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DAMAGE STATE OF NON-ENGINEERED RESIDENTIAL BUILDINGS OWING TO EARTHQUAKES: A CASE STUDY IN PACITAN REGENCY, INDONESIA

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Abstract

The Pacitan Regency of Indonesia is located in an area of high seismic activities. Regrettably, field surveys held in Pacitan Regency, Indonesia have revealed the prevalence of low-income housing structures that do not meet the engineering standards and are vulnerable to damage during earthquakes. The seismic evaluation of these buildings is urgently required. This study proposes realistic damage state limit values for the seismic damage evaluation of non-engineered buildings. For this purpose, an incremental dynamic analysis was conducted on a sample of non-engineered buildings in Pacitan to evaluate their probable seismic damage. As a result, curves showing the dynamic pushover responses of buildings to various input earthquakes were plotted. These curves were then summarized as three-linear curves, providing a single value for the average response of the structure. Based on this simplified curve, a more realistic damage-state limit for non-engineered structures are proposed. The damage levels of slight (DS1), moderate (DS2), extensive (DS3), and complete (DS4) were determined using the drift ratio at the linear elastic limit, midway point between the linear elastic and yield limits, yield state, and 80% rupture limit, respectively.

Keywords:

Non-engineered building;
Incremental dynamic analysis;
Damage state;
Earthquake;
Dynamic pushover curve.

1 Introduction

The islands of Indonesia have a unique geographical position as they are situated on the edges of three distinct tectonic plates. This location renders Indonesia prone to frequent and regular seismic activity. In November 2023, the Indonesian meteorological, climatological, and geophysical agency (BMKG) registered 1104 earthquakes of magnitude in the range 1.5–4.9 M along the tectonic boundaries (Fig. 1). Furthermore, during that year, Indonesia experienced multiple earthquakes with magnitudes higher than 5 M, including the earthquakes that occurred consecutively in Pacitan in June, July, and August 2023 with intensities of 6, 5.7, and 5 M, respectively.

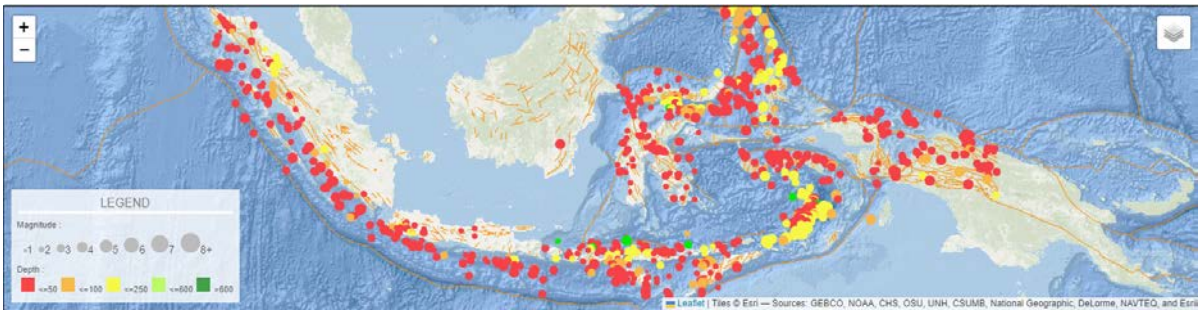

Fig. 1: Seismic activity in Indonesia in November 2023.

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In the Pacitan region, earthquakes have caused significant losses, primarily because of the damage to residential buildings, as several of them were built without proper technical planning, relying solely on local customs and practices. These non-engineered buildings often lack compliance with technical design standards, leading to various issues, such as sub-standard material quality, inadequate reinforcements and details, and poor execution/work processes, which affect the building quality [1]–[3].

The large number of residential houses classified as non-engineered buildings in Pacitan region raises concerns regarding the structural integrity of such buildings in the event of future earthquakes. Therefore, an evaluation of the potential risk of seismic damage to buildings must be carried out as a first step in developing a strategy to enhance building resilience. The potential risk of the collapse of non-engineered residential buildings owing to earthquakes should be assessed using a proper analysis [4] rather than prescriptive or engineering judgment [5]. For this reason, seismic fragility analysis can be employed to quantify the expected level of damage that these buildings may experience due to earthquakes [6], [7]. Once the expected damage is detemined, anticipatory measures that will boost their resilience to such hazards can be taken effectively [8], [9].

Fragility analysis is commonly used to determine seismic damage in engineered structures. Suppose this approach is applied to non-engineered buildings, it presents two major challenges: first, understanding the unique behavior of these structures and, second, accurately determining the various levels of damage states that may occur in non-engineered structures [6], [7], [10].

In contrast to standardized engineered buildings, non-engineered buildings exhibit variations in materials, structural details, and construction quality, making their seismic behavior more difficult to predict. Clustering non-engineered buildings by region can help minimize these discrepancies. The premise is that each location will have distinctive non-engineered characteristics based on local materials, construction practices, typical structures, and reinforcement details commonly found in local houses. Not surprisingly, in much of the literature, the discussion of the seismic performance of non-engineered buildings is always conducted within a localized scope [6], [11]. The article adopts this approach by surveying non-engineered conditions in the Pacitan area, focusing on residential houses as a case study.

Meanwhile, damage state threshold values in engineered buildings are well-established in the literature [12]–[15], allowing for general application in the fragility analysis of engineered buildings. The damage state is typically expressed through four categories, namely, slight, moderate, extensive, and complete damage [15]–[17]. However, the reference points are not readily available for non-engineered buildings, as their unique features prevent a one-size-fits-all. Therefore, to determine the damage states in non-engineered buildings, it must be strictly tailored to the local non-engineered features of the building. This article presents a method for assessing damage states in a non-engineered building in Pacitan Regency in Indonesia, which may serve as a model for other non-engineered building cases.

The principle presented in determining the damage states begins with describing the phyisical damage that will occur to the building due to the earthquake, ranging from slight damage to collapse. Some literature can reference the typical damage found in different types of buildings due to earthquakes. For example, FEMA - Hazus MH 2.1 [18] thoroughly documents the description of damage to various infrastructures, including different types of buildings. One building type, Reinforced Concrete Moment Resisting Frames (C1), indicates progressive seismic damage characterized by hairline cracking, reinforcement yielding, concrete crushing, and collapse. Gaudio et al [19] used the European Macroseismic Scale EMS-98 as the basis for categorizing the damage states for evaluating earthquake damage data on reinforced concrete (RC) buildings in Italy over the past 50 years. The damage description in EMS-98 consists of 5 levels DS0-DS5, starting from no damage, fine cracks, and so on to structural collapse. Meanwhile, Carrillo et al [20] introduced a damage index that relies on the fractal dimension of the observed cracking in 39 wall specimens subjected to seismic testing.These specimens were built with the standard features of thin, lightly reinforced concrete walls for housing. Among the various classifications of damage states, the ones deemed appropriate for adoption in this study must consider the compatibility of the structural system, functionality, and special conditions, especially related to non-engineered features.

