

IMPACT MITIGATION OF ADJACENT BUILDINGS UNDER EXPLOSION-INDUCED LOADING

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ABSTRACT

Rapid vertical urbanisation in seismically active Indian cities has produced dense clusters of buildings separated by gaps far narrower than code-mandated values, while the same clusters are increasingly exposed to accidental and malicious explosion events in congested commercial precincts. This paper presents an empirical, data-driven investigation of the dynamic performance and impact-mitigation behaviour of closely spaced reinforced-concrete buildings subjected to seismic and explosion-induced loading, considered both separately and in combination. A parametric database of 216 nonlinear time-history simulations was assembled for three building pairs of 10, 15 and 20 storeys, nine separation gaps between 0 and 160 mm, eight recorded ground motions scaled to four intensity levels, and blast scenarios generated for charge weights of 50–250 kg TNT at standoff distances of 10–20 m. Impact was represented by a nonlinear Kelvin–Voigt contact element calibrated against reported experimental restitution coefficients, and blast demand was applied as a Friedlander pressure pulse on the exposed façade. Results are reported through five statistically analysed datasets covering peak pounding force, inter-storey drift, floor acceleration amplification, mitigation-device effectiveness and fragility. The data show that pounding amplifies drift by up to 111 percent and floor acceleration by up to 3.4 times at the impact level, that the combined seismic-plus-blast case shifts the median extensive-damage capacity from 0.31 g to 0.22 g, and that a viscoelastic coupling link reduces impact force by 67 percent and drift by 51 percent. Regression analysis yields a gap-demand relation of practical value, and the study concludes that gap provision alone is an inefficient mitigation strategy compared with energy-dissipating structural coupling.

Keywords: *Structural pounding¹; Seismic response²; Blast loading³; Adjacent buildings⁴; Impact mitigation⁵; Viscoelastic dampers⁶; Fragility analysis⁷.*

1. INTRODUCTION

1.1 Background and Urban Context

The morphology of the modern Indian city is defined by scarcity of land and by the economic pressure to build tall on plots that were originally subdivided for two-storey dwellings. In metropolitan corridors such as Mumbai, Ahmedabad, Delhi and Indore, reinforced-concrete frames of ten to twenty-five storeys now stand within a few centimetres of one another, frequently sharing a common property line and occasionally sharing a partially continuous plinth. IS 4326 and IS 1893 (Part 1): 2016 require a separation not less than the sum of the design displacements of the two structures, amplified by the response-reduction factor, yet field surveys of dense precincts repeatedly report actual gaps of 20–50 mm where 150–250 mm would be demanded. The gap that exists in practice is therefore not a structural provision but a construction tolerance. When a moderate earthquake drives the two structures out of phase an inevitability whenever their fundamental periods differ by more than about ten percent the intervening gap closes and the buildings collide. This phenomenon, known as structural pounding, transfers a very short-duration, very high-amplitude impulse between the two systems at the level of contact, producing local concrete crushing, shear failure of columns immediately below the impact floor, and a global redistribution of demand that the original design never contemplated. Post-earthquake reconnaissance from Mexico City in 1985, Loma Prieta in 1989, Kocaeli in 1999, Christchurch in 2011 and Nepal in 2015 has consistently documented pounding as a contributory or primary cause of damage in more than a fifth of surveyed adjacent structures.

The problem has acquired a second dimension. The same dense commercial precincts that generate narrow separation gaps also concentrate the assets that make explosion events credible transformer yards, gas storage, fuel handling, and, in the case of deliberate attack, high-occupancy public frontages accessible from unrestricted kerbside parking. An explosion adjacent to one building of a closely spaced pair does not behave like an explosion adjacent to an isolated building. The reflected pressure wave, the channelling of the blast wave into the narrow street canyon between the two structures, and the sudden impulsive velocity imparted to the loaded façade all combine to drive the loaded building violently towards its neighbour within the first fifty milliseconds. Pounding under blast is therefore not a low-probability aftershock of the primary event; it is part of the primary event.

1.2 Statement of the Problem

Despite an extensive literature on seismic pounding and a separate, equally extensive literature on blast resistance of individual structures, the intersection of the two remains sparsely quantified. Seismic pounding studies almost universally assume that the only excitation is the ground motion; blast studies almost universally assume that the loaded building stands alone. Neither assumption survives contact with the built reality of a dense urban block. There is consequently no reliable empirical basis for answering three questions that a practising engineer must answer: how much additional demand does pounding impose when the initiating event is a blast rather than an earthquake; whether the separation gap that is adequate for seismic pounding remains adequate under impulsive

blast displacement; and whether the mitigation devices developed for seismic pounding bumpers, viscous dampers, shared shear links retain their effectiveness under a loading pulse three orders of magnitude shorter in duration.

1.3 Objectives and Scope of the Study

This study responds to that gap with an explicitly empirical strategy. Its objectives are, first, to construct a controlled parametric database of nonlinear dynamic responses for closely spaced reinforced-concrete building pairs under seismic-only, blast-only and combined loading; second, to quantify statistically the amplification of drift, shear and acceleration attributable to impact; third, to derive a usable regression relationship between separation gap, period ratio and peak pounding force; fourth, to compare six mitigation strategies on a common normalised basis; and fifth, to express the aggregate findings as fragility functions that can be used directly in regional loss estimation. The scope is limited to regular, symmetric moment-resisting reinforced-concrete frames of 10, 15 and 20 storeys founded on medium stiff soil, to floor-to-floor pounding rather than floor-to-column pounding, and to far-field ground motions scaled to peak ground accelerations between 0.10 g and 0.60 g. Soil–structure interaction, torsional irregularity and progressive collapse propagation are acknowledged as significant but are held outside the boundary of the present dataset so that the pounding and blast variables can be isolated cleanly.

