Earthquake vulnerability assessment of the 6.5 Mw Pidie Jaya earthquake: Analytical-based fragility curves

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Abstract. Nearly all residential houses were damaged due to 6.5 Mw earthquake in Pidie Jaya, 2016. The state of damage can be slight, moderate, and even can be extensive which lead to the collapsing. The confined masonry structure, which commonly found in Aceh, especially for housing construction, were seemingly prone to the extensive earthquake excitation. In this paper, analytical-based fragility curves are employed to the typical house structure. To account various uncertainty, 32 ground motion records are considered in the analysis. Based on the results, the fragility curve could render different interpretation if different definition of damage intensities is used.

1 Introduction

In the aftermath of the 6.5 Mw earthquake in Pidie Jaya Regency, Indonesia, severe damages are apparent in most of the buildings, and not to mention that several buildings are completely collapsed to the ground. Unexpectedly, the damages of structure were occurred in low-rise residential houses. Based on the field investigation, the typical structural system of the housing can be classified as Confined Masonry. Moreover, despite that the magnitude of the earthquake was considerably high, poor construction practices to meet the standard and minimum design of seismic details are the suspected glitches. Since most of the residential houses in Aceh province are quite typical to the damaged confined masonry that found in Pidie Jaya, its liability is certainly compromised, and in due course, the vulnerability of such structural system is pertinent to be investigated.

Confined masonry is a non-engineered structural system, which favourably chosen as typical house construction in Indonesia and mostly in developing countries, by virtue of its material availability, construction practicality, and relatively low-cost compared to the engineered structural system, such as Reinforced Concrete Frame with Masonry Infills (RCFMI).

Although for common people, the confined masonry appears to be quite similar with RCFMI, both of structural systems have different means in engineering perspectives. Unlike the confined masonry structure, a regular building with RCFMI system will be designed and analysed by the authorized engineer to meet the local building code requirements with special structural detailing, so it can perform satisfactorily under earthquake event.

In most practices, the building is analysed as a bare frame, where the structural frame is designed to carry the gravity load and to resist the lateral earthquake forces, while the stiffness contribution from the infill wall is neglected, despite several studies has investigated that the infill wall act as a diagonal strut which significantly increase the lateral strength and deformation. Contrastingly, the brick masonry in confined masonry structure is confined with light amount of concrete and its reinforcement, having a thickness mostly as thick as brick masonry. During the earthquake excitation, the confined masonry act as whole to resist the lateral force.

Based on the experimental study conducted by Sari et al [1], it can be observed that the diagonal crack was seemed on the specimen which has similar typology and structural configuration with typical housing in Pidie Jaya, Aceh, meaning that the confined masonry also has the in-plane diagonal strut action, similar with RCFMI. In addition, there were evidences where several damaged houses in the post-event of 6.5 Mw Pidie Jaya Earthquake also shown similar failure mode as can be seen on Figure 1.

Fig. 1. Damaged House of Confined Masonry System due to 2016's Pidie Jaya Earthquake

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Although the presence of the out-of-plane failure of masonry wall cannot be negligible, this study is limited to observe the in-plane behaviour of confined masonry. The structural analysis will be utilized by using SAP2000. The structural model will be developed, based on the typical housing in Pidie Jaya. In the absence of local building code in assessing the existing building structure, the modelling procedures will be in accordance with ASCE 41-17 [2].

There are several procedures to assess the proneness of structure due to earthquake. In the conventional procedures, the target performance of structure can be evaluated by performing the available variants of pushover analysis techniques [3], [4], and [5]. A more robust and tedious method by incorporating the probabilistic of earthquake records can be performed by using incremental dynamic analysis (IDA) [6]. In the later study, the demanding dynamic analysis that is required in the IDA, can be evaded via SPO2FRAG [7] and taking into account the seismic fragility assessment. Taking the idea of using an equivalent single-degree-of-freedom structural model to generate the damage fractions, in order to develop the fragility function, this study will combining the capacity spectrum method and then presenting it in the form of record-to-record variability via the analytical fragility function with the fitting approach by using Maximum Likelihood Estimation (MLE), which is based on the previous study from J. W. Baker [8].

