Use of walls in controlling detrimental effects of stiffness irregularity in RC buildings on hill slopes

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This paper discusses behaviour of archetypical reinforced concrete (RC) buildings on sloping ground with two possible types of column base fixity conditions. Twisting behaviour due to stiffness irregularity is a major concern for such buildings. This study demonstrates the use of RC walls to minimise torsional effects using results of modal analysis of building models. Based on building typology and ground slope, three configurations of RC walls are examined, and among them, the best configuration is identified. Also, earthquake behaviour of existing buildings in hilly regions is studied and compared with that of buildings designed using revised code provisions. Results of nonlinear time history analyses of buildings demonstrates poor behaviour of existing buildings, and corroborates the efficacy of RC walls towards ensuring seismic safety of buildings on hill-slope.

Keywords: *Coupling; torsion; step-back buildings; short column; shear wall.*

INTRODUCTION

Mountain formation includes compressional folding, large scale faulting and uplift of crustal blocks. During this process, elastic energy is released primarily in the form of shock waves, resulting in ground accelerations [1]. Past earthquakes indicate that the subducting plate boundaries can generate more intense ground shaking than plate boundaries with essential lateral deformation. Therefore, hilly regions, such as Himalayas, are at an acute risk of significant seismic damage to structures. Increase in population and old construction further aggravate the seismic risk in the region. In addition, steep ridges are known to amplify the ground accelerations. Consequently, buildings on top of them are subjected to amplified ground shaking [2]. This phenomenon was observed during 1985 Chilean earthquake, wherein buildings on ridges were severely damaged and had to be abandoned from further use, while buildings in valley were less affected, although both groups of buildings had similar structural properties. Therefore, to account for this effect, some building design codes [3, 4, and 5] define scaling factors (greater than one) to amplify the ordinates of elastic design spectrum; IS 1893 (1) [6] does not consider this yet.

Due to increase in population and perceived durability of concrete over other building materials, multistory reinforced concrete (RC) buildings are being preferred over traditional constructions in hilly regions. However, most of these buildings are constructed without engineering inputs. Consequently, they exhibit perilous features like poor material quality (poorly executed hand-mixed concrete is common), absence of seismic design and detailing, and inadequate member (columns in particular) sizes [7]. Such buildings have incurred severe damage during past earthquakes (*e.g.*, 2005 Kashmir earthquake, 2011 Sikkim earthquake, 2015 Nepal earthquake, and 2016 Imphal earthquake); some buildings collapsed due to brittle column failures forming storey mechanism [8, 9, 10, 11, 12]. Revised IS 13920 [13] limits maximum axial compressive stress ratio in columns to ensure minimum curvature ductility and thereby attempts to prevent such brittle failures.

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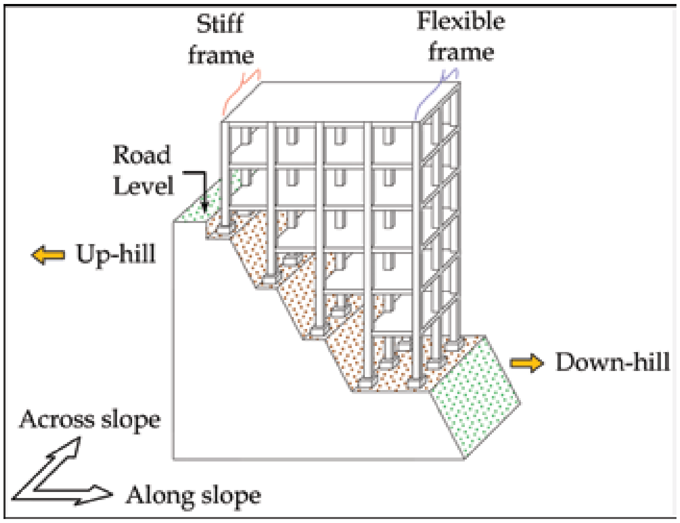

Figure 1. Step-back configuration of buildings on sloping ground

Depending on steepness of ground slope, various structural configurations are adopted in hilly areas of Northern India. Among them, step-back configuration is most prevalent [14]. Individual column base in such buildings rests on artificial flat ground, but step-down along hill slope (hereinafter referred to as X-direction) (Figure 1). Consequently, successive floors (below road level) are supported on columns of unequal heights. The shorter columns attract large portion of total storey shear, thus making them vulnerable to shear failure [15]. Further, the shear demand induced at the interface of foundation of short column and supporting soil may exceed the frictional capacity of soil, during strong ground shaking. This leads to loss of translation fixity at base of these columns. Alternatively, similar situation arises when loose soil on hill slopes subsides during strong vertical ground shaking. In such cases, columns start sliding off the ground towards the down-hill direction [7]. This increases compressive loads and bending moments on columns present in the down-hill side. Ultimately, the increase in loads leads to failure of these columns and overturning of buildings in the down-hill direction. Further, step-back buildings have *stiffness irregularity (in plan)* arising due to varying height of frames along X-direction. This causes torsional coupling in modes of oscillation in cross-slope direction (hereinafter referred to as Y-direction) [12]. The coupling grows stronger with increase in ground slope and length of the building along X-direction, due to increased difference in stiffness of frames in the up-hill and down-hill sides oriented in the Y-direction.

STIFFNESS TUNING

The efficacy of RC walls in resisting strong seismic actions is well-known. RC walls offer large in-plane stiffness and strength to buildings during seismic action. Thus, this study uses RC walls to minimise torsional effects in buildings with stiffness irregularity (Figure 2), and demonstrates the use of elastic modal analysis to identify optimum wall configuration for a given building typology. For modal analyses and subsequent designs, full fixity is assumed at the base of all columns. The in-plane rigidity of all floors is considered using rigid diaphragm constraints. The effective

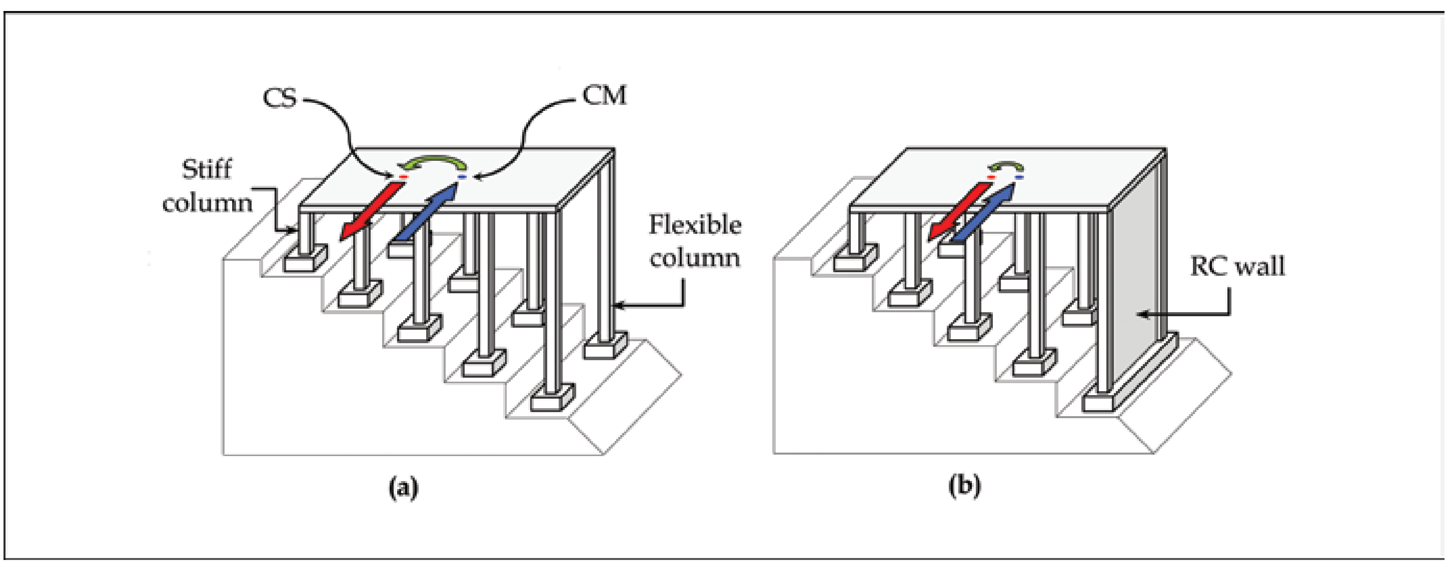

Figure 2. Degree of torsional coupling in irregular frame (a) without RC wall, and (b) with RC wall