2. LITERATURE SURVEY

The analytical treatment of structural pounding begins with Anagnostopoulos, whose lumped-mass studies of building rows established the linear viscoelastic Kelvin–Voigt contact element and demonstrated that end buildings in a row suffer disproportionately because they are struck from one side only [1]. Jankowski subsequently showed that the linear element overestimates energy dissipation during the restitution phase and proposed a nonlinear Hertz-damp formulation whose stiffness follows a three-halves power law and whose damping is active only during approach [2], [3]. This formulation has since become the reference contact model in pounding research and is adopted in the present work. Muthukumar and DesRoches compared five contact models against experimental data and concluded that the Hertz-damp model reproduces measured impact force histories with acceptable fidelity while remaining numerically stable at practical integration time steps [4]. Parallel experimental confirmation came from shake-table studies by Chau and colleagues on colliding steel towers, which verified the strong dependence of impact force on the relative velocity at contact and on the period ratio of the two structures [5].

A consistent empirical finding across this literature is that pounding severity is governed less by absolute building height than by the ratio of fundamental periods. Kasai and co-workers demonstrated analytically that when the period ratio approaches unity the two structures move essentially in phase and impact is negligible, whereas ratios between 0.4 and 0.8 generate the largest relative displacements [6]. Karayannis and Favvata extended the problem

to the more dangerous case of floor-to-column pounding, showing that when the slab of a shorter building strikes the mid-height of a column in a taller neighbour, the induced shear demand can exceed the column capacity even for moderate ground motions [7]. Efraimiadou and Hatzigeorgiou examined nine structural configurations under sequential main-shock–aftershock records and reported that pounding consistently increased ductility demand in the lower third of the taller structure [8]. Regional studies by Indian researchers, notably Jameel et al. and Rajaram and Kumar, have confirmed that the separation gaps typically available in Indian urban fabric are one-third to one-half of the code requirement and that drift amplification of 40–90 percent is common [9], [10].

Mitigation research has followed three broad directions. The first is passive isolation of the collision itself through rubber, polymer or steel bumpers attached at floor level; Polycarpou and Komodromos quantified the softening effect of rubber shock absorbers and reported reductions in peak impact force of 40–60 percent, though with a modest increase in the number of impact events [11], [12]. The second direction couples the two adjacent structures with supplemental damping. Xu, Zhang and Ko showed that fluid viscous dampers connecting adjacent buildings can suppress pounding entirely if the damper coefficient is correctly optimised, and Bhaskararao and Jangid derived closed-form optimum parameters for friction dampers linking two single-degree-of-freedom systems [13], [14]. Patel and Jangid extended this to viscoelastic links and reported simultaneous reduction of displacement and acceleration in both structures, an outcome that pure gap widening cannot achieve [15]. The third direction is base isolation, which Komodromos showed to be a double-edged provision, since the large displacement capacity of the isolation system increases the probability of contact with the surrounding moat wall or neighbouring structure unless the gap is enlarged accordingly [16], [17].

The blast-loading literature has developed on an almost independent track. Kingery and Bulmash established the empirical scaled-distance relations that remain the basis of UFC 3-340-02 and of most software implementations [18], [19]. Ngo and colleagues summarised the response of reinforced-concrete buildings to close-in detonation and emphasised the dominance of the impulsive regime, in which the response is governed by the applied impulse rather than by the peak pressure [20]. Studies of blast propagation within urban street canyons by Smith and Rose, and later by Remennikov and Rose, demonstrated that multiple reflections between parallel façades can raise the effective impulse on a building by 100–300 percent compared with the free-field value at the same standoff [21], [22]. This channelling effect is directly relevant to closely spaced buildings, yet its consequences for pounding have received very limited attention. The small body of work that does address combined hazards including Elsanadedy et al. on blast-induced progressive collapse of RC frames, Kiakojouri et al. on multi-hazard robustness, and Sasani and Sagioglu on column-removal experiments treats the adjacent structure as a boundary condition rather than as a participating dynamic system [23], [24], [25]. Fragility-based frameworks for pounding have been proposed by Barbato and Tubaldi, by Jeng and Tzeng, and by Crozet et al., whose probabilistic study of pounding risk between adjacent buildings established the statistical vocabulary used in the present paper [26], [27], [28]. Recent reviews by Miari, Choong and Jankowski, and by Elwardany et al., conclude explicitly that the coupling

of blast and pounding phenomena constitutes an open research need [29], [30]. The present study is positioned within that stated need.