2 Methodology

a. Structural model and material properties

The structural plan of house is defined by the typology study from the typical house in Pidie Jaya, whereabouts the map that showing the study location can be seen on Figure 2. The compressive strength of concrete is based on the average results of in-situ non-destructive test that is taken on field survey on several houses. Other material properties, such as brick masonry and reinforcement bar can be determined by the typical material characteristics in Aceh. The material properties are summarized in Table 1.

Fig. 2. Map of Pidie Jaya Regency, Aceh. Background map by © OpenStreetMap Contributors 2019. Distributed under a CC BY-SA License.

Table 1. Material properties

Parameters	Nominal
Compressive Strength of Concrete, f_c'	15 MPa
Yield Strength of Reinforcement, f_y	240 MPa
Tensile Strength of Reinforcement, f_u	300 MPa
Compressive Strength of Brick Masonry	5.3 MPa
Shear Strength of Brick Masonry	0.35 MPa
Elastic Modulus of Brick Masonry	2957 MPa

In order to construct the analytical model, simplified structural model is assumed. The confined structure of masonry is assumed as 1-dimensional frame elements of reinforced concrete column and beam with dimensions of 11x11 cm and 11x13 cm, respectively. The longitudinal reinforcement bars for column and beam are 4 ϕ 10, while the transverse reinforcement is ϕ 6-150. To put some note, restrained fixity is assumed at the base of the column. Furthermore, to represent the diagonal strut action from the masonry, equivalent diagonal compression struts are defined and located concentrically to the reinforced concrete columns. In summary, the structural model is shown in Figure 3. The nonlinear behaviour of structural model that is developed in this study is referring to ASCE 41-17 [2] procedures, as can be illustrated in Figure 4.

In the analytical model, the infill strut is modelled as 1-dimensional elements to carry the axial compressive strength. The tension limit is predefined, to neglect its tension stiffness and avoiding it to carry the tension load. In addition, the stiffness matrix of structure is modified, so thus the equivalent strut element will only transmit the axial force rather than bending moment, somewhat represented as a pinned condition.

(a)

(b)

Fig. 3. Structural model (a) Perspective view; (b) Plan view

Fig. 4. Generalized force-deformation relationship for deformation-controlled, (a) Column and beam; (b) masonry wall, Source: ASCE 41-17 [2]

Based on Figure 4, the terminology of column and beam forces are determined in terms of flexural moment strength, while for the masonry wall, the force terminology is in terms of axial compressive strength. The axial tension strength for the masonry wall is neglected, so thus its out-of-plane capacity. The nominal moment strength of column and beam can be computed accordingly, with the section analysis of reinforced concrete. The compression capacity of masonry wall is defined based on the previous study of Zarnic and Gostic [9]. In addition, the deformation capacity for column, beam, and unreinforced masonry wall elements are referring to the ASCE 41-17 [2].

b. Earthquake ground motions

Due to some circumstances to obtain the actual ground motion of the 2016's Pidie Jaya Earthquake, plausible approach is engaged, instead. To investigate the vulnerability of structural model due to the variability of earthquake excitations, 32 strong ground motion records are considered, in which based on PEER NGA-West2 Database [10], and enlisted on Table 2.

Table 2. Summary of selected ground motion from PEER NGA West 2 database

No	Earthquake Name	Year	Magnitude
1	Parkfield	1966	6.19
2	Coyote Lake	1979	5.74
3	Imperial Valley-06	1979	6.53
4	Livermore-01	1980	5.8
5	Livermore-02	1980	5.42
6	Anza (Horse Canyon)	1980	5.19
7	Mammoth Lakes-02	1980	5.69

8	Mammoth Lakes-03	1980	5.91
9	Mammoth Lakes-04	1980	5.7
10	Victoria_Mexico"	1980	6.33
11	"Westmorland"	1981	5.9
12	"Morgan Hill"	1984	6.19
13	"Hollister-04"	1986	5.45
14	"Chalfant Valley-01"	1986	5.77
15	"Chalfant Valley-02"	1986	6.19
16	"Chalfant Valley-03"	1986	5.65
17	"Chalfant Valley-04"	1986	5.44
18	"Superstition Hills-02"	1987	6.54
19	"Landers"	1992	7.28
20	"Double Springs"	1994	5.9
21	"Kobe_Japan"	1995	6.9
22	"Duzce_Turkey"	1999	7.14
23	"Manjil_Iran"	1990	7.37
24	"Hector Mine"	1999	7.13
25	"Chi-Chi_Taiwan-04"	1999	6.2
26	"Landers"	1992	7.28
27	"Tottori_Japan"	2000	6.61
28	Parkfield-02_CA	2004	6
29	Basso Tirreno Italy	1978	6
30	Potenza_Italy	1990	5.8
31	Joshua Tree CA	1992	6.1
32	Darfield New Zealand	2010	7

The selected ground motion is then scaled in the spectral acceleration-period domain, to be matched to the target response spectrum. The target response spectrum is referring to the Indonesia's Earthquake Map and Response Spectra [11], in the location of Pidie Jaya. The parameters of the corresponding design response spectrum are summarized in Table 3.

