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Seismic Vulnerability Assessment of Buildings at an Urban Scale

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Abstract

Recent earthquakes in Australia have underscored the vulnerability of the built environment, emphasizing the imperative of enhancing resilience. Understanding the vulnerability of critical structures is paramount for adequate strengthening and mitigation efforts. Vulnerability assessment falls into two categories: data-driven and analytical methods. While data-driven methods rely on limited statistical data, analytical approaches predict damage through structural analysis, which is often impractical for large building stocks. This study introduces an innovative approach which involves fast tracking of incremental dynamic analyses onto a bid database of building models. A comprehensive urban-scale seismic vulnerability assessment of Australian RC buildings is illustrated by a virtual city model. Addressing uncertainties related to earthquake excitations, building properties, seismic response, and damage interpretation, this approach offers a proactive means to enhance resilience in the face of future seismic events. The paper outlines resolutions to challenges and emphasizes the potential for widespread application across building types once integrated into the virtual city model.

Keywords: buildings; earthquake; seismic vulnerability; resilience; virtual city; damage level.

1 Introduction

Recent earthquakes in Australia have exposed the vulnerability of our built environment, underscoring the challenges in enhancing its resilience. Earthquakes have the potential to inflict widespread damage on Australian communities. Notably, the recognition of earthquake hazards in Australian building design only commenced around 1995. Given that new construction represents a small fraction of the national building stock, exemplified by the fact that approximately 95 percent of Melbourne's houses were built before 2005 (Harrison and Foliente, 2018), it is crucial to acknowledge the substantial number of buildings at high risk of earthquake damage. This susceptibility was starkly demonstrated during the 1989 Newcastle Earthquake, which reportedly damaged over 70,000 properties and incurred an estimated total economic loss of AU$4 billion (Lumantarna et al., 2014). The 2010 Kalgoorlie Earthquake and the 2011 Christchurch Earthquake further reinforce the evidence of how such buildings in communities pose a significant risk to loss of life, property, and economic functions during an earthquake. Understanding the vulnerability of critical structures is of paramount importance, as without this knowledge, these structures cannot be adequately strengthened to mitigate future earthquake damage.

Vulnerability assessment falls broadly into two categories: data-driven and analytical methods. The data-driven method relies on statistical data gleaned from damage observations during previous earthquakes. A key limitation of this method lies in the scarcity of statistical data and the practical application of such data for structural strengthening. To circumvent these limitations, analytical methods predict damage data for individual buildings based on structural analysis subjected to a specific seismic input. However, due to the large number of buildings in a city, conducting seismic simulations for each building individually is not feasible, given the substantial computational effort and cost involved. To address this challenge, most previous research on vulnerability assessment of building stocks in large-scale urban settings has primarily employed building typological approaches. These approaches categorize buildings into several groups, selecting representative buildings from each category for further seismic analysis, assuming that all buildings within a category share the same geometrical and mechanical properties. For example, the seismic vulnerability study of limited ductile RC buildings in Melbourne, Australia, conducted by Hoult et al. (2019), categorized buildings supported by limited ductile RC walls into four configurations and conducted seismic assessments. A similar approach was employed for unreinforced masonry buildings in Australia by Ryu et al. (2020). Although these approaches provide a rapid and simplified estimate, their results may not always align with the actual response of individual buildings, as they depend significantly on various parameters, including geometry, structural characteristics, materials, and more. While comprehensive seismic analysis at the individual building level has started gaining popularity (Marasco et al., 2021 and Lu and Guan, 2021), this approach is new to Australia. Building on existing developments, this paper proposes the comprehensive urban-scale (virtual city) seismic vulnerability assessment of RC buildings in Australia. Though the current scope of this study is the RC building with walls as the lateral load resistance, a similar procedure can be adopted for other buildings and critical infrastructure once they are integrated into the virtual city model.

Achieving such a comprehensive assessment necessitates addressing four categories of uncertainties related to earthquake excitations and intensity, building property uncertainties, seismic response prediction, and the interpretation of responses into seismic damage. Challenges concerning the generation of accurate and sufficient soil surface seismic excitation for Australia have been resolved by the authors through the past research (Hu et al., 2022; Hu et al., 2023; and Khatiwada et al., 2021). The application of these research outcomes to this study is elaborated upon in Section 2. This paper primarily focuses on resolving uncertainties in the building structural database or portfolio, which is discussed in Section 3, with a case study featured in Section 4. In addressing the third uncertainty related to having a reliable and efficient structural analysis technique, the authors have dedicated recent years to developing a macroscopic or macro model for the accurate yet rapid prediction of the nonlinear dynamic time history response of RC buildings (Khatiwada et al., 2023). A brief overview of the macro-model is provided in Section 5, with readers referred to the original papers for detailed information. Finally, regarding the fourth uncertainty concerning damage levels and their interpretation, the recommendations put forth by Ghaborah (2004) and Menegon et al. (2019), as discussed in Section 6 will be adopted. The implementation of the overall procedure is explained through the flowchart in Figure 1.

