Framework for Improving the Seismic Behaviour of RC MRF Buildings Resting on Hill Slopes

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by

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CERTIFICATE

It is certified that the work contained in this thesis titled "**Framework for Improving Seismic Behaviour of RC MRF Buildings Resting on Hill Slopes**" by Pammi Raghu Nandan Vyas has been carried out under by supervision and is not submitted elsewhere for a degree.

Date

Advisor: Prof. Pradeep Kumar Ramancharla

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Raghu Nandan Vyas Pammi

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List of Symbols and Abbreviations

f_c	Characteristic compressive cylinder strength of unconfined concrete
f_{ck}	Characteristics of compressive cube strength of unconfined concrete
f_{sc}	Stress in reinforcement steel in compression
f_u	Ultimate strength of reinforcement steel
f_y	Yield strength of reinforcement steel
g	Acceleration due to gravity
h_{eff}	Effective height of SDOF system
l_b	Clear span of beams
l_c	Clear span of columns
l_p	Effective length of plastic hinge
l_{pc}	Length of plastic hinge
A_c	Gross area of concrete
A_g	Gross area of RC section
A_h	Seismic coefficient
A_{sc}	Area of reinforcement steel in compression
A_{st}	Area of reinforcement steel in tension
D	Overall depth of beam
E_c	Modulus of elasticity of concrete
EI	Flexural rigidity of RC section
H	Lateral force
H_D	Design lateral force
H_e	Elastic maximum lateral force
H_{max}	Maximum lateral force
H_y	Lateral force at yield
I_b	Second moment of area of RC beam section
I_c	Second moment of area of RC column section
I_{eff}	The effective second moment of area of the RC section
I_g	Gross second moment of area of RC section
K_{eff}	Effective stiffness of SDOF system

K_i	Initial lateral stiffness of building
L	Span of beam
M	Flexural strength
M_u	Ultimate flexural strength
M_y	Flexural strength at yield
N_D	Design life of the structure
N_R	Return period of earthquake
P	Axial force
PF_1	Mode participation factor
P_u	Axial load capacity of RC section
R	Response reduction factor
R_μ	Ductility reduction factor
s	Scale factor
S_a	Spectral acceleration
S_a/g	Design acceleration spectrum value
S_d	Spectral displacement
S_v	Spectral velocity
t	Time instant
T	Fundamental natural period of a structure
V_B	Base shear
W	Seismic weight of the building
Z	Zone factor
γ_s	Partial safety factor of reinforcement steel
γ	Unit weight of material
γ_c	Partial safety factor of concrete
δ_u	Ultimate drift
δ_y	Yield drift
ε_c	Strain in concrete
$\varepsilon_{c,max}$	Maximum compressive strain in extreme compression fibre of concrete
ε_{csc}	Strain in concrete in compressive at the centre of reinforcement steel bars in compression

ε_r	Fracture strain of longitudinal reinforcement steel
ε_{sc}	Strain at the centre of reinforcement steel bars in compression
ε_{sh}	Tensile strain at the commencement of strain hardening in reinforcement steel bars
ε_{st}	Strain at the centre of an extreme layer of reinforcement steel in tension
ε_{su}	Fracture strain of transverse reinforcement steel bars
ε_u	Ultimate strain in longitudinal reinforcement steel bar
ε_0	Strain in unconfined concrete at extreme compression fibre at peak stress level (from Mander Model)
ε_{cc}	Strain in extreme compression fibre of core concrete
ε_{cu}	Ultimate strain in extreme compression fibre of unconfined concrete
ε_{co}	Strain in extreme compression fibre of unconfined concrete at peak stress level
ε_{sc}	Strain in reinforcement steel in compression
ε_s	Strain in reinforcement steel
ε_y	Yield strain in reinforcement steel
ε_{ccu}	Ultimate strain in extreme compression fibre of confined concrete
η	Damping correction factor
φ	Curvature of a RC section
Φ	Wavelet function
μ	Ductility ratio
θ_p	Plastic rotation of a frame member
θ_y	Yield rotation of a frame member
θ_u	Ultimate rotation of a frame member
ξ	Percentage of critical damping
ξ_{eff}	Effective damping
ω	Fundamental natural frequency of the SDOF
Δ	Lateral deformation
Δ_{roof}	Lateral deformation at the roof level of the building
Δ_u	Ultimate lateral deformation of the building