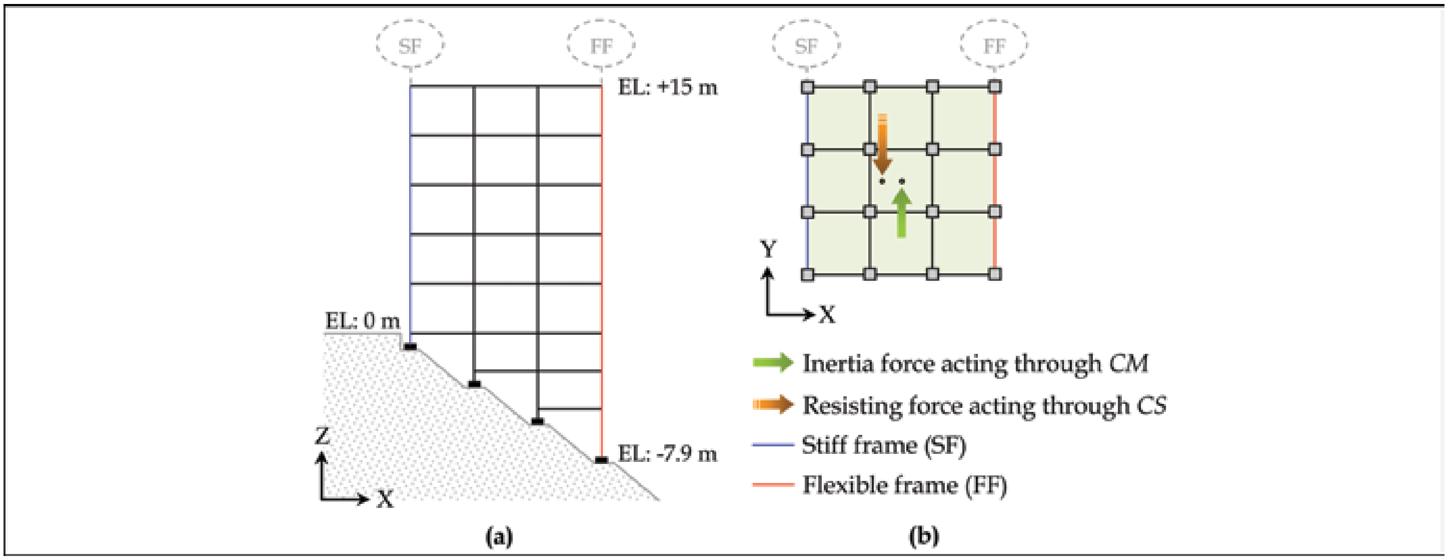

Figure 3. 3-bay step-back building on 30° ground slope (a) side elevation, and (b) plan

stiffness of beams and columns are assumed as 35% and 70% of gross stiffness, respectively [2].

This study investigates earthquake behaviour of step-back buildings with (i) three cases of ground slope (*30°, 40°* and *50°*), (ii) two cases of number of bays along X-direction (*3* and *4*) , and (iii) two cases of column base fixity conditions (*fixed-base and roller-base*); all cases have three bays in Y-direction and typical bay length of 4m in both plan directions. A typical 3-bay step-back building on 30° ground slope is shown in Figure 3. Each building is assigned a three-digit model number; the first two digits denote the *ground slope* (in degrees) while the last digit denotes the *number of bays* along X-direction. Thus, building model *303* represents a step-back building, resting on ground with a slope of 30° and with 3-bays along X-direction. The fundamental periods in three directions of all building models are listed in Table 1. It is seen that the modes along X- and Y-directions are close to each other, *i.e.,* difference between their natural periods is less than 10% of the higher natural period (values in columns (2) and (3) in Table 1). However, IS 1893 [6] requires a minimum separation of these lower modes of oscillation,

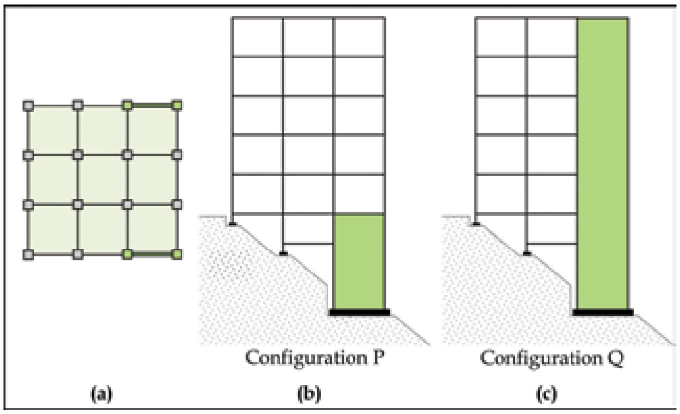

Figure 4. Location of RC walls along X-direction (a) plan at road level, (b) side elevation of configuration P, and (c) side elevation of configuration Q

that govern overall dynamic behaviour of buildings. One way of ensuring this is by increasing lateral stiffness of buildings in one direction. Thus, two RC walls are placed symmetrically in X-direction on down-hill side (Figure 4). Providing walls up to road level alone (*i.e.,* Configuration P) does not help as the fundamental modes remain closely

Table 1. Fundamental periods of oscillation of study buildings

Building Model (1)	Moment Frame			Configuration P			Configuration Q		
	T_x (2)	T_y (3)	T_θ (4)	T_x (5)	T_y (6)	T_θ (7)	T_x (8)	T_y (9)	T_θ (10)
303	1.20	1.22	1.02	1.18	1.22	1.00	0.63	1.22	0.55
304	1.22	1.27	1.05	1.21	1.27	1.04	0.63	1.27	0.60
403	1.21	1.24	1.04	1.20	1.24	1.03	0.58	1.24	0.51
404	1.18	1.30	1.08	1.10	1.30	0.99	0.70	1.30	0.67
503	1.21	1.23	1.03	1.08	1.23	0.96	0.56	1.23	0.49
504	1.19	1.27	1.02	1.16	1.27	1.01	0.67	1.27	0.58