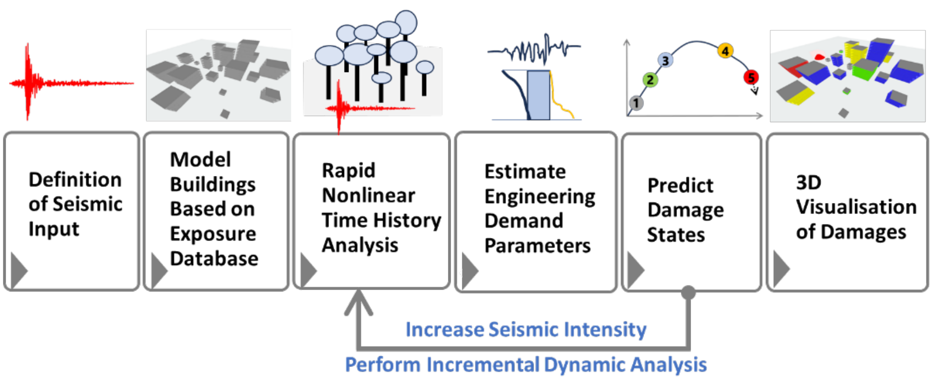


Figure 1. Flowchart of the proposed procedure.

2 Earthquake Intensities and Ground Motions

A wide range of site-specific soil surface accelerograms, corresponding to various earthquake intensities, is required to account for record-to-record dispersion when investigating seismic vulnerability. For this purpose, the Conditional Mean Spectrum (CMS) approach, which matches the code spectrum at four reference periods (T* of 0.2, 0.5, 1, and 2 seconds), will be used. This approach identifies controlling magnitude-distance combinations through hazard disaggregation analyses and sources accelerograms from the NGA-West2 strong motion database (Ancheta et al., 2014). These sourced accelerograms are then scaled so that their calculated response spectra closely match the CMS within the period range of 0.2T* to 2T*. For each CMS, the 'best six' accelerograms, determined by minimizing the sum of squares error (SSE), are chosen. These scaled bedrock accelerograms are then transformed into soil surface accelerograms using the dynamic soil column analysis method detailed in Hu et al. (2023). One soil column from twenty different soil sites, each with borelog information provided by Hu et al. (2023) and collected from various sites across Australia, is to be selected as input in the proposed assessment tool. To streamline this process, the authors have developed an online tool, as mentioned in Khatiwada et al. (2021), accessible at 'quakeadvice.org'. For more details, readers are encouraged to consult the aforementioned references.

3 Building Exposure Database

Urban seismic simulations require an exposure database, also known as a building structural database. This task can be daunting, particularly because structural information for existing buildings, especially older ones, is often unavailable. In such cases, generic building parameters are typically used to estimate a structure's capacity. Many studies rely on parameters developed by Hazus (FEMA, 2010). However, building codes and construction practices can vary significantly from one country to another, and different countries experience varying levels of seismic activity. Thus, it is not reasonable for a country in a low to medium seismicity region, such as Australia, to adopt building parameters similar to those used in the United States, which is in a higher seismicity region. Therefore, this study employs a four-stage analysis to determine exposure parameters and mitigate related uncertainties. Once these uncertainties are resolved, the final structural input parameters are used into the tim ehistory analysis, as explained in Section 4. The four-stage analytical procedure is explained below.

In **Stage 1**, the basic geomatic and geometric building information are gathered by analyzing existing databases from various public and accessible sources. For example, the Census of Land Use and Employment (CLUE) dataset (Melbourne City Council, 2015) provides comprehensive information, including general details like building classification (residential, commercial, etc.), construction year, number of floors, building materials, Global Positioning System (GPS) location, and gross floor area. Figure 2 illustrates the CLUE database of RC buildings in the Melbourne CBD region, as used in the study by Hoult et al. (2019). Similarly, geometric information such as the shape, the number of bays, their spans, and the constituent structural elements, including gravity and lateral load-resisting systems of the building, are collected from either architectural floor plans or obtained through rapid visual surveys of the buildings. The floor plan may be obtained in local councils, public archives, and libraries. Likewise, the sizes of structural elements can often be gleaned from the floor plan. However, in cases where the floor plan is unavailable, a reasonable range of element sizes is used, and any uncertainties involved are resolved using the probabilistic approach discussed in Stage 4.

