite element model

In this study, 3-, 6-, and 9-story RC frames representing the typical RC buildings in Malaysia were selected. All frames had four similar spans with a length of 6 m. The height of the ground floor in all frames was 4 m. However, the height of stories above the ground floor was 3 m. The selected frames had different numbers of stories mainly because it has been shown that the total height of a structure affects its seismic vulnerability [18, 19]. Besides, considering the fact that the majority of existing buildings in Malaysia do not comply with the requirements of seismic codes, it was assumed that all frames had a low ductility level.

All frames were designed for the effect of gravity loads using the dead and live loads shown in Table 1. The sizes of beams and columns were determined using the specifications of the BS 8110 [20] code. The beams had a rectangular cross-section with dimensions of 350x250 mm. However, columns had a square cross-section with various dimensions of 300, 400, and 450 mm. All the beam-to-column and column-to-foundation connections were assumed to be fixed.

Table 1. Detailed description of low ductile RC bare frames.

Number of stories	3, 6, 9
Height of buildings	10 m, 19 m and 28 m
Dead load	Finishes = 1.2 kN/m^2 RC slab with 150mm thickness = 3.75 kN/m^2 M&E Services = 0.5 kN/m^2 150 mm thick brick wall with plaster finishes at both sides = 2.9 kN/m^2 per 1 m height.
Live load	Typical floor = 1.5 kN/m^2 Roof = 1.0 kN/m^2
Notional load	Equivalent to 1.5% of the characteristic dead load

The material properties shown in Table 2 were used to determine the required reinforcing bars of beams and columns.

Table 2. Summary of material properties used in the design and FE models.

Property	Concrete	Steel
Weight per unit volume	25 kN/m^3	76.97 kN/m^3
Mass per unit volume	2549 kg/m^3	7849kg/m^3
Modulus of elasticity, E	30000 MPa	199948MPa
Poisson's ratio, U	0.2	0.3
Shear modulus, G	14583 MPa	-
Compressive strength	20 MPa	-
Yield tensile strength	-	400 MPa
Ultimate tensile strength	-	600 MPa

As shown in Fig. 1, the finite element models were developed using ETABS [21] software. This software is able to conduct the nonlinear static (i.e., pushover) and nonlinear time history analysis. The nonlinear behaviour of beams and columns was simulated through the lumped plasticity model [22]. The lumped plasticity model has been suggested for the nonlinear analysis of structures by several codes like ASCE/SEI 41-13 [23] and has been widely employed by many researchers [23-25]. This study extracted the modelling parameters and acceptance criteria for plastic hinges from the tables provided in ASCE/SEI 41-13 [26]. It should be mentioned that columns with inadequate splice lengths have a smaller plastic rotation angle than columns controlled by flexural or shear failure. ASCE/SEI 41-13 [26] has included the effect of inadequate lap splice length of columns in the recommended values for plastic hinges' modelling parameters and acceptance criteria. Therefore, in this study, the effects of inadequate splice length of columns were taken into account using the recommended modelling parameters of ASCE/SEI 41-13. Plastic hinges were assigned to both ends of beams and columns where the maximum bending moment is expected. These locations provide higher accuracy for the obtained results as suggested by other researchers [22].

Fig. 1. Finite element model of the 3-storey RC frame.

Following the recommendation of ASCE/SEI 41-13 [26], the elastic stiffness of beams and columns was decreased by multiplying their moment of inertial to 0.3 and 0.7, respectively. Besides, three different damage states were included in the analysis. The first damage state was immediate occupancy (IO) and represented minor damage to structural elements. Life safety (LS) was the second damage state and showed moderate damage. The third damage state was collapse prevention (CP) and indicated severe damage to structural elements. All frames were subjected to nonlinear static and nonlinear time history analyses.

Following the recommendation of ASCE/SEI 41-13, the nonlinear static analysis was conducted using two different lateral load patents. The first lateral load pattern followed the product of mass in each floor and was referred to as Uniform while the second lateral load pattern followed the shape of the first vibration mode of frames and was referred to as Mode. The lateral forces were gradually increased until the frames reached their ultimate load. The nonlinear time history analysis was performed using 15 natural earthquake records, as explained in the next section.

3. Selected Earthquake Records

Tso et al. [27] studied a set of 45 earthquake records and analysed the significance of the peak ground acceleration to peak ground velocity (PGA/PGV) ratio of records as a parameter to indicate the dynamic characteristics of earthquake ground motions. They found that the PGA/PGVV ratio of ground motions is a suitable indicator for categorising their dynamic characteristics. For instance, they noted that ground motions associated with small or moderate earthquakes have high PGA/PGV ratios and large earthquakes usually show low PGA/PGV ratios.

In addition, they revealed that ground motions with high PGA/PGV ratios have a short duration with seismic energy in their high-frequency range, and ground motions with low PGA/PGV ratios have a long duration with seismic energy in their low-frequency range. Considering that Malaysia is mainly affected by far-field earthquakes, 15 ground motions from the low category of investigated records by Tso et al. [27] were selected and used to develop seismic fragility curves. Table 3 shows the details of selected earthquakes.

Table 3. Details of selected earthquake records.

Name	Year	Location	Duration (sec.)	PGA (cm/s^2)	PGV (cm/s)	PGA/PGV (1/s)
L1	1933	Long Beach	30	95.6	23.7	4.04
L2	1933	Long Beach	30	62.3	17.3	3.59
L3	1934	Lower California	25	156.8	20.9	7.52
L4	1971	San Fernando	20	98.8	19.3	5.12
L5	1971	San Fernando	25	129.8	21.6	6.01
L6	1971	San Fernando	25	126.9	18.6	6.81
L7	1971	San Fernando	20	111.8	18.6	6.02
L8	1971	San Fernando	20	114.9	21.5	5.34
L9	1971	San Fernando	20	116.9	17.3	6.76
L10	1971	San Fernando	20	103.8	16.9	6.12
L11	1968	Undefined	45	221.5	33.4	6.63
L12	1973	Undefined	35	200.9	27.5	7.31
L13	1985	Undefined	35	101.3	15.0	6.74
L14	1985	Undefined	30	32.0	7.4	4.34
L15	1985	Undefined	60	38.8	7.4	5.27

4. Results and Discussion

The obtained results from the nonlinear static and dynamic analyses have been presented in the subsequent sections. At first, the failure mode of RC frames has been investigated through pushover analysis. Then, the RC frames' drift demand and capacities that have been obtained from nonlinear time history analysis are presented. Finally, the developed fragility curves have been plotted and discussed.