Once the categorization of damage states and their descriptions are selected, the next issue is how to identify the damages and correlate them with the observed changes in the seismic response of the structure, such as stiffness degradation, signs of reinforcement yielding, collapse, etc. In this way, damage states may be quantified in terms of engineering parameters such as drift ratio, spectral displacement, etc.[17], [21]. In this research, the response of the structure is determined by Incremental dynamic analysis (IDA) using several earthquake sources, which were previously processed into artificial accelerograms compliant with the spectral response of the Pacitan region. This analysis method

was selected because it can generate information that can be used to quantify the variability of the structural response owing to earthquake uncertainty.

Based on the preceding paragraphs, the issue that needs to be addressed in assessing the likelihood of damage to non-engineered structures is defining damage states realistically. It is insufficient to rely on existing damage states from engineered buildings; instead, they must be tailored to suit the unique characteristics of the non-engineered buildings under investigation. This study suggests establishing realistic damage states that align with the structural response conditions of non-engineered buildings in Pacitan.

2 Methods

This study examined the potential seismic damage to non-engineered buildings, considering the residential housing in the Punung Kendal Subdistrict, Pacitan Regency, as an example. At this stage, the goal is to specify threshold values of damage states that can later be used in determining probabilistic damage to non-engineered buildings in this region through fragility analysis. The following subsection describes the identification of non-engineered features of residential buildings in Pacitan through a field survey, details of the building structures, and methods used for the analysis to obtain the seismic response of the structure. The response, presented as base shear versus displacement, will serve as a reference to identify the point at which progressive damage to the structure occurs.

2.1 Non-engineered features of residential buildings in Pacitan Regency

We conducted an on-site inspection of the residential buildings in the Pacitan area to study the non-engineered buildings, including but not limited to:

a) Use of plain reinforcements

In residential construction, plain reinforcing bars (Fig. 2) are commonly used because of their affordability; however, their inadequate bond strength with concrete makes them unsuitable for seismic events, during which the structure is subjected to cyclic loading. A low bond strength means that the concrete cannot grip the plain reinforcement adequately, leading to slippage prior to reaching its yield strength; thus, the seismic performance of any structure using this reinforcement type must consider its tendency to slip [22], [23].

Fig. 2: Use of plain reinforcements.

b) Non-conforming reinforcement diameter

In the Pacitan region, residential structures are generally built using reinforced concrete, which is a combination of concrete and a reinforcement, which provides compressive and tensile capabilities. However, these buildings often utilize reinforcement diameters smaller than the recommended size (Fig. 3). This is because the actual size of the reinforcement available in the market is less than the recommended size. For example, when a reinforcement with a diameter of 10 mm was ordered, the actual size received was only 9.8 mm. This condition undermines the design principle, which should provide conservative material strength, as stipulated in the earthquake-resistance guidelines [24].

Fig. 3: Actual diameter (less than recommended diameter).

c) Insufficient stirrup spacing

The seismic design code for buildings [24] mandates a maximum stirrup spacing for earthquake resistance. This regulation ensures that the structural elements can withstand large shear forces during an earthquake. In principle, stirrups will work effectively to bear the shear forces in structural elements when the concrete begins to crack. The stirrups also play a role in preventing the propagation of these shear cracks. Shear cracks generally form a 45° to the axis of the structural element. When the stirrup spacing exceeds the element's height, the stirrups become ineffective due to the possibility of not being crossed by shear cracks. In addition, sufficient stirrups have a confining effect on the concrete core, protecting it from crushing [25]. However, in the construction of residential houses in the Pacitan region, the stirrup spacing was found to be in the range of 150–200 mm in columns with a cross-section of 110 x 110 mm (Fig. 4).

Fig. 4: Insufficient stirrup spacing.

d) Non-compliance joint

A well-designed joint is paramount for preventing the failure of other components of the structure. The joint must be stronger than the connected elements to transmit the internal forces. To ensure the strength of a joint, the reinforcement details must be carefully considered because they are often complicated. Regrettably, these details are frequently overlooked, resulting in insufficient stirrups or reinforcement anchorage details to satisfy the mandatory anchorage length (Fig. 5). This increases the likelihood of the total collapse of the building, which is extremely dangerous for the safety of occupants [26], [27].

Fig. 5: Non-compliance reinforcements detail at the joint.

e) Non-measurable concrete mixing practices

In the Pacitan region, workers often mix concrete without following the correct proportions and instead rely on their traditions and convenience (Fig. 6). This practice yields inconsistent concrete mixes with compressive strength of only 3–4 MPa, which is low, and is prone to brittle collapse.

Fig. 6: Mixing of concrete without considering proper proportions.

f) Poor concrete casting process

After mixing, the concrete is cast. Poor casting has many indications, such as voids in the concrete and concrete that only partially cover the reinforcement (Fig. 7).

Fig. 7: Reinforcement not fully covered with concrete.

2.2 Non-engineered building structure model

The building under investigation is a single-story reinforced concrete portal with an earthen tile roof. The technical details are listed in Table 1, and a side view and floor plan of the structure are shown in Fig.8.

Table 1: Technical data of non-engineered building.

Description	Technical data
Number of stories	1 story
Structural system	Reinforced concrete frame
Length x width of building	11.3 x 6 m
Height of the structure	3.7 m
Dimension and reinforcement of columns	110 x 100 mm with plain 8 mm diameter longitudinal reinforcements, 4 mm diameter stirrup reinforcement
Dimensions and reinforcement of sloof beams	110 x 200 mm with plain 8 mm diameter longitudinal reinforcements, 4 mm diameter stirrup reinforcement
Dimensions and reinforcement of lattice beams	110 x 150 mm with plain 8 mm diameter longitudinal reinforcements, 4 mm diameter stirrup reinforcement
Dimensions and reinforcement of ring beams	110 x 150 mm with plain 8 mm diameter longitudinal reinforcements, 4 mm diameter stirrup reinforcement
Concrete compressive strength	4.53 MPa
Reinforcement yield strength	412 MPa