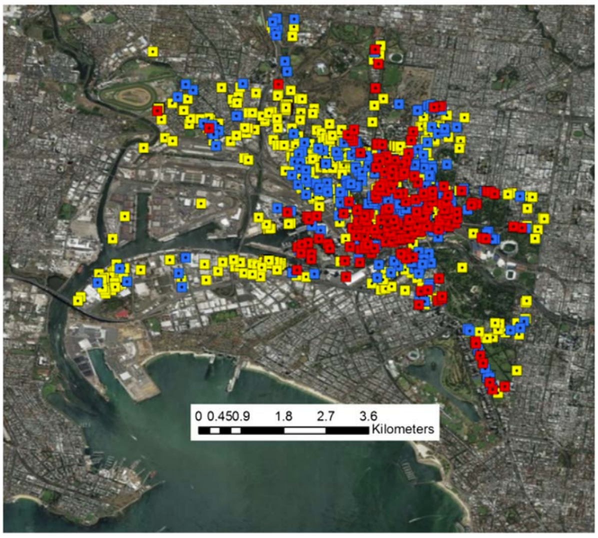


Figure 2. The CLUE data set of the locations of RC buildings in Melbourne (Hoult et al., 2019). In the figure, the yellow, blue, and red colours represent buildings of 2-3, 4-7, and 8-12 storeys in height.

In **Stage 2**, if possible, the structural information about the building is extracted from the database used in Stage 1. If this information is not available, a correlation study based is conducted on the data processed in Stage 1 to make an educated estimate of the structural information. The construction year serves as the primary parameter for establishing correlations with the structural parameters required in Stage 2. For instance, by referencing the construction year, the design standards used can be tracked. Using building functionality and the live load rating, axial load in the wall and the floor seismic weights are determined. Similarly, mechanical properties, such as the material (e.g., steel and concrete) and their strength grades, can be defined by considering the construction year and manufacturing trends. Two different correlation scenarios are considered based on the year of construction. The study by Munter and Lume (2018), which discusses the history of the evolution of reinforcement bars and concrete in Australia, serves as a reference. In relation to the typical range of wall reinforcement percentage (Pv), the data were collected based on the engineering judgment of the authors and input from expert consultations. Scenario 1 as detailed in Table 1, is employed when deformed bars are used. Scenario 2 as detailed in Table 2, is employed when normal round bars are used, or the type of reinforcement is unknown.

In **Stage 3**, a sensitivity analysis is performed to examine how various structural parameters, also referred to as mechanical parameters, including percentage of reinforcement, axial load, concrete strength, yield and ultimate strength of reinforcement, can affect the seismic capacity of buildings. In a 2018 study, Menegon conducted a parametric analysis of rectangular reinforced concrete walls. The study explored variations in parameters such as wall length, percentage of reinforcement, axial load ratio, and concrete strength. For walls with an axial load ratio greater than 0.05, Menegon made the following conclusions:

- Wall length exhibited a dominant influence on both moment capacity and curvature.
- The percentage of reinforcement exhibited a strong impact on moment capacity, but it had minimal effect on curvature capacity.
- Concrete strength had a minor effect on moment capacity but had negligible influence on displacement capacity.
- Axial load ratio had a significant effect on moment and curvature capacity while it had very little impact on yield curvature.

Table 1. Structural parameters based on year of construction (when deformed bars are used).

Year of Construction	Design Criteria	Concrete Code	f_y (MPa)	f_u (MPa)	e_{sy}	e_{su}	P_v	f'c (MPa)
till 1963	WLS (Gravity Load)	SAA CA.2 (1934)	200	400	0.001	0.002	0.002-0.005	10 to 20
1963-1973	WLS (Minimum Design Loads_SAA Int 350_1952)	S.A.A. CA.2 (1963)	400	440	0.002	0.004	0.0025-0.005	15 to 20
1973-1993	ULS (Wind_AS 1170.2_1973)	S.A.A. CA.2 (1973)	410	440	0.00205	0.0041	0.0025-0.007	20 to 30
1993-2001	ULS (Wind_AS 1170.2_1989 + Earthquake AS 1170.4_1993)	AS 3600 (1988)	400	440	0.002	0.004	0.0025-0.008	25 to 40
2002-2007	ULS (Wind_AS 1170.2_2002 + Earthquake AS 1170.4_1993)	AS 3600 (2001)	500	600	0.0025	0.005	0.0025-0.009	25 to 40
2007-Present	ULS (Wind_AS 1170.2_2002 + Earthquake AS 1170.4_2007)	AS 3600 (2001)	500	600	0.0025	0.005	0.0025-0.01	30 to 50