4.1. Pushover analysis

As shown in Fig. 2, in the 3-story frame with inadequate lap splice length, the first IO damage state was observed at the base of a column located at the ground level. As the lateral load was increased, more columns at the ground level reached the IO damage state.

As can be seen from Figs. 3 and 4, the first LS and CP damage state also occurred at the base of a column located at the ground level. In other words, the seismic-induced damage was mostly concentrated at the base of the ground floor's columns.

The plastic hinge formation in the same 3-story frame when columns have adequate lap splice length is shown in Figs. 5 and 6. Similar observations were made for the 6- and 9-story RC frames.

As shown in Figs. 7 and 8, the first CP damage state in the 6-story frame with inadequate lap splice length occurs at the base of columns located on the ground floor. However, when the columns of the same frame have adequate lap splice length, the location of CP damage state is relocated to the beams of the first and second floors indicating better seismic performance.

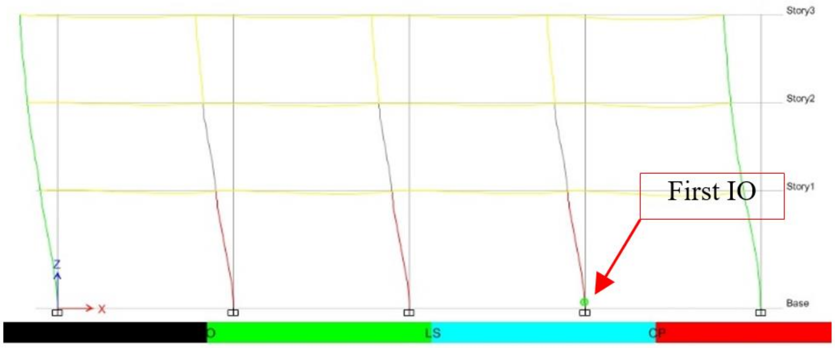

Fig. 2. Formation of first IO damage state in the 3-storey frame with inadequate splicing in columns.

Fig. 3. Formation of first LS damage state in the 3-storey frame with inadequate splicing in columns.

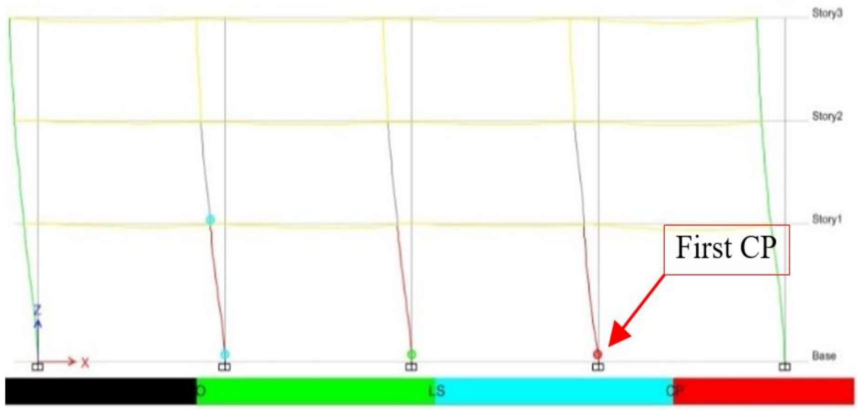

Fig. 4. Formation of first CP damage state in the 3-storey frame with inadequate splicing in columns.

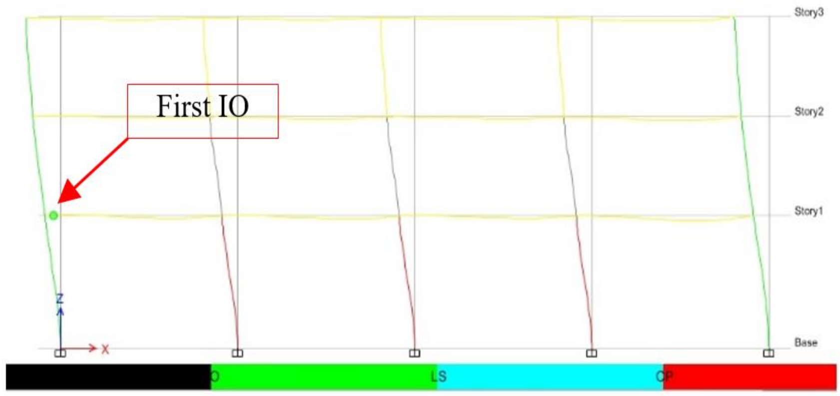

Fig. 5. Formation of first IO damage state in the 3-storey frame with adequate splicing in columns.