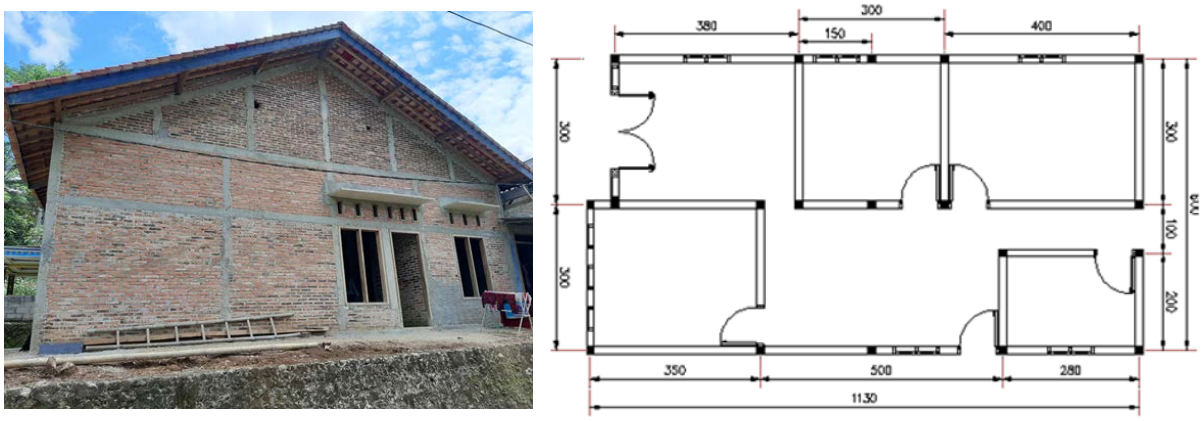

Fig. 8: Side view and floor plan of the structure.

2.2 Incremental dynamic analysis

2.2.1 Structure dead load

The self-weight of the structure was determined based on the measurements and density of the materials used. The density of construction materials can be assessed in terms of SNI 1727-2019. The self-weight of the building's structural elements was automatically generated while modeling using the SeismoStruct software. Nevertheless, parts of the building not included in the SeismoStruct model were calculated manually and viewed as an additional dead load. In this study, the components of the building that were not modeled were those located above the ring beam (Fig. 8). These components include the triangular walls, truss roofs, battens, stake wood, roof coverings, and ceilings. The truss was 2.8 m high and 11.3 m long, with the trusses spaced 6 m apart.

2.2.2 Seismic load

The earthquake was transmitted to the buildings in the form of ground movement. Ground movements caused by several strong earthquakes worldwide have been recorded and presented as accelerograms. Accelerograms display the ground acceleration versus time during earthquakes. Accelerogram data are essential for evaluating the seismic performance of buildings. Seven accelerograms were selected from records of earthquake events in the Loma Prieta, Imperial Valley,

and Northridge areas. Accelerograms were obtained from the Pacific Earthquake Engineering Research Centre (PEERC) database. The Loma Prieta, Northridge, and Imperial Valley earthquakes occurred in different regions with varying soil conditions, resulting in diverse ground motion intensities. Using data from multiple earthquake events, we can gain insights into how structures respond to different seismic intensities and types of ground motion. To incorporate the accelerograms from Loma Prieta, Northridge, and Imperial Valley into the seismic analysis of this study, they must be adjusted to the design response spectrum values in the Pacitan, taking into account the site class of the soil in the region. The earthquake intensities from these sources after matching to the response spectra in the Pacitan region (Region 3) are shown in Fig. 9. Earthquake intensities were matched using the Seismomatch software. The matched accelerogram (artificial accelerogram) was then used as the input for the IDA. The accelerogram input in the IDA analysis starts from low intensity, i.e., by taking a small scale factor that will give a structural response still within linear limits. This scale factor is incrementally increased until the structure collapses.

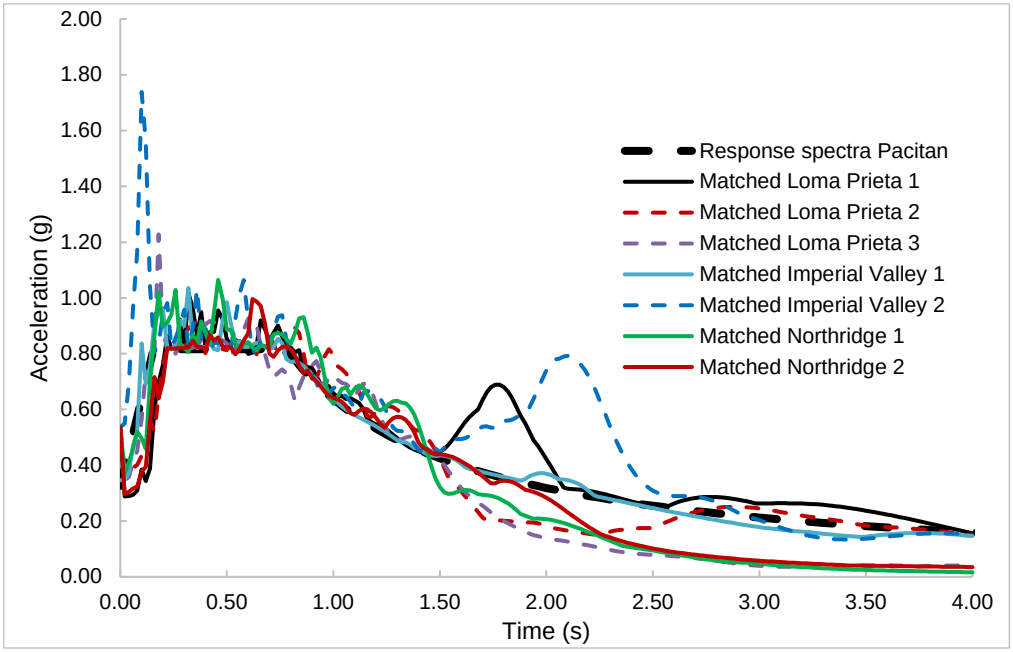

Fig. 9: Matched response spectra to Pacitan.

2.2.3 Modeling and dynamic analysis of structure

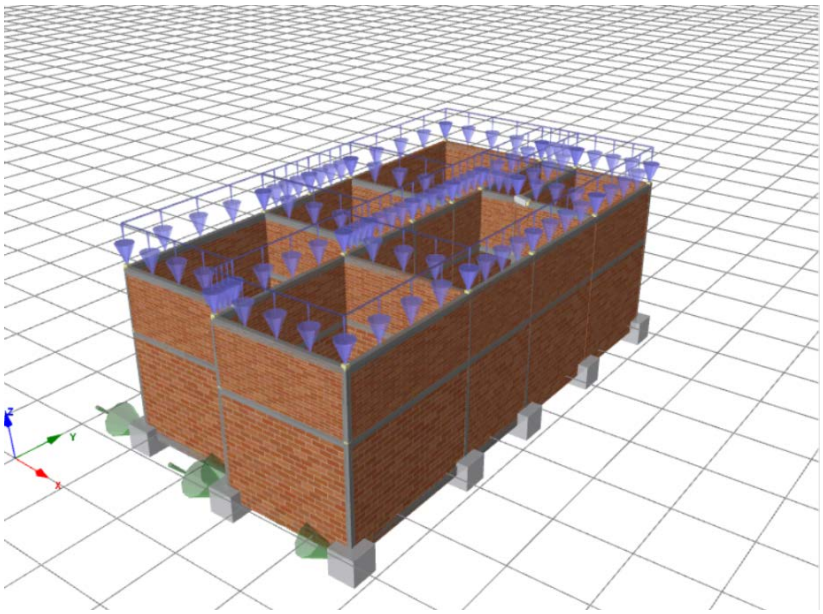

Fig. 10: Structure modeling in SeismoStruct software.