Δ_y	Lateral deformation at yield of load-deformation curve of the building
SU_x	sum of mass participation in the X-direction considered from the first three modes
SU_y	sum of mass participation in the Y-direction considered from the first three modes
SR_x	sum of mass participation in rotation about X-axis
SR_y	sum of mass participation in rotation about Y-axis
SR_z	sum of mass participation in rotation about Z-axis
$R_{1,z}$	Mass participation in rotation about Z-axis in the first mode
AR	Ratio of the length of the building to the width of the building
SR	Ratio of the height of the building to the width of the building
BAS	Beam Axial stress ratio
BSS	Beam Shear stress ratio
BTS	Beam Torsional stress ratio
BFS	Beam Flexural stress ratio
CAS	Column Axial stress ratio
CSS	Column Shear stress ratio
CFS_M3	Column Flexural stress ratio in the major axis
CFS_M2	Column Flexural stress ratio in the minor axis

Abstract

In the past decade, India witnessed a surge in concentrated urban growth to manifolds. The same trend is visible in hilly regions where the seismic safety of buildings is partially answered. Also, the peak ground accelerations observed in the past earthquakes are in accordance with the design PGA, the associated damages observed are brittle, which is undesirable. This can be mainly attributed to the current design codes not providing sufficient recommendations for the safety of buildings on hill slopes. For example, the code suggests modifications to consider the height of buildings resting on slopes in calculating the lateral forces, but do not discuss the ambiguity in the shear force distribution that is inevitable at the shorter column.

The parameters responsible for their ill behaviour must be well understood to improve the safety of hill buildings. Therefore, a methodology is formulated for understanding the effect of varying building dimensions on (i) design stress ratios, (ii) dynamic response, i.e., drifts, and (iii) dynamic characteristics, i.e., modal properties. Correlation matrices are plotted to identify the parameters influencing the behaviour. Further, nonlinear dynamic analysis is performed using a reference structure to detect the failure pattern. It is observed that the predominant failure is due to shear in all uphill columns, followed by the yielding of an immediate story. Based on the parameters identified, a framework is proposed to (i) restrict the shear failure in the uphill columns and (ii) improve the base shear distribution, flexural deformations, and modal properties along and across the valley. A similar nonlinear analysis is performed to confirm the improvement in the behaviour of buildings resting on slopes.

Chapter 1 Introduction and Literature Review

1.1 Introduction

In the past decade, India witnessed a surge in concentrated urban growth to manifolds, and the North-eastern region, which has scenic beauty, is no exception. This urban sprawl, especially in the north-eastern part, can be attributed to increased population, recognition as a tourist destination, pleasant climatic conditions, etc. Urban growth has increased the construction of multistorey reinforced concrete buildings on mild to steep slopes with weak soil underneath. Past earthquakes, i.e., Imphal (2016), Nepal (2015), Sikkim (2011), Kashmir (2005), Chamoli (1999), and Uttarkashi (1991), provided an opportunity to understand the impending danger for the buildings located in the Himalayan region. Table 1.1 explains the PGA ranges recorded in different earthquakes and the PGA used to design buildings.

Though the observed PGAs in these earthquakes are on par with design PGAs, damages caused by these earthquakes are not proportional to observed PGA because of the following reasons: (a) landslides, (b) foundation failures because of slope instability, and (c) irregular configuration of buildings (d) poor construction practices. All these factors found coherence with reported damage during the earthquake reconnaissance survey.