Table 3. Summary of design response spectrum

Parameters	Nominal
Location	Pidie Jaya, Aceh Regency
Longitude	96.201 degrees
Latitude	5.241 degrees
PGA	0.54 g
Ss	1.275836 g
S1	0.557154 g
SDs	0.756712 g
SD1	0.774701 g
Site Class	SE

As noted in SNI 1726 2019 [12] and also in accordance with ASCE 7-16 [13], the design response spectrum shall be scaled by 1.5, to achieve the Risk-Targeted Maximum Considered Earthquake (MCER) Response Spectrum. The ground motion records were spectrum matched, at least over a period range between 0.2T and 1.5T for a damping ratio of 5%, as suggested in NEHRP/NIST GCR 11-917-15 [14]. Based on the modal analysis, the building fundamental period resulting in T=0.343 sec. Henceforth, rather than by using a single period scaling, the 32 ground motion records are averagely scaled in arithmetic mean, by the scaling method of the minimize Mean Squared Error (MSE) within the period range of 0.05 sec and 0.08 sec (more conservative). The generated scaling of ground motion records in this study is depicted on Figure 5.

In this study, the pushover analysis is conducted by using SAP2000 to obtain the capacity curve. As the usual capacity spectrum method, the capacity curve is translated into Acceleration-Displacement Response

Spectrum (ADRS) format to enable direct interpretation between capacity spectrum curve with response spectrum curve, where the intersection between those two curves is defined as the performance point (PP).

Fig. 5. Ground motion scaled to the target response spectrum within period of 0.05 sec and 0.08 sec (Re-Illustrated from the scaling results of PEER NGA-West2 Website)

Previously, equivalent linearization method, based on FEMA 440 [15] is used. However, the analysis results cannot render good results, specifically in the elastic range of the capacity spectrum. As such, the analysis is reiterated by using conventional capacity spectrum method, based on ATC-40 [16]. Due to practicality consideration, only the Arithmetic Mean pSa, Arithmetic Mean + sigma pSa, and Arithmetic Mean - sigma pSa is used as response spectrum set to be analysed with the corresponding capacity spectrum curve. The PP is estimated by the intersection between the capacity spectrum curve with the construction of inelastic response spectrum is in accordance with ATC-40 [11] method.

c. Formulation of analytical-based fragility curve function

In this study, the acceptance criteria from ASCE 41-17 are assumed as the Damage states (DS), corresponded to the later creation of fragility function's family curve, which can be classified into three categories, e.g., Life Safety (LS) as DS-1, Immediate Occupancy (IO) as DS-2, and Collapsed Prevention (CP) as DS-3. In order to render the probability earthquake intensity levels, each response spectrum set is scaled up and scaled down by using scaled factor with the increments of 0.1. The number of PPs generated is then observed, to determine the Intensity Measures (IM) of spectral acceleration and the level of DS. The construction of fragility curve is derived from the set of IM and DS pairs by adopting the fitting approach by using MLE [3]. The general

cumulative lognormal distribution can be computed as follows:

$$P(DS \geq DS_i | pSa) = \Phi \left(\frac{\ln(pSa / \theta_{DS_i})}{\beta_{DS_i}} \right) \quad (1)$$

Where:

Φ = standard norm. cumulative dist. func.