Seismic analysis of the non-engineered structure was conducted using the SeismoStruct software. Fig. 10 shows a three-dimensional (3D) structural frame of the investigated structure where the triangular walls and openings due to the door/window were eliminated. Triangular walls only exist on the two outer sides (i.e., the left and right sides of the building plan) and are not part of the moment-resisting frame structural system. Meanwhile, the walls fill the frame structure, i.e., ignoring the openings due to doors and windows. This approach is a simplification, but as the size and number of openings are insignificant compared to the total wall area, this simplification is acceptable. In addition to its self-weight, the structure had an extra dead load in the form of a fixed gravity load from the building components above the ring balks, described in the structural dead-load subsection. The incremental dynamic method was applied to complete the dynamic analysis of the structure, with the ground movements imitated based on the artificial accelerogram described in the previous subsection. This type of analysis is a time-history analysis that incrementally applies earthquake intensity (artificial accelerogram) to the structure, starting from a low intensity that enables the structure to act elastically to a high intensity that causes the structure to collapse. In this study, the multiplier for the artificial accelerogram was set as 0.02. The SeismoStruct software could then automatically perform incremental dynamic analysis and produce a dynamic pushover curve that shows the maximum response (top drift) versus the base shear of every dynamic loading step.

3 Results and Discussion

3.1 Dynamic pushover curve

The dynamic pushover curve shown in Fig. 11 illustrates the relationship between the magnitude of the base shear force and peak displacement of the structure (top drift). The structure was generally observed to be linearly elastic up to 23.7 kN. The linear elastic behavior at this stage indicates that no significant damage occurs in the structure.Thereby, the structural frame can maintain its stiffness. Above this linear elastic limit, the structure began to exhibit damage. According to Nakano et al [28] above the linear elastic limit, the damage may be initiated by visible narrow cracks on concrete surface. These cracks continue to propagate and to cause stiffness degradation of the overall structural system. For non-ductile structure, remarkable wide cracks and possible concrete crushing may be observed at this stage. When the top drift reached the range of 0.0138–0.0239 m, the structure showed signs of reinforcement yielding; therefore, the base shear force above this limit caused the structure to experience a significant increase in displacement until the structure collapsed. For a ductile structure, the peak value of base shear correlated to the initiation of compression failure of the concrete elements. After this, the structure continues to exhibit large displacement at the lowering base shear [28]. However, this is not true for the non-engineered buildings. This study indicates that the base shear's peak value corresponds to the structure's collapse. This may be due to the fact that the non-engineered building has a low concrete quality; thereby, crushing of the concrete can occur at an early stage. The collapse limit corresponds to the top drift of 0.0185–0.0644, depending on the input earthquake. The range of the top drift at collapse is significantly higher than at the linear elastic or yield limits. This has implications for determining the damage state, which will be explored in a subsequent section.

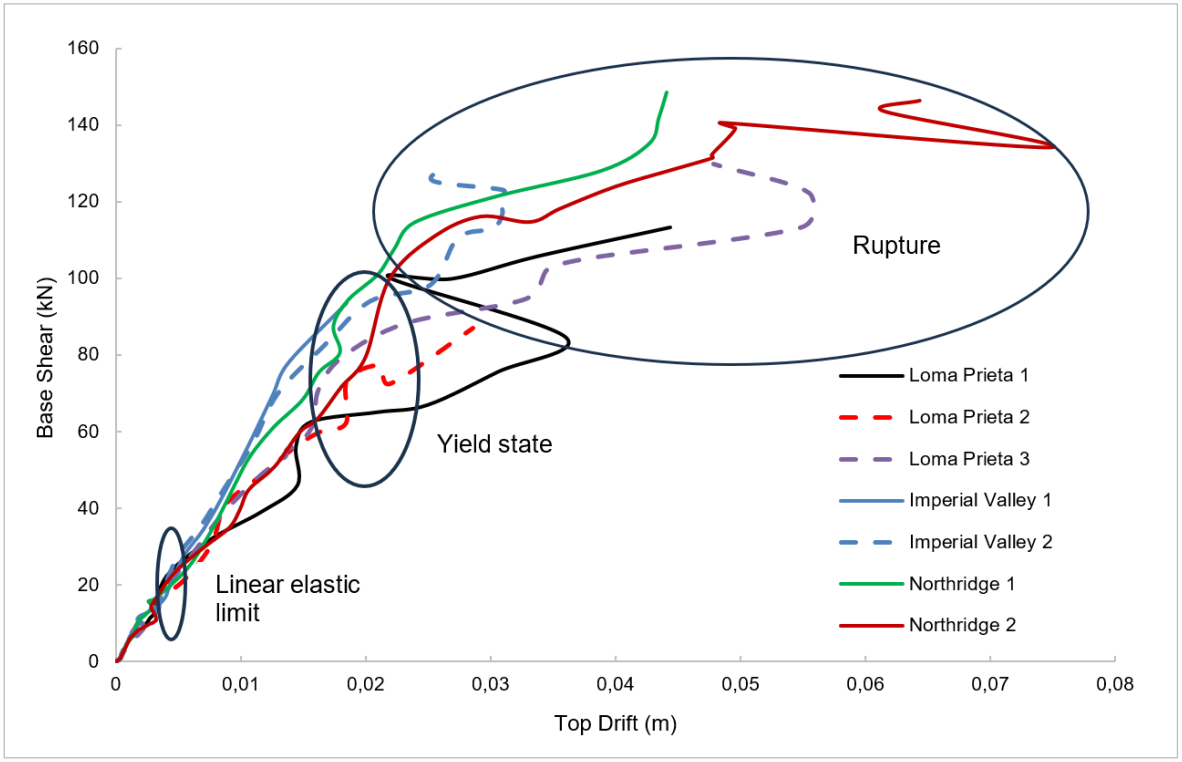

Fig. 11: Dynamic pushover curve of structure obtained from IDA.

