An Investigation into Adequacy of Separation Gap to Preclude Earthquake-induced Pounding

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Abstract. Pounding happens when contiguous structures with differing heights vibrate out of line caused by a seismic activity. The situation is aggravated due to the insufficient separation gap between the structures which can lead to the crashing of the buildings or total collapse of an edifice. Countries around the world have compiled building standards to address the pounding issue. One of the strategies recommended is the introduction of the separation gap between structures. AS1170.4-2007 is an Australian standard that requires 1% of the building height as a minimum separation gap between buildings to preclude pounding. This article presents experimental and numerical tests to determine the adequacy of this specification to prevent the occurrence of seismic pounding between steel frame structures under near-field and far-field earthquakes. The results indicated that the recommended minimum separation gap based on the Australian Standard is inaccurate if low-rise structure in a coupled case is utilised under both near and far field earthquakes. The standard is adequate if a tall building is involved but only when a far-field earthquake happens. The research likewise presents results derived by using the ABS and SRSS methods.

Keywords: Seismic Response, Structural Pounding; Steel Structure; SAP2000; Separation Gap; Seismic Code; Shaking Table, Moment Resisting Building Frames

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1. Introduction

Investigations from the past up to the present have shown that the collision of adjacent structures can cause severe damage during an earthquake excitation (Shrestha & Hao 2018). The resulting collision is commonly known as 'structural pounding'. Pounding of adjacent edifices has caused a lot of damage and in many instances led to the total collapse of structures. According to Raheem (2013), pounding is a phenomenon in which two buildings strike due to their lateral movements induced by lateral forces. Cole et al. (2011) consider seismic pounding as the collision of adjacent buildings during earthquakes. Jeng et al. (1992) stated that the difference in the dynamic characteristics of each structure and the existing distance between the buildings were common causes of structural pounding. The said factors are often the results of an out of phase vibration.

The effects of structural pounding have been elucidated in various researche. For instance, statistical records indicate that over 40% of the damaged or collapsed buildings during the 1985 Mexico earthquake are attributed to structural pounding (Rosenblueth & Meli 1986). The Loma Preita earthquake in 1989 which had a moment magnitude scale (Mw) of 7.1 caused destruction of over 200 structures (Kasai & Maison 1997) . In the said earthquake, two adjacent ten story and five story buildings situated at 90 kilometres away from the epicentre experienced pounding. The gap between both structures was about 4 cm. Pounding transpired at the sixth story of the ten story building and at the top story of the five story edifice (Kasai & Maison 1997; Lin & Weng 2002) . Structural pounding was also observed in the 1999 Chi Chi earthquake in Taiwan. A school building with new classrooms built near an old structure experienced pounding due to insufficient gap (Lin & Weng 2002). Some other notable earthquakes which caused pounding are Sequenay earthquake in Canada, 1988, Cairo earthquake in 1992, Northridge earthquake in 1994, Kobe earthquake in 1995, Tohoku earthquake in 2011 and others.

Researchers have studied the above mentioned earthquakes to highlight the reasons and causes of massive damage that occurred due to pounding effects. They classified pounding as the following:

- Floor-to-floor collision (Cole et al. 2010; Kazemi et al. 2021)
- Floor-to-column collision (Efraimiadou et al. 2013; Kazemi et al. 2018)

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- Eccentric or non-eccentric pounding (Leibovich et al. 1996; Polycarpou et al. 2014; Raheem et al. 2019)
- Pounding of heavier building with adjacent lighter building (Jankowski 2008b, 2010; Kazemi et al. 2020)
- Pounding between buildings in series (end building pounding) (Anagnostopoulos 1988; Raheem et al. 2018; Skrekas et al. 2014).

Jankowski (2008a) described the structural pounding as a relatively complex phenomenon which basically involved plastic deformations at any given contact point, and in turn can cause local cracking and crushing during the resulting impact of an earthquake. Over the years, researchers considered insufficient separation gap as the main reason for structural pounding (Jankowski & Mahmoud 2016; Jeng et al. 1992; Lopez-Garcia & Soong 2009; Far& Flint 2017). Many studies conducted proved that providing sufficient gap is a reasonable approach to mitigate the incidence of pounding (Hao 2015). Based on these studies, providing sufficiently larger gap between adjacent buildings appears to be a reasonable solution to prevent collision.

Many studies were conducted to mitigate the pounding effects. Solutions are divided into two types and according to the buildings statuses, if buildings are not constructed yet, then creating enough separation gap between the adjacent buildings is the appropriate solution (Kamal & Inel 2022; Khatami et al. 2020). However, if the buildings are already constructed, then engineers need to think of other solutions (Abdel Raheem 2014; Jankowski & Mahmoud 2016). For non-constructed buildings, researchers have suggested a separation gap solution by deriving advance mathematical equations using various technique and parameters. Some parameters were used like the short building height, tall building height, natural period and others. Favvata (2017) examined inter-story pounding cases between an 8-storey RC frame adjacent to a 3-storey RC frame buildings in order to determine the minimum separation gaps for three intensity levels of seismic hazard. The required separation distance has been determined to prevent shear failure in the exterior column where the pounding is occurred. Moreover, it was established that the minimum separation distance is dependent on the level of the seismic hazard. For constructed buildings, researchers have suggested various solutions, such as: building shear walls, using soft material layers, or connecting adjacent structures together with links. These solutions will be further discussed in Section 4.

Even though providing a sufficient gap is considered as one of the best solutions in decreasing the occurrence of collision between structures, many property owners and engineers do not adopt this strategy because it is costly and architecturally difficult. Within this purview, there are other techniques recommended to reduce the incidence of pounding, which will be discussed later.

This study aims to determine the adequacy of the minimum separation gap prescribed by AS1170.4, to mitigate the incidence of structural pounding between adjacent structures. Specifically, the main purpose was to find out whether or not the minimum separation gap of 1% between two adjacent steel frame structures is adequate to preclude pounding under earthquake ground motions.

In the study, experimental and numerical tests will be carried out to measure separation gap between adjacent structures to avoid earthquake-induced pounding. Testing will be conducted in an independent lab platform based on the records of past earthquakes on the scaled models to ascertain the lateral deflection and acceleration on the shake table. Experimental data will be measured using the accelerometers and laser displacement sensors. A full nonlinear time history dynamic analysis will be performed on the scaled structural models to produce numerical results using SAP2000.

Fig. 1 illustrates two different examples for adjacent buildings in Sydney, Australia. Fig. 1(a) shows that the 1% separation gap has been applied in recently constructed buildings. The said application has not been implemented for the old buildings as shown in Fig. 1(b).

(a) (b)

Fig. 1 Case of neighbouring buildings in Sydney, Australia (a) with separation gap; (b) with zero separation gap (image by Yazan Jaradat)

2. Review on Codal Provisions

Building standards in seismically active regions around the world take into consideration the influence of earthquake induced pounding on structural frames by recommending some construction guidelines to mitigate its adverse effect. The most common provision integrated in the building codes is to separate the structures to prevent interactions between adjacent edifices. Some building standards based their separation gap requirements on the resulting displacements while other code provisions take into consideration the building height or a combination of the building height and the separation gap requirements. Other countries went further by taking into account the type of soil where the edifices are constructed as well as the design of the structure. As stated in International Building Code (IBC 2009) and Eurocode 8 (Eurocode-8 2005), the required separation gap is given by:

$$S = U_a + U_b \tag{1}$$

$$S = \sqrt{{U_a}^2 + {U_b}^2} \tag{2}$$

Eqs. (1)-(2) are commonly referred to as the absolute sum (ABS) and square-root-of-sum-of-squares (SRSS) methods, respectively. If the adjacent building separated by a proper line or located on the same property, the ABS and SRSS methods will be used accordingly (Lopez-Garcia & Soong 2009). Moreover, in Canadian Standard (NBCC 2010) the formula is calculated using the following expression:

$$S = \sqrt{U_a + U_b} \tag{3}$$

where S, U_a, U_b are separation distance, peak displacement response of the adjacent structures A and B, respectively, in the location where pounding is expected to occur (Lopez-Garcia & Soong 2009). A similar requirement can be observed in American Society of Civil Engineers ASCE7–10 (ASCE 2013).

Referring to Chinese earthquake standard GB5001, the minimum gap in 15 m high building or less is 0.07m, which increases 0.02m for seismic intensity level of 6 to 9 (GB50011 2001). However, the provision has been upgraded in GB50011-2010 to 100 mm in concrete framed buildings.

The 1997 Taiwan Building Code (TBC 1997) suggested the following formulae when considering the construction of the same type of structures:

$$S_{code} = 0.6(\Delta u_a + \Delta u_b) \tag{4}$$

where

$$\Delta u_a = 1.4\alpha_y R_a \Delta e_a \tag{5}$$

$$\Delta u_b = 1.4\alpha_y R_b \Delta e_b \tag{6}$$

where $\Delta u_a$and $\Delta u_b$are the displacements with 0.6 and 1.4 representing the factor of reduction and over strength, respectively. R_a and $R_b$are the ductility factors, α_y is the amplification factor, Δe_a and $\Delta e_b$are the elastic displacements (Lin & Weng 2002). In other words, the required separation distance is equal to 60% of the absolute sum of peak inelastic displacements of the two adjacent buildings.

The Iranian Code of Practice for Seismic Resistant Design of Buildings (ISC 2005) and Australian Earthquake Standard (AS1170.4 2007) both follow a similar approach by recommending a gap of 0.01 of the building height. In response to this, Hao (2015) agreed with this approach, which is similar to ISC-2005 specification, and is expressed in Eq. (7).

$$S = 0.01H \tag{7}$$

where H is the building height.

The gap required to preclude earthquake-induced collision between closely spaced buildings has be en investigated for many decades. Many cases were studied during this period with researchers sugges ting various methods, one of which is to separate adjacent buildings by single-degree-of-freedom (SD OF) oscillators (Anagnostopoulos 1988; Garcia 2004; Hao & Liu 1998; Kasai et al. 1996) or multiple -degree-of-freedom (MDOF) oscillators (Abdel Raheem 2014; Anagnostopoulos & Spiliopoulos 1992 ; Jeng et al. 1992; Lin & Weng 2001; Maison & Kasai 1992; Far & Far 2018) while considering structural responses in either elastic or inelastic phase.

3. Experimental Campaign

3.1 Tested Frames

The experimental program has tested 1/30 scaled single-bay moment resisting steel-frame models – as 15-storey, 10-storey and 5-storey structures – on an MTS 354.20 multi-axial simulation table of size 4.84 m2 at the University of Technology, Sydney. The shake table is capable of testing samples of 2 tonnes at 5 g accelerations, 1000 mm/s velocity and up to +/– 200 mm stroke. Associated building frames were analysed based on the requirements of AS4100 (Steel structures) with the connecting base plates in accordance to AS 3678–2011. The tested frames were designed following a similar approach as reported by Tabatabaiefar (2016), Tabatabaiefar & Mansoury (2016), and Tabatabaiefar et al. (2014). The overall floor plan dimensions of all models are 0.4 m × 0.4 m. The height of the 15-storey frame, 10-storey frame and 5-storey frame are 1.5 m, 1.0 m and 0.5 m, respectively. Flat bars of 0.04m wide and 0.002m thick as columns with 0.4m by 0.005m thick floors were selected as the respective scaled structural members. Detailed drawings of scaled models are illustrated in Appendix A. The laboratory models are presented in Fig. 2.