In addition to the above factors, Topography is another critical factor. The Indian subcontinent is majorly divided into 4 seismic regions viz., (a) Himalayan region, (b) Andaman Nicobar Islands, (c) Kutch region, and (d) Peninsular Indian region[1]. The Himalayan region is the most vulnerable because of the well-known theory of the Indian-Eurasian plate collision. Table 1.2 shows the percentage of the geographical area of the Himalayas shared by the Indian states. 12 states of India share the Indian Himalayan Region (IHR), with Jammu and Kashmir occupying 41%. Other Indian states like Himachal Pradesh, Uttarakhand, and West Bengal hills share 10.43%, 10.02%, and 0.59%, respectively. The rest 37.21% of the geographical area is shared by eight states of the North-eastern Region (NER), notifying the importance of understanding the tectonic setup of the North-Eastern Region (NER) and studying the vulnerability of building stock located in these areas by choosing appropriate ground motions.

The Seismo-tectonic setup of northeast India is one of the most complex geological systems. Figure 1.1 explains the complex tectonic setup of northeast India. The region consists of 3 significant plates, viz., India, Eurasia, and Sunda, which interact with two convergent boundaries, i.e., the Himalayas to the north and Indo-Burman ranges to the southeast along with a variety of intraplate domains of India.

Indian and Eurasian plate convergence is well accepted, and convergence prevails from Eastern Himalayas through Mikir and Assam valleys. The E-W to ENE-WSW Eastern Himalayan collision belt includes the southeast Tibet plateau, Main Central Thrust (MCT), and Main Boundary Thrust (MBT). Also, the East-West extension of Tibet is related to this Indian-Eurasian convergence. Similarly, India-Burma ranges are under compression due to oblique convergence between Sunda and the Indian plates. The compressive stress between the plates results in rotation across the northern arc. This rotation results in a relative movement from an SSW-directed Sunda-Burma motion to a WSW-directed Burma-India motion. Assam syntaxes form in the northeast portion of the Indian plate bounded by major thrust zones like MBT-MCT to the north, Lohit-Mishmi thrusts to the northeast, Naga, Disang, and Eastern boundary thrusts to the east, Arakan Yoma belt to the southeast. Though there is a common agreement regarding convergence between the Indian and Eurasian plates, the variation in stress parameters along different directions is unknown. Hence, it is essential to study the uncertainties associated with the fault movements, though not part of the current study.

Table 1.1: Observed PGA's in past earthquakes

Earthquake	**Year**	**PGA**	**Design PGA**
Imphal	2016	0.11-0.34g	0.36g (Zone V)
Nepal*	2015	0.15-0.35g	-
Sikkim	2011	0.23-0.55g	0.24g (Zone IV)
Kashmir	2005	0.03g-0.23g	-
Chamoli	1999	0.03g-0.36g	0.36g (Zone V)
Uttarkashi	1991	0.035	0.36g (Zone V)

* Indicates epicentre is outside India territory

Table 1.2: Percentage of the geographical area of the Himalayas shared by the Indian States [2]

S.no	State	% of the geographical area in the Himalayan region
1	Jammu & Kashmir	41.65
2	Arunachal Pradesh	15.69
3	Himachal Pradesh	10.43
4	Uttarakhand	10.02
5	Meghalaya	4.20
6	Manipur	4.18
7	Mizoram	3.95
8	Nagaland	3.11
9	Assam Hills	2.87
10	Tripura	1.97
11	Sikkim	1.33
12	West Bengal Hills	0.59