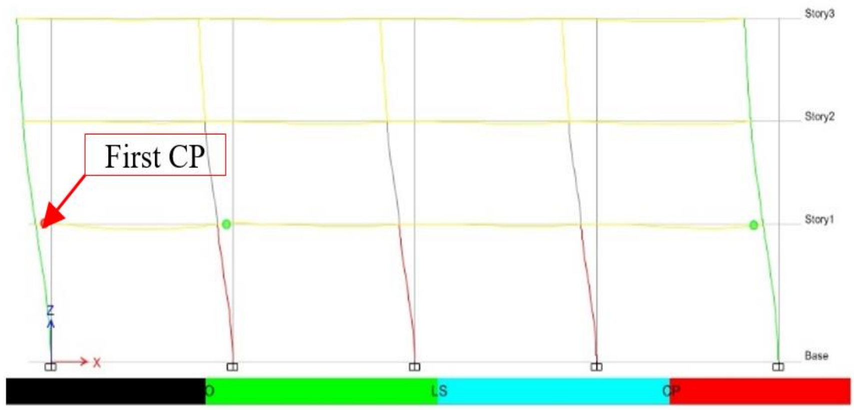

Fig. 6. Formation of first CP damage state in the 3-storey frame with adequate splicing in columns.

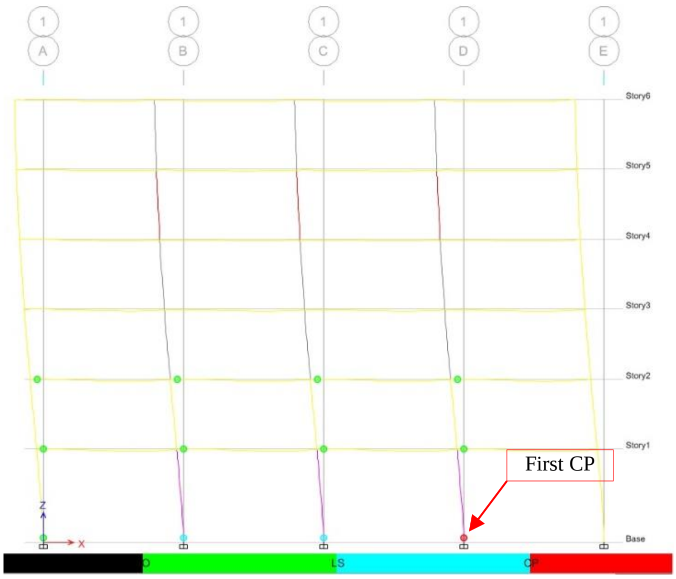

Fig. 7. Formation of first CP damage state in the 6-storey frame with inadequate splicing in columns.

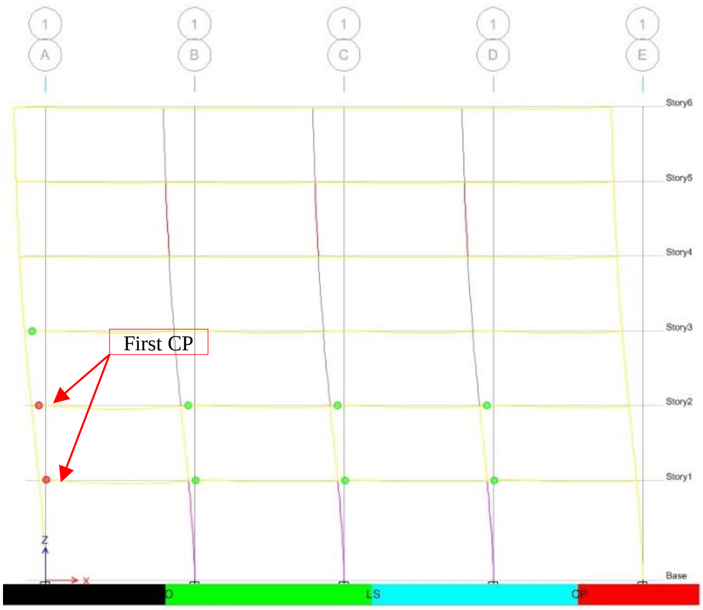

Fig. 8. Formation of first CP damage state in the 6-storey frame with adequate splicing in columns

Similarly, the comparison between Figs. 9 and 10 shows that the first CP damage state in the 9-story frame with inadequate lap splice length occurs at the base of the ground floor's columns, while in the 9-story frame with adequate lap splice length occurs in the beams of the third floor. Thus, it is evident from these figures that when columns have adequate lap splice length, the damage has mainly concentrated on beams rather than at the base of columns on the ground floor. Therefore, the presence of inadequate lap splice length in columns has changed the failure mode of the frames from a ductile mechanism (i.e., beams failure) to a brittle mechanism (i.e., columns failure).

As shown in Table 4, the inter-story drift capacities of frames were calculated based on pushover analysis results for different damage states and types of frames. It is evident from this table that frames with inadequate lap splice length have significantly smaller drift capacities for all damage states. For example, the drift capacities of the 3-story frame with an adequate lap splice length for IO, LS, and CP damage states are, respectively, 61%, 62%, and 61% larger than that of the frame with inadequate lap splice length. The main reason for this observation relies on the failure mode of frames.

As mentioned earlier, frames with inadequate lap splice length exhibited a brittle failure mode (i.e., column failure), while frames with adequate lap splice length had a ductile failure mode (i.e., beam failure). Table 4 also shows that the inter-story drift capacities of different damage states are close to each other in all frames. In other words, the frames have low reserved strength and ductility. The reason is mainly related to the fact that all frames were designed only for gravity, and notional loads and seismic actions were not included in their design.