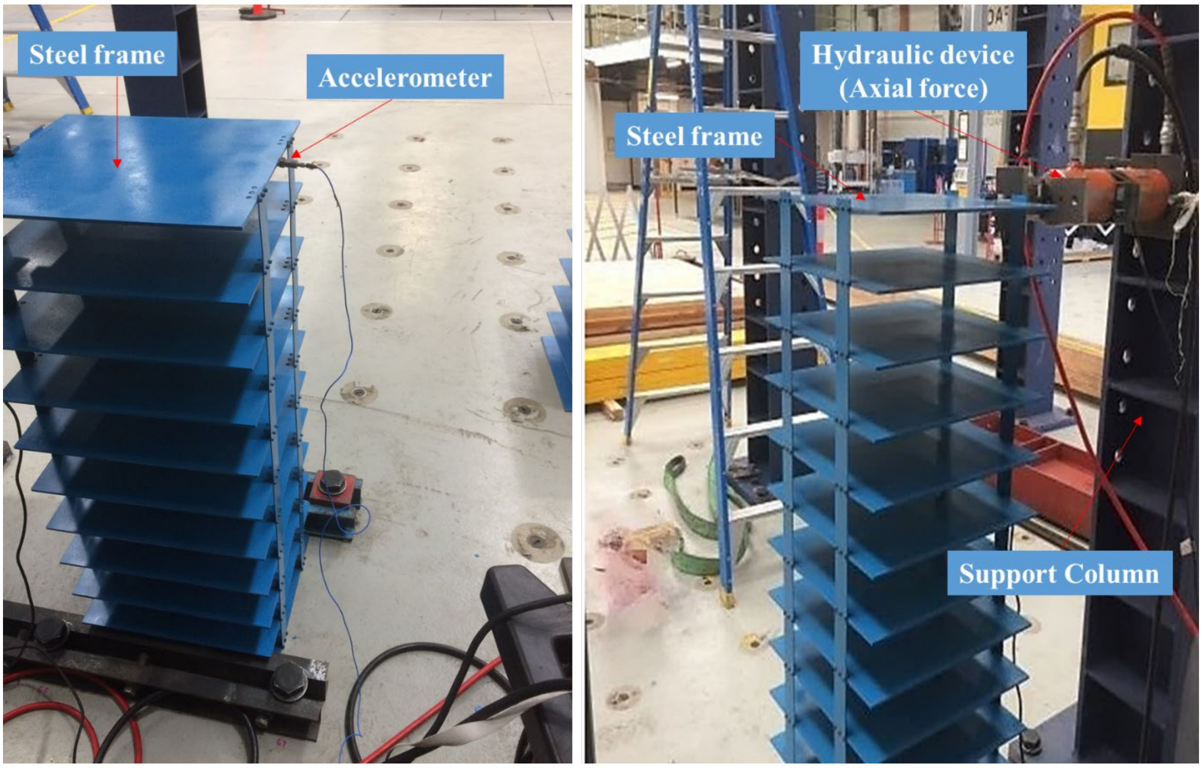
Fig. 2 15-storey, 10-storey, and 5-storey steel structural models

3.2 *Preliminary Identification Tests*

The dynamic characteristics of each steel frame were identified by conducting several preliminary tests including free vibration test, stiffness test and sine sweep test (Saleh et al. 2018). In the free vibration tests, the experiment aimed to measure the fundamental period and damping of the structures, each structure was excited manually in its first mode by displacing and releasing its roof level. The fundamental periods and natural frequencies were established from the acceleration decay time-histories using an accelerometer attached at the to the structure's top level as presented in Fig. 3(a). Fourier amplitude spectra and frequency response curves were generated from the recorded data of the free-vibration tests, and the natural frequencies were determined from the peaks of these plots which are presented in Fig. 4. Here, damping is calculated by using the half-power bandwidth method (Chopra 2007; Papagiannopoulos & Hatzigeorgiou 2011).

3.3 *Frame Stiffness Tests*

Load-deflection tests were carried out on all the models. The experiment focused on calculating the stiffness parameter of the frame structures. A hydraulic pressure device was used to measure this value, as shown in Fig. 3(b). The hydraulic pressure device is placed on a support beam placed on the right side of the picture with a red colour. The scaled models were subjected to a lateral loading at top level to deduce a resulting deflection in the three cases, where frame stiffness is expressed as $K = \frac{Lateral\ Load}{Deflection}$



(a) (b)

Fig. 3 (a) Natural frequency test using accelerometer; (b) Load-deflection test using hydraulic pressure device

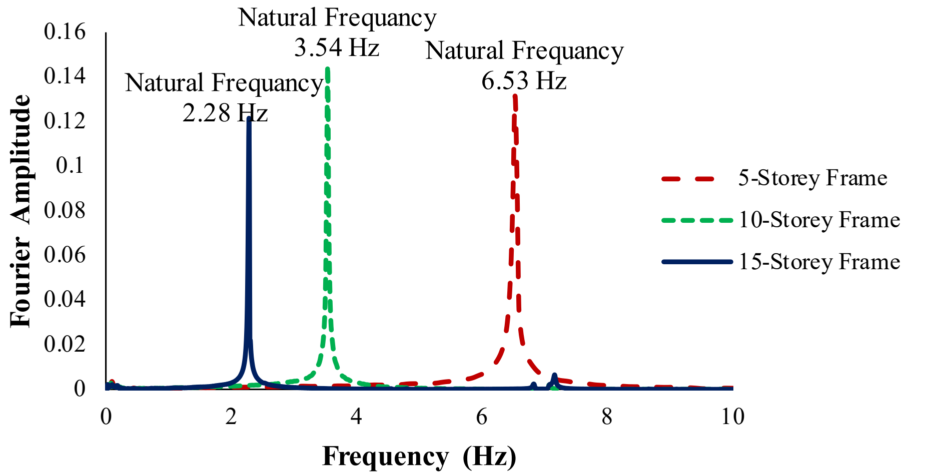


Fig. 4 Fourier amplitude frequency response curve for the 5-storey, 10-storey and 15-storey frames

3.4 Sine Sweep Tests

Finally, Sine sweep tests were performed on these scaled-models. The purpose of the sine sweep tests was to determine the natural frequency and modes of vibration particularly in modes 1, 2, and 3 as these could not be verified during the free vibration test. Sine sweep test involves a logarithmic frequency sweep holding a specified acceleration constant at the base of the structure. The frequency of the shaking table has increased from 0.1 Hz to 50 Hz, in order to achieve the aim of the Sine sweep test. The first resonance between the shaking table and structural model frequencies showed the fundamental natural frequency of the model. Several attempts were made required to achieve more accurate results which are tabulated in Table 1. The comparison showed that these results were similar to the models' frequency obtained from the free vibration tests presented in Table 1.

Table 1 presents the summary of the dynamic characteristics of the experimental and numerical results for the 15-storey, 10-storey and 5-storey models. Results are closely similar in natural period and stiffness values. Details about the numerical investigation are reflected in the succeeding sections of the study.

Table 1 Experimental and numerical dynamic characteristics of the structural models

	Experiment						***Numerical***				
	Free Vibration		Sine Sweep Test			Stiffness *kN/mm*	Modal Load Analysis			Stiffness *kN/mm*	Mass *Kg*
	Natural Frequency *Hz*	Damping %	Mode 1 *Hz*	Mode 2 *Hz*	Mode 3 *Hz*		Mode 1 Hz	Mode 2 Hz	Mode 3 Hz		
5-Storey	6.53	0.467	7.05	21.15	36.83	0.0275	6.76	20.31	33.24	0.0278	104.25
10-Storey	3.54	0.431	3.61	11.26	18.70	0.0144	3.53	10.57	17.56	0.0149	72
15-Storey	2.27	0.503	2.33	7.11	11.76	0.0081	2.29	6.87	11.44	0.009	34.85

3.5 Selected Earthquake Acceleration Records

Four scaled earthquake acceleration records, namely: El Centro 1940 (Fig. 5(b)), Hachinohe 1968 (Fig. 6(b), Northridge 1994 (Fig. 7(b)) and Kobe1995 (Fig. 8(b)) are utilised in the shake table tests. The time duration of the original ground motion was scaled by $\sqrt{\lambda}$, where λ is the scale factor of 1 /30 resulted in a time scale factor of 0.182. More details about scaling of adopted earthquake time histories can be found in (Tabatabaiefar & Mansoury 2016). The four mentioned earthquakes have been chosen by the International Association for Structural Control and Monitoring to benchmark seismic studies (K-Karamodin & H-Kazemi 2010). Frequencies, time-history and accelerations were included in the adopted results. The four mentioned earthquakes were diverse in relation to epicentre distance. The first two mentioned earthquakes (El Centro in 1940 and Hachinohe earthquake in 1968) were far-field occurrences whereas the other two (Northridge in 1994 and Kobe in 1995) were near-field in nature. Adverse behaviour of adjacent buildings during these types of earthquakes was the main factor in the seismic design (Yaghmaei-Sabegh & Jalali-Milani 2012). Hatzigeorgiou (2010) and Yaghmaei-Sabegh & Tsang (2011) emphasised the significance of these motions under the effect of dynamic elastic and inelastic analysis.

The Northridge earthquake (1994) had the highest Peak Ground Acceleration (PGA) among the four cases discussed, with a PGA equalling to 0.843 g and an epicentre distance of less than 9.2 km. The PGA of the Kobe earthquake (1995) was lesser in both PGA and the distance from the epicentre, measuring 0.833 g and 7.4 km, respectively. However, the PGA values decreased even further for the El Centro earthquake (1940) and the Hachinohe earthquake (1968) equating 0.343 g and 0.229 g, respectively, but farther from their respective epicentres, measuring at 15.7 and 14.1km in comparison to the other two.