3.2 Determining Damage State of Non-Engineered Structures

The first step in the fragility analysis of non-engineered structures is to determine the damage state of the structure. The dynamic pushover curve discussed in the previous section indicates progressive damage to the structure from when the structure behaves linearly elastically to when it yields and ruptures. Hence, the global behavior of the structure may be used to define the damage. The quantification of damage based on the global behavior of the structure has been proposed by researchers. Park and Ang [12] developed a model that links expected damage in reinforced concrete structures to maximum deformation and hysteretic energy. This model takes into account various types of deformations, including flexural, elastic, inelastic shear, and slip (bond failure) deformations. The Park-Ang damage model has been widely used by researchers to assess damage in reinforced concrete structures. Kappos [29] focused on the role of energy in the Park-Ang model, using rotation to represent deformation and the product of moment-rotation to represent energy. He found that the energy component has a minimal impact. Fu and Liu [30] investigated the effect of displacement amplitude and load sequence on adjusting the Park-Ang model. Other researchers used the drift ratio value [28], [31] or spectral displacement [15], [32] as an indicator to express damage states.

The research outlines the results of the incremental dynamic analysis in the form of curves that illustrate the relationship between base shear and top drift. As such, the top drift ratio emerges as a suitable indicator for expressing various levels of structural damage. If the top drift is used as a criterion, it can be seen from the dynamic pushover curve that the linear elastic and yield limits of the seven curves are within a relatively small range of top drift values. However, the rupture limit has a significantly wider range of values. Therefore, in this study, the dynamic pushover curve obtained from the incremental dynamic analysis of the various earthquake inputs was first converted into a single linear curve (Fig. 12). This three-linear curve was obtained by averaging the linear elastic, yield, and rupture limits. The result of this approach yields a three-linear curve that can be considered representative of the seismic behavior of the non-engineered structure.

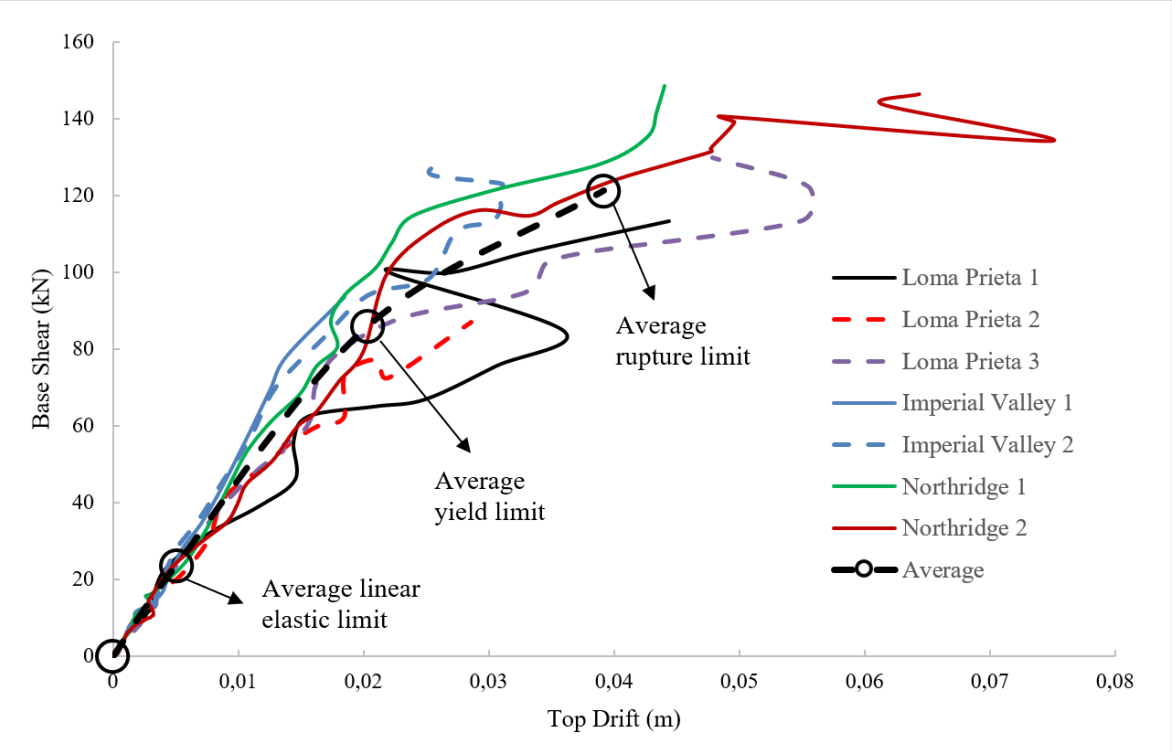

Fig. 12: Three-linear curve as an average value of dynamic pushover curves.

Table 2: Description of damage according to FEMA-Hazus MH 2.1 for C1.

Damage state	Description of damage
DS1 (Slight)	Flexural or shear type hair cracks in some beams and columns near joints or inside joints.
DS2 (Moderate)	Most of the beams and columns show hairline cracking. In ductile frames, some frame elements have reached their yield capacity as indicated by larger flexural cracks and some concrete spalling. Non-ductile frames may exhibit greater shear cracking and spalling.
DS3 (Extensive)	Some frame elements have reached their ultimate capacity, which is demonstrated in ductile frames by the presence of large flexural cracks, chipped concrete, and buckled main reinforcement; non-ductile frame elements may experience shear failure or bond failure at reinforcement joints, or broken bonds or buckled main reinforcement in columns, which may result in partial collapse.
DS4 (Complete)	The structure collapses or is in danger of collapse owing to brittle failure of non-ductile frame elements or loss of frame stability. Approximately 13% (low buildings), 10% (medium buildings), and 5% (high buildings) of the total building area of C1 with this level of damage is expected to collapse.

Table 3: Drift ratio limit at each damage state.

Seismic design level	Drift ratio			
	Slight	Moderate	Extensive	Complete
High-code	0.0050	0.0100	0.0300	0.0800
Moderate-code	0.0050	0.0087	0.0233	0.0600
Low-code	0.0050	0.0080	0.0200	0.0500
Pre-code	0.0040	0.0064	0.0160	0.0400

FEMA-Hazus MH 2.1 [18] describes the damage levels in reinforced concrete frame structures (category C1 structures, i.e., Reinforced Concrete Moment Resisting Frames), as shown in Table 2. The damage levels, as listed in Table 2, are correlated with the drift ratio in the structure, as shown in Table 3 for C1 structures having a low height (6 m), and they are applied to various design levels ranging from pre-code to high code. The pre-code indicates that the design of the structure does not consider seismic design. These structures represent older structures that predate seismic design regulations. The damage state limit of the pre-code structure is considered to be 80% of the damage state value of the low-code design-level structure.