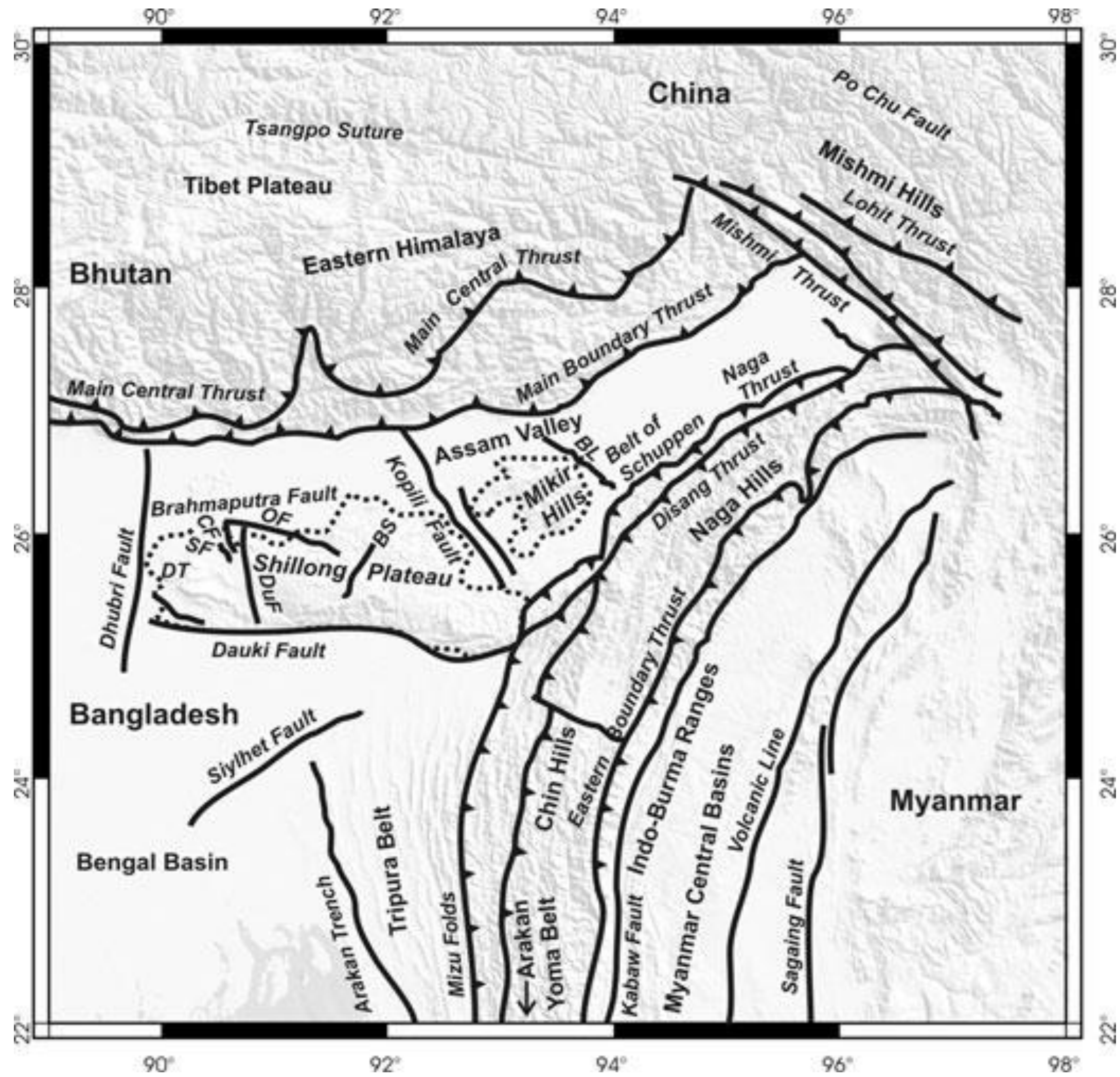


Figure 1.1: Seismo-tectonic setup of the north-eastern region in India [3]

1.2 Building Catalogues in the Hilly Regions

As per the Planning Commission report [4], out of 32.78 lakh sq. km of land, 7.08 lakh sq. km falls under hilly districts. The Hilly region is called a district when more than 50 percent of the geographical area falls in a hill taluka [4]. Table 1.3 shows the proportion of land under hilly terrain for different states. Except for Assam, all the Northern-Eastern states have a complete proportion of land under hilly terrain. Table 1.4 shows the typical reinforced concrete building catalogs in the north-eastern region.