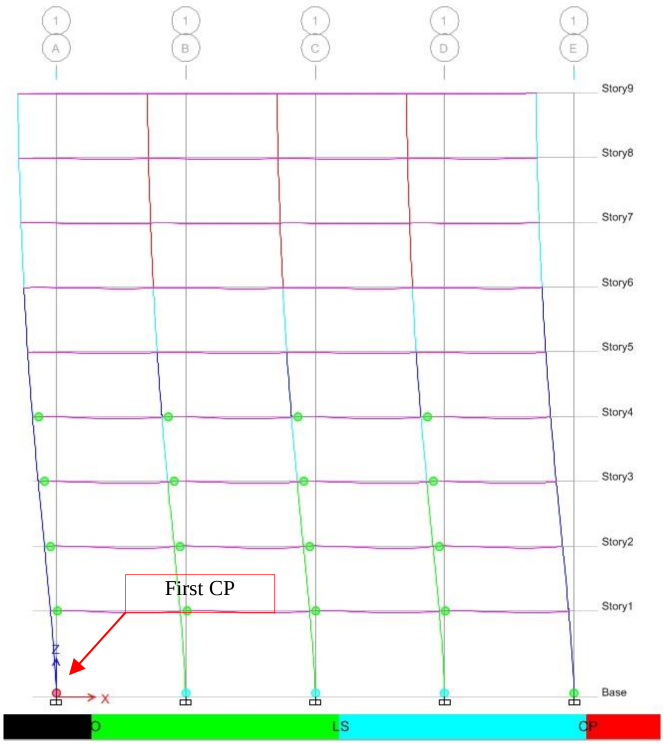

Fig. 9. Formation of first CP damage state in the 9-storey frame with inadequate splicing in columns

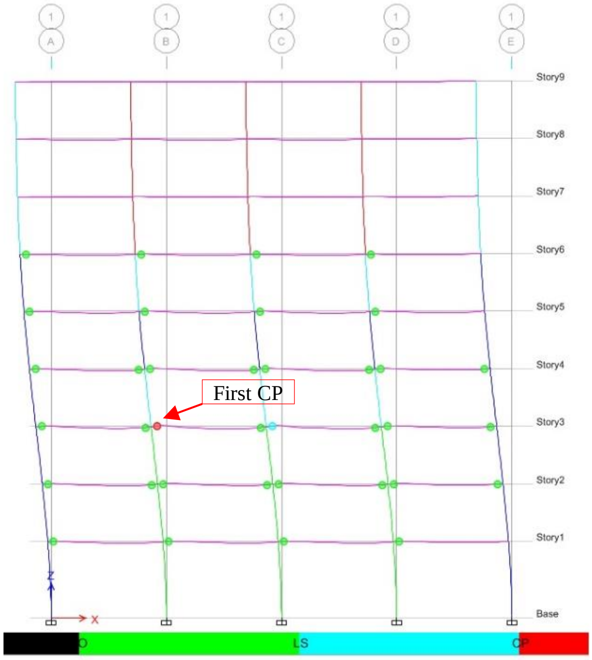

Fig. 10. Formation of first CP damage state in the 9-storey frame with adequate splicing in columns

Table 4. Obtained inter-story drift capacities for different damage states and types of frames.

Columns splice length	Frame	IO	LS	CP
inadequate	3-storey	0.012	0.014	0.016
adequate	3-storey	0.031	0.037	0.041
inadequate	6-storey	0.008	0.012	0.012
adequate	6-storey	0.018	0.024	0.026
inadequate	9-storey	0.008	0.009	0.011
adequate	9-storey	0.014	0.022	0.023

4.2. Incremental dynamic analysis (IDA)

The incremental dynamic analysis was conducted using the 15 earthquake records shown in Table 3. The peak ground acceleration (PGA) of all records was scaled from 0.1g to 0.5g with an increment of 0.1g. Then, the frames were subjected to the scaled earthquake records, and their maximum inter-story demand was calculated for different PGAs. Tables 5 to 10 present the obtained results for different frames. As can be seen from these tables at large PGAs (e.g., 0.3g to 0.5g), the median of inter-story demand of frames with adequate lap splice length is larger than that of frames with inadequate lap splice length. However, at small PGAs, the median of inter-story demands for frames with adequate lap splice length is close to frames with inadequate lap splice length. The main reason for this observation is that at small PGAs, all frames exhibit almost an elastic behaviour. Therefore, the adequacy of splice length has an insignificant effect on the response of frames. On the other hand, as the PGA of records increases, plastic hinges form and frames' inelastic behaviour controls their inter-story drift demand. Since frames with inadequate lap splice length have smaller plastic rotation capacity, they reach their ultimate load at smaller PGAs, and therefore, their drift demand at large PGAs remains below that of frames with adequate lap splice length.

It is also noteworthy that an increase in the PGA of records has not increased the median of inter-story demand in some frames. This observation is more pronounced in frames with inadequate lap splice length because they reach their ultimate capacity at smaller PGAs. Therefore, when they are subjected to records with larger PGAs, they fail before the earthquake record reaches its maximum excitation. Comparing the median of inter-story demand shows that 3-story RC frames have a greater median drift demand than 6- and 9-story frames, especially at large PGAs. This observation is related to the smaller inter-story drift capacity of taller frames. For example, Tables 7 and 9 shows that the inter-story drift capacity of the 6- and 9-story RC frames are around 0.014. However, as can be seen from Table 5, the inter-story drift capacity of the 3-story frame is around 0.021.

Table 5. Maximum inter-storey drift demands for the 3-storey RC frame with inadequate splicing.

Earthquake record	**Peak Ground Acceleration (PGA)**				
	0.1	**0.2**	**0.3**	**0.4**	**0.5**
L1	0.024	0.023	0.019	0.020	0.024
L2	0.016	0.023	0.024	0.025	0.019
L3	0.007	0.013	0.016	0.018	0.021
L4	0.012	0.022	0.015	0.020	0.023
L5	0.011	0.021	0.018	0.023	0.024
L6	0.007	0.013	0.024	0.019	0.016
L7	0.011	0.016	0.020	0.022	0.016
L8	0.009	0.018	0.016	0.018	0.021
L9	0.007	0.013	0.019	0.019	0.019
L10	0.014	0.019	0.017	0.020	0.021
L11	0.005	0.010	0.016	0.021	0.018
L12	0.010	0.021	0.019	0.025	0.020
L13	0.016	0.024	0.023	0.024	0.018
L14	0.012	0.021	0.017	0.015	0.022
L15	0.020	0.021	0.023	0.023	0.022
Median	**0.011**	**0.021**	**0.019**	**0.020**	**0.021**

Table 6. Maximum inter-storey drift demands for the 3-storey RC frame with adequate splicing.