The damage state limits listed in Table 3 can be used as an initial reference to evaluate the damage level of reinforced concrete frame structures. If the damage state limit of the pre-code structure

listed in Table 3 is applied to the non-engineered structure in this study, the top drift in the three-linear curve shown in Fig. 12 must first be converted to drift ratio. With the height of the structure being 3.7 m, Fig. 12 becomes the three-linear curve shown in Fig. 13. In this new three-linear curve, the damage-state limit of the pre-code structure is plotted in Fig. 13. At a drift ratio of 0.004, the pre-coded structure just reaches the slight damage level (DS1), whereas the non-engineered structure almost attains yielding. At the moderate damage limit (DS2), the non-engineered structure crosses the yielding limit. Meanwhile, at higher damage limits (DS3 and DS4), the drift ratio values were outside the three-linear curves of the non-engineered structure. This means that non-engineered structures are very fragile and prone to collapsing compared with the pre-code buildings.

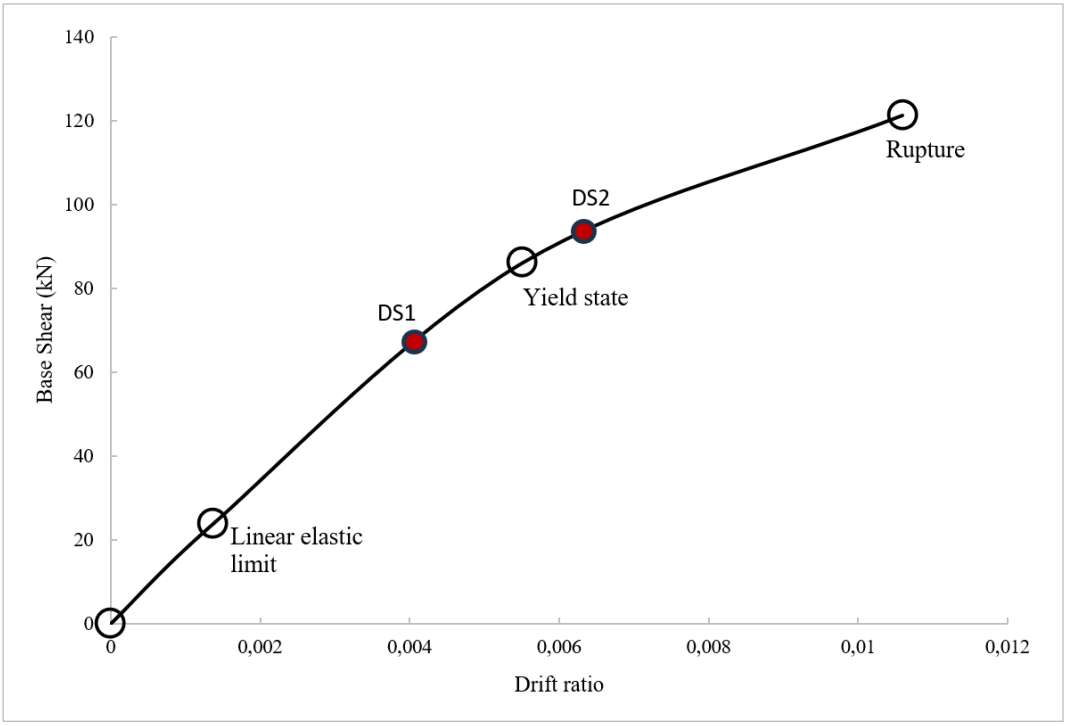

Fig. 13: Damage state limit of pre-code within three-linear curve of non-engineered structure.

A realistic damage state must be proposed for the fragility evaluation of non-engineered structures. The approach for determining this damage state refers to the structural damage description presented in Table 2. Hairline cracks in either the flexural or shear-type cracks in the beams and columns near the joints are characterized by slight damage (DS1). If the cracks continue to develop, they lead to stiffness degradation. On this basis, DS1 in non-engineered structures can be determined to be close to or at the linear elastic limit. This limit is also more realistic than that proposed by Vargas [33] for reinforced-concrete structures, where DS1 is equivalent to 70% of the yield value. Furthermore, Table 2 indicates that DS2 was determined based on the condition that some of the reinforcements had reached their yield capacity. However, this criterion may not be suitable for non-engineered structures because their concrete quality is very low (below 5 MPa); thus, before the reinforcement yields, there is a possibility of severe damage to the concrete, even though the concrete core is not yet crushed. Therefore, a moderate level of damage (DS2) is proposed midway between the linear elastic and yield limit values.

Meanwhile, the occurrence of yielding reinforcement in non-engineered structures may indicate that extensive damage has occurred, with many concrete elements chipped. Therefore, DS3 is proposed because the reinforcement begins to yield. Finally, the complete damage level (DS4) is characterized by the possibility that 13% of the low-rise buildings in the earthquake zone have collapsed. Suppose that this figure is used as a reference. The collapse point on the three-linear curve is the average drift ratio value at rupture (or 50% collapse upon reaching this drift ratio value) for non-engineered structures. In that case, DS4 can be set at the range of the drift ratio value at collapse minus its standard deviation, i.e., 0.000498–0.000175= 0.000323. However, this limit may be too low owing to the considerable standard deviation. Alternatively, DS4 could be assigned to 80% of the rupture point. Thus, the DS1–DS4 limits for the NEB structure can be formulated as:

$$\left.\begin{aligned} DS1 &= dr_e \\ DS2 &= 0{,}5(dr_e + dr_y) \\ DS3 &= dr_y \\ DS4 &= 0.80\ dr_r \end{aligned}\right\} \quad (1)$$

where dr_e, dr_y, and dan dr_r are the drift ratios at the linear elastic limit, yield, and rupture, respectively. The DS1–DS4 values are plotted in Fig. 14 to illustrate the damage progression of the structure in response to an increase in the earthquake intensity.