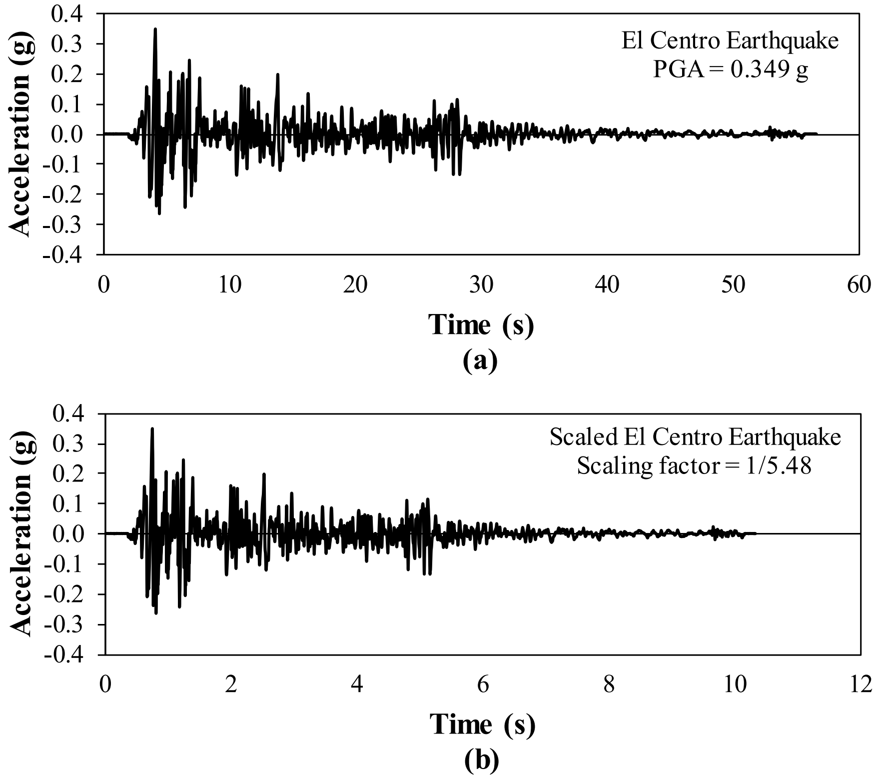


Fig. 5 El Centro earthquake 1940; a) Original record; b) Scaled record

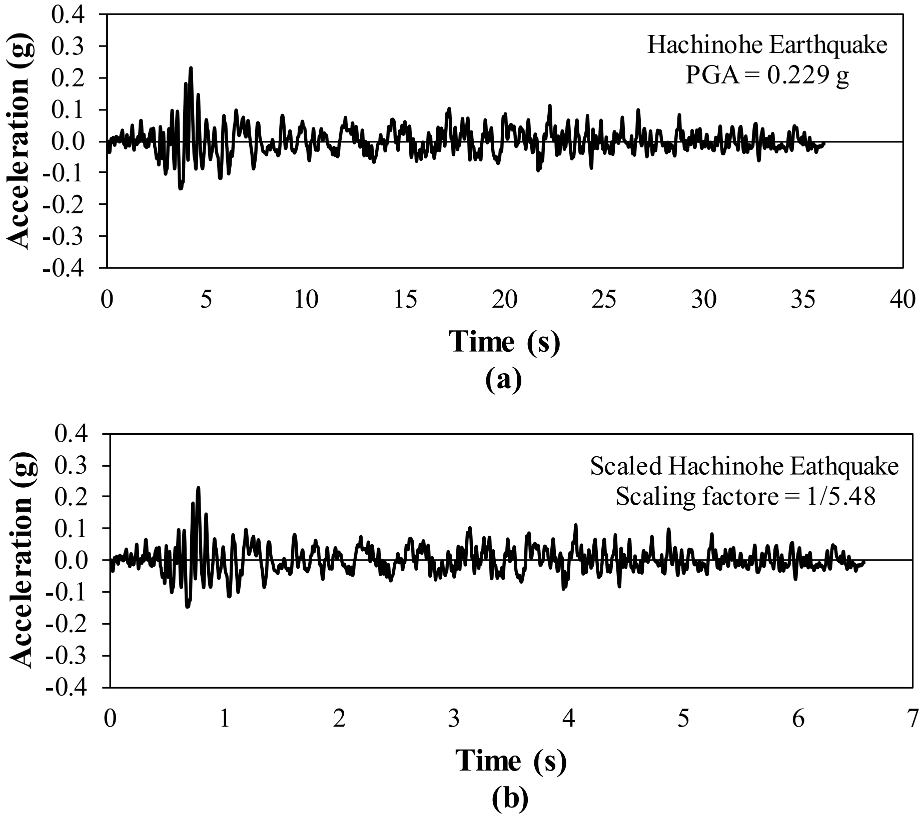


Fig. 6 Hachinohe earthquake 1968; a) Original record, b) Scaled record

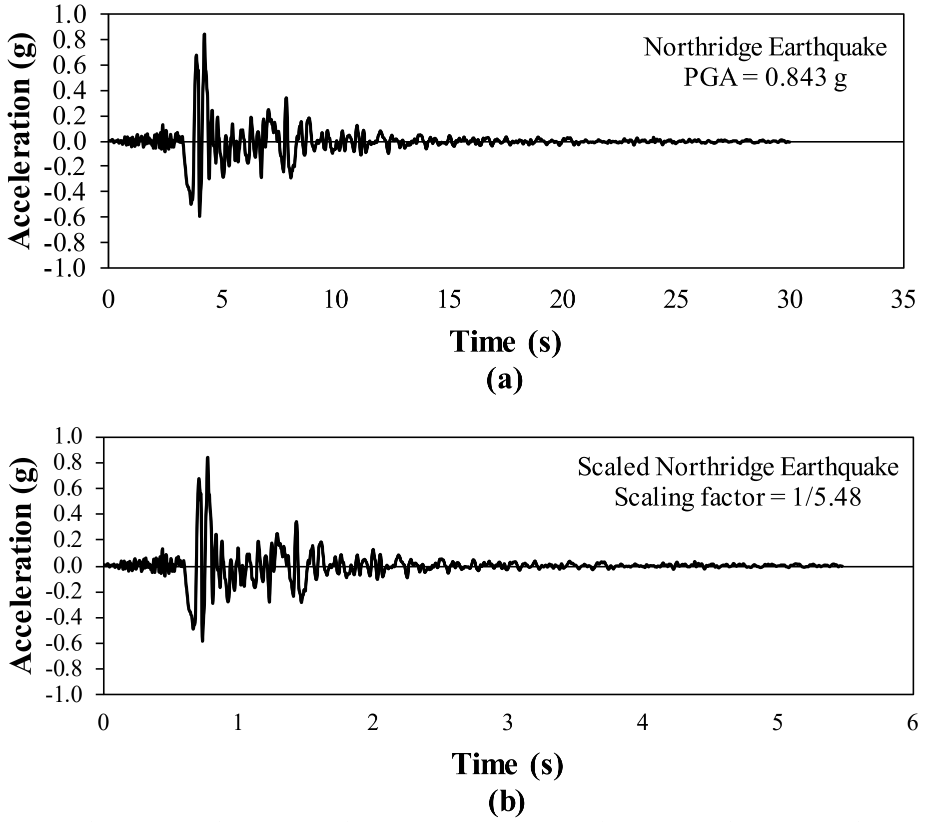


Fig.7 Northridge earthquake 1994; a) Original record, b) Scaled record

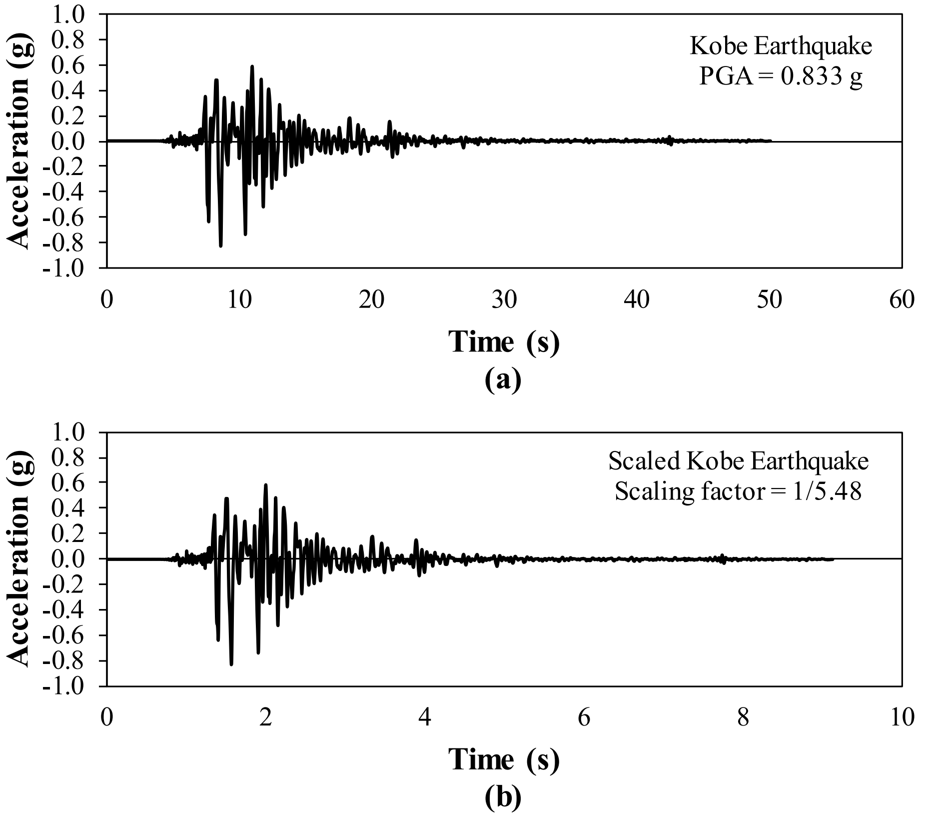


Fig. 8 Kobe earthquake 1995; a) Original record, b) Scaled record

3.6 Experimental Testing Program

During the experiment, the structural models were fixed and secured to the shake table with the coupled configuration of 15-storey & 10-storey, 15-storey & 5-storey, and 10-storey & 5-storey. Upon securing the models, the accelerometers and laser displacement (LD) sensors were connected to the first combination of 10-storey and 5-storey, as shown in Fig. 9. PCB 352C34 (±50 g) accelerometers were mounted to the frames and laser displacement sensors were installed on reference frames range from (±50 mm) to (±200 mm), experimental data were collected and digitised using National Instrument® acquisition data system (Fig. 10). The procedure was repeated for coupled 15-story & 10-storey, and 15-storey & 5-storey, from which the acceleration and displacements were recorded. An additional accelerometer was connected to the shake table platform to measure the applied acceleration. The recorded time-history was inputted to the computer model, to prevent any errors. Shake table tests were carried out by applying the above mentioned scaled earthquake acceleration records which are depicted in (Fig. 5(b), Fig. 6(b), Fig. 7(b) and Fig. 8(b)).

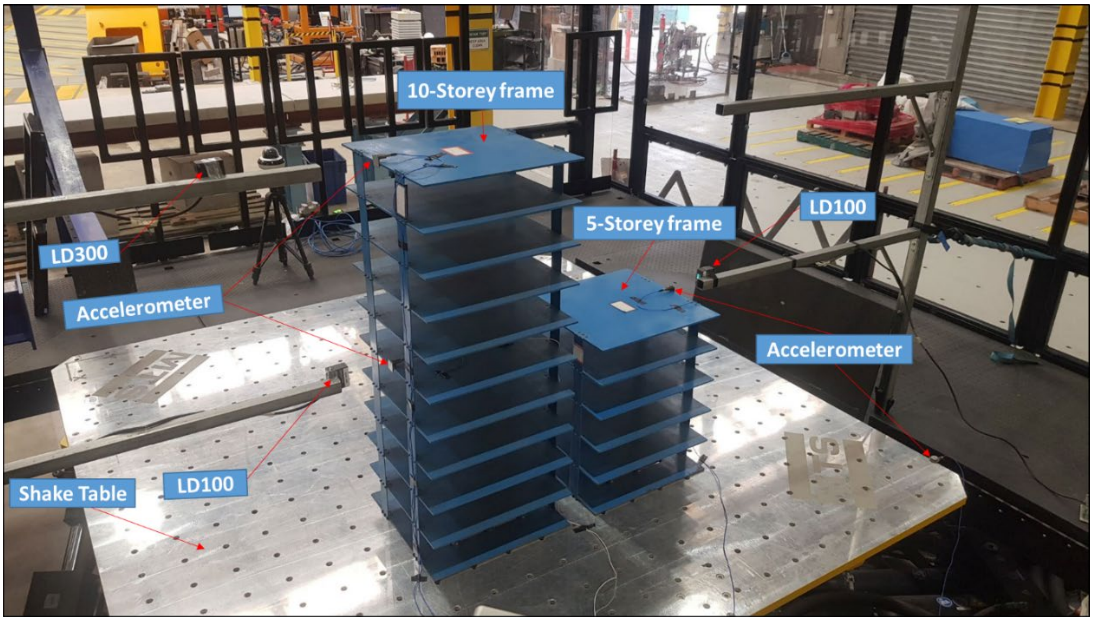


Fig. 9 Test frames on shaking table

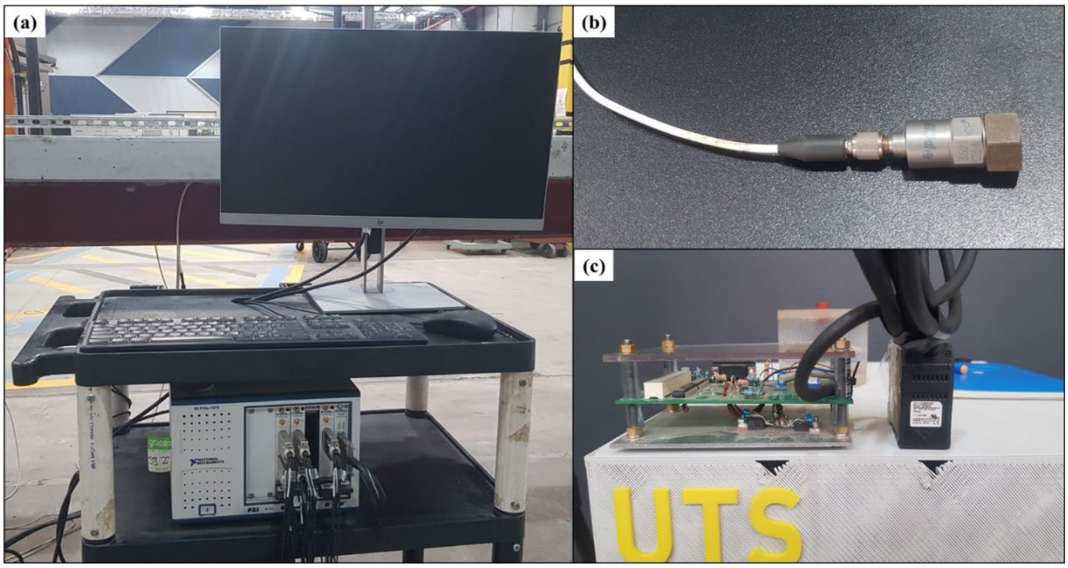


Fig. 10 a) Acquisition data system, b) Accelerometer, c) Laser displacement

4. Shaking Table Test Results and Numerical Investigation

Three-dimensional numerical models were created in SAP2000 version 20 (SAP 2000) utilising two-dimensional shell elements to model columns and floors. The models consist of five, ten and fifteen storey frames. The frames consist of four columns, which are modelled using vertical steel plates. Moreover, the slabs/floors are represented using horizontal steel plates as shown in Fig. 11 (Tabatabaiefar & Mansoury 2016; Tabatabaiefar et al. 2014). Numerical analysis involving time-history used the Ritz Modal Loading Analysis (Jaradat & Far 2020) to measure lateral deflection and acceleration. Mode numbers were selected targeting dynamics check modal participating mass ratios. Nonlinear time history dynamic analyses (fast nonlinear analysis or FNA) was conducted by applying a range of 6000-11000 time steps from the subject earthquakes.

Results are depicted in Figs. 12-17. In Fig. 12, experimental and numerical relative displacement time history subjected to the four scaled seismic excitations utilised in this study were compared to each other. For the top floor of the 5-storey frame, the highest relative displacement was caused by the Kobe earthquake. In Fig. 14 it can be seen that the peak relative displacement of the 10-storey frame was caused by the Northridge earthquake. Fig. 17, shows time histories and lateral deflection at the top floor of the 15-storey frame. The highest relative overall displacement belongs to the Northridge earthquake (1994).