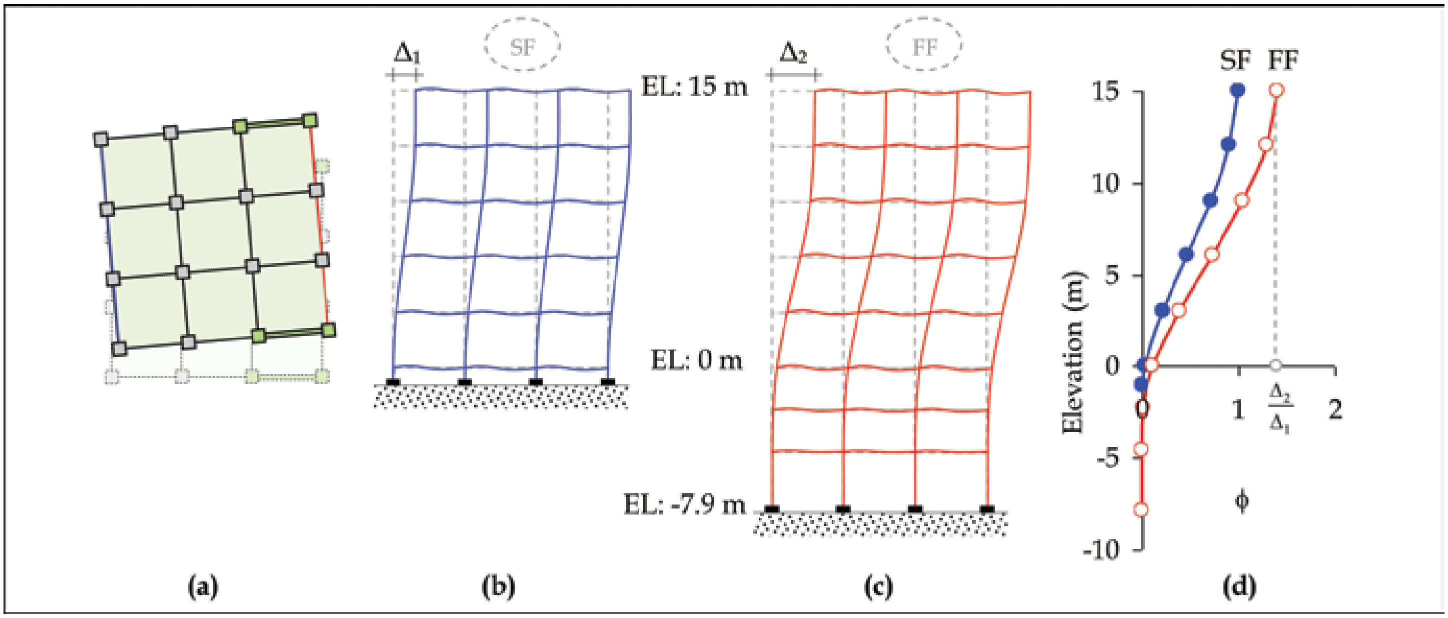

Figure 5. Fundamental mode of oscillation of building model 303: (a) plan at roof level, (b) elevation of stiff frame, (c) elevation of flexible frame, and (d) graphical presentation of normalised mode shape ϕ along building elevation

spaced (values in columns (5) and (6) in Table 1). Therefore, these walls are continued till the roof level (*i.e.,* Configuration Q); this ensures that the modes along X and Y-directions are separated, and the torsional mode remains the third mode of oscillation (values in columns (8), (9), and (10) in Table 1). However, the mode of oscillation along Y-direction, which is the first mode, is torsionally coupled (Figure 5). The terms Δ_1 and Δ_2 in Figure 5 represent the ordinates of the mode shape at roof level of the stiff and flexible frame, respectively. Thus, the ratio Δ_2/Δ_1 is a quantitative measure of torsional coupling, which is aimed to be minimised with the addition of RC walls oriented in the Y-direction of the building (Figure 6). The ratio Δ_2/Δ_1 is calculated for different building configurations using results of modal analysis (Table 2). It can be seen that the ratio consistently increases with increase in ground slope and number of bays along X-direction (values in column (2) in Table 2). Consequently, larger cross-sectional area of wall is required to decouple the torsional mode from the translational mode in the Y-direction (or to minimise the ratio Δ_2/Δ_1). Therefore, while configuration

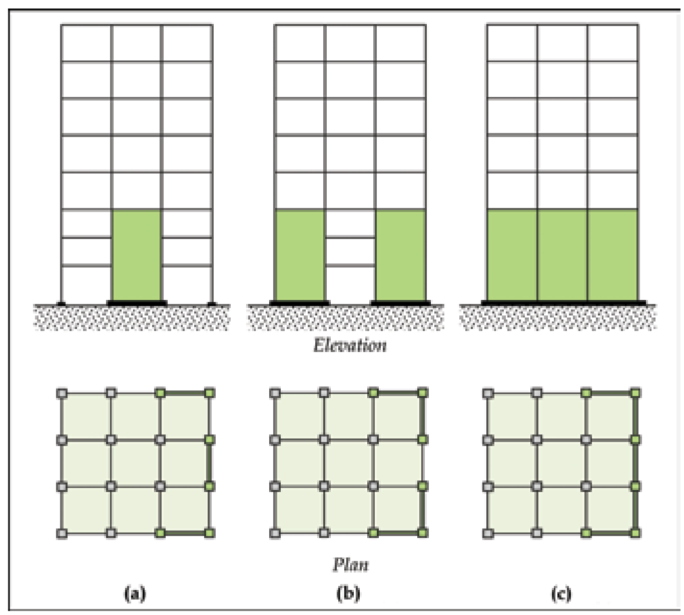

Figure 6. Elevation and plan of buildings with different configurations of RC walls: (a) Configuration A, (b) Configuration B, and (c) Configuration C

Table 2. Ratio Δ_2/Δ_1 at roof level of buildings for different wall configurations as shown in Figure 6

Building Model (1)	Δ_2/Δ_1			
	Moment Frame (2)	Configuration A (3)	Configuration B (4)	Configuration C (5)
303	1.41	**1.02**	0.98	0.97
304	1.73	**1.06**	0.99	0.95
403	1.41	1.16	**1.07**	1.06
404	1.83	1.14	**1.04**	0.99
503	1.42	1.08	1.04	**1.01**
504	1.82	1.17	1.10	**1.02**

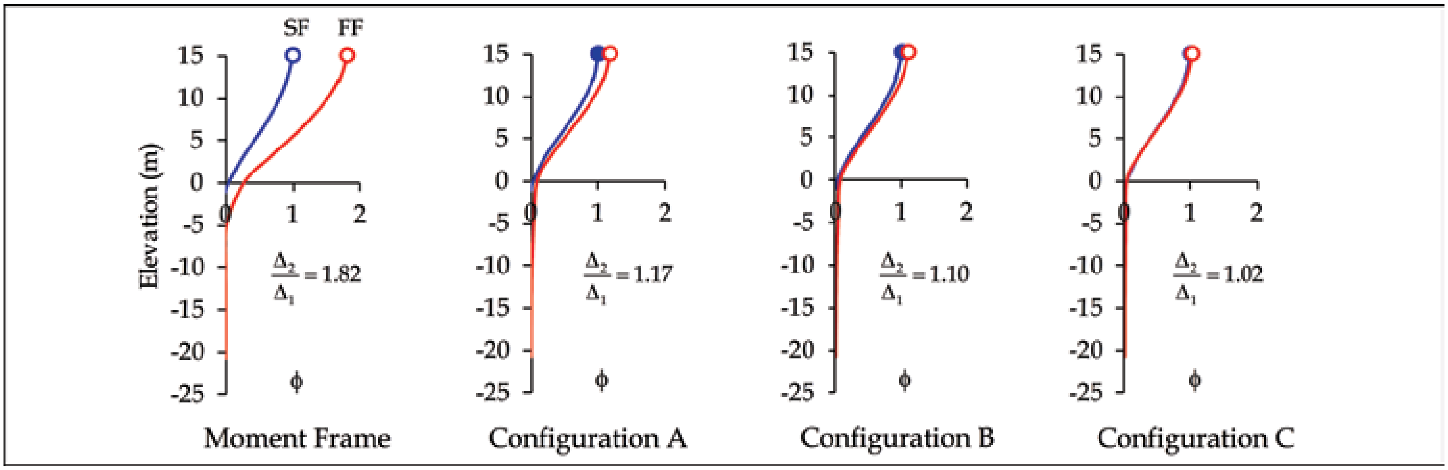

Figure 7. Typical normalised mode shapes of building model 504 in Y-direction

A is adequate for buildings on *30°* ground slope, at least configurations B and C are required for buildings on *40°* and *50°* slopes, respectively (values in columns (3), (4) and (5) in Table 2). This is illustrated in Figure 7, where it can be seen that, wall configurations A and B are not sufficient enough to minimise torsional coupling in a building on 50° ground slope.