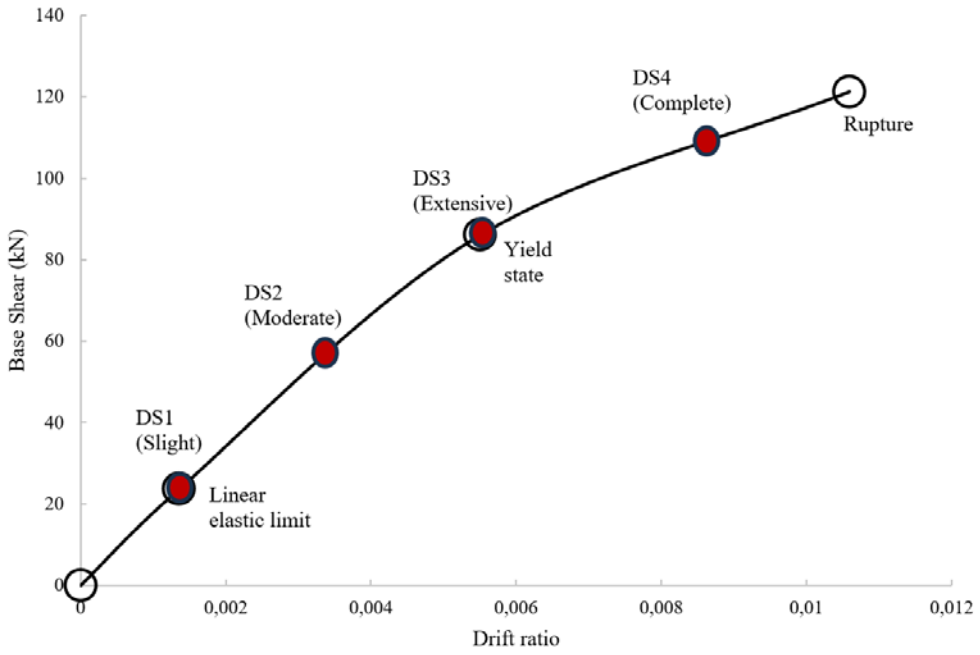

Fig. 14: Damage state limit of non-engineered structures.

4 Conclusion

This study first presents the survey results that show the indications of the non-engineered structure of residential buildings in Pacitan Regency, Indonesia. Furthermore, this study presents a dynamic analysis of the NEB structures and uses the results of the analysis in the form of dynamic pushover curves to determine the limits of realistic seismic damage levels in non-engineered structures. The following conclusions were drawn:

a) Residential buildings in the Pacitan Regency have been constructed with inadequate structural reinforcement. Plain reinforcement with diameters smaller than those recommended has been used, with the stirrups spaced too far apart. The joint reinforcement details have not been properly followed, and the concrete quality is extremely low owing to improper mixing. In addition, the construction implementation has led to porous concrete and exposed reinforcement. These non-engineered features potentially compromise the seismic performance of the residential structures in the region.

b) The incremental dynamic analysis results are displayed as dynamic pushover curves for multiple earthquake inputs. These curves show the behavior of non-engineered structures from when they act linearly elastically to when the reinforcement yields, and finally, when the structure collapses. Therefore, progressive damage to the structure can be identified.

c) From the dynamic pushover curves, the response of the non-engineered structure can be presented as an average value in the form of a three-point linear curve. This simplified curve makes it easier to identify the limits of seismic damage in non-engineered structures.

d) From the three-linear curve, it can be concluded that the non-engineered structure is very fragile because various levels of damage can occur at a significantly lower top drift ratio than the damage state limit of the pre-code building. The findings alert stakeholders to take action to improve the resilience of residential houses in Pacitan.

e) This study proposes a more realistic damage state limit for non-engineered structures based on the damage descriptions contained in FEMA-Hazus MH 2.1, where the damage level criteria of slight (DS1), moderate (DS2), extensive (DS3), and complete (DS4) are based on the drift ratio at the linear elastic limit, midway between the linear elastic and yield limits, yield state, and 80% of the rupture limit, respectively. Damage states formulated from this research can be used to analyze the fragility of residential houses in the Pacitan region.

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References

[1] WATANABE,S. - SHIMA,N. - FUJITA, K.: Research on Non-Engineered Housing Construction Based on a Field Investigation in Jakarta, Journal of Asian Architecture and Building Engineering, 2018, pp. 1347-2852, Doi: 10.3130/jaabe.12.33.

[2] CHARLESON, A. - NONOGUCHI, A. – MAESHIMA, A. – VILLACIS, C. – MINOWA, C. – ZAMANI, G. - IMAI, H. - OKAZAKI, K. - YAMAGUCHI, K. - SHAW, R. - FUKUSHIMA, S. - KITA, S. - SAITO, S. - MATSUZAKI, S. – ANDO, S. – YASUKAWA, S. - NARAFU, T., YOKOI, T. - HANAZATO, T. – ISHIYAMA, Y.: Towards resilient non-engineered construction: Guide for risk-informed policy making. UNESCO, 2016, 166 p.

[3] BOEN, T. - PRIBADI, K.S.: Engineering the Non Engineered Houses for Better Earthquake Resistance in Indonesia, Asian Journal of Environment and Disaster Management, 2011 pp. 65-70.

[4] RAVICHANDRAN, N - LOSANNO, D - PARISI, F. Comparative Assessment of Finite Element Macro - Modelling Approaches For Seismic Analysis of Non - Engineered Masonry Constructions, Bulletin of Earthquake Engineering, 2021, vol. 19, no. 13. pp. 5565–5607, Doi: https://doi.org/10.1007/s10518-021-01180-3.

[5] JEAN, M - UPADHYAY, R. Vulnerability Assessment on Non-Engineered Building in Earthquake Prone Area . Case Study : Klaten District , Central Java , Indonesia. IOP Conf. Series: Materials Science and Engineering, 2019, vol. 650 p. 012040, Doi: 10.1088/1757-899X/650/1/012040.

[6] GIORDANO, N - DE RISI, R- VOYAGAKI, E - KLOUKINAS, P- NOVELLI, V- KAFODYA, I - NGOMA, I- GODA, K- MACDONALD, J. Seismic Fragility Models for Typical Non-Engineered URM Residential Buildings in Malawi, Structures, 2021, vol. 32, no. March, pp. 2266–2278, Doi: 10.1016/j.istruc.2021.03.118.

[7] NOVELLI, V.I - DE RISI, R - NGOMA, I - KAFODYA, I - KLOUKINAS, P. Fragility Curves for Non-Engineered Masonry Buildings in Developing Countries Derived from Real Data Based on Structural Surveys and Laboratory Tests, Soft Computing, 2021, vol. 25, no. 8, pp. 6113–6138, Doi: 10.1007/s00500-021-05603-w.

[8] TRAYKOVA, M - GIARLELIS, C - LAMPROPOULOS, A - MOSELEY, J. Simple Strengthening Techniques for Non-Engineered Buildings Against Seismic Actions, 40th IABSE Symposium,19-21 September 2018, Nantes, France, pp. 19–21.

[9] PURWANTO, E- KRISTIAWAN, S.A – SANGADJI, S - SAIFULLAH, H.A. Strengthening of Non-Engineered Building with Steel Plate to Increase Seismic Performance, IOP Conf. Series: Earth and Environmental Science, 2021, vol. 832 p. 012020, doi: 10.1088/1755-1315/832/1/012020.

[10] PARISSE, F - POLETTI, E - DUTU, A - RODRIGUES, H. Numerical Modeling of the Seismic Performance of Romanian Timber-Framed Masonry Walls, Engineering. Structures, 2021,vol. 239, no. March, p. 112272, Doi: 10.1016/j.engstruct.2021.112272.