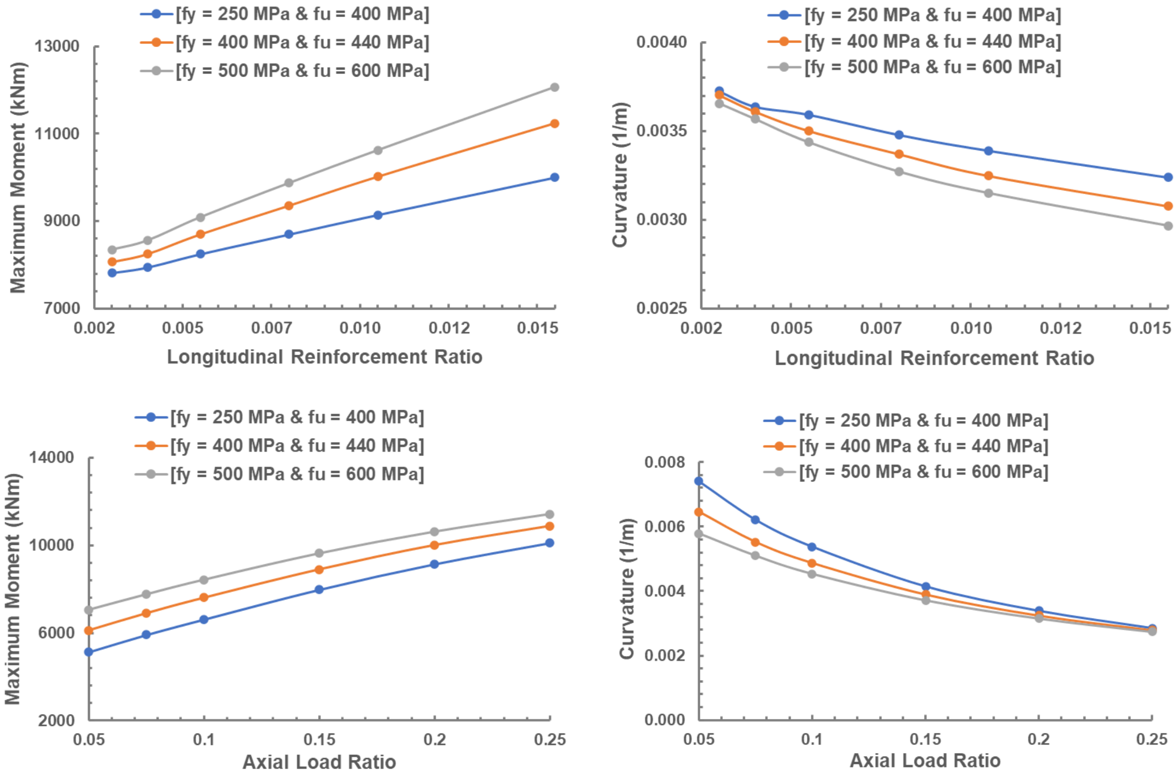
Table 2. Structural parameters based on year of construction (when round bars are used, or material is unknown).

Year of Construction	Design Criteria	Concrete Code	f_y (MPa)	f_u (MPa)	e_{sy}	e_{su}	P_v	f'c (MPa)
till 1973	WLS (Gravity Load)	S.A.A. CA.2 (1934)	200	400	0.001	0.002	0.002-0.007	10 to 20
1973-1993	ULS (Wind_AS 1170.2_1973)	S.A.A. CA.2 (1973)	230	400	0.0015	0.003	0.0025-0.007	15 to 25
1993-2007	ULS (Wind_AS 1170.2_1989 + Earthquake AS 1170.4_1993)	AS 3600 (1988)	250	400	0.002	0.004	0.0025-0.008	20 to 30
2007-Present	ULS (Wind_AS 1170.2_2002 + Earthquake AS 1170.4_2007)	AS 3600 (2001)	300	440	0.0015	0.003	0.0025-0.001	30 to 40

Where,

- f_y: characteristic yield strength of reinforcement
- f_u: ultimate strength of reinforcement
- e_{sy}: yield strain of reinforcement
- e_{su}: ultimate strain of reinforcement
- P_v: reinforcement ratio
- f'c: characteristic compressive strength of concrete

As Menegon did not investigate the yield and ultimate strengths of the reinforcement, a sensitivity analysis was conducted on 3-meter-long walls using the in-house MATLAB-based sectional analysis tools developed by the authors. For this analysis, three common types of bars typically used in Australia were considered, each with combinations of yield and ultimate strengths of 250 MPa and 400 MPa, 400 MPa and 440 MPa, and 500 MPa and 600 MPa, respectively. Firstly, the reinforcement ratio was varied from 0.0025 to 0.015 while maintaining a constant axial load ratio of 0.2. Then, the axial load ratio was varied from 0.05 to 0.25, maintaining a constant reinforcement ratio of 0.01. For both cases, the maximum moment capacity and the corresponding curvature were determined, and the results are presented in Figure 3(a) and (b), respectively. Figure 3 illustrates that the reinforcement strengths have more significant influence on moment capacity and a less significant influence on curvature, particularly at higher reinforcement ratios (> 0.005) and lower axial load ratios (< 0.1).