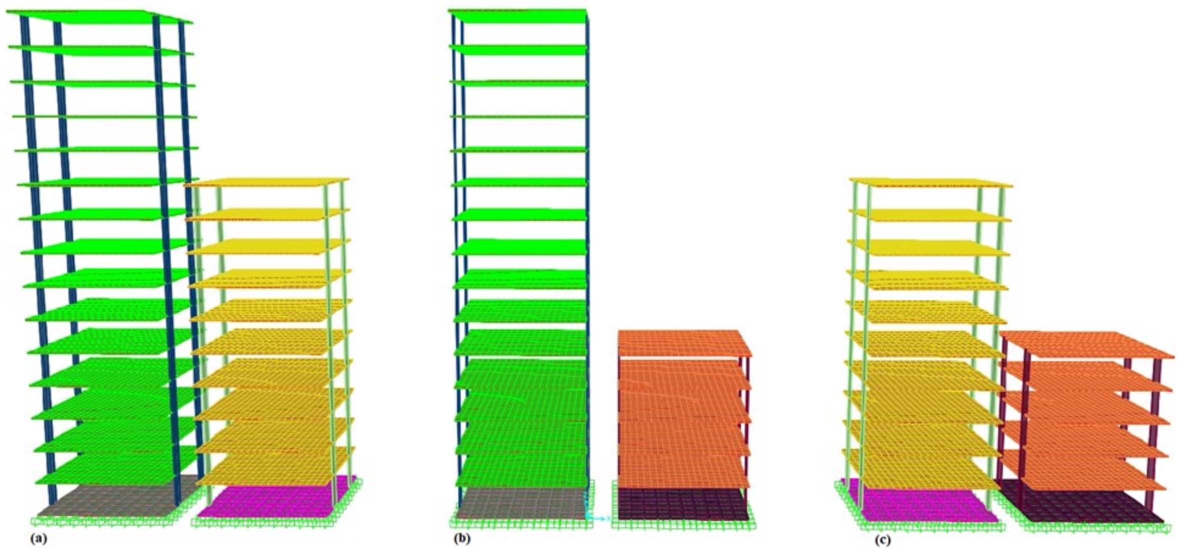


Fig. 11 3D numerical model of the structural models in SAP2000; a) 15-storey adjacent 10-storey; b) 15-storey adjacent 5-storey; c) 10-storey adjacent 5-storey

Table 2 tabulates peak displacement values for each earthquake of the minimum and maximum displacements. The results are classified according to the combination of the building heights, which are level 5 of the 5-storey frame, level 5 and level 10 of the 10-storey frame and level 5, level 10 and level 15 of the 15-storey frame.

The building response increases when the characteristic period of the ground motion is close to its fundamental period (Abdel Raheem 2006; Yaghmaei-Sabegh & Jalali-Milani 2012). Table 2 illustrates similar concept; it is apparent that the minimum and maximum displacements in Northridge earthquake (1994) were higher than the rest of the earthquakes in most cases. This explains Abdel Raheem (Abdel Raheem 2006) concepts of ground motion and fundamental period. The concept demonstrates the impacts of earthquakes on the building's response. If the earthquake movement happens in harmony with the building motion, then the displacement will be higher because of the momentum exerted into the building motion. Moreover, it is apparent from Fig. 18(c) that the response of the 15 and 10-storey buildings increases when dominant and fundamental period values are close during Northridge earthquake (1994). This can be seen in 5-storey building during Kobe earthquake (Fig. 18(d)), and Northridge earthquake (1994). This is not the case for the other two earthquakes as depicted in Figs. 18(a) and 18(b).

Table 2 Peak relative displacement, in mm

	El Centro				***Hachinohe***				***Northridge***				***Kobe***			
	Experiment		*Numerical*		*Experiment*		*Numerical*		*Experiment*		*Numerical*		*Experiment*		*Numerical*	
	Max	(-)Min	Max	(-)Min	Max	(-)Min	Max	(-)Min	Max	(-)Min	Max	(-)Min	Max	(-)Min	Max	(-)Min
15S/ Level 15	12.9	12.0	14.2	14.2	17.2	16.6	16.6	17.0	36.8	36.3	37.7	37.5	12.3	17.3	12.7	19.0
15S/ Level 10	12.1	9.5	10.5	10.9	15.2	13.3	14.3	14.6	32.7	32.5	33.3	33.5	9.6	12.2	11.6	11.8
15S/ Level 5	6.5	5.5	6.5	6.8	8.8	8.1	9.5	9.2	20.0	21.2	22.6	22.7	8.7	8.3	9.8	9.5
10S/ Level 10	5.8	6.8	5.8	6.1	3.3	4.7	3.5	3.3	26.4	23.5	27.8	28.6	21.4	18.3	24.8	22.1
10S/ Level 5	4.5	5.5	4.0	4.2	2.5	3.5	2.7	2.6	21.6	20.9	21.8	22.1	17.5	15.2	17.1	15.7
5S/ Level 5	4.5	4.9	4.8	5.0	3.1	3.1	3.2	3.0	8.2	9.0	8.1	8.9	13.1	14.4	13.7	14.5

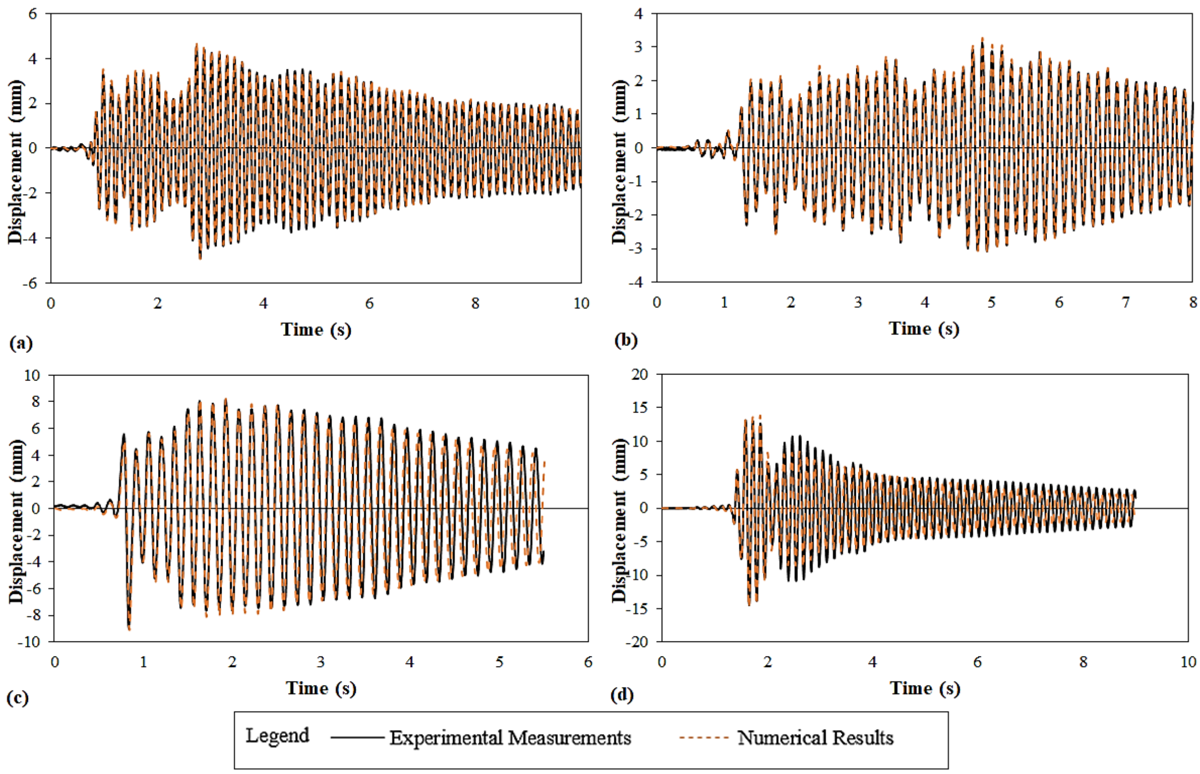


Fig. 12 Experimental and numerical relative displacement time histories for 5-Storey frame (fifth floor) under scaled; a) El Centro earthquake; b) Hachinohe earthquakes; c) Northridge earthquake; d) Kobe earthquake

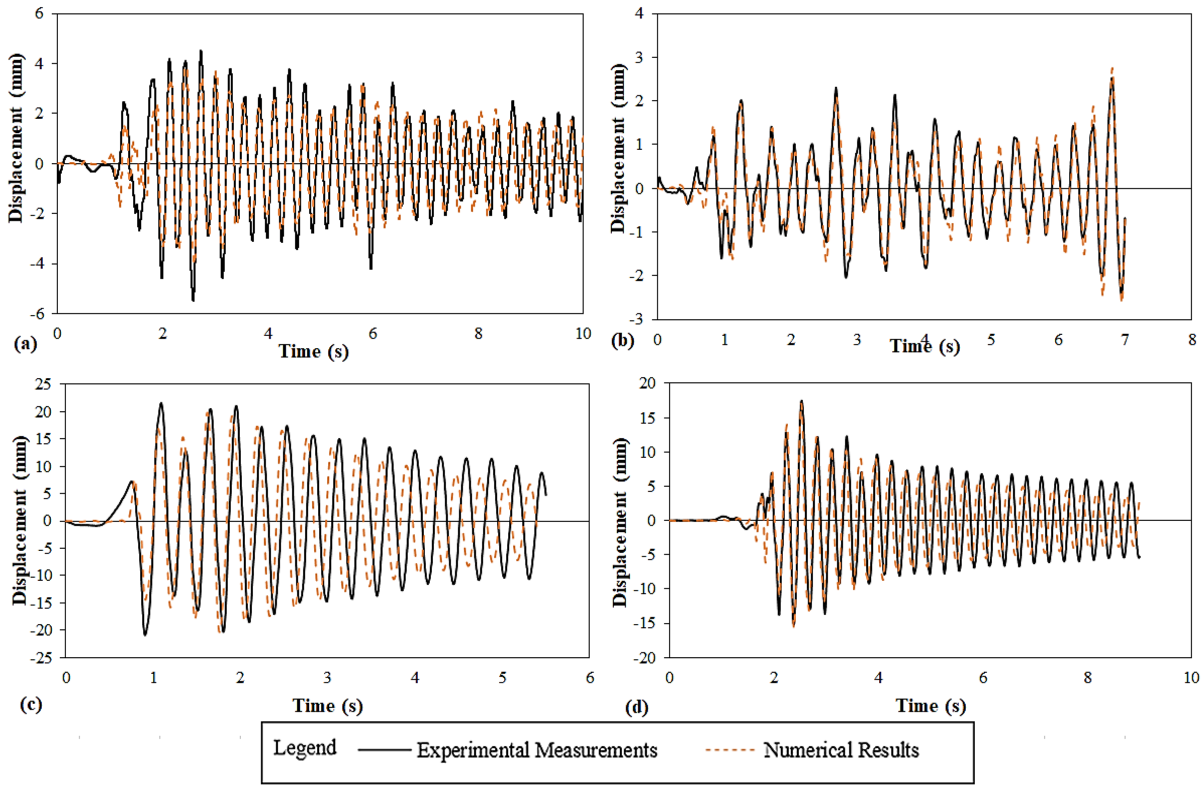


Fig. 13 Experimental and numerical relative displacement time histories for 10-Storey frame (fifth floor) under scaled; a) El Centro earthquake; b) Hachinohe earthquakes; c) Northridge earthquake; d) Kobe earthquake

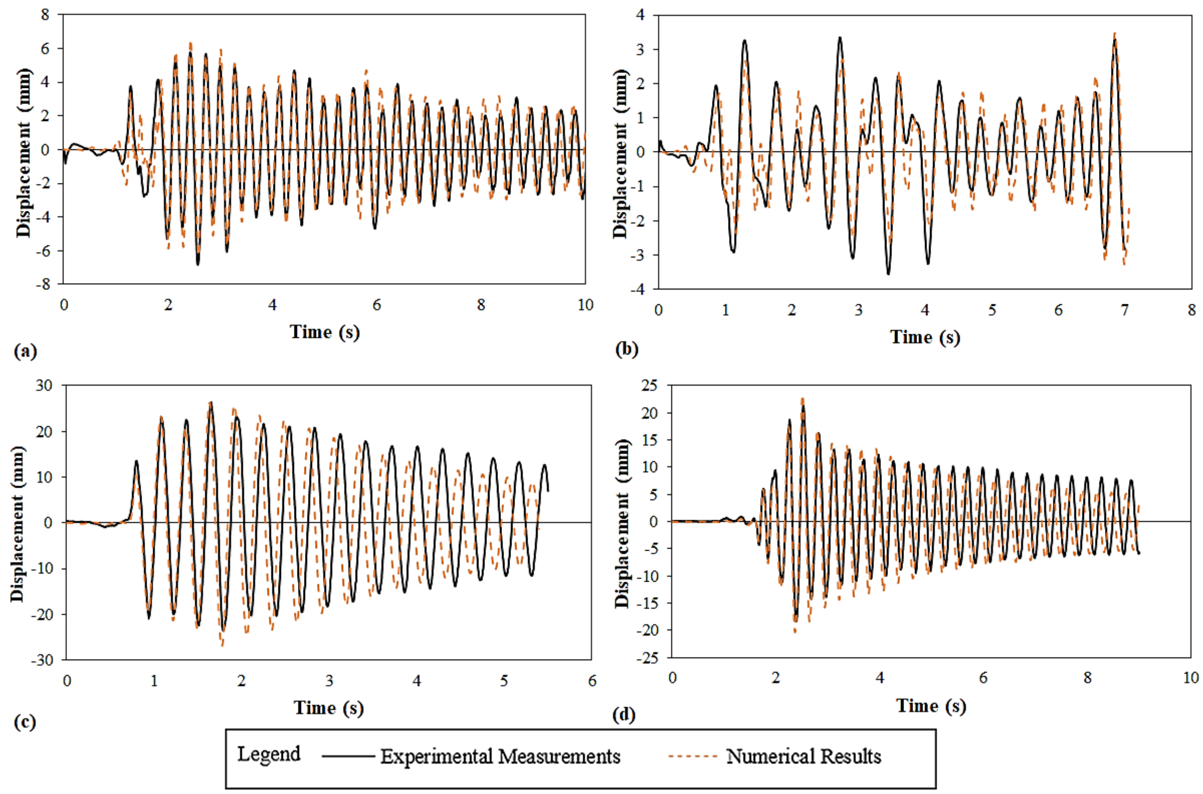


Fig. 14 Experimental and numerical relative displacement time histories for 10-Storey frame (tenth floor) under scaled; a) El Centro earthquake; b) Hachinohe earthquakes; c) Northridge earthquake; d) Kobe earthquake