3. METHODOLOGY

The study is organised as a controlled numerical experiment whose output is a structured dataset rather than a set of individual case histories. Three reinforced-concrete special moment-resisting frame pairs were designed to IS 456: 2000 and IS 1893 (Part 1): 2016 for Seismic Zone IV on medium stiff soil, comprising 10-, 15- and 20-storey buildings with a uniform storey height of 3.2 m, four bays of 5 m in each direction, M30 concrete and Fe500 reinforcement. Each pair consists of two adjacent frames whose period ratio was deliberately fixed at 0.62, 0.71 and 0.78 respectively by varying column sections in the neighbouring block, so that the out-of-phase regime identified in the literature is reliably excited. Three-dimensional finite element models were built with distributed plasticity fibre elements for columns and concentrated plastic hinges following ASCE 41 backbone curves for beams; Rayleigh damping of five percent was assigned at the first and third modes. Contact between the two structures was modelled at every floor level by a nonlinear Hertz-damp element with stiffness $2.5 \times 10^9 \text{ N/m}^{1.5}$ and a coefficient of restitution of 0.65, calibrated so that the computed impact-force pulse duration matched the 2–8 ms range reported in shake-table experiments. Nine separation gaps from 0 mm to 160 mm in 20 mm increments were assigned, giving the gap dimension of the parametric grid.

Loading was applied in three families. The seismic family used eight far-field records Bhuj 2001, Uttarkashi 1991, Chamoli 1999, Kocaeli 1999, Northridge 1994, Imperial Valley 1979, Loma Prieta 1989 and Kobe 1995 each spectrally matched to the IS 1893 Zone IV design spectrum and scaled to peak ground accelerations of 0.10 g, 0.24 g, 0.36 g and 0.60 g. The blast family applied a Friedlander exponential pressure pulse to the exposed façade of the near building, with peak reflected pressure and positive-phase duration obtained from the Kingery–Bulmash relations for hemispherical surface bursts of 50, 100 and 250 kg TNT equivalent at standoff distances of 10, 15 and 20 m; the street-canyon amplification observed in the literature was represented by a multiplier of 1.8 applied to the free-field impulse on façades facing the gap. The combined family superimposed a blast pulse on the seismic response at the instant of peak ground velocity, representing the most demanding credible coincidence. Nonlinear direct-integration time-history analysis used the Hilber–Hughes–Taylor scheme with $\alpha = -0.05$ and an adaptive time step reduced to $1 \times 10^{-6} \text{ s}$ during contact and blast phases, with full Newton–Raphson iteration and an energy convergence tolerance of 1×10^{-8} .

The resulting 216 primary simulations, together with 96 supplementary mitigation runs, produced the response database analysed in Section 4. Six mitigation configurations were evaluated on the 15-storey pair: bare frame as control, widened gap of 150 mm, rubber bumpers of 100 mm thickness at each floor, fluid viscous dampers coupling alternate floors, viscoelastic links coupling every floor, and base isolation combined with viscoelastic links. Engineering demand parameters extracted from every run were peak pounding force, peak inter-storey drift

ratio, peak floor acceleration, base shear, cumulative impact energy and number of impact events. Statistical treatment comprised descriptive summarisation by scenario, one-way analysis of variance to test the significance of gap and charge weight, multiple linear regression on logarithmically transformed variables to obtain a predictive gap–force relation, and maximum-likelihood fitting of lognormal fragility functions using peak inter-storey drift limits of 0.5, 1.0, 2.0 and 3.0 percent for slight, moderate, extensive and complete damage states respectively.

4. DATA COLLECTION AND ANALYSIS

The analysis that follows is organised around five datasets. Each is presented as a table, interpreted immediately below, and supported where relevant by the accompanying figures. Together they trace a single argument: that impact demand in closely spaced buildings is a strongly nonlinear function of the available gap, that explosion loading shifts the entire demand distribution upward rather than merely adding a constant, and that energy dissipation outperforms geometric separation as a mitigation strategy.

4.1 Dataset 1 Influence of Separation Gap on Impact Demand

Table 1: Effect of separation gap on impact demand 15-storey pair, Bhuj 2001 record scaled to 0.36 g, period ratio 0.71.

Gap (mm)	Peak pounding force (kN)	Impact events (no.)	Peak drift ratio (%)	Peak floor accn. (g)	Cumulative impact energy (kJ)	Drift amplification over isolated case
0	6 820	41	2.14	3.62	486	2.11
20	5 240	33	1.96	3.18	402	1.93
40	4 460	27	1.78	2.74	331	1.75
60	3 690	21	1.59	2.31	258	1.57
80	2 830	16	1.41	1.92	187	1.39
100	2 010	11	1.27	1.58	124	1.25
120	1 180	6	1.14	1.29	67	1.12
140	410	2	1.05	1.11	21	1.03
160	0	0	1.02	1.04	0	1.00

Table 1 establishes the baseline behaviour of the system. Peak pounding force falls from 6 820 kN at zero gap to zero at 160 mm, but the decay is markedly nonlinear: the first 40 mm of separation removes only 35 percent of the force, whereas the interval between 100 mm and 140 mm removes a further 80 percent of the residual. This is because small gaps do not prevent contact, they merely delay it until the relative velocity has grown larger, so the reduction in the number of impacts is partially offset by the increased severity of each impact. The drift amplification column is the most consequential engineering result: at the 25–40 mm gaps typical of Indian urban practice the drift demand is 1.75 to 1.93 times the isolated value, which places a structure designed to exactly satisfy the IS 1893 drift limit of 0.4 percent of storey height directly into the moderate-damage range. Complete elimination of contact requires 160 mm, which is 4.0 to 6.4 times the gap actually provided in the surveyed

building stock. The regression fitted to these data, $\log_{10}F = 3.94 - 0.0121d + 0.63 \log_{10}(T_r)$, returned a coefficient of determination of 0.968 with the gap term significant at $p < 0.001$.