(a) Maximum Moment Capacity *(b) Curvature at Maximum Moment*

Figure 3. Effect of reinforcement strengths on maximum moment (a) and the corresponding curvature (b).

From the findings of Menegon (2018) and this study, it can be concluded that the capacity of a RC wall is primarily affected by the vertical reinforcement ratio and the axial load ratio, while material strengths only have minor effects. Through such sensitivity studies, the dominant structural parameters that govern the capacity of the building under consideration can be identified, and more emphasis can be placed on addressing their uncertainties.

Stage 4 involves resolving uncertainties in the dominant input parameters identified in Stage 3 and finalizing these parameters for calculating building capacity. For less dominant parameters, the mean value from the range identified in Stage 2 can suffice. When dealing with dominant parameters, there are two approaches: deterministic and probabilistic. In cases where the structural layout, including geometric parameters of structural elements such as wall length and thickness, is well-defined, the members are redesigned using the original Australian Standards employed for the building's initial design. This approach is referred to as the deterministic approach and typically results in a design similar to the original construction. However, when the structural layout or geometric parameters of the structural elements are unknown, a probabilistic approach such as Monte Carlo Simulation (MCS) and/or machine learning (ML) are used. Such a probabilistic approach helps to reduce uncertainty in geometric and mechanical input parameters. Both MCS and ML assume input parameter uncertainties follow a normal distribution, with selected mean and standard deviation values for each parameter. In step 2, in MCS, various input parameter combinations are used to generate simplified moment-curvature or force-displacement capacity curves (see Khatiwada et al., 2023, for details), while in ML, machine learning algorithms based on existing data from numerous test specimens are used. In step 3, mean and standard deviation values for capacity curves are determined. In step 4, input parameters corresponding to mean capacity curves are backtracked using reverse ML.

A building in Melbourne has been selected as a case study in Section 4 to illustrate the preparation of building structural information using the proposed four-stage method.

4 Case Study

A 9-story reinforced concrete institutional building, constructed in 2002 in Melbourne, was chosen as a case study to demonstrate the four-stage method described in Section 3. Basic geomatic and geometric building data, including the architectural floor plan and elevation details, were obtained from the University of Melbourne's online library resources. The building's total height is 36.4 meters, with a floor height of 6 meters at the ground floor and 3.8 meters for the other stories. The floor plan was translated into the structural layout, depicted in Figure 4, which illustrated the building's shape, the number of bays and their spans, the constituent structural elements, encompassing both gravity and lateral load-resisting systems, along with the dimensions of these structural elements. Since no mechanical properties of the walls could be found in the drawings, they were initially estimated based on the construction year, using Table 1. In light of the results of the sensitivity analysis, the uncertainty in material strengths was disregarded. Consequently, a reinforcement yield strength of 500 MPa and a concrete characteristic strength of 32 MPa were adopted. Additionally, the axial load ratios at the base of walls 'W1,' 'W2,' and 'W3' were calculated as 0.05, 0.1, and 0.05, respectively, considering a floor live load of 2.5 kPa and a superimposed dead load of 1 kPa (AS 1170.1 1989). Furthermore, the approximate seismic floor weight (dead load + 0.3 live load) was calculated as 4000 kN per storey.