Table 1.3: Portion of land under hilly terrain [4]

S.no	State	Geographical Area (2009)	Geographical Hill Districts (2009)	The proportion of Land under Hilly Terrain
1	Arunachal Pradesh	83,743	83,743	1.00
2	Assam	78,438	19,153	0.24
3	Manipur	22,237	22,237	1.00
4	Meghalaya	22,429	22,429	1.00
5	Mizoram	21,081	21,081	1.00
6	Nagaland	16,579	16,579	1.00
7	Sikkim	7,096	7,096	1.00
8	Tripura	10,486	10,486	1.00

Table 1.4: Building catalogue in Hilly regions

S. No	Image of the building	Characteristics
1	 *Image Source:* [5]	***Stepped building***: This type of configuration is preferred for low to medium slopes. This configuration causes foundation instability when implemented for steep slopes. This is the most common building catalog.

S. No	Image of the building	Characteristics
2	 *Image Source:* [5]	***Split-foundation buildings:*** In this type of building, the foundation is split into different levels. Such kind of configuration is implemented when slopes are steep.
3	 *Image Source:* [5]	***Step-Set building:*** Buildings are similar to stepped buildings apart from the additional setback. It is preferred when the building is large and heavy. It is believed if set back is properly utilized, it will reduce the instability.
4	 *Image source:* [6]	***Geometric irregular structure:*** A stepped Building possesses vertical geometric Irregularity.
5	 *Image source:*[6]	***Floating column structure:*** Some of the columns are abruptly starting from the second floor. Also, the building is located on a hill slope.

S. No	Image of the building	Characteristics
6	 *Image source:* [6]	***Open ground story structure:*** The building does not have infill walls on the ground floor to facilitate car parking. Also, the building is located on a hill slope.

Most of the states in the North-Eastern region have similar seismicity, built environment, building byelaws, and construction practices. The building typologies in the North-Eastern region consist of reinforced concrete, brick masonry, and other traditional constructions, depending on the available local materials. With reinforced-concrete construction techniques, irregular multi-storied buildings are constructed in these hilly regions following the design procedures applicable to flat-ground buildings. This practice may not be correct since the sloped building design requirements and assumptions differ. Many irregular structural configurations exist in the hilly areas, i.e., stepped buildings, sloped buildings, stepback-setback buildings, split foundation buildings, etc. The current work focuses on buildings constructed based on the natural topography, i.e., slopped buildings constructed on mild to moderate slopes.

1.3 Failure of Buildings in Hilly Regions During Past Earthquakes

Earthquakes are not new, and they have been occurring for the past many centuries. Regional catalogs provide more significant insights into past earthquakes that occurred in India. From the listed regions, earthquakes in the Himalayan region are the focus of the current study. Table 1.5 shows the list of significant earthquakes in the region chronologically.

Each earthquake provides an opportunity to improve the existing design and construction practices. Though every earthquake played a crucial role in exposing the vulnerability of building stock in a hilly region, the 21st century Sikkim earthquake

(2011) is the differentiator for understanding the damage pattern. Later, the Nepal earthquake (2015) further intensified the fear regarding the safety of the built environment in the hilly region. Some unanticipated damages noticed in the hilly areas are described below by dividing the damages into the following categories.

Crushing of Ground Story: Crushing of ground story is one of the significant damages noticed in the Nepal (2015) earthquake. The probable reason for such failure would be the weak stories below the road level. Figure 1.2 shows a few such losses reported in the Nepal (2015) earthquake and Sikkim (2011) earthquake.