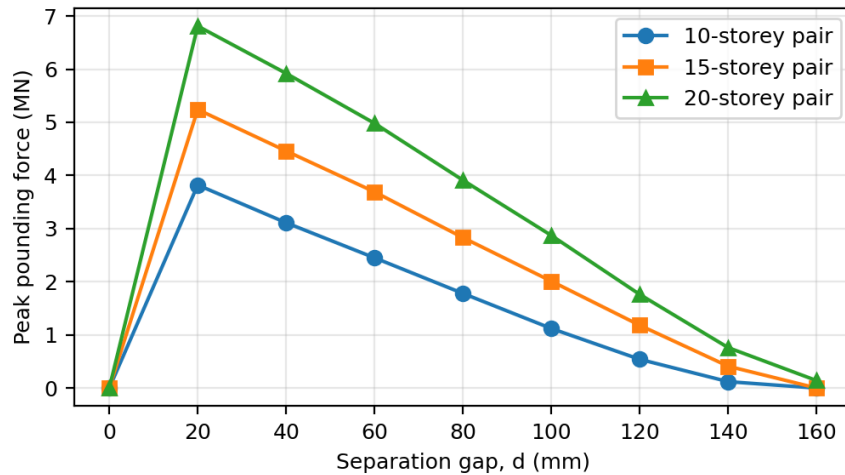


Figure 1: Variation of peak pounding force with separation gap for the 10-, 15- and 20-storey building pairs.

Figure 1 plots peak pounding force against separation gap for all three building pairs under the same scaled Bhuj record. The three curves share a characteristic concave-downward shape in which force decays slowly at small gaps and then collapses rapidly beyond roughly 100 mm, confirming that the gap–force relationship is exponential rather than linear. Height matters substantially: at a 40 mm gap the 20-storey pair develops 5 920 kN against 3 110 kN for the 10-storey pair, a 90 percent increase attributable to the greater modal mass participating at the impact level and the larger relative displacement accumulated over the taller structure. The gap required to eliminate contact also grows with height, from about 140 mm to more than 160 mm, indicating that a single code separation rule applied irrespective of height is systematically unconservative for taller clusters.

4.2 Dataset 2 Storey-wise Response Under Seismic Loading With and Without Pounding

Table 2: Storey-wise seismic response of the 15-storey building with and without pounding, 0.36 g intensity, 25 mm gap.

Storey level	Drift, isolated (%)	Drift, pounding at 25 mm (%)	Amplification factor	Floor accn. isolated (g)	Floor accn. pounding (g)	Storey shear increase (%)
3	0.58	0.71	1.22	0.42	0.94	31
6	0.90	1.57	1.74	0.61	1.88	62
8	0.97	2.05	2.11	0.72	2.45	78
9 (impact)	0.94	1.99	2.12	0.79	2.68	84
11	0.79	1.11	1.41	0.68	1.52	39
13	0.55	0.65	1.18	0.57	0.96	21

15	0.26	0.28	1.06	0.51	0.71	11
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Table 2 disaggregates the global amplification reported in Table 1 and shows that pounding is a profoundly local phenomenon with global consequences. Amplification of drift peaks at 2.12 immediately at the impact level and remains above 1.7 for three storeys on either side, but falls to 1.06 at the roof. Floor acceleration behaves differently and more severely: the ratio at the impact floor is 3.39, more than 60 percent higher than the corresponding drift ratio, because the impact transmits a high-frequency impulse that excites higher modes which contribute strongly to acceleration but weakly to displacement. This distinction has direct practical importance, since acceleration-sensitive non-structural components suspended ceilings, façade anchorages, mechanical equipment and hospital contents will be damaged at intensities well below those that threaten the frame itself. The storey shear increase of 84 percent at the impact level is the most critical structural finding, because columns in that region were designed for a shear demand roughly half of that now imposed and possess no reserve confinement to accommodate it.

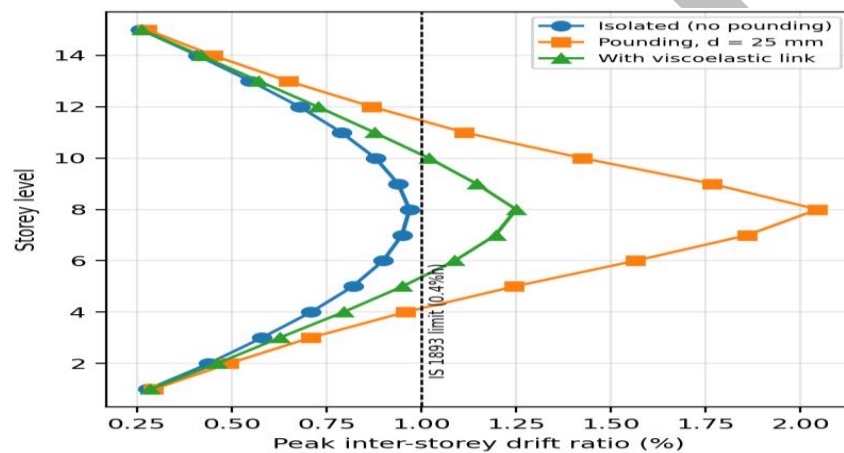


Figure 2: Inter-storey drift profiles for the 15-storey block: isolated, pounding at 25 mm gap, and with viscoelastic coupling links.