Earthquake record	**Peak Ground Acceleration (PGA)**				
	0.1	**0.2**	**0.3**	**0.4**	**0.5**
L1	0.023	0.046	0.069	0.092	0.115
L2	0.003	0.005	0.008	0.011	0.013
L3	0.006	0.012	0.017	0.023	0.029
L4	0.012	0.026	0.041	0.056	0.122
L5	0.008	0.014	0.023	0.032	0.074
L6	0.007	0.014	0.020	0.030	0.085
L7	0.009	0.016	0.027	0.057	0.187
L8	0.009	0.019	0.028	0.037	0.046
L9	0.007	0.013	0.020	0.027	0.033
L10	0.013	0.027	0.040	0.054	0.067
L11	0.005	0.010	0.015	0.020	0.025
L12	0.007	0.015	0.022	0.029	0.036
L13	0.017	0.034	0.052	0.069	0.086
L14	0.012	0.023	0.035	0.047	0.058
L15	0.024	0.047	0.071	0.094	0.118
Median	**0.009**	**0.016**	**0.027**	**0.037**	**0.067**

Table 7. Maximum inter-storey drift demands for the 6-storey RC frame with inadequate splicing.

Earthquake record	Peak Ground Acceleration (PGA)				
	0.1	**0.2**	**0.3**	**0.4**	**0.5**
L1	0.013	0.017	0.013	0.014	0.010
L2	0.013	0.013	0.011	0.013	0.014
L3	0.005	0.008	0.010	0.013	0.016
L4	0.009	0.016	0.016	0.015	0.018
L5	0.007	0.014	0.013	0.012	0.012
L6	0.006	0.011	0.010	0.012	0.012
L7	0.007	0.014	0.013	0.014	0.012
L8	0.007	0.012	0.011	0.014	0.009
L9	0.006	0.012	0.012	0.011	0.014
L10	0.008	0.016	0.010	0.015	0.016
L11	0.004	0.007	0.011	0.014	0.008
L12	0.006	0.011	0.014	0.015	0.013
L13	0.010	0.015	0.016	0.018	0.010
L14	0.007	0.009	0.013	0.014	0.011
L15	0.015	0.013	0.014	0.015	0.016
Median	**0.007**	**0.013**	**0.013**	**0.014**	**0.012**

Table 8. Maximum inter-storey drift demands for the 6-storey RC frame with adequate splicing.

Earthquake record	Peak Ground Acceleration (PGA)				
	0.1	**0.2**	**0.3**	**0.4**	**0.5**
L1	0.021	0.027	0.028	0.037	0.043
L2	0.013	0.049	0.126	0.010	0.013
L3	0.004	0.008	0.013	0.016	0.018
L4	0.008	0.017	0.023	0.026	0.166
L5	0.005	0.011	0.022	0.025	0.082
L6	0.004	0.008	0.012	0.051	0.019
L7	0.006	0.012	0.025	0.026	0.032
L8	0.007	0.014	0.023	0.011	0.012
L9	0.004	0.008	0.012	0.063	0.152
L10	0.010	0.014	0.025	0.033	0.055
L11	0.003	0.006	0.009	0.010	0.011
L12	0.004	0.008	0.013	0.018	0.026
L13	0.011	0.018	0.036	0.039	0.032
L14	0.008	0.014	0.018	0.022	0.022
L15	0.012	0.013	0.014	0.015	0.016
Median	**0.007**	**0.013**	**0.022**	**0.025**	**0.026**

Table 9. Maximum inter-storey drift demands for the 9-storey RC frame with inadequate splicing.

Earthquake record	Peak Ground Acceleration (PGA)				
	0.1	**0.2**	**0.3**	**0.4**	**0.5**
L1	0.015	0.187	0.008	0.014	0.018
L2	0.014	0.016	0.021	0.018	0.003
L3	0.003	0.006	0.009	0.013	0.015
L4	0.006	0.013	0.019	0.013	0.017
L5	0.007	0.012	0.013	0.016	0.010
L6	0.009	0.016	0.013	0.013	0.013
L7	0.007	0.010	0.014	0.011	0.008
L8	0.008	0.012	0.012	0.011	0.014
L9	0.008	0.016	0.025	0.012	0.009
L10	0.007	0.015	0.009	0.012	0.013
L11	0.003	0.006	0.009	0.012	0.011
L12	0.005	0.012	0.018	0.024	0.018
L13	0.008	0.014	0.016	0.014	0.016
L14	0.008	0.017	0.025	0.014	0.017
L15	0.017	0.017	0.018	0.026	0.034
Median	**0.008**	**0.014**	**0.014**	**0.013**	**0.014**

Table 10. Maximum inter-storey drift demands for the 9-storey RC frame with adequate splicing.

Earthquake record	Peak Ground Acceleration (PGA)				
	0.1	**0.2**	**0.3**	**0.4**	**0.5**
L1	0.007	0.032	0.050	0.030	0.028
L2	0.014	0.021	0.013	0.011	0.014
L3	0.003	0.007	0.010	0.011	0.013
L4	0.006	0.013	0.023	0.043	0.026
L5	0.006	0.011	0.015	0.021	0.041
L6	0.008	0.019	0.031	0.045	0.013
L7	0.007	0.014	0.008	0.033	0.028
L8	0.007	0.010	0.017	0.016	0.021
L9	0.008	0.018	0.032	0.051	0.016
L10	0.006	0.015	0.018	0.037	0.036
L11	0.003	0.005	0.007	0.009	0.011
L12	0.005	0.011	0.018	0.027	0.029
L13	0.009	0.014	0.016	0.020	0.040
L14	0.007	0.014	0.015	0.014	0.017
L15	0.009	0.017	0.034	0.015	0.009
Median	**0.007**	**0.014**	**0.017**	**0.021**	**0.021**