For all buildings considered, six moment frames and six dual-systems (each with most suitable wall configuration as mentioned above) are designed for seismic zone V, using response spectrum method, as per IS 1893 (1) [6]. The preliminary analysis is performed using commercial structural analysis software [16], to arrive at member forces. Design and detailing of structural members (beams, columns, and RC walls) are carried out using provisions of IS 456 and IS 13920 and other design aids [17, 13, 18, and 19].

NUMERICAL STUDY

In addition to the set of designed buildings, a second set of buildings with moment frame alone is considered. It is supposed to represent key structural characteristics of existing stock of step-back buildings in hilly regions; the structural details of this set of buildings is obtained from literature [20]. The former set of building is labeled as 'Designed Buildings' and the later as 'Reference Buildings'; only structural properties vary in terms of member sizes and reinforcement details while typologies of buildings in both the sets are same. Also, study of linear elastic behaviour of a structure alone is not sufficient to understand key factors that define its ultimate dynamic response such as, failure mechanism, damage states, energy dissipation etc., during strong earthquake shaking. Therefore, 3D computational models of the buildings that can capture their nonlinear behaviour are developed using a commercial software [21].

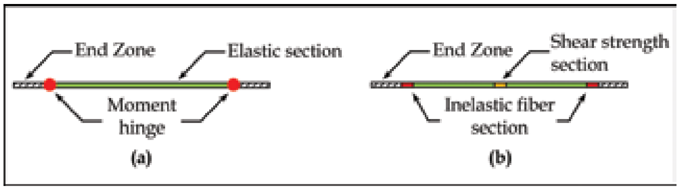

Figure 8. Numerical model of (a) beam element and (b) column element

Details of Computational Model

Material properties of M30 grade concrete and Fe415 grade steel are used to define elastic section properties of frame and RC wall elements. The nonlinear stress-strain behaviour of confined concrete is determined using Mander's model [22], while nominal stress-strain curve given in IS 456: 2000 [17] is used for unconfined concrete and Fe415 grade steel reinforcement. Beam elements are modeled using lumped plasticity approach (Figure 8(a)), with the hinge property (*i.e.*, moment-curvature relationship) idealised as a bilinear curve [23]. For columns and RC wall elements, fiber sections are used to define their inelastic behaviour (Figure 9). These sections use constitutive properties of materials used to define fibers, to develop their force-deformation response. In addition, shear strength sections

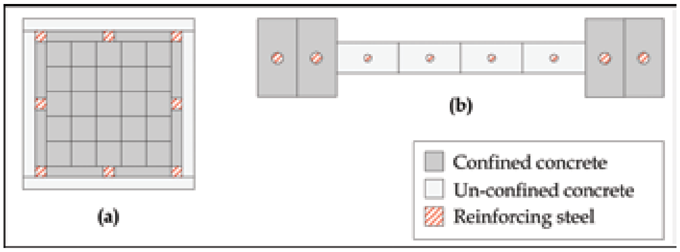

Figure 9. Typical fiber section of RC (a) column element, and (b) wall element

Table 3. Details of earthquake records used in analysis

Sr. No. (1)	Event (2)	M_w (3)	Maximum PGA (g) (4)	Fault Type (5)	Recording Station	
					Name (6)	Epicentral Distance (km) (7)
1	1976 Friuli	6.4	0.35	Thrust	Tolmezzo	28
2	1979 Montenegro	6.9	0.27	Thrust	Veliki	143
3	1985 Algarrobo	8.0	0.22	Thrust	Rapel	108
4	1988 Armenia	6.7	0.18	Thrust	Gukasian	36
5	1992 Big Bear	6.4	0.16	Strike-slip	Snow creek	37
6	1999 Chamoli	6.6	0.36	Thrust	Gopeshwar	17
7	1999 Chi-Chi	7.6	0.04	Thrust	Chiyai	45
8	1999 Hector Mine	7.1	0.08	Strike-slip	Heart Bar	69
9	2002 Denali	7.9	0.08	Strike-slip	Fairbanks	139
10	2011 Sikkim	6.8	0.16	Thrust	Gangtok	71

are defined in column elements (Figure 8(b)). They monitor shear demand-to-capacity ratio, and thus, help investigate possible shear failure mode.

In present study, nonlinear time history analysis is performed of all buildings, using 20 natural ground motion records from ten past earthquakes (Table 3), as obtained from Ground Motion Database [24 to 27]. Selection of these records is based on three parameters namely:

(i) Fault type: Mountainous zones are sites of mainly *thrust* type earthquakes [28]. However, *strike-slip* type earthquakes also occur in hilly regions. Therefore, past earthquakes with only these two types of fault mechanisms are considered;

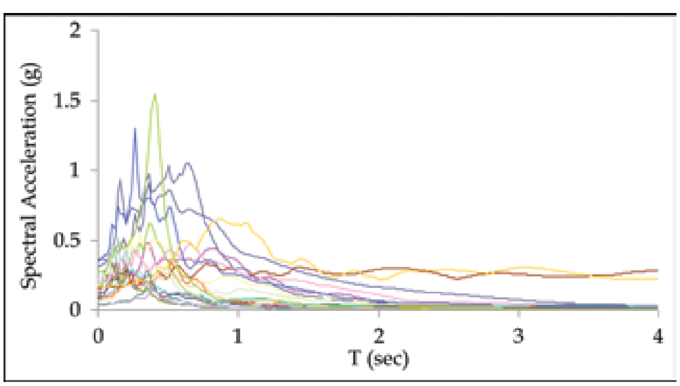

Figure 10. Elastic acceleration response spectra of 20 ground motion records from 10 earthquakes

(ii) Epicentral distance: The ground motion parameters of a particular earthquake depend on geological site characteristics and source to site distance. The selected data contains both near field and far field ground motions;

(iii) Station elevation and location: Due to topographic features, seismic ground motion in hilly regions encounters amplification in its frequency and amplitude [14]. In order to capture this effect, records from stations situated at least 1000 m above mean sea level on sloping ground was considered over those from other stations that recorded the same earthquake.

Each earthquake record consists of two horizontal and one vertical component of ground motion. However, only horizontal components are used in the analyses. The 20 ground motion records from 10 earthquakes are applied along each X- and Y-directions of building models. The elastic acceleration response spectra for the 20 ground motions are shown in Figure 10. The acceleration histories are scaled with respect to 5% damped design spectral acceleration value at fundamental natural period of each building model in the respective direction (*i.e.*, X and Y) for nonlinear dynamic analyses of the building models. Further, to identify possible failure, two collapse states are defined as: (i) normal compressive strain in any confined concrete fiber exceeding the ultimate strain capacity; and (ii) shear demand in any column element exceeding its shear capacity. Using these, an analysis is terminated when either of the two collapse states is reached. If none of the collapse states is reached during the analysis, a building model is said to have 'survived' a particular ground motion.