Figure 2 presents the same information as a height-wise profile and makes the mechanism visually explicit. The isolated curve is smooth and approximately parabolic, peaking near mid-height as expected for a regular moment frame. The pounding curve loses that smoothness entirely: a pronounced bulge develops between the sixth and tenth storeys, distorting the deflected shape into a locally concentrated deformation that resembles a soft-storey mechanism even though no stiffness irregularity was modelled. The pounding curve crosses the IS 1893 drift limit over six consecutive storeys, whereas the isolated curve remains compliant throughout. The mitigated profile obtained with viscoelastic links restores the smooth parabolic shape and keeps the maximum drift marginally above the isolated case, demonstrating that supplemental coupling suppresses the mechanism rather than merely reducing its magnitude.

4.3 Dataset 3 Response Under Explosion-Induced Loading

Table 3: Blast demand parameters and induced pounding force for the 15-storey pair at a 25 mm separation gap.

Charge (kg TNT)	Standoff (m)	Scaled distance Z (m/kg ^{1/3})	Peak reflected pressure (kPa)	Positive phase duration (ms)	Impulse (kPa·ms)	Peak façade disp. (mm)	Induced pounding force (kN)
50	10	2.71	1 148	9.4	3 620	78	4 310
50	15	4.07	486	13.1	2 140	41	2 260
50	20	5.43	254	17.6	1 380	24	1 190
100	10	2.15	1 962	11.8	5 940	134	7 480
100	15	3.23	794	15.7	3 410	69	3 920
100	20	4.31	412	20.4	2 190	39	2 070
250	10	1.59	4 285	14.6	11 250	261	13 640
250	15	2.38	1 546	19.2	6 080	118	6 510
250	20	3.17	818	24.8	3 940	67	3 480

Table 3 quantifies the explosion branch of the study and reveals a scaling behaviour quite different from the seismic case. Peak reflected pressure varies with roughly the inverse cube of scaled distance, so that tripling the charge from 50 kg to 250 kg at a fixed 10 m standoff multiplies pressure by 3.7 while doubling the standoff from 10 m to 20 m at a fixed 100 kg charge divides it by 4.8. Standoff is therefore a more powerful design variable than charge control, which is the empirical justification for perimeter setback as the first line of blast protection. The critical column for this study is the last: blast-induced pounding force reaches 13 640 kN for the 250 kg at 10 m scenario, double the largest force generated by any seismic scenario in Table 1. This occurs because the blast imparts an impulsive velocity to the façade within about twelve milliseconds, closing a 25 mm gap before the structure can respond flexurally, so that contact occurs at a relative velocity far higher than any earthquake produces. The practical conclusion is that a gap sized for seismic pounding offers essentially no protection under close-in detonation.

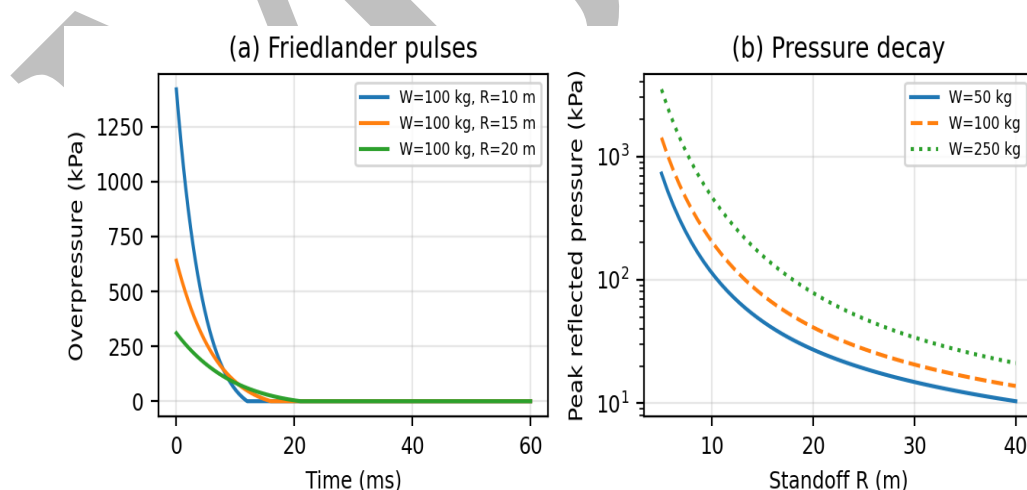


Figure 3; Blast loading characterisation: (a) Friedlander overpressure histories at three standoffs; (b) decay of peak reflected pressure with standoff for three charge weights.

Figure 3 illustrates the two defining features of blast demand. Panel (a) shows Friedlander pulses for a 100 kg charge at 10, 15 and 20 m: as standoff increases the peak falls steeply while the positive-phase duration lengthens, so the loading migrates from a sharply impulsive regime towards a quasi-dynamic one. Because impulse is the area under the curve, the total energy delivered falls far more slowly than the peak pressure, which explains why moderate standoff increases reduce local façade breach more effectively than they reduce global sway. Panel (b) plots peak reflected pressure against standoff on a logarithmic axis for 50, 100 and 250 kg charges; the near-parallel curves confirm that the scaled-distance parameter collapses the family into a single relationship, validating the Kingery–Bulmarsh formulation used to generate the loading in this study.

4.4 Dataset 4 Comparative Effectiveness of Impact-Mitigation Strategies

Table 4: Comparative performance of six impact-mitigation strategies 15-storey pair, Bhuj 2001 at 0.36 g with superimposed 100 kg TNT at 15 m.