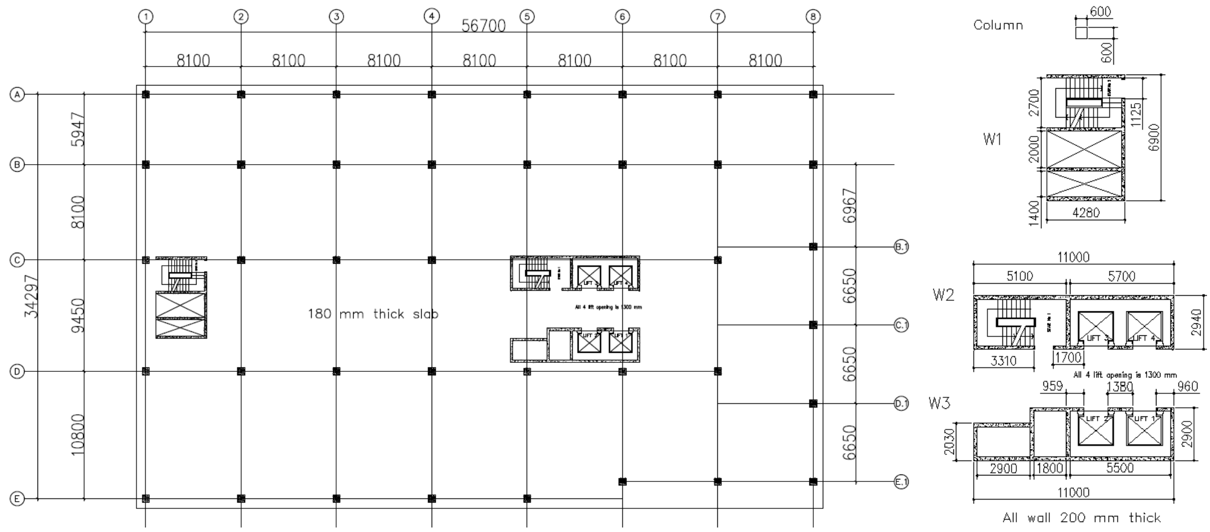


Figure 4. Floor plan and structural details of the case study building.

In this instance, since the architectural drawings already provided dimensions for structural elements such as columns and walls, both deterministic and probabilistic approaches were employed to design the amount of vertical reinforcements, as discussed in Stage 4. Both approaches are presented here for comparison.

Deterministic approach: The building was redesigned using AS 1170.4 (1993) and AS 3600 (2001) standards. The response spectral acceleration (RSA) value of 0.175g (~$1.25aS/T^{0.67}$, where 'a' is 0.08 for Melbourne, 'S' is 1.5 for soft soil site, and 'T' is the fundamental mode of vibration building = height/46 = 0.8 sec) was used. The seismic base shear in the building was calculated as RSA × seismic weight / R_f = 2400 kN (where seismic weight is 9 × 4 kN and 'R_f' is the response reduction factor equal to 4 for limited ductile walls). The corresponding bending moments at the base of the walls were determined as 27 MNm, 7 MNm, and 11 MNm for 'W1,' 'W2,' and 'W3,' respectively. To satisfy this design moment, the walls were required to have a vertical reinforcement ratio of 0.0053.

Probabilistic approach: A machine learning algorithm, as discussed in Section 3, was used to analyze wall 'W1' assuming a 'C' shaped wall with web length of range 4 m-8 m, flange width of 4.5 m (because the function of the wall is a staircase), uniform thickness of 200 mm, 32 MPa grade concrete, and reinforcement ratio range of 0.003 to 0.009. The axial load ratio was kept constant at 0.05, and the reinforcement yield strength of 500 MPa was used. The moment-curvature results of the wall were determined for all possible combinations of wall length and reinforcement ratio and the obtained curves are shown, along with the mean curve, in Figure 5. The standard deviation divided by the mean (at the ultimate state) was calculated as 0.23, which is within an acceptable range (Marasco et al., 2021). Reverse machine learning was used to predict the optimum input parameters, resulting in two combinations: a wall length of 5 m with reinforcement ratio of 0.009, and a wall length of 6.7 m with reinforcement ratio of 0.0055. This shows that, both the deterministic and probabilistic approaches are found to predict similar wall length and reinforcement ratio.

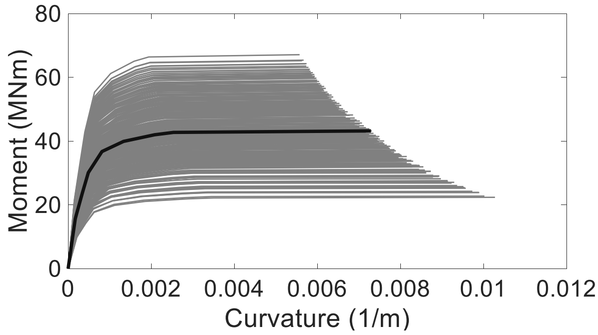


Figure 5. Use of machine learning in finding out the mean moment-curvature capacity curve (black).

5 Structural Analysis and Predictions of Engineering Demand Parameters

Considering the large number of buildings in urban areas, it is necessary to develop a simplified seismic response prediction model for buildings to minimize the required computational time for analysis. In this paper, an efficient macro model-based rapid nonlinear time history analysis, as developed by the authors (Khatiwada et al., 2023) and illustrated in Figure 6, is employed to achieve this objective. In contrast to a refined Finite Element Model, this approach reduced the computational cost by restricting the degree of freedoms (DOFs) to a minimum of 4 DOFs (comprising 3 DOFs representing linear translation due to the first three modes and an additional DOF to account for the inelastic behaviour of the plastic hinge), while still providing reasonably accurate predictions of the seismic response of the structure.