4.3. Fragility assessment

In literature, several methodologies have been proposed for the development of seismic fragility curves. However, as shown below, in this study, the proposed equation by Wen et al. [28] was used mainly because of its simplicity and wide application.

$$P(DS|SI) = 1 - \phi\left(\frac{\lambda_c - \lambda_{D|SI}}{\sqrt{{\beta_{D|SI}}^2 + {\beta_C}^2 + {\beta_M}^2}}\right) \tag{1}$$

$$\beta_{D|SI} = \sqrt{\ln\,(1 + S^2)} \tag{2}$$

$$\beta_C = \sqrt{\ln(1 + Cov^2)} \tag{3}$$

where, Φ is the standard normal distribution; DS is the damage state; SI is the seismic intensity; $\lambda_{D/SI}$ is the natural logarithm of the median drift demand given the ground motion intensity from the best-fitted power law equation; λ_c is the natural logarithm of the median drift capacities for a particular damage state; β_c and β_m are the uncertainties related to capacity and modelling. In equation (2), S^2 is the standard error, and in equation (3), Cov is the coefficient of variation. Following the other researchers' recommendation [29, 30], the value of β_m was assumed 0.3.

The developed seismic fragility curves for 3-, 6-, and 9-story buildings are shown in Figs. 11, 12, and 13, respectively. The provided graphs in each figure compare the fragility curves of frames with adequate lap splice length (i.e., dashed lines) with frames with inadequate lap splice length (i.e., solid lines). The obtained results indicate that frames with inadequate lap splice length have a higher probability of exceeding low (IO), medium (LS), and severe (CP) damage compared with frames with adequate lap splice length. For example, at PGA=0.2g that represented the design level acceleration for a city like Ranau in Sabah, Malaysia, the probability of exceeding IO, LS, and CP damage sates for the 3-story frame with inadequate lap splice length are, respectively, 7.8, 10.1, and 17.3 times larger than frames with adequate lap splice length.

Similarly, the probability of exceeding IO, LS, and CP damage states of the 6-story frame with inadequate lap splice length are, respectively, 3.8, 5.6, and 3.6 times larger than the 6-story frame with adequate lap splice length. At PGA=0.2g, the 9-story frame with inadequate lap splice length shows, respectively, 2.1, 6.9, and 4.7 times larger probability of exceeding IO, LS, and CP damage states compared with the frame with adequate frame lap splice length. These observations are mainly because frames with inadequate lap splice length exhibited a lower drift capacity than frames with adequate lap splice length (see Table 3).

It is also noteworthy that as the PGA increases, the probability of exceeding IO, LS, and CP damages in the 3- and 6-story frames with inadequate lap splice length get closer to that of frames with adequate lap splice length. For example, at PGA=0.5 g, the probability of exceeding CP damage state in the 3-and 6-story frames with inadequate lap splice length are, respectively, 6.3% and 7.6%, larger than their counterparts with adequate lap splice length.

A similar observation can be made for the IO damage state of 9-story frames. However, for LS and CP damage states, the difference in the probability of exceeding damage states is still significant when frames with inadequate lap splice length are compared with frames with adequate lap splice length. The reason is related to the frames' inter-story drift demand and capacities. As can be seen from Table 3, the inter-story drift capacities of frames with adequate lap splice length are significantly larger than frames with inadequate lap splice length.

On the other hand, at small PGAs, the inter-story drift demands of all frames are close to each other (see Tables 5 to 10). Therefore, at small PGAs, the frames with adequate lap splice length will have a lower probability of exceeding damage states. On the other hand, at large PGAs, the inter-story drift demand of frames with adequate lap splice length is significantly greater than frames with inadequate lap splice length. Therefore, when the PGA increases, the probability of exceeding damage states will have a sharper rise in frames with adequate lap splice length than frames with inadequate lap splice length.

As a result, at large PGAs, the frames have a closer probability of exceeding damage states. The obtained fragility curves for frames with adequate lap splice length show that, within the expected PGAs for Malaysia (i.e., 0.1g to 0.2g), as the height of frames increases, the probability of exceeding IO, LS, and CP damage states increases. For example, at PGA=0.2 g, the probability of exceeding the IO damage state for the 3-, 6-, and 9-story frames are, respectively, 10%, 20%, and 40%. Similarly, at PGA=0.2g, the probability of exceeding medium damage state for the 3-, 6-, and 9-story frames are, respectively, 5%, 7%, and 9%. It is evident that from these results that the majority of RC frames with adequate lap splice length can survive the expected earthquakes with minor damage to structural elements.

On the other hand, an increase in the height of frames has not increased the probability of exceeding IO, LS, and CP damage states in frames with inadequate lap splice length. For instance, at PGA=0.2g, the probability of exceeding LS damage state in 3-,6-, and 9-story frames are, respectively, 64%, 41%, and 67%.

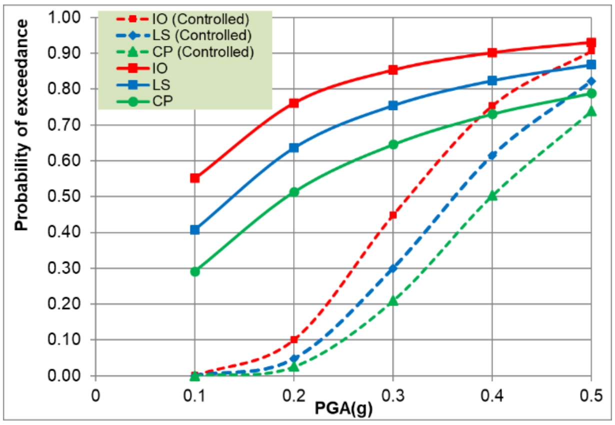

Fig. 11. Fragility curves for a 3-storey RC frame with adequate and inadequate lap splice lengths of columns.