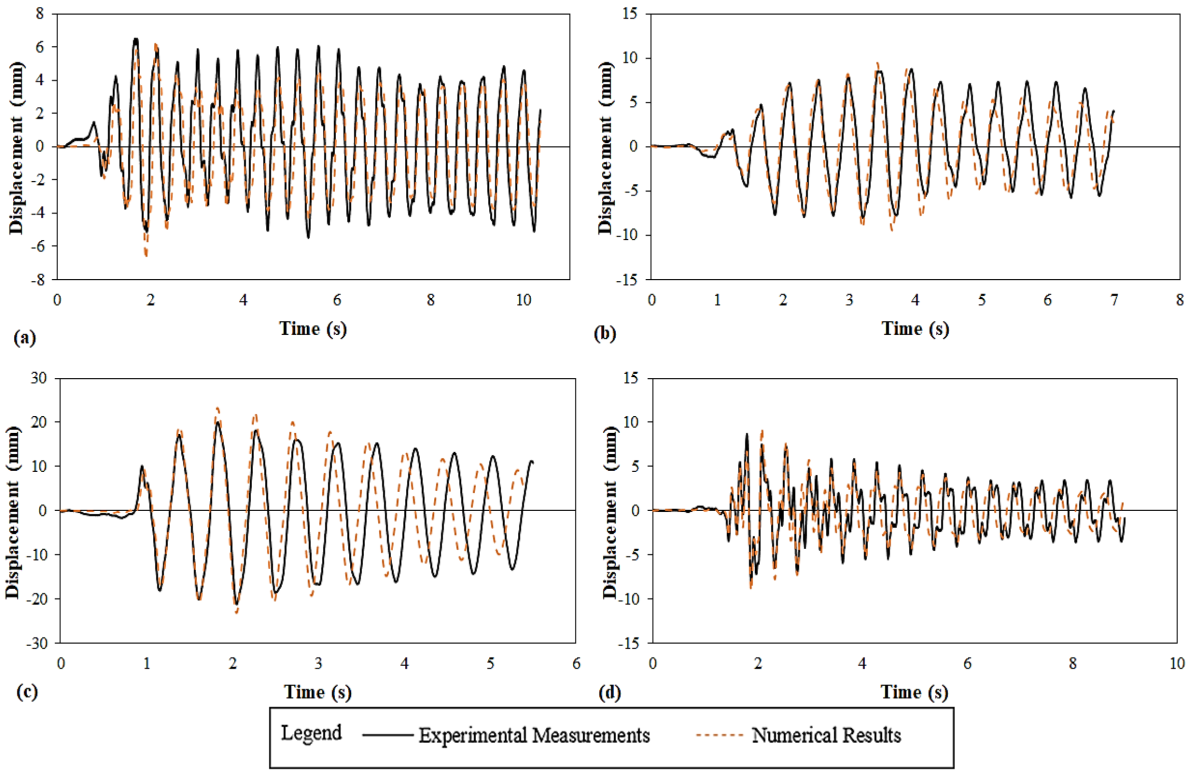


Fig. 15 Experimental and numerical relative displacement time histories for 15-Storey frame (fifth floor) under scaled; a) El Centro earthquake; b) Hachinohe earthquakes; c) Northridge earthquake; d) Kobe earthquake

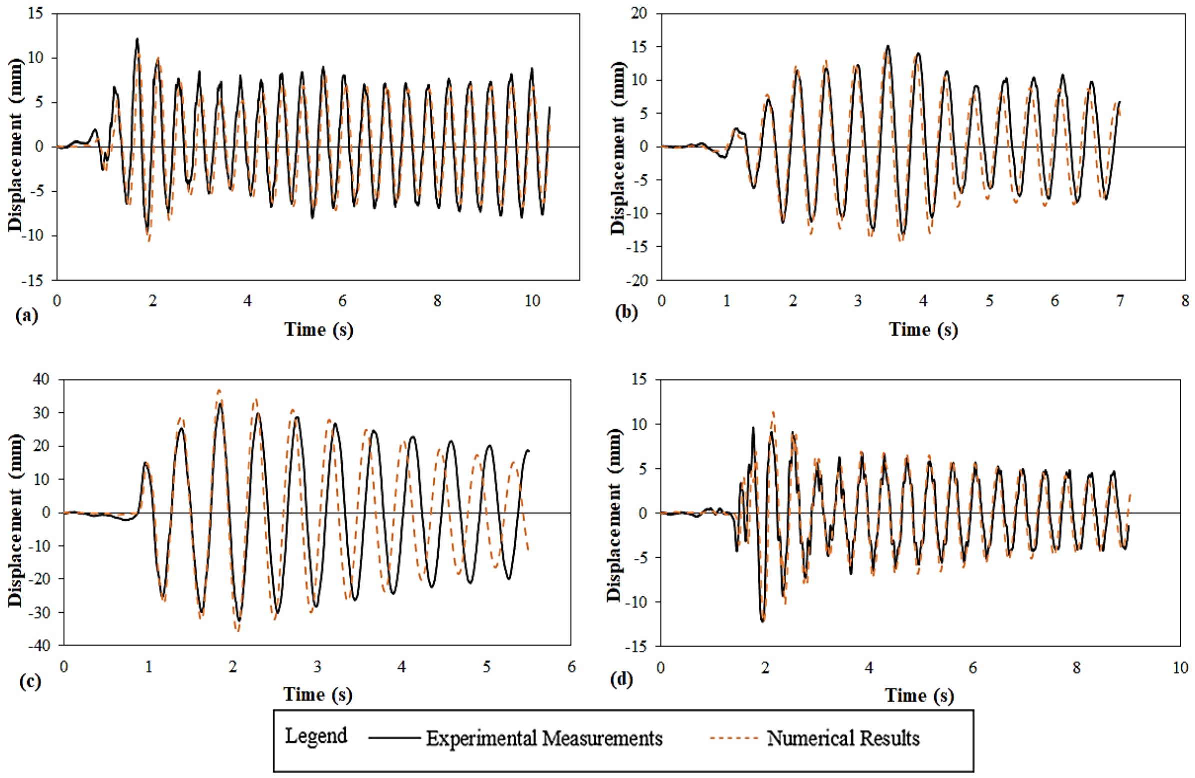


Fig. 16 Experimental and numerical relative displacement time histories for 15-Storey frame (10th floor) under scaled; a) El Centro earthquake; b) Hachinohe earthquakes; c) Northridge earthquake; d) Kobe earthquake

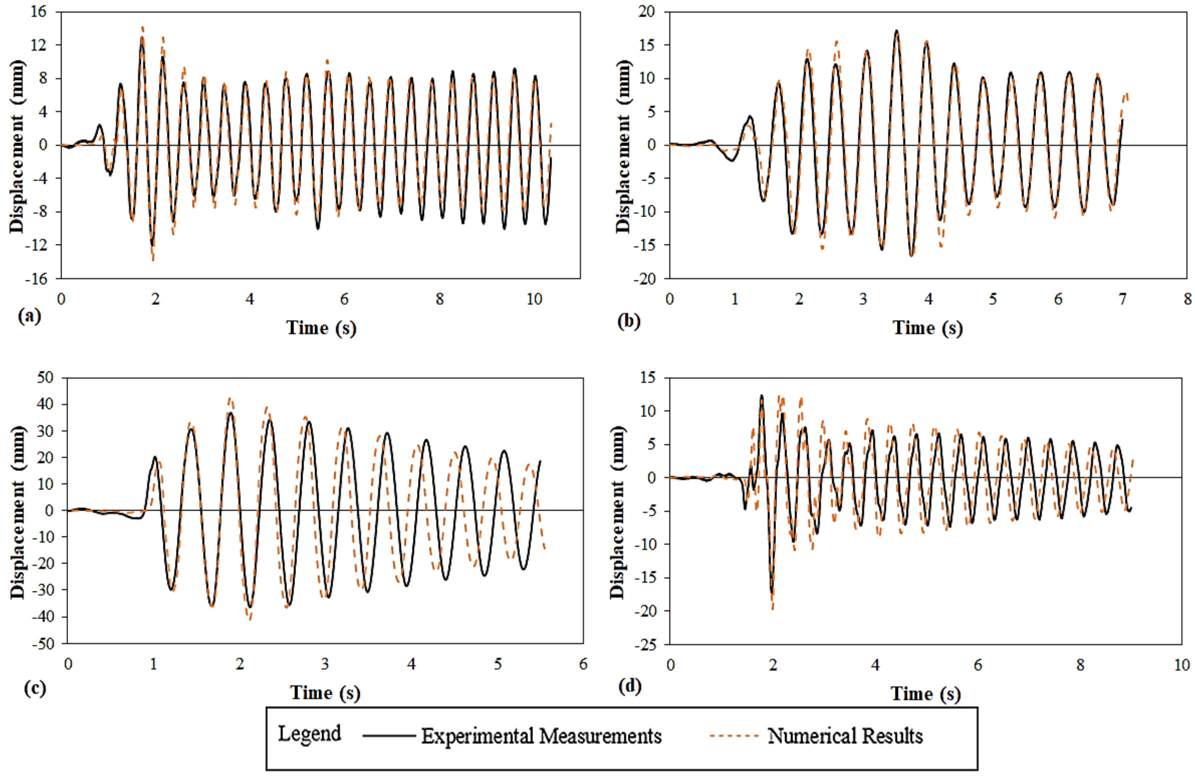


Fig. 17 Experimental and numerical relative displacement time histories for 15-Storey frame (15th floor) under scaled; a) El Centro earthquake; b) Hachinohe earthquakes; c) Northridge earthquake; d) Kobe earthquake

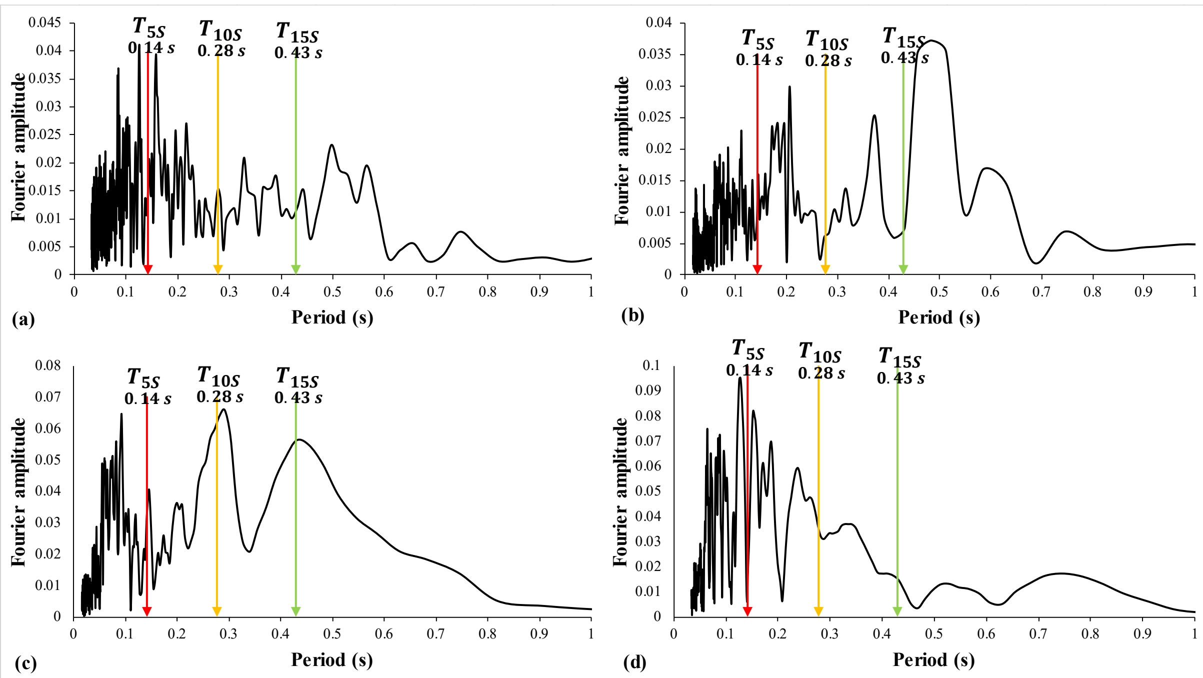


Fig. 18 Fundamental period of 5-storey, 10-storey, 15-storey frames and Fourier spectrum of ground motion of scaled a) El Centro earthquake; b) Hachinohe earthquake; c) Northridge earthquake; d) Kobe earthquake

4.1 Required Separation Distance to Avoid Structural Pounding

Lateral movement in adjacent buildings has been acknowledged as an important factor in earthquake-induced structural pounding issues. Lin & Weng (2002) generated a relationship between these two in which they assumed $u_a(t)$ and $u_b(t)$ as the lateral deflection and time histories of the building A and building B at the collision point are depicted in Fig. 19. On that, the least gap required S was expressed as:

$$S = max|u_a(t) - u_b(t)|_{T_D} \tag{8}$$

where T_D is the duration of vibration. Hence, the collision occurs once the deducted displacement of the two buildings from the gap value is greater than zero and is avoided once the value is negative. In other words, the minimum gap is the maximum value of the difference between $u_a(t)$ and $u_b(t)$. The minimum separation distance to avoid pounding between the 15-Storey and 10-Storey frames, 15-storey and 5-storey frames and 10-storey and 5-storey frames under the aforementioned scaled earthquakes are presented in Figs. 20-22.