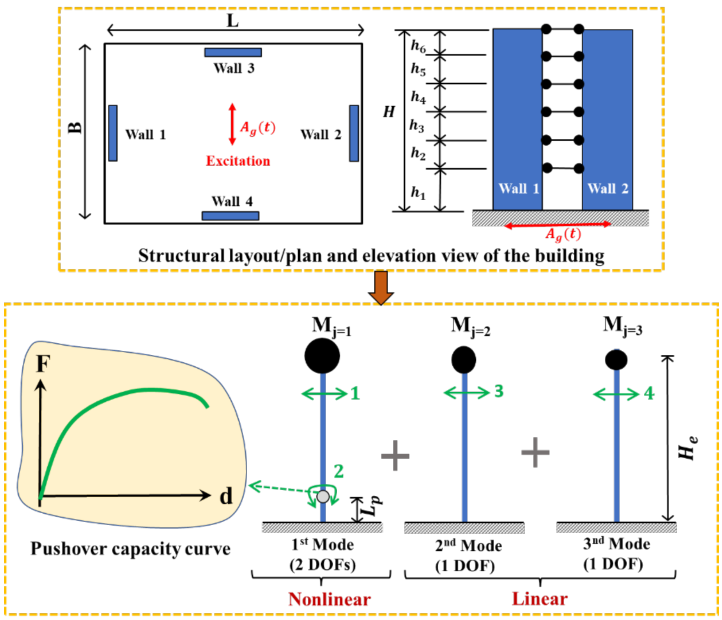


Figure 6. RNLTHA procedure utilising macroscopic model (Khatiwada et al., 2023).

After obtaining the structural input parameters of the building as explained in Section 3, the nonlinear sectional analysis procedure is applied to generate the inelastic force-displacement curve for the entire structure or system. Subsequently, the incremental dynamic analysis utilizing the Rapid Nonlinear Time History Analysis (RNLTHA) procedure, as described in Khatiwada et al. (2023), is performed for each site-specific accelerogram (discussed in Section 2), and various engineering demand parameters (EDPs) are estimated, including maximum storey and interstorey drift, as well as maximum concrete and reinforcement strains in the critical elements of the structure. This process is then repeated for the remaining buildings within the urban-scale model. Using the generated EDPs, the level of damage to the analysed structures is identified, as discussed in Section 6.

6 Interpretation of the Engineering Demand Parameters to Seismic Damage

The estimated engineering demand parameters are used to determine damage levels based on predefined limits for drift and material strain corresponding to each level of damage. Five damage levels, as outlined in the Hazus report (FEMA, 2010), are applied: (1) no damage, (2) slight damage, (3) moderate damage, (4) extensive damage, and (5) complete damage. The criteria of Ghobarah (2004) are applied to define the 'no damage' level, while the criteria of Menegon et al. (2019) are applied to the other four levels. Additionally, a 'collapse' state is introduced, indicating a condition beyond complete damage, where the structure is either on the verge of collapse or prone to total collapse. The definitions of these five damage levels in terms of the force-displacement curve of the building are depicted in Figure 7. Similarly, the definitions of the damage levels and the associated drift and material strain limits are summarized in Table 3.