[11] GIORDANO, N - NORRIS, A - MANANDHAR, V - SHRESTHA, L - PAUDEL, D.R - QUINN, N - REES, E - SHRESTHA, H - MARASINI, N.P - PRAJAPATI, R - GURAGAIN, R - DE LUCA, F – SEXTOS, A. Financial Assessment Of Incremental Seismic Retrofitting of Nepali Stone-Masonry Buildings, International Journal of. Disaster Risk Reduction, 2021 vol. 60, p. 102297, Doi: 10.1016/j.ijdrr.2021.102297.

[12] PARK, Y - ANG, A.H. Mechanistic Seismic Damage Model for Reinforced Concrete, Journal of Structural Engineering, 1985, vol. Ill, no. 4, pp. 722–739.

[13] AHMED, B - MANGALATHU, S - JEON, J. Seismic Damage State Predictions ff Reinforced Concrete Structures Using Stacked Long Short-Term Memory Neural Networks, Journal of Building Engineering, 2022, vol. 46, p. 103737, Doi: 10.1016/j.jobe.2021.103737.

[14] DURGA, M - SATYABRATA, C. Effect of Damage Limit States on the Seismic Fragility of Reinforced Concrete Frame-Shear Wall Buildings, Disaster Advances, 2021, vol. 14, no. 9, pp. 57–68.
[15] KRISTIAWAN, S.A - HAPSARI, I.R - PURWANTO, E - MARWAHYUDI, M. Evaluation of Damage Limit State for RC Frame Based on FE Modeling, Buildings, 2022, vol. 12, p. 21, Doi: 10.3390/buildings12010021.
[16] SOHN, J - CHOI, I - KIM, J. Development of Limit States for Seismic Fragility Assessment of Piloti-Type Structures Verified With Observed Damage Data, Engineering. Structures, 2022, vol. 251, p. 113562, Doi: 10.1016/j.engstruct.2021.113562.
[17] HAPSARI, I.R - KRISTIAWAN, S.A - SANGADJI, S - GAN, B.S. Damage States Investigation of Infilled Frame Structure Based on Meso Modeling Approach, Buildings, 2023, vol. 13, p. 298., Doi: 10.3390/buildings13020298.
[18] FEMA Hazus®-MH 2.1 Technical Manual: Multi-hazard loss estimation methodology: Earthquake model. Department of Homeland Security, Federal Emergency Management Agency, Mitigation Division, 2012, 718 p.
[19] DEL GAUDIO, C - DI LUDOVICO, M - POLESE, M - MANFREDI, G - PROTA, A - RICCI, P - VERDERAME, G.M. Seismic Fragility for Italian RC Buildings Based on Damage Data of the Last 50 Years. Bulletin of Earthquake Engineering , 2019, Doi: https://doi.org/10.1007/s10518-019-00762-6.
[20] CARRILLO, J - DOMINGUEZ, D - PRADO, N. Seismic Damage Index Based on Fractal Dimension of Cracking on Thin Reinforced Concrete Walls, ACI Structural Journal, 2017, vol. 114, no. 6, pp. 1649–1658, Doi: 10.14359/51700919.
[21] OZMEN, H.B. Relation Between the Rapid Evaluation Method Scores and the Damage States of Buildings, Natural Hazards Earth System Sciences, 2013, vol. 13, no. 3, pp. 763–770, Doi: 10.5194/nhess-13-763-2013.
[22] ZHAO, J. - YIN, L. - LI, X. - YUE, X.: Bond Behavior of Plain Bars in Concrete under Reversed Cyclic Loading, Materials, vol. 16, p. 4836., 2023, Doi: 10.3390/ma16134836.
[23] PADERNO, A. - PELUCCO, S. - METELLI, G. - PRETI, M.: Analytical And Numerical Modelling of Existing RC Frames With Smooth Rebars," Engineering Structure, vol. 249, p. 113160, 2021, Doi: 10.1016/j.engstruct.2021.113160.
[24] SNI 1726:2019: Tata cara perencanaan ketahanan gempa untuk struktur bangunan gedung dan non-gedung. Badan Standarisasi Nasional Indonesia, 2019, 238 p.
[25] WANG, P. - SHI, Q. - WANG, F. - WANG, Q.: Seismic Behaviour of Concrete Columns With High-Strength Stirrups, Earthquake and Structure, 2020, pp. 15–25, 2020, Doi: 10.12989/eas.2020.18.1.015.
[26] RIZWAN, M. - AHMAD, N. - KHAN, A.N.: Seismic Performance of Compliant and Non-Compliant Special Moment-Resisting Reinforced Concrete Frames, ACI Structural Journal, 2018, pp. 1063-1073, Doi: 10.14359/51702063.
[27] AHMAD, N. - SHAHZAD, A. - ALI, Q. - RIZWAN, M. - KHAN, A.N.: Seismic Fragility Functions For Code Compliant and Non-Compliant RC SMRF Structures In Pakistan, Bulletin of Earthqake Engineering, 2018, pp. 4675–4703, Doi: 10.1007/s10518-018-0377-x.
[28] NAKANO, Y - MAEDA, M - KURAMOTO, H - MURAKAMI, M. Guideline for Post-Earthquake Damage Evaluation and Rehabilitation of RC Buildings in Japan," 13th World Conference on Earthquake Engineering, Vancouver, Canada, 1-6 August 2024, p. 124.
[29] KAPPOS, A.J. Seismic Damage Indices for RC Buildings: Evaluation of Concepts and Procedures, Progress in Structural Engineering and Materials, 1997, vol. 1 no.1, pp. 78–87, Doi: 10.1002/pse.2260010113.
[30] FU, G - LIU, B. The Research on Modified Park-Ang Damage Model Based on Effective Energy Dissipation, Advanced. Materials Research., 2013, vol. 742, pp. 46–50, Doi: 10.4028/www.scientific.net/AMR.742.46.
[31] Saruddin, S.N.A. - Nazri, F.M. Fragility Curves for Low- and Mid-Rise Buildings in Malaysia, Procedia Engineering, 2015, vol. 125, pp. 873–878, Doi: 10.1016/j.proeng.2015.11.056.
[32] Kumar, P - Samanta, A. Seismic Fragility Assessment of Existing Reinforced Concrete Buildings in Patna, India, Structures, 2019, vol. 27, pp. 54–69, 2020, Doi: 10.1016/j.istruc.2020.05.036.
[33] VARGAS, Y.F. - PUJADES, L.G. - BARBAT, A.H. - HURTADO, J.E.: Incremental Dynamic Analysis and Pushover Analysis of Buildings: A Probabilistic Comparison, Computational Methods in Stochastic Dynamics, Computational Methods in Applied Sciences 26, 2013, pp. 293–308, Doi: 10.1007/978-94-007-5134-7_17.