Table 4. Number of ground motions survived by study buildings

Building Model (1)	Column base fixity (2)	Along X-direction			Along Y-direction		
		Reference building (Moment frame) (3)	Designed building		Reference building (Moment frame) (6)	Designed building	
			Moment frame (4)	Dual system (5)		Moment frame (7)	Dual system (8)
303	Fixed	0	20	20	0	16	18
304		0	16	19	0	16	19
403		0	18	19	0	17	19
404		0	18	19	0	13	18
503		0	18	20	0	18	19
504		1	18	19	0	17	19
303	Roller	2	7	19	0	6	20
304		0	12	19	0	7	19
403		0	13	19	0	11	19
404		0	13	19	0	9	19
503		5	16	19	6	16	19
504		1	16	19	4	15	18

RESULTS AND DISCUSSION

The number of ground motions 'survived' by the study building models is listed in Table 4. It is observed that designed moment frame buildings perform better than reference (moment frame) buildings. The reference buildings sustain severe damage to the columns and undergo large residual displacements of 60 mm and 30 mm for shaking in X- and Y-directions respectively (Figures 11(a) and 12(a)). In particular, for ground motion in Y-direction, corner columns at road level fail due to flexural crushing of concrete under combined action of axial compression and biaxial bending. Elastic analysis showed that these columns carry axial compressive stress, as high as 0.8-1.0f_{ck}, whereas the limiting value of 0.4f_{ck} is stipulated by IS 13920 [13]. The high axial compressive stress in these columns limits their ductility capacity causing brittle failure. Further, for ground

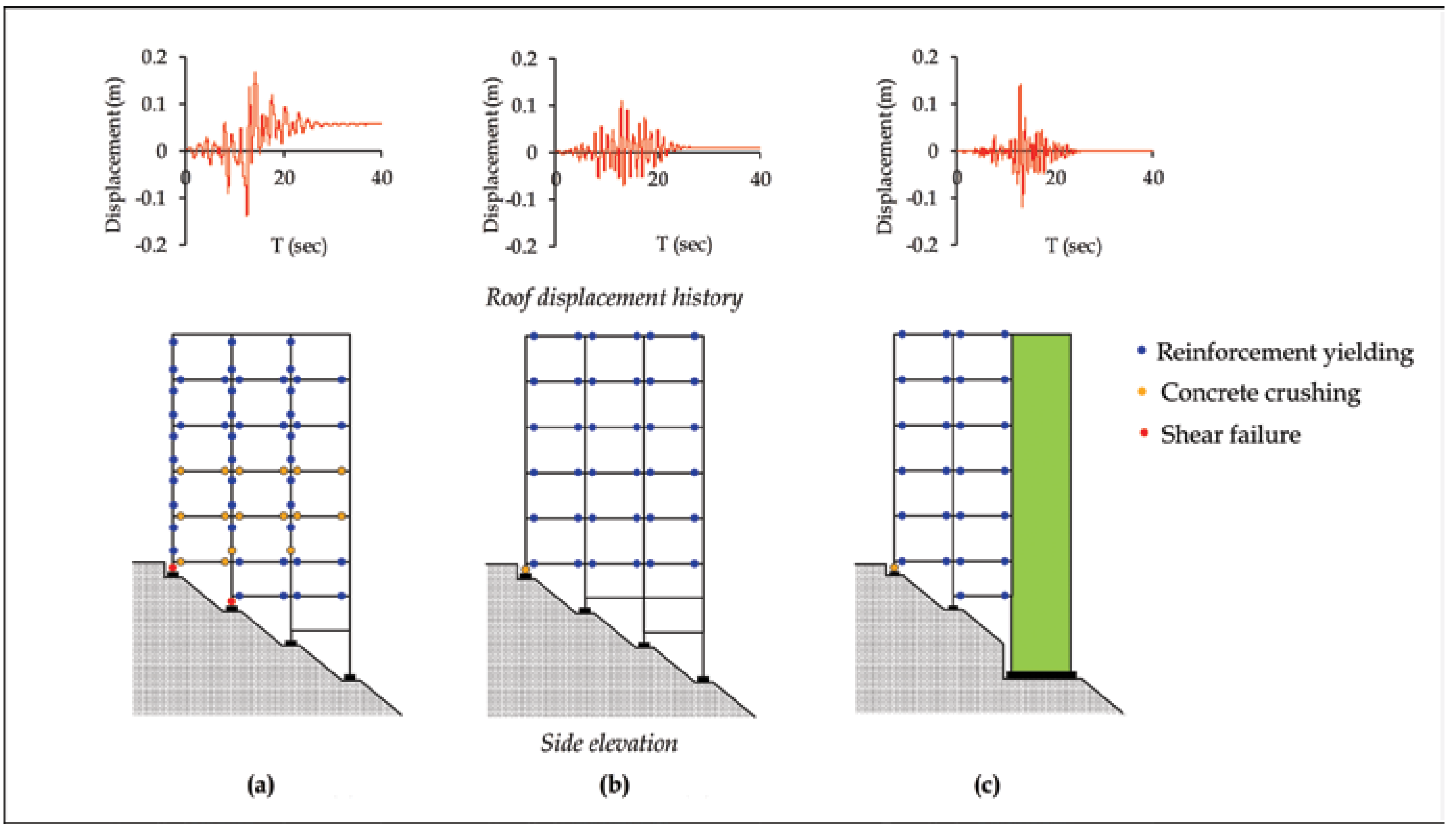

Figure 11. Roof displacement history and damage pattern in fixed-base building model 303 under E-W component of 1988 Armenia earthquake ground motion acting in X-direction: (a) reference building, (b) design moment frame, and (c) design dual-system

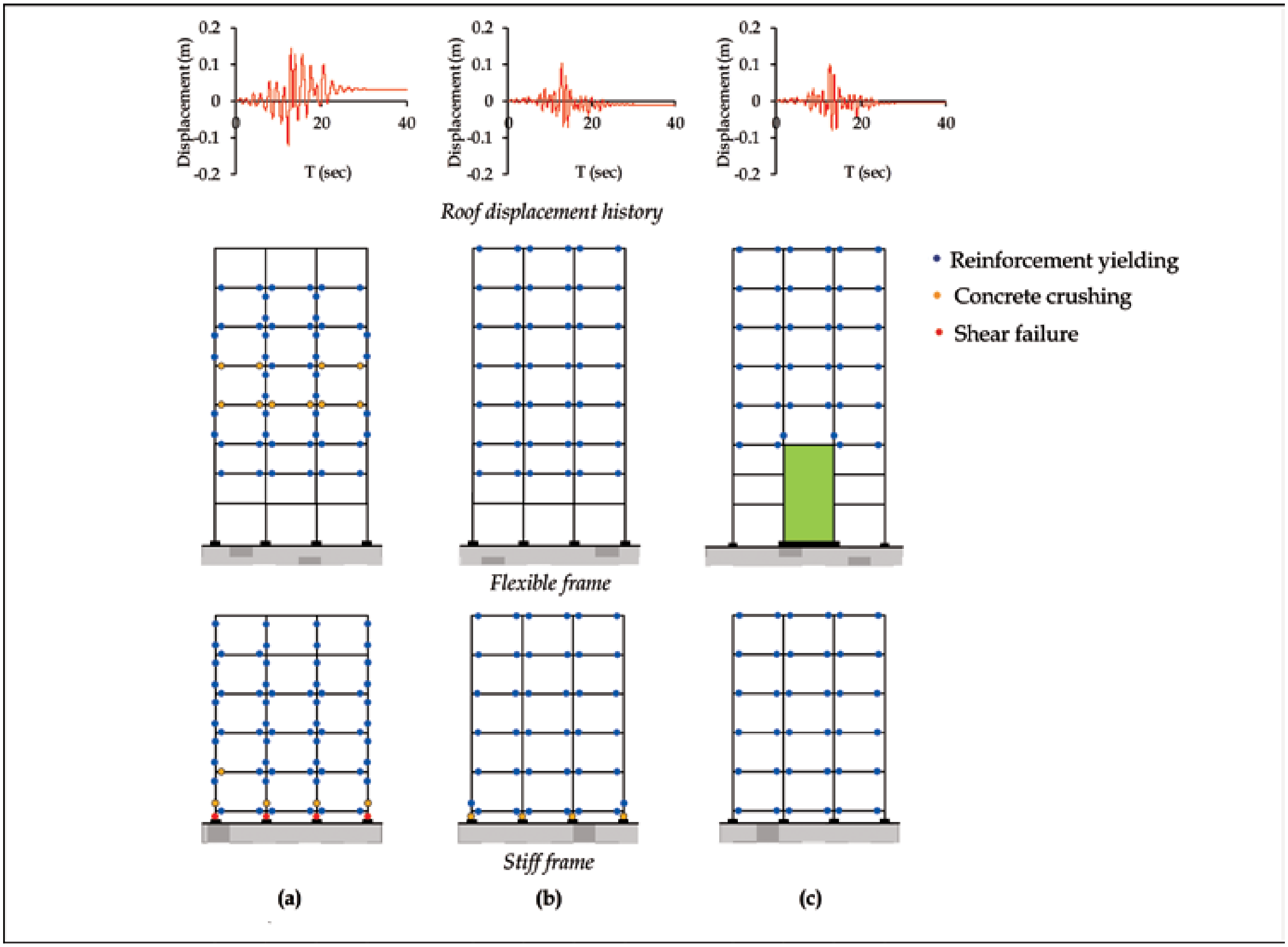

Figure 12. Roof displacement history and damage pattern in fixed-base building model 303 under N-S component of 1988 Armenia earthquake ground motion acting in Y-direction: (a) reference building, (b) design moment frame, and (c) design dual-system