Table 1.5: List of the earthquakes that occurred in the Himalayan region of India and Nepal

S.no	Earthquake	Year	Magnitude	Damage
1.	Kashmir Earthquake	1885	-	-
2.	Shillong Earthquake	1897	8.7	-
3.	Kangra Earthquake	1905	8.5	> 1,00,000 buildings damaged [7]
4.	Bihar-Nepal Earthquake	1934	8.3	In this earthquake, the towns of Monghyr in India and Bhatgaon in Nepal were entirely in ruins, large parts of the cities of Motihari, Muzaffarpur, and Darbhanga in India, and Patna and Kathmandu in Nepal suffered severe damage [8]
5.	Assam Earthquake	1950	8.5	12,000 buildings and 2,000 granaries were damaged [9]
6.	Bihar-Nepal Earthquake	1988	6.5	1,50,000 buildings were damaged [10]
8.	Uttarkashi Earthquake	1991	7.0	> 40,000 buildings were damaged [11]
9.	Chamoli Earthquake	1999	6.8	About 2,500 buildings collapsed, and 10,800 were partially damaged [12]
10.	Kashmir Earthquake	2005	7.6	> 4,50,000 damaged in Pakistan [13]
11	Sikkim Earthquake	2011	6.9	95,000 buildings have been entirely, partially, or severely damaged [14]
12	Nepal Earthquake	2015	7.8	> 5,00,000 buildings damaged [15]
13	Imphal Earthquake	2016	6.7	Extensive damage: 70 Moderate damage: 2,000 [16]

(a) (b)

Figure 1.2: (a) Crushing of ground storey witnessed during (a) *2015 Nepal*[17], (b) 2011 Sikkim *earthquake*[18].

(a) (b)

Figure 1.3: Crushing of intermediate storey observed during (a) 2011 Sikkim earthquake[14] *and (b) 2015 Nepal earthquake*[17].

Crushing of Intermediate Story: Crushing of intermediate stories is among the classic damages observed during past events. In this damage category, the ground storey collapses, but other stories remain intact. Figure 1.3 shows a few typical examples of crushing of intermediate storey. 3(a) shows the crushing of 2nd storey during the 2011 Sikkim earthquake, and 3(b) shows the crushing of the first storey during the 2015 Nepal earthquake.

Yielding of Intermediate Story: With increasing irregularity, lower stories of buildings resting on hill slopes become rigid, and thus, the columns connecting the rigid parts may yield, as shown in Figure 1.4.

Tilting: The most significant failure reported in 2015 Nepal earthquake is excessive tilting of buildings due to loss of foundation systems, as shown in Figure 1.5.

(a) (b)

Figure 1.4: Yielding of columns observed in (a) 2015 Nepal earthquake [12] and (b) the 2016 Imphal earthquake[16]

(a) (b)

Figure 1.5: Tilting of buildings during 2015 Nepal earthquake[19]

The damages from past earthquakes provide sufficient evidence that the failure of buildings on slopes is brittle, and damage is concentrated mainly in the ground and intermediate stories without inelastic deformations. One exception is the foundation failures, where tilting with permanent deformation occurs. However, these failures may also be due to poor construction and design practices. Hence, the behaviour must be understood in detail with sufficient analytical justification.

1.4 Literature Review on Sloped Buildings

Literature on the behaviour of buildings on slopes is limited. However, the available literature can be divided into two groups viz., (a) Pre-Sikkim studies and (b) post-Sikkim studies. While the pre-Sikkim studies primarily focus on the influence of different parameters, post-Sikkim studies focus on the vulnerability assessment of building stock. Thereby, understanding the actual behaviour of sloped buildings gained importance.