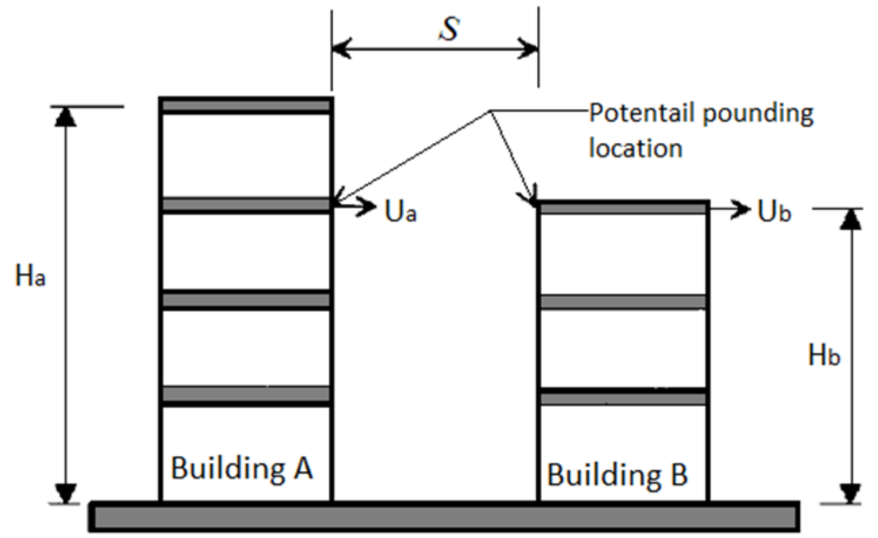


Fig. 19 Potential pounding location between adjacent buildings having different height

Numerical minimum separation distance to preclude pounding when 15-storey adjacent to 10-storey frames is 16 mm for El Centro (1940), 19 mm for Hachinohe (1968), 52 mm for Northridge (1994) and 30 mm for Kobe (1995). The minimum required separation gap in 15-Storey and 5-Storey coupled case was reduced by 44-49% being 9 mm, 10.5 mm, 25 mm, and 17 mm, respectively. However, in the coupled case of 10-Storey and 5-Storey, this number hardly changed for El Centro (1940), but reduced significantly for Hachinohe (1968), and remarkably increased in Northridge (1994) and Kobe (1995) earthquakes with 8.5 mm, 5 mm, 31 mm, and 24.5 mm, respectively. A comparison of these values with the actual experiment results are shown in Table 3.

In the experiment, each adjacent pair is kept close to one another for pounding to occur. The results showed that the pounding has finally occurred when the separation distance was less than 18 mm for the coupled case of the 15-storey and 10-storey while subjected to El Centro earthquake (1940), but reduced to less than 21 mm, 53 mm, and 29 mm while under the influence of Hachinohe (1968), Northridge (1994), and Kobe (1995), respectively. For the 15-storey and 5-storey coupled case, the pounding has occurred when the distance was less than 11 mm and 13 mm for El Centro (1940) and Hachinohe (1968), while 28 mm and 17 mm under the influence of Northridge (1994) and Kobe (1995) excitations, respectively. For the 10-storey and 5-storey case, though, the pounding occurred when the separation gap was less than 12 mm under El Centro earthquake (1940), reduces more than half of previous case with less than 6 mm for Hachinohe (1968), but remained unchanged with 28 mm for Northridge (1994) and increased to 26 mm for Kobe (995), respectively. All the experimental results for pounding and no-pounding cases have been recorded and listed in the references (Jaradat & Far 2021).

It is worth noted, the results of the present study are valid for the case of buildings responding elastically with different buildings' heights. Also, soil–structure interaction has not been taken into account assuming that the soil underneath the foundations is infinitely rigid.

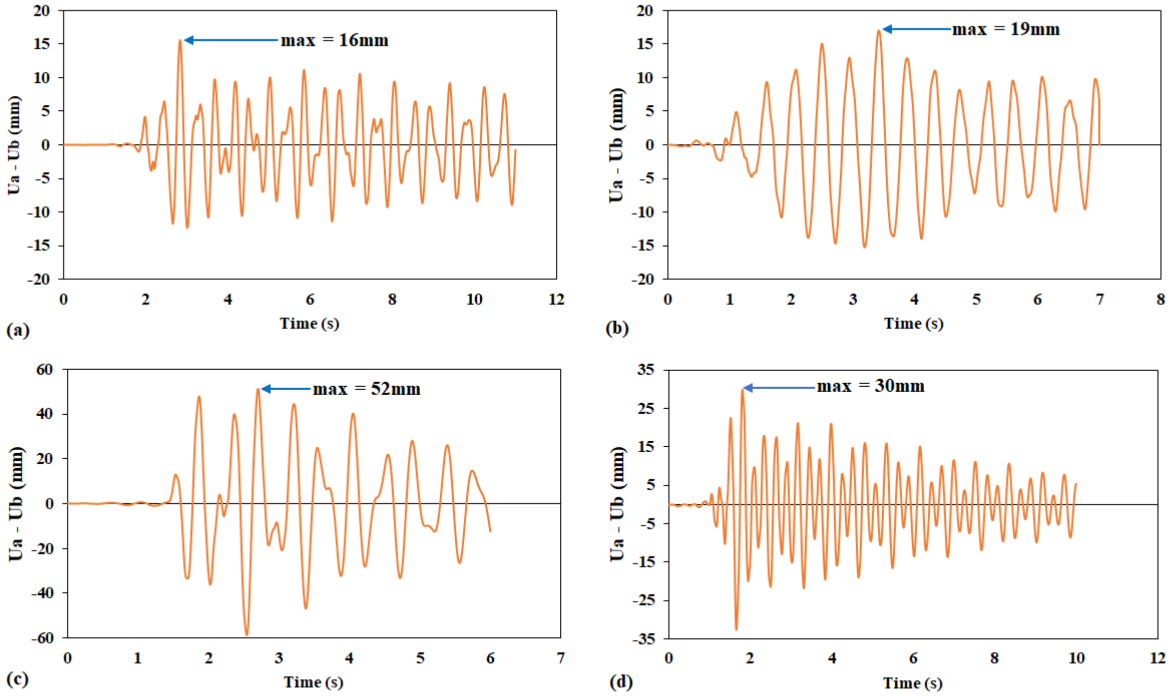


Fig. 20 Numerical minimum separation distance to avoid pounding between the coupled 15-storey and 10-storey under scaled; a) El Centro earthquake; b) Hachinohe earthquake; c) Northridge earthquake; d) Kobe earthquake

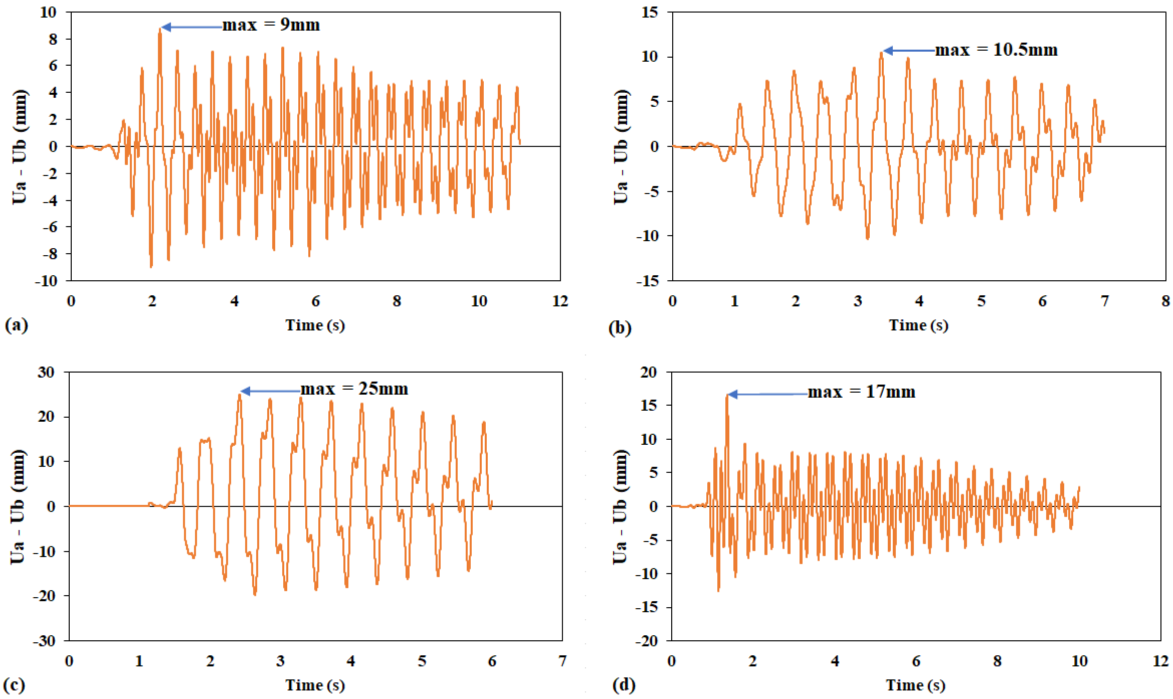


Fig. 21 Numerical minimum separation distance to avoid pounding between the coupled 15-storey and 5-storey under scaled; a) El Centro earthquake; b) Hachinohe earthquake; c) Northridge earthquake; d) Kobe earthquake

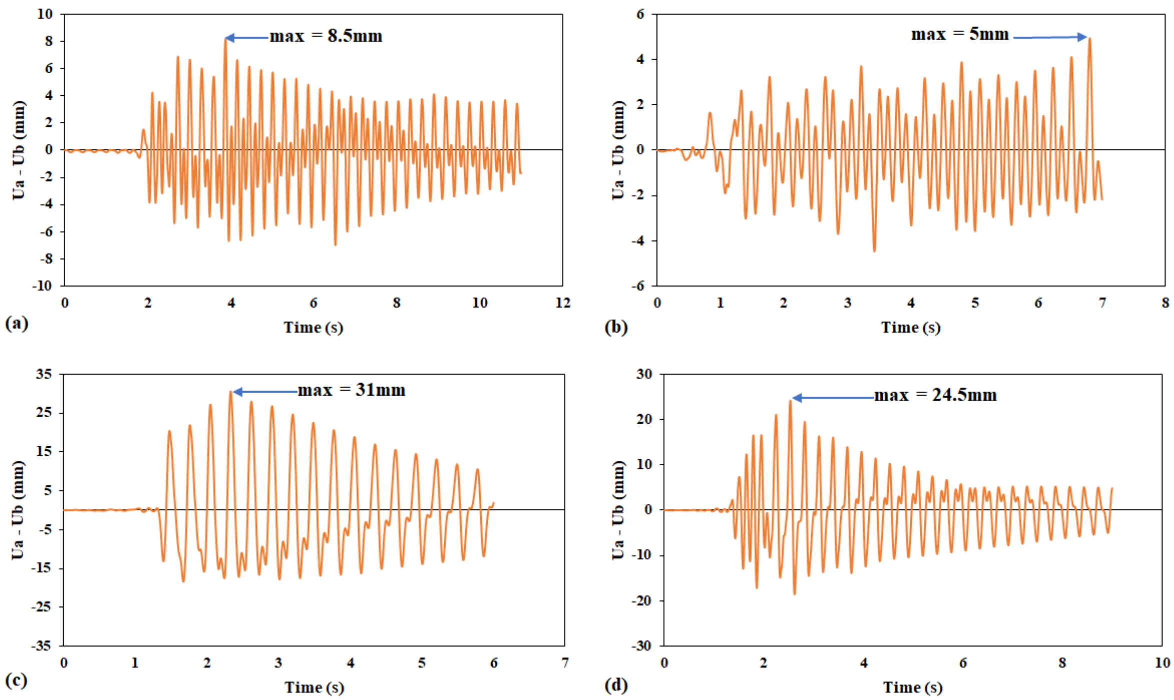


Fig. 22 Numerical minimum separation distance to avoid pounding between the coupled 10-storey and 5-storey under scaled; a) El Centro earthquake; b) Hachinohe earthquake; c) Northridge earthquake; d) Kobe earthquake

4.2 Comparison with code specifications

The Australian Standard, AS 1170.4-2007, requires that any adjacent buildings affiliated with category II design, with the height greater than 15m, and adjacent buildings associated with category III design, must be separated by 0.01H to prevent collision impact.