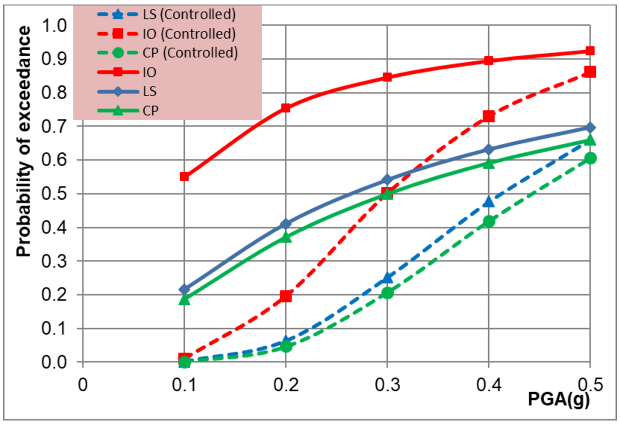

Fig. 12. Fragility curves for a 6-storey RC frame with adequate and inadequate lap splice lengths of columns.

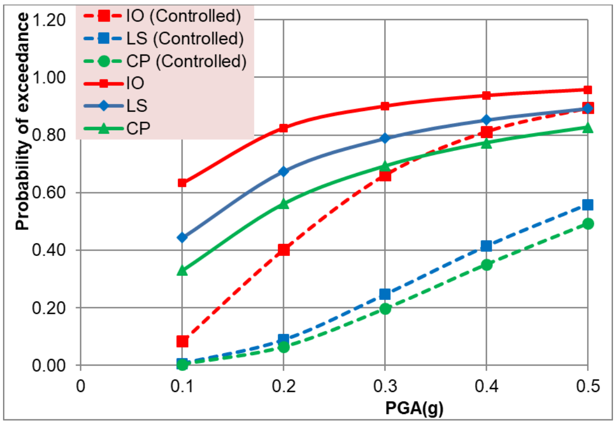

Fig. 13. Fragility curves for a 9-storey RC frame with adequate and inadequate lap splice lengths of columns.

In other words, while there is almost a similar probability of exceeding LS damage states for the 3- and 9-story frames, the 6-story frame has exhibited a lower probability of exceeding LS damage level. A similar observation can be made for the IO and CP damage states of these frames. The reason behind this observation relies on the obtained inter-story drift demand and capacities for these frames. Although the inter-story capacity of the 3-story frame is larger than the 9-story frame, the inter-story demand of the 9-story frame is smaller than the 3-story frame. Therefore, they have exhibited almost a similar probability of exceeding at small PGAs.

On the other hand, the inter-story drift capacity of the 6-story frame is slightly larger than that of the 9-story building, and its inter-story demand is smaller than the 9-story frame. Therefore, the 6-story frame has exhibited a smaller probability of exceeding damage states than the 9-story frame. It is worth mentioning that the obtained results from this study indicate that regular RC frames with inadequate lap splice length in Malaysia that are located in areas with a relatively high PGAs (i.e., like Ranau with a PGA=0.2g) have a low to medium chance of collapse but a high chance of minor to moderate damage.

5. Conclusions

This study investigated the effect of inadequate lap splice length on the seismic vulnerability of 3-, 6-, and 9-story regular bare RC frames in Malaysia. The frames were subjected to pushover and incremental dynamic analyses. The failure mode and inter-story drift demand and capacities of all frames were obtained and discussed. The following observations and conclusions were made:

- The inadequate lap splice length in columns altered the failure mode of all frames from a ductile mechanism (i.e., failure of beams) to a brittle mechanism (i.e., failure of columns).
- Frames with inadequate lap splice length exhibited a smaller inter-story drift capacity than frames with adequate lap splice length.
- An increase in the height of frames with inadequate lap splice length decreased their inter-story demand.
- At small PGAs, the probability of exceeding IO, LS, and CP damage states in frames with inadequate lap splice length was significantly larger than frames with adequate lap splice length. However, as the PGA was increased, the effect of inadequate lap splice length on the probability of exceeding the damage states was decreased.
- It was concluded that regular bare RC frames with inadequate lap splice length located in the areas with PGA=0.2g had a great chance of experiencing minor to moderate damage; however, their collapse chance was low to moderate.

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Nomenclatures

E	Modulus of Elasticity, MPa
G	Shear Modulus, MPa
U	Poisson's ratio, unitless

Greek Symbols

ϕ	Standard normal distribution.
λ_c	Natural logarithm of the median drift capacities for a particular damage state
$\lambda_{D/SI}$	Natural logarithm of the median drift demand given the ground motion intensity from the best-fitted power law equation
β_c	Uncertainties related to capacity
β_m	Uncertainties related to modelling

Abbreviations

IO	Immediate Occupancy
LS	Life Safety
CP	Collapse Prevention
PGA	Peak Ground Acceleration, m/s^2
PGV	Peak Ground Velocity, m/s

References

1. Alih, S.C.; and Vafaei, M. (2019). Performance of reinforced concrete buildings and wooden structures during the 2015 Mw 6.0 Sabah earthquake in Malaysia. *Engineering Failure Analysis*, 102, 351-368.
2. Marto, A.; Soon, T.C.; Kassim, F.; and Yunus, N.Z.M. (2013). Seismic impact in Peninsular Malaysia. *Proceedings of the* 5*th International Geotechnical Symposium*, Republic of Korea. 237-242.
3. Loi, D.W.; Raghunandan, M.E.; and Swamy, V. (2018). Revisiting seismic hazard assessment for Peninsular Malaysia using deterministic and probabilistic approaches. *Natural Hazards and Earth System Sciences*, 18(9), 2387-2408.
4. Wang, Y.; Wei, S.; Wang, X.; Lindsey, E.O.; Tongkul, F.; Tapponnier, P.; Bradley, K.; Chan, C.-H.; Hill, E.M.; and Sieh, K. (2017). The 2015 M w 6.0 Mt. Kinabalu earthquake: an infrequent fault rupture within the Crocker fault system of East Malaysia. *Geoscience Letters*, 4, 6, 1-12.
5. Vafaei, M.; Alih, S.C.; and Abdul Rahman, Q. (2016). Drift demands of low-ductile moment resistance frames (MRF) under far field earthquake excitations. *Jurnal Teknologi*, 78(6), 82-92.
6. Aisyah, S.; Vafaei, M.; Alih, S.C.; and Aljwim, K. (2019). Seismic fragility of tall concrete wall structures in malaysia under far-field earthquakes. *The Open Civil Engineering Journal*, 13, 140-146.
7. Ghazali, A.; Alaydrus, H.A.; Alih, S.C.; and Vafaei, M. (2019). Seismic fragility of concrete box girder bridges in Malaysia. *IOP Conference Series: Materials Science and Engineering*, 513, 012019, 1-8.
8. Amiera, N.; Vafaei, M.; and Alih, S.C. (2018). *Seismic fragility of low ductile*