θ_{DS_i} = expected median value of pSa at a certain damage state; and

β_{DS_i} = natural logarithm standard dev. of pSa at a certain damage state

Meanwhile, to estimate the fragility function parameters by using the maximum likelihood function, it can be obtained as follows:

$$\{\hat{\theta}, \hat{\beta}\} = \arg \arg \max \sum_{i=1}^m \Phi \left(\frac{\ln \left(\frac{pSa}{\theta_{DS_i}} \right)}{\beta_{DS_i}} \right) + (n - m) \ln \left(1 - \Phi \left(\frac{\ln(pSa / \theta_{DS_i})}{\beta_{DS_i}} \right) \right) \quad (2)$$

Where:

m = ground motion at specified damage state

n = ground motion used in the analysis

3 Results and discussion

a. Results of fragility curve

The result of analytical based fragility curve is shown in Figure 6. Based on the following figure, it can be said that the probability to reach the DS-1 is relatively yield on lower pseudo spectral acceleration compared to the other DSs. Contrastingly, the DS-2 and DS-3 is likely can be triggered in relatively close range of pseudo spectral acceleration. It can be means as well, there is a high probability that the structure is having low ductility, since the extensive damage state (DS-3) can be reached sooner, upon reaching the moderate damage state (DS-2).

Fig. 6. Analytical-Based Fragility Curve with Acceptance Criteria of ASCE 41-17 as DS

In comparison to Figure 6, fragility curve with introducing the Inter-story Drift (IDR) as DS indices is represented in Figure 7. It can be seen that from the following curve, defining the IDR as DS can differ the intensity of damage probabilities. In fact, the DS-1 curve in Figure 7 highlight more disperse probability of failure, compared to Figure 6. Contrastingly, perhaps, the distinguish features by adopting the IDR as DS are how the DS-2 and DS-3 curves can intersect each other, which can lead to a bias to determine a firm judgement, associated to the DS probabilities. Apparently, unlike in Figure 6, where the observed fractions of DS-2 and DS-3 are solid, in Figure 7 the observed fractions of DS-2 and DS-3 are assorted within the pSa range of 1.5-1.6 g, with an intermediate probability.

Fig. 7. Analytical-based fragility curve with inter-story drift (IDR) as DS

b. Extended discussion for the structural response and behaviour

To observe the structural response and behaviour, the capacity spectrum curve in ADRS format is shown in Figure 8. In the following figure, the structural model discussed in this study (Frame + Infill Strut) is depicted in comparison with the bare frame model without infill strut. It can be concluded that the structural model incorporating infill strut is having a higher stiffness and lateral strength capacity compared with the bare frame model. Whereas, the bare frame model is only having 10% lateral strength of the Frame + Infill strut model.

However, it should be highlighted as well that the bare frame model shows higher deformation capacity by roughly twice as much, compared the structure with infill strut model. This signifies the model with infill strut resolute brittle behaviour, compared to the bare frame model. The capacity spectrum also indicated strength deterioration type which is not preferable for earthquake-resistant structure system, where the sudden drop of strength occurred rapidly after the failure point.

To sum up, although the masonry infill contributes to the general in-plane stiffness of the structure frame, due to the lacks of reinforcement details, the column drift capacity is considerably minimum, and resulting in a brittle response manner, which is indicated from the insignificant dissipation of hysteretic energy.

In addition, the analytical results show that the failure mechanism of the structure is anticipated by the compressive strut action of infill wall. Nonetheless, the column member begins to yield, followed by the infill wall dropping its load, upon it reached the peak strength capacity, and then tailed by the sudden drop of lateral strength. The observed response from the analytical works is also compared to the field investigation, and signify seemingly similar failure mechanism.

Fig. 8 Analytical-based fragility curve

4 Conclusion

This study is limited to a particular typology of the “type 36” regular house, which is commonly found in Aceh. The writers are motivated to study this typical house due to its proneness to the notably 2016’s Pidie Jaya Earthquake. The structural model is developed under presumptive modelling parameters, that is also be based on plausible evidence to some extent, taking into account the common practices and also based on field observation.

The reliability of structural analysis can be somewhat influenced by the basis of analysis assumption, from the sophistication of modelling approaches to the uncertainty of earthquake characteristics. In order to eliminate the biases, in this study, earthquake vulnerability assessment is conducted by involving 32 ground motion records that is scaled to the specified target response spectrum, to include the randomness and also probabilistic consideration.

However, the writers perceive that there is always a room for improvement to enhance the results of study. Perhaps, the flaws of this study can be asserted in the limitation of selecting pertinent ground motion record. Under some circumstances, the corresponding ground motion of Pidie Jaya Earthquake is not available to be used in this study. Further study by utilizing empirical fragility curve and including analytical-based fragility curve with Peak Ground Acceleration (PGA), for more intuitive and direct interpretation.

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