1.4.1 BEHAVIOUR STUDIES

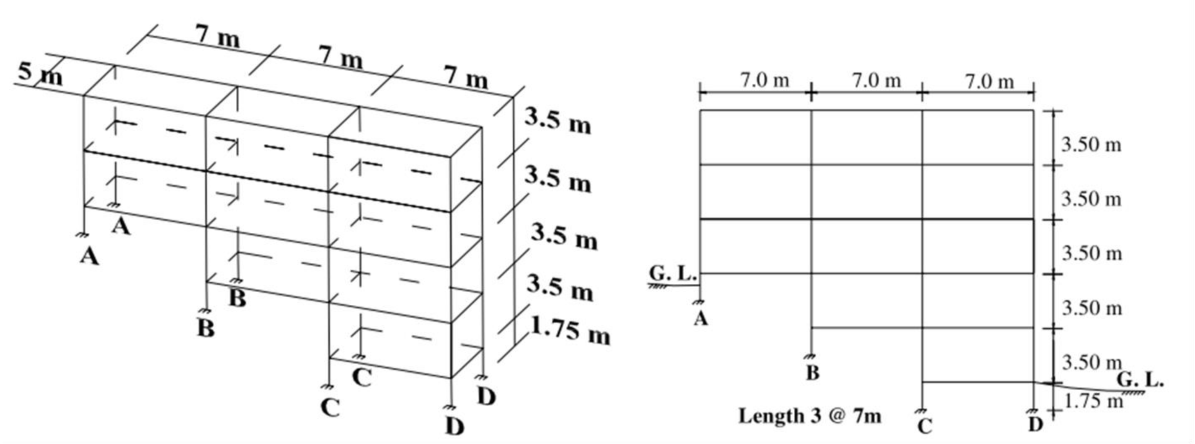
Birajdar and Nalawade (2004) [20] studied the performance of 3 building categories, viz., (a) step-back buildings of different heights resting (4-11 storeys) on sloped ground, (b) step back- set back buildings of different heights (4-11 storeys) resting on sloped ground and (c) step back-set back buildings of different heights (4-11 stories) resting on flat ground as shown in Figure 1.6. The dynamic response of the buildings is represented as normalized base shear, natural period (T), and top roof displacements and compared along and across the configurations to arrive at the suitable configuration. It is observed that (a) the performance of step-back buildings is more vulnerable than other building configurations, the reason being higher torsional moments, and (b) Shear actions induced in step-back buildings are moderately higher as compared to step-back-stake buildings. Therefore, if the cost of cutting the sloping grounds is within acceptable limits, setback buildings are suitable compared to step-back-setback buildings. However, the effect of variation of support conditions on the dynamic response is unanswered, in addition to the discussion on damage corresponding to the nonlinear response of the structure.

Absolute attention to understanding the behaviour of sloped buildings was given only after the 2011 Sikkim earthquake. *Singh et al. (2012)* [21] studied the behaviour of sloped buildings shown in Figure 1.7. The first building, Type S-1, is a back building resting on 45^o upto 6 storey and has 3 stories above road level. The second building, Type S-II, is a stepping-back building on the sixth floor only and has three stories above the road level. The 9 and 3-storeyed regular buildings on flat ground are labelled as 'Type P-III' and 'Type P-IV', respectively. The focus of the study is to correlate the damages observed during the 2011 Sikkim earthquake. Analytical studies are used to understand the dynamic response of sloped buildings and compare them with that of buildings on flat ground in terms of the natural period of building, inter-story drift pattern, column shear, and of particular interest is the plastic hinge formation. It is observed that hill buildings have significantly different dynamic properties when compared to flat buildings.