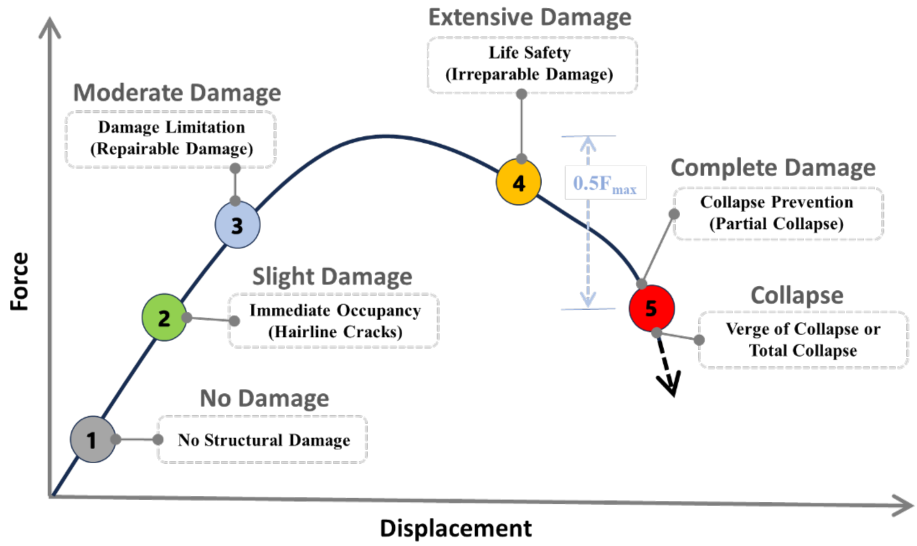


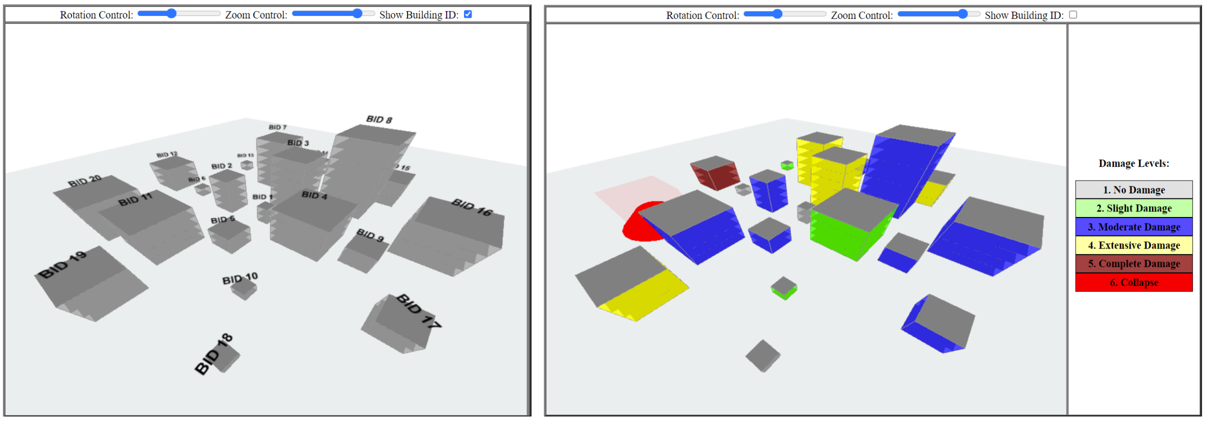
Figure 7. Definition of damage states for buildings with RC elements.

Table 3. Interpretation of the engineering demand parameters to different damage states.

Damage State	Description	Drift Limit (%)	Concrete Strain	Steel Strain
No Damage	No structural damage is observed. Some fine cracks in plaster may exist.	0.1	0.0008	0.5 × yield strain
Slight Damage	Immediate occupancy is possible. Concrete and reinforcement strains are within the elastic limit. Hairline cracks are expected in columns/walls.	0.2	0.0015	yield strain
Moderate Damage or Repairable Damage	The structure has reached its yield capacity in the critical lateral load resisting elements (either concrete has reached peak strain or steel has reached its yield strain). Damage to structural elements is limited (flexural and shear cracking in columns and walls) and the building is repairable.	0.5	0.002	2 × yield strain

Extensive Damage	The structure has reached its ultimate capacity. Large cracks in concrete or concrete spalling may occur. Buckling or fracturing of the reinforcement may occur. The building is nonrepairable (very serious damage) but no loss of life.	1	0.006	0.05
			strain when force capacity = 0.8 F_{max}	
Complete Damage	Partial collapse of the lateral and gravity load carrying elements are observed. However, axial load carrying capacity still remains therefore loss of life may be prevented.	1.5 - (0.75 × axial load ratio)	strain when force capacity = 0.5 F_{max}	
Collapse	The structure is on the verge of collapse or may experience total collapse.	exceeds complete damage	exceeds complete damage	exceeds complete damage

The graphical user interface of the input system and the easy-to-interpret 3D visualization of damage states will be enabled through an online virtual city model hosted on a website, controlled by JavaScript scripting. An example of this is depicted in Figure 8, where the before and after assessment of the virtual city model are visualized. This 3D visualization will assist decision-makers in identifying vulnerable structures and implementing seismic retrofitting measures to enhance the seismic resilience of these buildings.



(a) Before assessment *(b) After assessment*

Figure 8. Damage visualisation through 3D graphical display.

7 Conclusions

Recent seismic events in Australia have highlighted the vulnerability of our built environment to earthquakes. The majority of existing structures are not designed with seismic resilience in mind, posing substantial risks to both structures and human life. To address this issue, this study proposes a comprehensive urban-scale seismic vulnerability assessment for reinforced concrete buildings in Australia, utilizing a virtual city model. This approach aims to overcome the challenges posed by large building stocks and limited data availability. A case study has been presented to demonstrate how such challenges can be effectively resolved. By addressing uncertainties related to earthquake excitations, building properties, seismic response prediction, and damage interpretation, proactive steps can be taken toward strengthening critical structures and enhancing preparedness for future seismic events in Australia.

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