motion along X-direction, shear failure of short columns on uphill side initiates collapse, followed by crushing of intermediate road level columns, followed by hinging in beams (Figure 11(a)). This failure mechanism is undesirable for two reasons; (i) shear is a brittle mode of failure, and hence, should be prevented in any structural member, and (ii) columns should not fail prior to formation of plastic hinges in beams, as such failure mechanism dissipates little amount of energy input to structure during earthquakes. This premature failure of columns in the reference buildings is due to lower values of column-to-beam flexural design strength ratio (0.5 to 1.3) provided, against a minimum value of 1.4 prescribed by IS 13920 [13].

In contrast, moment frame buildings designed following recommendations of the revised IS 13920 [13] incur damages confined in beams predominantly, with yielding of reinforcement in corner columns under biaxial bending (Figures 11(b) and 12(b)). This is due to stiffness irregularity induced twisting of these buildings. Also, short columns in the stiff up-hill frame undergo failure by crushing of core concrete in some cases in buildings with full fixity of column bases (Figure 12(b)). This is because larger portion of the inertia forces mobilized in a building gets transferred through the short columns in stiff frame. On the other hand, when multiple column bases lose their translational and rotational fixity (roller support condition), the load path

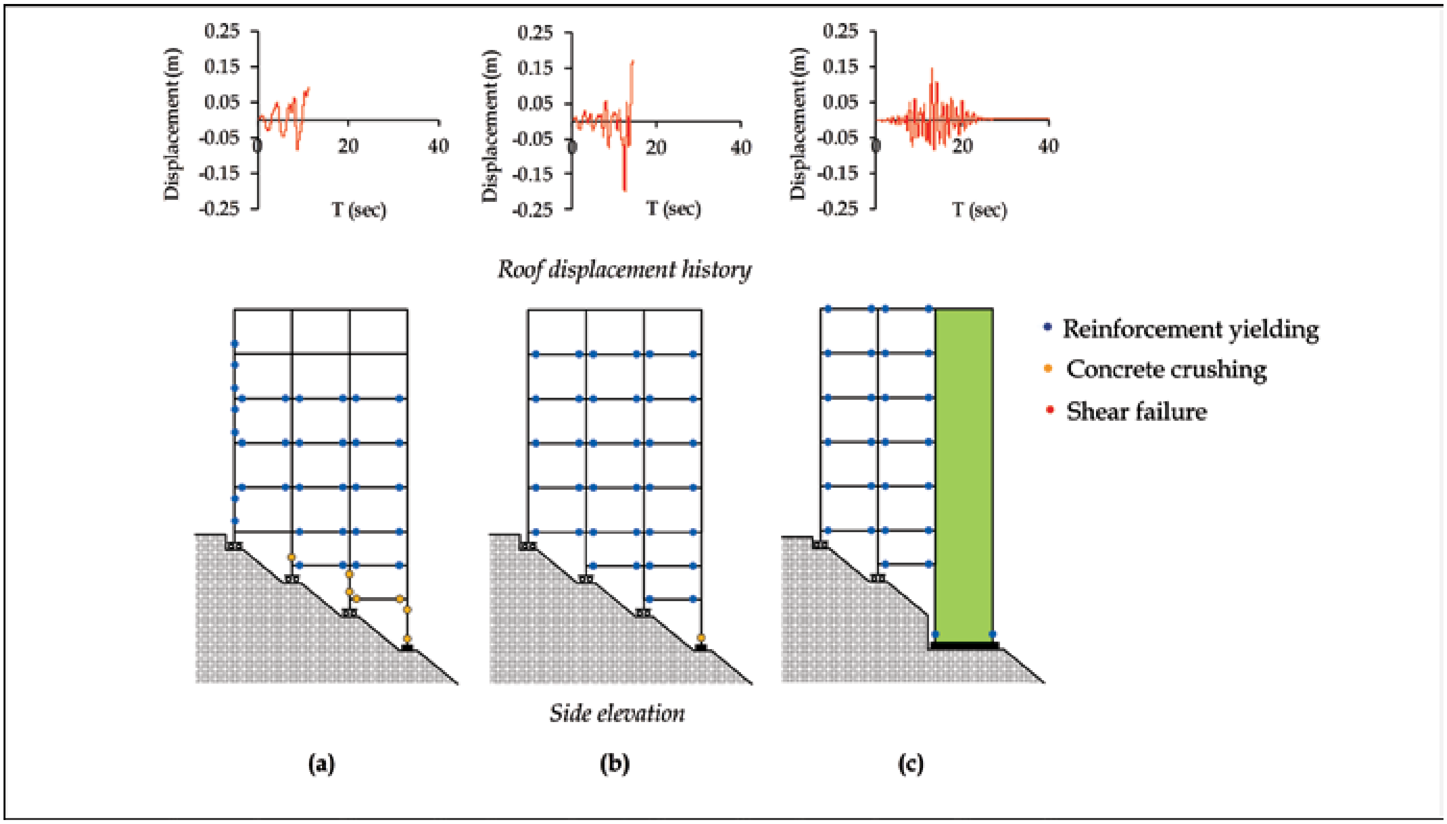

Figure 13. Roof displacement history and damage pattern in roller-base building model 303 under E-W component of 1988 Armenia earthquake ground motion acting in X-direction: (a) reference building, (b) design moment frame, and (c) design dual-system

for transferring horizontal inertia forces changes, causing severe distress in columns in the flexible frame on the down-hill side; these columns fail due to crushing of concrete (Figures 13 and 14). As such, the overall performance of these buildings, particularly those on 30° and 40° ground slope with roller support condition, is not satisfactory (values in columns (4), and (7) of Table 4) and thus, suggests a need for improved seismic behaviour.

Addition of RC walls significantly changes the earthquake behaviour of step-back buildings (values in columns (5) and (8) of Table 4). For shaking in the X-direction, the RC walls carry the additional force demand arising due to loss of column base fixity and thereby prevent compression failure of columns in down-hill frames. This is extremely important as the buildings continue to carry the gravity loads without collapse, although during the analyses, RC walls in the X-direction incurred damage due to yielding of reinforcement (Figure 13(c)). However, the yielding was confined in the outermost reinforcement fibers in boundary elements only (Figure 9(b)). Also, the walls in the Y-direction minimise torsional coupling and thereby prevents damage to corner columns due to significant biaxial moment (that is common in moment frame buildings during shaking in the Y-direction). To illustrate this behaviour, the displacement history is shown of one control node of corner column at roof level of all building models, for one ground motion applied along Y-direction (Figure 15). It can be seen that, addition of wall (of the best configuration as discussed using results in Table 2) minimises lateral displacement of control node, thus rendering columns to bend predominantly in one direction (*i.e.*, Y- direction). The dual-system behaves like a regular moment frame building in Y-direction, with effective fixity at road level. Therefore, the overall translation along Y-direction at roof level in dual-system remains more or less same as that in designed moment frame buildings. However, minimal residual displacements are observed in dual-system buildings after earthquake shaking (Figures 11(c), 12(c), 13(c), and 14(c)), with ductile plastic hinges primarily in beams, and thereby, ensure seismic safety of buildings on hill-slopes.