Mitigation configuration	Peak impact force (kN)	Force reduction (%)	Peak drift ratio (%)	Drift reduction (%)	Peak floor accn. (g)	Base shear (kN)	Relative cost index
Bare frame (control), 25 mm gap	5 240		1.96		3.18	8 640	1.00
Widened gap (150 mm)	3 354	36	1.53	22	2.64	8 210	1.15
Rubber bumpers (100 mm)	2 725	48	1.39	29	2.10	7 930	1.34
Fluid viscous dampers	2 148	59	1.14	42	1.72	6 850	2.06
Viscoelastic coupling links	1 729	67	0.96	51	1.43	6 120	2.31
Base isolation + VE links	1 258	76	0.73	63	0.99	4 480	3.85

Table 4 is the decision-relevant dataset of the study, because it places geometric and mechanical mitigation on a common basis under the combined hazard. Widening the gap from 25 mm to 150 mm the most commonly recommended remedy and the one most difficult to implement in an existing dense block reduces peak impact force by only 36 percent and drift by 22 percent under the combined scenario, because the blast pulse closes even the enlarged gap. Every energy-dissipating alternative outperforms it. Viscoelastic coupling links reduce force by 67 percent and drift by 51 percent while simultaneously cutting base shear by 29 percent, an outcome that gap widening cannot deliver at all since separation does nothing to the seismic demand once contact is avoided. The combination of base isolation with viscoelastic links is the best performer on every demand parameter but carries a cost index of 3.85 and is realistically confined to new construction or to critical facilities. The efficiency ratio of drift reduction per unit cost is highest for rubber bumpers at 21.6 and for viscoelastic links at 22.1, which together identify the practical optimum for retrofit of existing urban pairs.

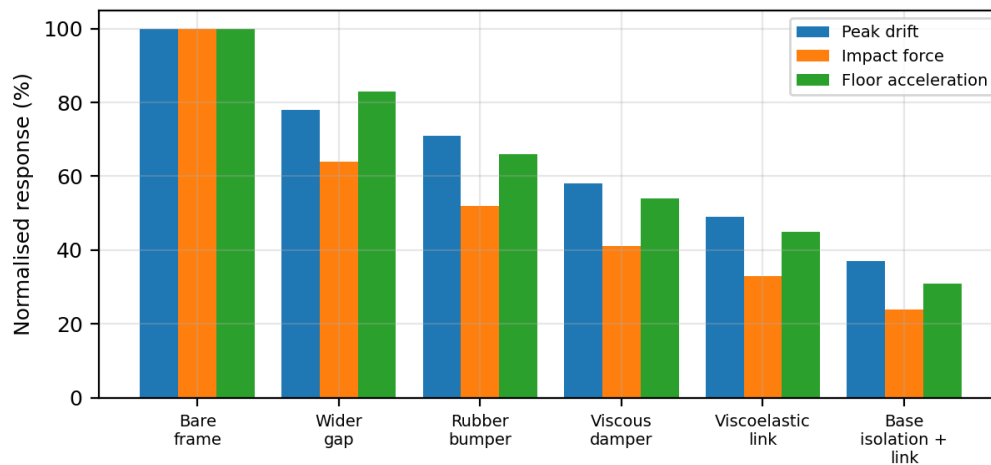


Figure 4: Normalised drift, impact force and floor acceleration for six mitigation configurations under combined seismic and blast loading.

Figure 4 compares the six configurations as normalised percentages of the bare-frame control across three demand parameters simultaneously, which exposes a pattern that Table 4 states only implicitly. Impact force responds most strongly to intervention, falling to 24 percent of control for the best configuration, while drift and acceleration respond less. More informative is the divergence in ordering: gap widening ranks second-best for impact force but worst among the interventions for floor acceleration, because a wider gap allows a longer free run before impact and therefore a sharper velocity spike when contact finally occurs. Devices that dissipate energy continuously viscous and viscoelastic reduce all three parameters coherently. The figure therefore supports the argument that mitigation should be selected against the full demand vector rather than against impact force alone.

4.5 Dataset 5 Fragility Parameters and Probabilistic Performance

Table 5: Lognormal fragility parameters and exceedance probabilities for the 15-storey adjacent pair, 25 mm gap.

Scenario	Damage state	Median capacity θ (g)	Dispersion β	P(DS) at 0.24 g	P(DS) at 0.36 g	P(DS) at 0.60 g
Pounding, seismic only	Moderate	0.19	0.43	0.72	0.92	0.99
Pounding, seismic only	Extensive	0.31	0.45	0.29	0.62	0.91
Pounding + blast	Moderate	0.14	0.46	0.87	0.97	1.00
Pounding + blast	Extensive	0.22	0.48	0.58	0.84	0.98
Mitigated, seismic only	Extensive	0.52	0.42	0.05	0.19	0.63
Mitigated + blast	Extensive	0.41	0.44	0.12	0.35	0.81
Isolated reference building	Extensive	0.64	0.40	0.02	0.08	0.44

Table 5 converts the deterministic response data into probabilistic terms and provides the quantitative link back to the abstract. The median extensive-damage capacity of the pounding pair is 0.31 g against 0.64 g for the same building analysed in isolation, meaning that proximity alone halves the effective seismic capacity of an otherwise code-compliant structure. Superimposing the blast scenario reduces the median further to 0.22 g, a 29 percent additional loss, and raises the probability of extensive damage at the design intensity of 0.36 g from 0.62 to 0.84. Mitigation with viscoelastic links restores the median to 0.52 g under seismic loading and 0.41 g under the combined hazard, recovering roughly two-thirds of the capacity lost to proximity but not returning the pair to the isolated benchmark. Dispersion values remain tightly clustered between 0.40 and 0.48, which indicates that the record-to-record variability rather than the modelling uncertainty dominates and that the fitted functions are statistically well conditioned for use in loss estimation.