Considering two adjacent buildings, the required separation gap in this standard, is calculated as a setback distance from the second building. In the case of adjacent buildings with equal heights, the code still recommends 0.01H. With that though, uncertainty arises on which structure's height is to be used when these heights vary. Therefore, in practice and reality, the required separation distance can be determined by considering either height (Hao 2015). As a provision, this study considers the requirements of the Australian Standard AS1170.4 to calculate the required gap utilising both numerical and experimental methods. The results are then compared to verify the adequacy of the standard requirement, as depicted in Table 3.

As illustrated in Table 3, separation distance calculated using 1% of the taller adjacent structure always underestimates the required separation distance to avoid pounding under near-field earthquakes. It also underestimates the required separation distance to avoid pounding under far-field earthquakes except in two cases. In case one, the code required separation distance of 15 mm (1% of the 15-storey frame height) which is deemed adequate to preclude pounding between 15-storey and 5-storey frames under scaled El Centro and Hachinohe earthquakes. While, in case two the code required separation distance of 10 mm (1% of the 10-storey frame height) as adequate to preclude pounding between 10-storey and 5-storey frames under scaled Hachinohe earthquake.

However, the underestimation of gap values is true for both near-field and far-field if the shorter building is considered. These results indicate that the code specifications are inadequate if the shorter adjacent building is used to estimate the seismic gap under both near and far field earthquakes. Moreover, the specifications are also inadequate if the height of the taller building is utilised during near-field earthquakes. The standard specifications become adequate, in some cases, if the same building is contemplated during far-field earthquakes only.

Table 3 Experimental, numerical and Australian Standard seismic gap for all four scaled earthquakes, in mm

	El Centro				*Hachinohe*				*Northridge*				*Kobe*			
	Experiment	**Numerical**	**AS 1170.4** $0.01 \times H_a$	**AS 1170.4** $0.01 \times H_b$	**Experiment**	**Numerical**	**AS 1170.4** $0.01 \times H_a$	**AS 1170.4** $0.01 \times H_b$	**Experiment**	**Numerical**	**AS 1170.4** $0.01 \times H_a$	**AS 1170.4** $0.01 \times H_b$	**Experiment**	**Numerical**	**AS 1170.4** $0.01 \times H_a$	**AS 1170.4** $0.01 \times H_b$
15S adjacent 10S	18	16	15	10	21	19	15	10	53	52	15	10	29	30	15	10
15S adjacent 5S	11	9	15	5	13	10.5	15	5	28	25	15	5	17	17	15	5
10S adjacent 5S	12	8.5	10	5	6	5	10	5	28	31	10	5	26	24.5	10	5

Many building codes specify a minimum separation gap between two adjacent buildings in order to avoid collision during an earthquake. In the absolute sum (ABS) method, the square-root-of-sum-of-squares (SRSS) method and Australian Standard AS 1170.4-2007 have provided formulas to measure required separation distance, based on the maximum lateral displacement in Eqs. (1)-(2), and the height in Eq. 7, respectively. Table 4, compare the calculated separation distances using ABS along with SRSS methods by considering experimental relative displacements, subjected to the given excitations. ABS method appears to be the safest among all, but slightly exaggerating the final outcomes. Results given by the SRSS method are reasonably accurate as it is not as conservative as those given by the ABS method. These findings are consistent with what reported by Jeng et al. (1992), Kasai et al. (1996) and Lopez-Garcia & Soong (2009). It is worth mentioning that results obtained by the SRSS method are relatively similar to the experimental outputs This is only true for coupled case of 15-storey and 5-storey, when the natural frequency of both buildings are different, also described by Shrestha (2013).

Moreover, it appears that the Australian Standard has based the gap requirement on earthquakes with far-field condition because of the absence of active tectonic plates. Hence, the requirement should revolve around both structure and earthquake's characteristics.

Table 4 Gap distance results for all four scaled earthquakes, in mm, using experimental relative displacement

	El Centro			*Hachinohe*			*Northridge*			*Kobe*		
	Experiment	**ABS**	**SRSS**	**Experiment**	**ABS**	**SRSS**	**Experiment**	**ABS**	**SRSS**	**Experiment**	**ABS**	**SRSS**
15S adjacent 10S	18	19	13.9	21	19.8	15.9	53	59.1	42	29	33.6	24.6
15S adjacent 5S	11	11.4	8.1	13	11.9	9.3	28	30.2	23.1	17	23.1	16.8
10S adjacent 5S	12	10.4	7.3	6	6.6	4.7	28	30.6	23.4	26	31.9	22.7

As mentioned before increasing the separation gap is an expensive solution. To compromise between cost and safety there are other solutions that can be used as alternatives. These are highlighted as follows.

- The first technique is to utilise the collision shear walls and bracing systems (Anagnostopoulos & Karamaneas 2008; Barros & Khatami 2012).
- The second technique is the adoption of the soft material layers made of rubber for installation at the specific locations between adjacent buildings (Raheem 2013; Sołtysik et al. 2017).
- The third technique is to connect adjacent structures together with links (such as spring links, dashpot links or viscoelastic links) to produce in-phase vibrations (Jankowski & Mahmoud 2016; Richardson et al. 2013)

There are inherent advantages and disadvantages among the three techniques. Anagnostopoulos & Karamaneas (2008)) and Lopez-Garcia & Soong (2009) stated that, using shear walls decreases the top displacements and number of impacts but will increase the maximum impact force. Abdel Raheem (2014) and Sołtysik et al. (2017) suggested that filling the gap with rubber pad may reduce the peak impact force but will increase the number of poundings. Furthermore, Jankowski & Mahmoud (2016) and Richardson et al. (2013) stated that connecting the two adjacent structures together is beneficial to the flexible adjacent structure while increasing the responses of the stiffer building.

4.3 Impact effect of the separation gap between the adjacent structures

In order to study impact effect on the adjacent structures response in term of lateral displacement, the gap between the adjacent structures was considered based on AS 1170.4-2007. The separation distance between the 15-Storey and 10-Storey frames, 15-storey and 5-storey frames and 10-storey and 5-storey frames were 15 mm, 15 mm and 10 mm respectively. However, only the impact of a 15 story building on an adjacent 10 story building was selected for description, which tolerated the most intensive impact effect.

The envelopes of maximum lateral displacements measured in the coupled configuration of 15-storey & 10-storey excited by the above mentioned four scaled earthquakes are shown in Figure 23. Due to impact, peak lateral displacement of the 15 storey structure was slightly increased under scaled El Centro earthquake that maximum value was 10.29%. Similar observation was seen in the 10 storey structure. However, lower floors has shown a decrease in lateral displacement up to 4.76%, as shown in Fig. 23(a). The lateral displacement of the stories in the 15 story structure recorded under the scaled

Hachinohe accelerograph were decreased throughout the height, the maximum value was 20.73%. Differently, the lateral displacement due to pounding in the 10 storey structure substantially increased to 157.69%, as shown in Fig. 23(b). The response of the 15 storey and 10 storey structures due to pounding that occurred under scaled Northridge earthquake was decreased to 26.87% and 49.54% respectively, as shown in Fig. 23(c). This explain from Table 3, as the required separation gap for no pounding between the coupled 15 storey and 10 storey is more than 50 mm.

Fig. 23(d) depicts the lateral displacement response of the coupled 15 storey and 10 storey structures excited by the scaled Kobe earthquake. The lateral displacement in the 15 storey was increased throughout the height. Conversely, the lateral displacement in the 10 storey structure has decreased to maximum value of 21.74%.

The result shows that the response in term of lateral displacement due to impact will be decreased in both adjacent building only when both buildings fundamental period and the characteristic period of the ground motion are close as in Northridge case. Moreover, the results show that the peak lateral displacement response due to pounding in the shorter structure are generally less than those from the no-pounding case at most elevations.

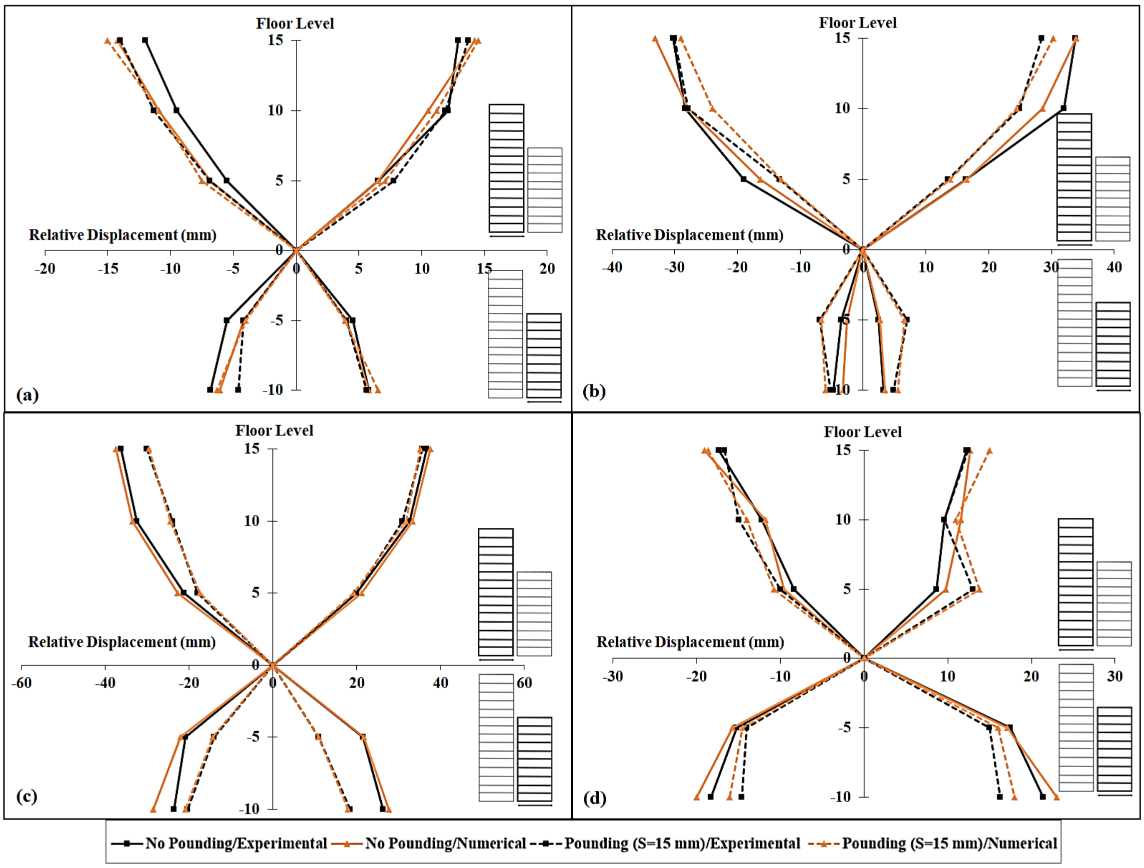


Fig. 23 Envelopes of lateral displacements for pounding between floor diaphragm of the coupled 15-storey and 10-storey under scaled earthquake; a) El Centro; b) Hachinohe; c) Northridge; d) Kobe

5. Conclusion

This study is focused in comparing the results of the separation gap using numerical and experimental approaches to the 0.01H requirement by AS1170.4. The objective was assess the degree of accuracy of the suggested expressions based on the specifications adhered to by the Standard.