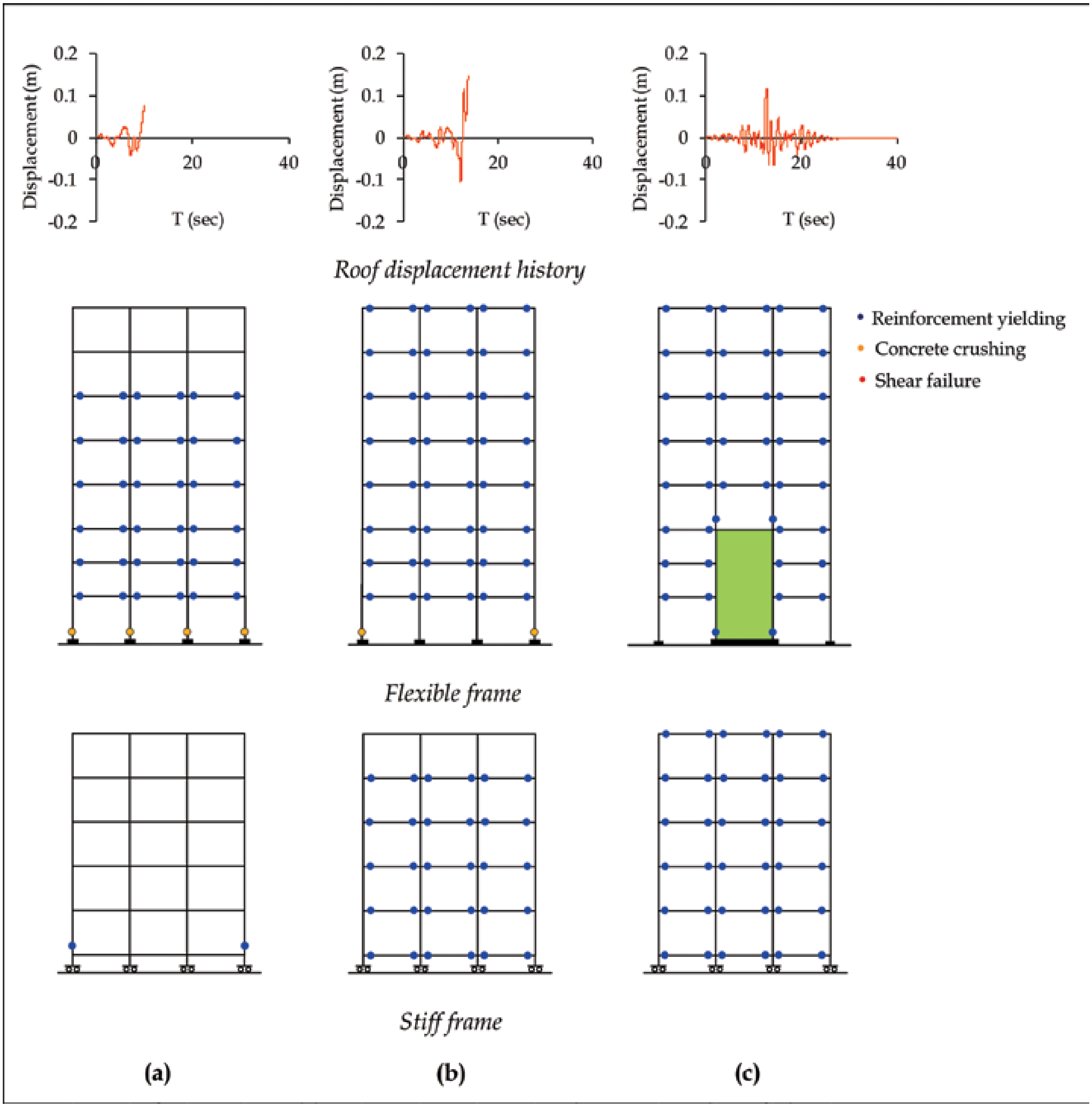

Figure 14. Roof displacement history and damage pattern in roller-base building model 303 under N-S component of 1988 Armenia earthquake ground motion acting in Y-direction: (a) reference building, (b) design moment frame, and (c) design dual-system

CONCLUDING REMARKS

The key findings of the study are:

1. In step-back buildings, the fundamental mode of oscillation in cross-slope direction is torsionally coupled. Therefore, under cross-slope excitation, corner columns in the building are susceptible to failure under combined action of axial compressive force and biaxial bending.

2. Loss of column base fixity has profound effect on seismic performance of step-back buildings, as it changes load path for transferring earthquake induced

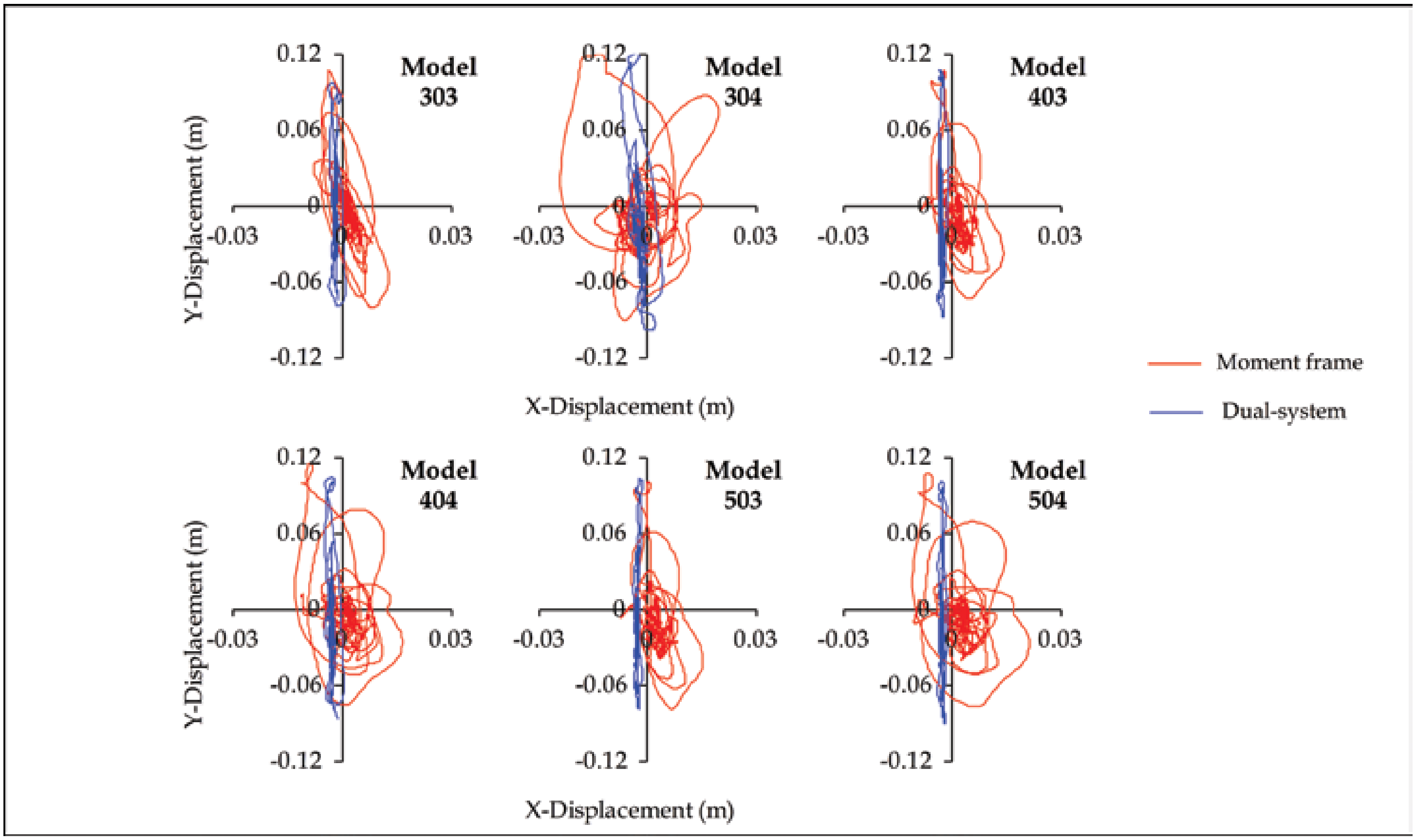

Figure 15. Displacement of the control node on corner column at roof level under N-S component of 1988 Armenia earthquake ground motion applied along Y-direction of building models

inertia forces and thus, can cause catastrophic collapse of the building. Such brittle failure can be prevented by use of RC walls as a part of lateral force resisting system. In addition, RC walls located suitably in plan help minimise stiffness irregularity induced torsional effects. However, this requires a study using results of modal analyses to identify the most appropriate wall configuration.