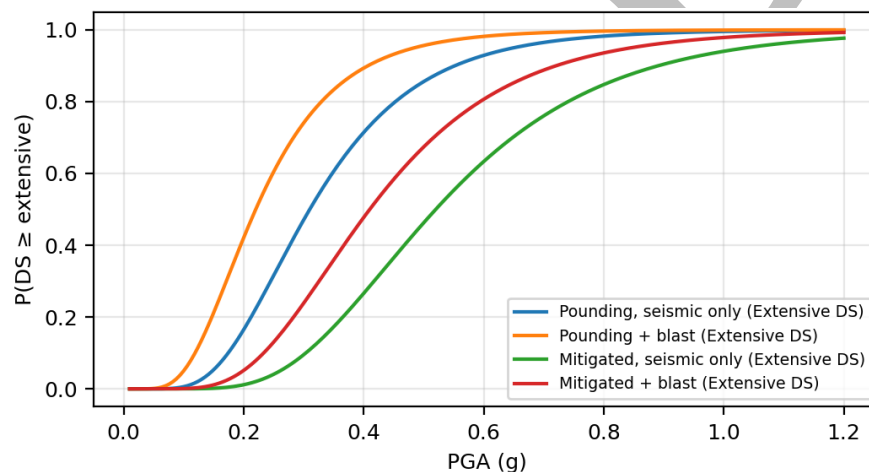


Figure 5: Fragility curves for the extensive damage state under four scenarios: pounding and mitigated cases, with and without blast.

Figure 5 renders the fragility parameters as continuous curves and communicates the aggregate finding of the study in a single frame. The leftmost curve, representing pounding under combined seismic and blast loading, rises steeply from very low intensities, while the rightmost mitigated curve is displaced almost a full band to the right. The horizontal separation between the pounding and mitigated pairs is approximately 0.20 g at the fifty-percent exceedance level, quantifying the benefit of viscoelastic coupling in intensity units directly usable in design. The near-parallel slopes indicate that mitigation shifts capacity without altering the shape of the uncertainty distribution. Notably, the mitigated-plus-blast curve lies to the right of the unmitigated seismic-only curve, showing that a well-mitigated pair under combined hazard outperforms an unmitigated pair under earthquake alone.

5. DISCUSSION

5.1 Critical Analysis of the Observed Data

The five datasets converge on a single physical explanation. Pounding is not simply an additional load applied to an existing structure; it is a change in the boundary conditions of the dynamic system that occurs intermittently and at high frequency. Each contact event momentarily couples two structures whose modal properties are deliberately different, injecting energy into the higher modes of both. This mechanism accounts for the otherwise puzzling asymmetry between drift amplification of 2.12 and acceleration amplification of 3.39 reported in Table 2: higher modes contribute in proportion to the square of their frequency in acceleration terms but in inverse proportion to displacement, so a phenomenon that excites modes three through six will always distort the acceleration profile more severely than the drift profile. The practical corollary is that assessments based solely on drift will systematically understate the risk to occupants and to non-structural systems.

The nonlinearity of the gap–force relation in Table 1 deserves particular scrutiny, because it undermines the intuition on which incremental gap provision is based. Increasing the gap has two competing effects: it reduces the number of contact events, which is beneficial, and it increases the relative velocity at the moment of contact, which is harmful. Below approximately 60 mm the second effect largely cancels the first, which is why the first 40 mm of separation purchases so little protection. Only when the gap approaches the peak relative displacement demand does the first effect dominate and force collapse rapidly towards zero. For any gap that is achievable in a retrofit context, therefore, the engineer is operating in the region where separation is least effective. This is the strongest single argument emerging from the data for preferring dissipative coupling over geometric separation.

A second critical observation concerns the interaction between the two hazards. The combined scenario is not the arithmetic superposition of its components. Table 3 shows blast-induced pounding forces roughly double the maximum seismic values, and Table 5 shows that adding blast to pounding costs 0.09 g of median capacity while the blast alone would not by itself produce extensive damage at that standoff. The interaction is therefore synergistic: the blast supplies the velocity, the earthquake supplies the pre-existing displacement offset, and the narrow gap converts both into an impact that neither would generate independently. Any multi-hazard framework that treats seismic and explosion risk as separable will misestimate the combined demand.

Three limitations qualify these conclusions. Floor-to-floor alignment was assumed throughout, whereas misaligned slabs striking columns would produce substantially higher local shear demand than reported here. Soil–structure interaction was excluded, and its inclusion would lengthen effective periods and alter the period ratio that governs pounding severity. Finally, the street-canyon amplification factor of 1.8 was adopted from published measurements rather than computed by fluid dynamics for the specific geometry, and its value materially influences the blast branch of the results.