Testing was conducted in an independent lab platform based on the records of past earthquakes on the scaled models to ascertain the lateral deflection and acceleration. Experimental data were measured using the accelerometers and laser displacement sensors. A full nonlinear time history dynamic analysis was performed on the scaled structural models to produce numerical results. From there, the absolute acceleration and relative displacement along with time histories were compared with the experimental measurements. Based on the numerical results and experimental measurements conducted in this study, it can be concluded that:

- The standard-based separation gap prescribed in AS 1170.4-2007 is inadequate when the shorter building height in a coupled case is considered under the given earthquake excitation. This requirement also becomes inadequate when the taller building height is utilised under a near-field earthquake.
- The adequacy of the standard-based separation gap prescribed in AS 1170.4-2007 returns, only if the height of the taller structure is contemplated under a far-field earthquake.
- ABS method is the most reliable method in determining the separation gap. However, it tends to overestimate the values.
- SRSS method produces more veracious results while overestimating the values at times.
- The SRSS method is conservative when the natural frequencies of both buildings are close to each other and reasonably accurate for determining the separation gap if the natural frequencies are different.
- The earthquake characteristics of near-filed and far-field earthquakes have significant impact on the gap requirements to prevent collision.
- Lateral displacement response due to pounding will be decreased in both adjacent buildings only when both buildings fundamental period and the dominant period of the ground motion are closely.
- Compared to the no-pounding state, building top floor pounding can decrease the lateral displacement over the entire building height, whereas pounding at building mid-height can increase the peak lateral displacement response over the entire building height.

It is also highly recommended that more extensive experimental studies are needed to evaluate the range of the model's parameters more accurately for various types of structures with different material and contact surface geometry properties. Several parameters can be considered in the future studies which have not been covered in this paper, e.g. P-delta effect, soil structure interaction, direction of incidence of earthquake, structural system..., etc. Further studies, taking these parameters into consideration, are recommended. This study did not consider the structural size variation; therefore, it is recommended to consider this issue in future studies.

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Appendix A

Construction details of the structural models

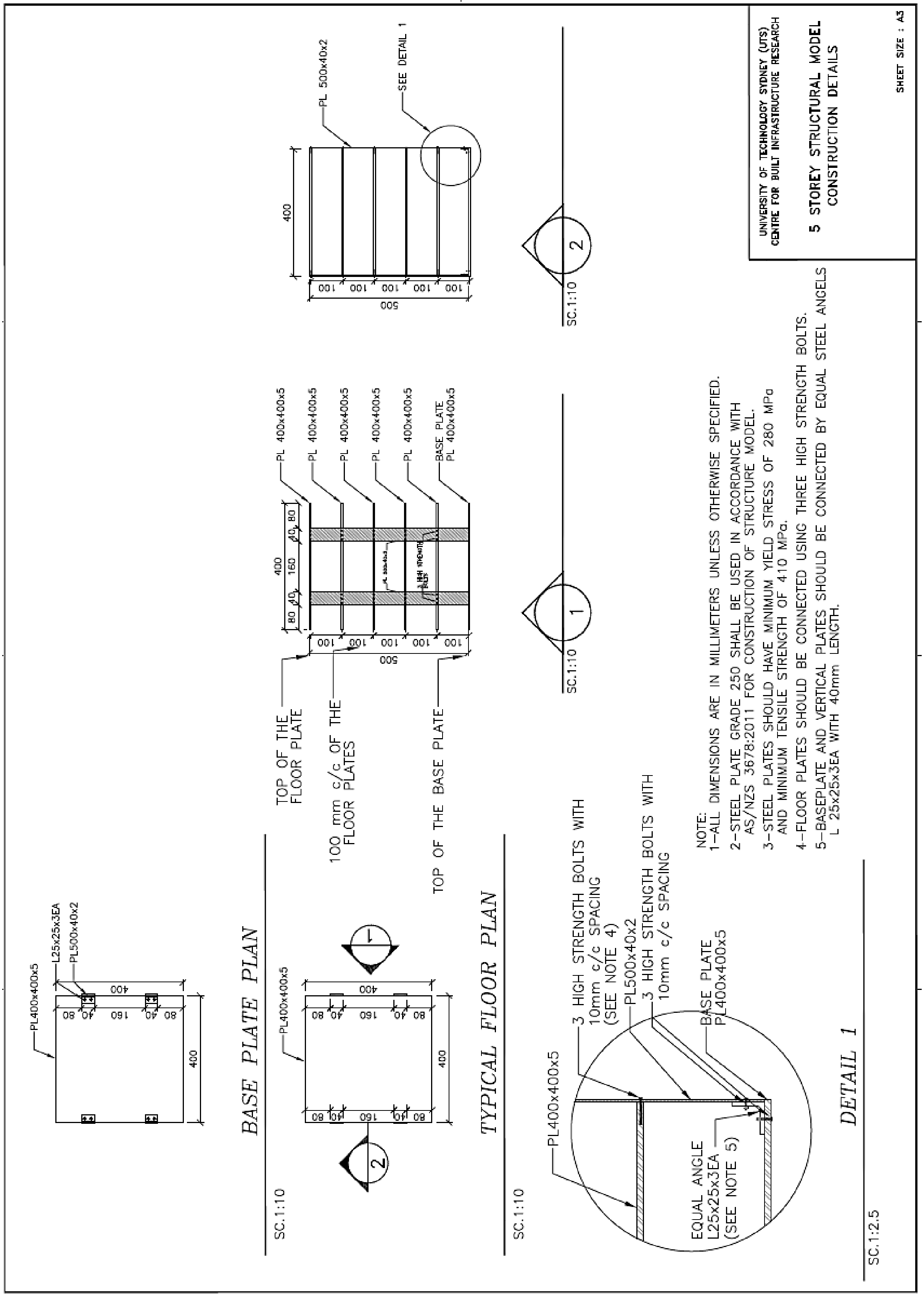

Fig. A.1 Construction detail drawings of the 5-storey structural model

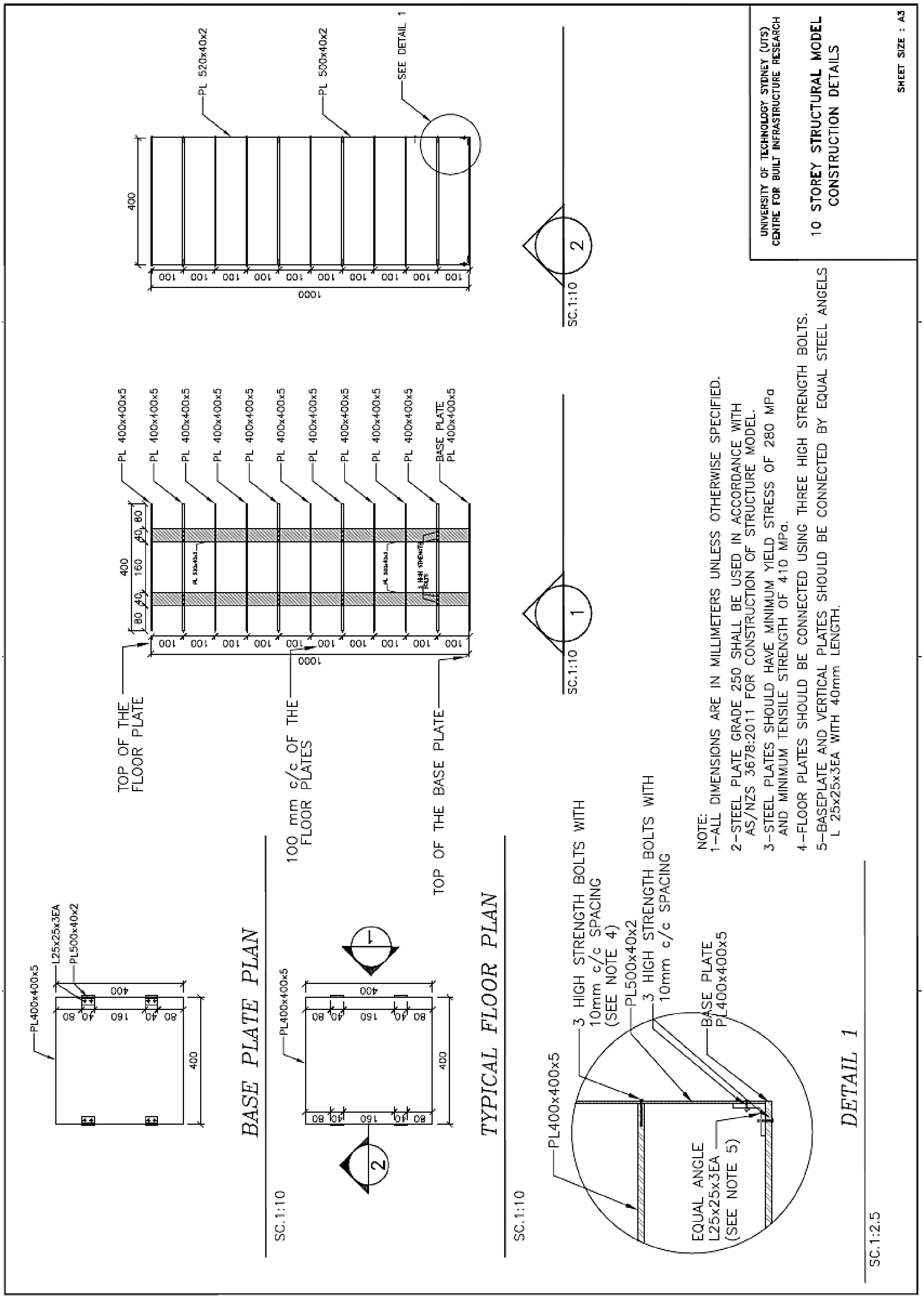

Fig. A.2 Construction detail drawings of the 10-storey structural model

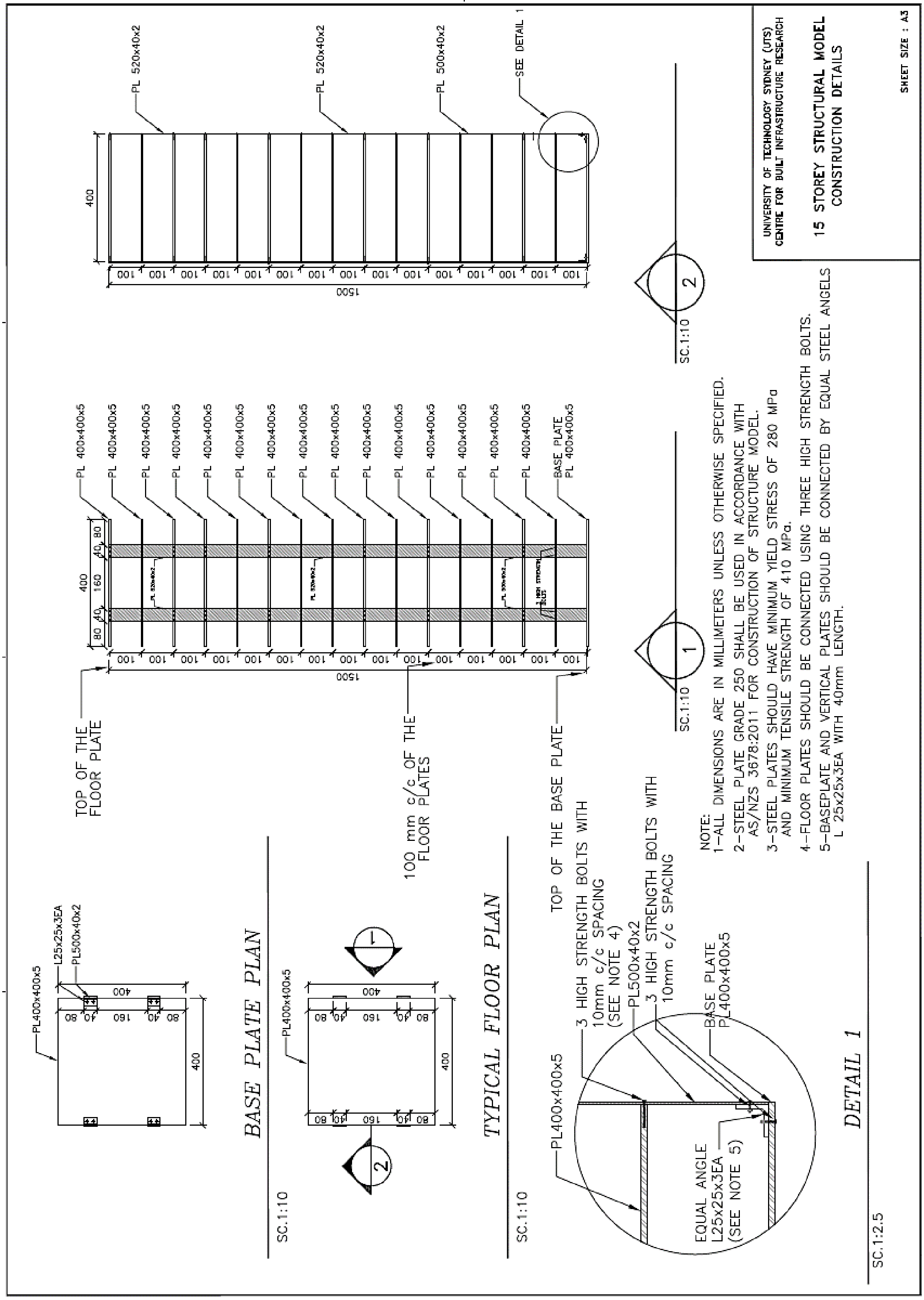

Fig. A.3 Construction detail drawings of the 15-storey structural model