3. Based on results of linear and nonlinear dynamic analyses, three RC wall configurations are proposed (Table 5). They are classified depending on the level difference between most up-hill and down-hill footing in the building.

Table 5. Recommendation on selection of RC wall configuration

Level difference	Wall configuration
≤ 10 m	A
10-15 m	B
> 15 m	C

References

1. Gerrard A.J., *Mountain Environments: An Examination of the Physical Geography of Mountains*, MIT Press, Boston, 1990, pp. 3-35.
2. Paulay T. and Priestley M.J.N., *Seismic Design of Reinforced Concrete and Masonry Buildings*, John Wiley and Sons, New York, 1992, pp. 49-50, 57-58, 162-163.
3. _______*Eurocode 8: Design of structures for earthquake resistance-part 5: Foundations, retaining structures and geotechnical aspect*, EN 1998-5: 2004, CEN European Committee for Standardization, Brussels.
4. _______*Recommendations for the redaction of rules relative to the structures and installations built in regions prone to earthquakes*, AFPS 90, 1990, French Association for Earthquake Engineering, France.
5. _______*Guidelines and Criteria for the Seismic Microzonation*, Gdl MS, 2008, Department of Civil Protection, Rome.
6. _______*Indian standard criteria for earthquake resistant design of structures*, IS 1893 (Part 1): 2002, General provisions and buildings, Bureau of Indian Standards, New Delhi.
7. Vijaya Narayanan A.R., Goswami R., and Murty C.V.R., Performance of RC buildings along hill slopes of Himalayas during 2011 Sikkim earthquake, *Proceedings of 15th World Conference on Earthquake Engineering,* Lisbon, Portugal, 2012, Paper No. 621.
8. Rai D.C. and Murty C.V.R., *First Report on the Kashmir Earthquake of October 8, 2005 - India*, EERI Special Earthquake Report, EERI

Newsletter, Earthquake Engineering Research Institute, USA, 2005.
9. Murty C.V.R., Raghukanth S.T.G., Menon A., Goswami R., Vijayanarayanan A.R., Gandhi S.R., Satyanarayana K.N., Sheth A.R., Rai D.C., Mondal G., Singhal V., Parool N., and Pradhan T., Jaiswal A., Kaushik H.B., Dasgupta K., Chaurasia A., Bhushan S., Roy D., and Pradeep Kumar R., The M_w 6.9 Sikkim-Nepal border earthquake of September 18, 2011, *EERI Newsletter, EERI Special Earthquake Report,* Earthquake Engineering Research Institute, USA, 2012.
10. Kaushik H.B., Bevington J., Jaiswal K., Lizundia B., Shrestha S.N., *M7.8 Gorkha, Nepal Earthquake on April 25, 2015 and its aftershocks,* EERI Earthquake Reconnaissance Team Report, Chapter 15, Earthquake Engineering Research Institute, USA, 2016, pp. 73-98.
11. Rai D.C., Kaushik H.B., and Singhal V., *The 2016 Imphal Earthquake of January 04, 2016,* Preliminary Report, National Information Center for Earthquake Engineering, India, 2016.
12. Singh Y., Gade P., Lang D. H. and Erduran E., Seismic behavior of buildings located on slopes–An analytical study and some observations from Sikkim earthquake of September 18, 2011, *Proceedings of 15th World Conference on Earthquake Engineering*, Lisbon, Portugal, 2012, Paper No. 4201.
13. _______*Indian Standard Code of Practice for Ductile Design and Detailing of Reinforced Concrete Structures Subjected To Seismic Forces*, IS 13920: 2016, Bureau of Indian Standards, New Delhi.
14. Singh Y., Lang D. H., and Narsimha D. S., Seismic risk assessment in hilly areas: case study of two cities in Indian Himalayas, *Proceedings of SECED 2015 Conference: Earthquake Risk and Engineering towards a Resilient World*, Cambridge, UK, 2015.
15. Birajdar B. G. and Nalawde S. S., Seismic analysis of buildings resting on sloping ground, *Proceedings of 13th World Conference on Earthquake Engineering*, Vancouver, B.C., Canada, 2004, Paper No. 1472.
16. CSI, *Structural Analysis Program* (SAP) 2000, Version 15, Computers and Structures Inc., Berkeley, USA, 2012.
17. _______*Indian standard code of practice for plain and reinforced concrete*, IS456: 2000, Bureau of Indian Standards, New Delhi.
18. Aysha Z.M., Goswami R., and Murty C.V.R., Mechanics-driven hand calculation approach for obtaining design P-M interaction curves of RC sections, *The Indian Concrete Journal*, 2015, Vol. 89, No. 9, pp. 59-68.
19. Deshmukh A. and Goswami R., Unified strain-based procedure to obtain design P-M interaction curves of slender RC wall sections, *Proceedings of 16th World Conference on Earthquake Engineering,* Santiago, Chile, 2017, Paper No. 864.
20. Welsh-Huggins S.J., Rodgers J., Holmes W., and Liel A.B., Seismic vulnerability of reinforced concrete hillside buildings in northeast India, *Proceedings of 16th World Conference on Earthquake Engineering,* Santiago, Chile, 2017.
21. CSI, *Nonlinear Analysis and Performance Assessment for 3D Structures* (PERFORM-3D), Version 5, Computers and Structures Inc., Berkeley, USA, 2011.
22. Mander J. B., Priestley M. J. N., and Park R., Theoretical stress-strain model for confined concrete, *Journal of Structural Engineering*, Proceedings of ASCE, 1988, Vol. 114, No. 8, pp. 1804-1826.
23. Sunitha P., Goswami R., and Murty C.V.R., Idealised bilinear moment-curvature curves of RC sections for pushover analysis of RC frame buildings, *The Indian Concrete Journal*, April 2016, Vol. 90, No. 4, pp. 43-54.
24. *PEER NGA Database*, Pacific Earthquake Engineering Research Center, University of California, Berkeley, California, 2010.
25. Luzi L., Puglia R., Russo E., and ORFEUS WG5, *Engineering Strong Motion Database*, version 1.0, Istituto Nazionale di Geofisica e Vulcanologia, Observatories & Research Facilities for European Seismology, Istanbul, Turkey, 2016.
26. COSMOS, *Consortium of Organizations for Strong-Motion Observation Systems*, Pacific Earthquake Engineering Research Center, University of California, Berkeley, California, USA, 2012.
27. Mittal H., Kumar A., and Ramhmachhuani R., Indian Strong Motion Instrumentation Network and its site characterization, *International Journal of Geosciences*, 2012, Vol. 3, No. 6, pp. 1151-1167.
28. Bolt B.A., *Earthquakes*, W. H. Freeman and Company, New York, 1988, pp. 57-58.

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