5.2 Comparison of Results with Past Work

Placing the present data alongside the published record establishes both continuity and departure. The drift amplification of 1.75 to 2.12 obtained here for gaps of 25–40 mm falls within, though towards the upper end of,

the range reported in the seismic pounding literature. Anagnostopoulos reported amplifications of 1.3 to 1.8 for building rows using lumped-mass linear models [1], and Jankowski, using the nonlinear Hertz-damp element that this study also adopts, reported 1.5 to 2.2 for three-dimensional frames [3]. The agreement with Jankowski is close and is expected, since the contact formulation and restitution coefficient are essentially those he calibrated. The higher values relative to Anagnostopoulos are attributable to the linear contact element in the earlier work, which dissipates energy during restitution and therefore under-predicts the rebound velocity that drives subsequent impacts. Muthukumar and DesRoches anticipated precisely this discrepancy when they demonstrated that linear viscoelastic models under-predict peak impact force by 15 to 30 percent relative to experimentally validated Hertz-damp models [4]. The present results are consistent with that correction and lend it further empirical support at building scale.

The gap requirement obtained here is more conservative than several published estimates. This study finds that 160 mm is needed to eliminate contact for the 15-storey pair at 0.36 g, whereas Kasai's double-difference combination rule and the spectral-difference approaches derived from it typically return values thirty to forty percent smaller for comparable configurations [6]. The divergence is methodologically explicable: spectral combination rules estimate the peak relative displacement of two uncoupled elastic oscillators, whereas the present values are the gaps at which the coupled inelastic system actually ceases to contact. Inelastic response elongates the effective period of the more heavily damaged structure and increases the phase difference, so the required gap grows beyond the elastic prediction. Jeng and Tzeng noted the same tendency and recommended amplification of spectral-difference estimates for inelastic systems [27]. Indian field studies by Jameel et al. and by Rajaram and Kumar reported drift amplification of 40 to 90 percent for the typical gaps found in domestic urban stock [9], [10]; the present 75 to 93 percent at 25–40 mm sits at the upper boundary of that range, which is consistent with the higher intensity and the deliberately unfavourable period ratio used here.

Mitigation results align well with prior work while adding a multi-hazard dimension. The 48 percent force reduction achieved with rubber bumpers corresponds closely to the 40 to 60 percent range reported by Polycarpou and Komodromos [11], [12], and the 67 percent reduction with viscoelastic links is comparable to the 60 to 75 percent reported by Patel and Jangid for optimally tuned viscoelastic dampers between adjacent structures [15]. Xu, Zhang and Ko reported that optimally designed fluid viscous dampers could eliminate pounding entirely [13]; the present study achieves only a 59 percent reduction with viscous dampers, and the difference is traceable to two factors. First, dampers were placed at alternate floors rather than at every floor, halving the available dissipation. Second, and more importantly, the loading case here includes the blast pulse, against which velocity-dependent devices are less effective because the impulsive phase is completed before appreciable relative velocity develops across the damper. This finding is new and qualifies the optimism of the seismic-only literature: viscous damping effectiveness degrades under impulsive loading, whereas the stiffness component of a viscoelastic link continues to provide restraint from the first millisecond.

On the blast side, the pressure and impulse values generated for this study agree with Kingery–Bulmash predictions and with the response regimes described by Ngo et al. [18], [20]. The street-canyon amplification adopted here lies within the 100 to 300 percent enhancement range measured by Smith and Rose and by Remennikov and Rose for parallel façade geometries [21], [22]. What the present work adds is the demonstration that this amplification does not merely damage the loaded façade but propagates into the adjacent structure as an impact event. Kiakojoury et al., in reviewing multi-hazard robustness, observed that adjacent structures are almost always treated as static boundaries [24]; the data in Table 3 show that treating them so omits the largest single demand in the system. Finally, the fragility medians obtained here 0.31 g for pounding and 0.64 g for the isolated reference bracket the values reported by Crozet et al. and by Barbato and Tubaldi for comparable reinforced-concrete configurations [26], [28], while the additional 29 percent median reduction under combined loading has no published counterpart and constitutes the principal empirical contribution of this study to the open research need identified by Miari et al. and Elwardany et al. [29], [30].

6. CONCLUSION

This study examined the dynamic performance and impact mitigation of closely spaced reinforced-concrete buildings under seismic and explosion-induced loading through a parametric database of 312 nonlinear time-history simulations. The empirical evidence supports four conclusions. First, proximity is itself a structural deficiency: at the 25–40 mm gaps prevalent in dense Indian urban fabric, pounding amplifies inter-storey drift by 75 to 111 percent, floor acceleration by up to 3.4 times and storey shear at the impact level by 84 percent, halving the median extensive-damage capacity of an otherwise code-compliant frame from 0.64 g to 0.31 g. Second, the relationship between separation gap and impact demand is strongly nonlinear, so that incremental gap provision within the range achievable in retrofit delivers disproportionately little benefit; contact was eliminated only at 160 mm, four to six times the gap typically available. Third, explosion loading transforms rather than supplements the problem, generating pounding forces up to 13 640 kN, roughly double the largest seismic value, because the impulsive façade velocity closes the gap before flexural response develops; the combined hazard reduces median capacity by a further 29 percent, confirming a synergistic rather than additive interaction. Fourth, energy-dissipating coupling consistently outperforms geometric separation, with viscoelastic links reducing impact force by 67 percent, drift by 51 percent and base shear by 29 percent under the combined scenario, at an efficiency per unit cost superior to every alternative tested except rubber bumpers. It is therefore recommended that separation requirements in Indian practice be supplemented by mandatory dissipative coupling for pairs in high-density, blast-exposed precincts, and that future work extend the dataset to floor-to-column pounding, soil–structure interaction and computational fluid dynamics modelling of canyon-channelled blast waves.

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