fully infilled reinforced concrete frame in Malaysia. Master Thesis, Faculty of Civil Engineering, Universiti Teknologi Malaysia

9. Fazilan, N.N.; Rosman, N.A.; Anuar, N.A.; and Alih, S.C. (2018). Seismic fragility of low ductile reinforced concrete frame in Malaysia. *International Journal of Civil Engineering and Technology*, 9(4), 1559-1571.
10. Ahmadi, R.; Mulyani, R.; Nazri, F.M.; Pilakoutas, K.; and Hajirasouliha, I. (2014). Seismic vulnerability assessment of an industrial building in peninsular Malaysia. *Proceedings of the* 5*th Brunei International Conference on Engineering and Technology* (*BICET* 2014), Brunei. 307-313.
11. Eshghi, S.; and Zanjanizadeh, V. (2008). Cyclic behavior of slender R/C columns with insufficient lap splice length. *Proceedings of the* 14*th World Conference on Earthquake Engineering*. Beijing, China, 12-17.
12. Chowdhury, S.R.; and Orakcal, K. (2013). Analytical modeling of columns with inadequate lap splices. *ACI Structural Journal*, 110(5), 735-744.
13. Goksu, C.; Yilmaz, H.; Chowdhury, S.R.; Orakcal, K.; and Ilki, A. (2014). The effect of lap splice length on the cyclic lateral load behavior of RC members with low-strength concrete and plain bars. *Advances in Structural Engineering*, 17(5), 639-658.
14. Rave-Arango, J.F.; Blandón, C.A.; Restrepo, J.I.; and Carmona, F. (2018). Seismic performance of precast concrete column-to-column lap-splice connections. *Engineering Structures*, 172, 687-699.
15. Koon, F.S.; Vafaei, M.; and Alih, S.C. (2021). Effect of inadequate lap splice length on the seismic fragility of ground soft-story reinforced concrete frames. *IOP Conference Series: Earth and Environmental Science*, 682, 012009.
16. Barkhordary, M.; and Tariverdilo, S. (2012). Vulnerability of ordinary moment resistant concrete frames. *Earthquake Engineering and Engineering Vibration*, 10(4), 519-533.
17. Barkhordary, M.; Farjadi, A.; and Tariverdilo, S. (2009). Fragility Curves for reinforced concrete frames with lap-spliced columns. *International Journal of Engineering Transactions A: Basics*, 22(3), 213-224.
18. Kalantari, A.; and Roohbakhsh, H. (2020). Expected seismic fragility of code-conforming RC moment resisting frames under twin seismic events. *Journal of Building Engineering*, 28, 101098.
19. Kumar, P.; and Samanta, A. (2020). Seismic fragility assessment of existing reinforced concrete buildings in Patna, India. *Structures*, 27, 54-69.
20. British Standards Institution (BSI) 1997 BS 8110-1 : Structural use of concrete - Part 1: Code of practice for design and construction.
21. CSI, ETABS. “V-16” (2016): Integrated finite element analysis and design of structures basic analysis reference manual. Computers and Structures Inc.
22. Vafaei, M.; Alih, S.C.; and Fallah, A. (2019). The accuracy of the lumped plasticity model for estimating nonlinear behavior of reinforced concrete frames under gradually increasing vertical loads. *Structural Concrete*. *Journal of the Fib*, 21, 65-80.
23. Bruschi, E.; Calvi, P.M.; and Quaglini, V. (2021). Concentrated plasticity modelling of RC frames in time-history analyses. *Engineering Structures*, 243, 112716.

24. Berry, M.P.; Lehman, D.E.; and Lowes, L.N. (2008). Lumped-plasticity models for performance simulation of bridge columns. *ACI Structural Journal*, 105(3), 270-279.
25. Rahaie, A.R.; and Nafari, S.F. (2013). A comparison between lumped and distributed plasticity approaches in the pushover analysis results of a pc frame bridge. *International Journal of Civil Engineering*, 11(4), 217-225.
26. ASCE/SEI 41-17 (2017), *Seismic evaluation and retrofit of existing buildings*. Virginia, USA: Amercian Society Of Civil Engineering.
27. Tso, W.K.; Zhu, T.J.; and Heidebrecht, A.C. (1992). Engineering implication of ground motion A/V ratio. *Soil Dynamics and Earthquake Engineering*, 11(3), 133-144.
28. Wen, Y.K.; Ellingwood, B.R.; and Bracci, J.M. (2004). Vulnerability function framework for consequence-based engineering. *MAE Center Project DS*-4 *Report*, University of Illinois.
29. Mwafy, A. (2012). Analytically derived fragility relationships for the modern high-rise buildings in the UAE. *Structural Design of Tall and Special Buildings*, 21, 824-43.
30. Vafaei, M.; and Alih, S.C. (2018). Seismic vulnerability of air traffic control towers *Natural Hazards*, 90, 803-822.