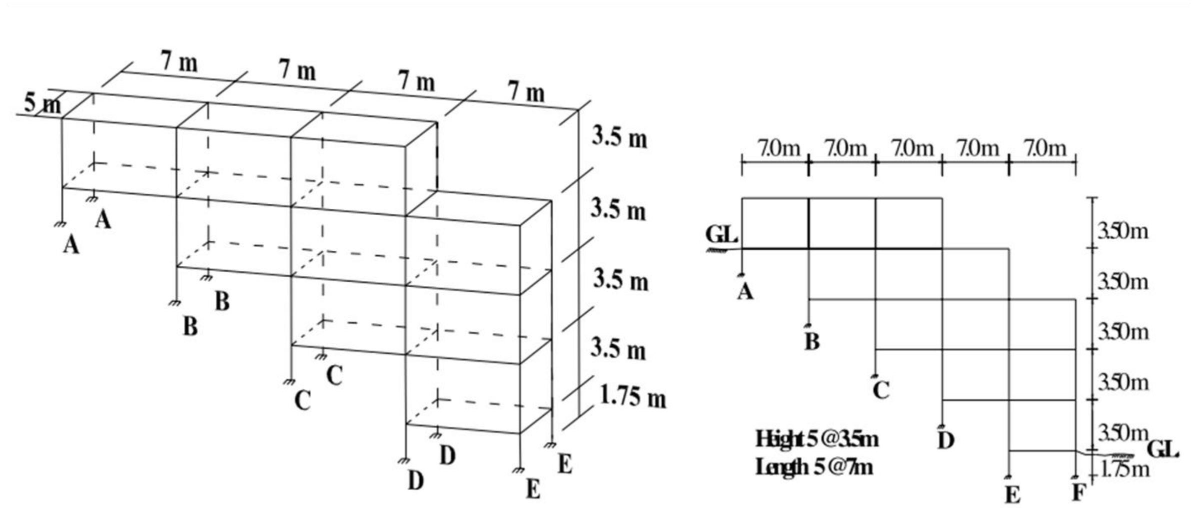
Figure 1.8 shows the hinge pattern observed in hill buildings of type S-I subjected to earthquake excitation independently along and across the valley. In type S-I configuration, most damage is concentrated in the top three stories, and the hinge pattern develops a mechanism indicating collapse. The columns on the rigid side exceeded the collapse limit state, and in stories below the road, level hinges are developed only in short columns and adjacent beams.

Figure 1.9 shows the hinge pattern observed in hill buildings of type S-II subjected to earthquake excitation independently along and across the valley. In type S-II configuration subjected to a long-the-valley excitation, hinges are developed in beams at all levels, columns at the base, and road level. Similarly, under the slope excitation, all the elements in the shorter frame reach the collapse limit state.

From the study, the authors concluded that the hill buildings are subjected to significant torsional effects across slope excitation. Under the slope excitation, the varying heights of columns cause stiffness irregularity, and the short columns attract maximum shear force. The authors justified the analytical finding with the damage pattern noticed in 10 RC framed buildings that collapsed during the Sikkim earthquake.



(a)



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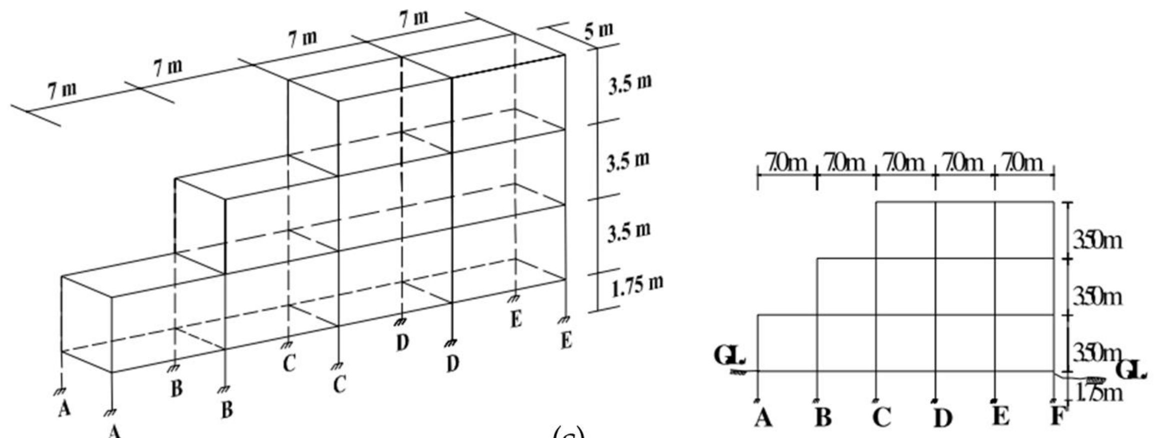


(c)

Figure 1.6: Building models (a) STEP BACK building on the sloped ground (b) STEP BACK-SET BACK building on the sloped ground (c) STEP BACK-SET BACK buildings on flat ground considered by Birajdar and Nalawade (2004)

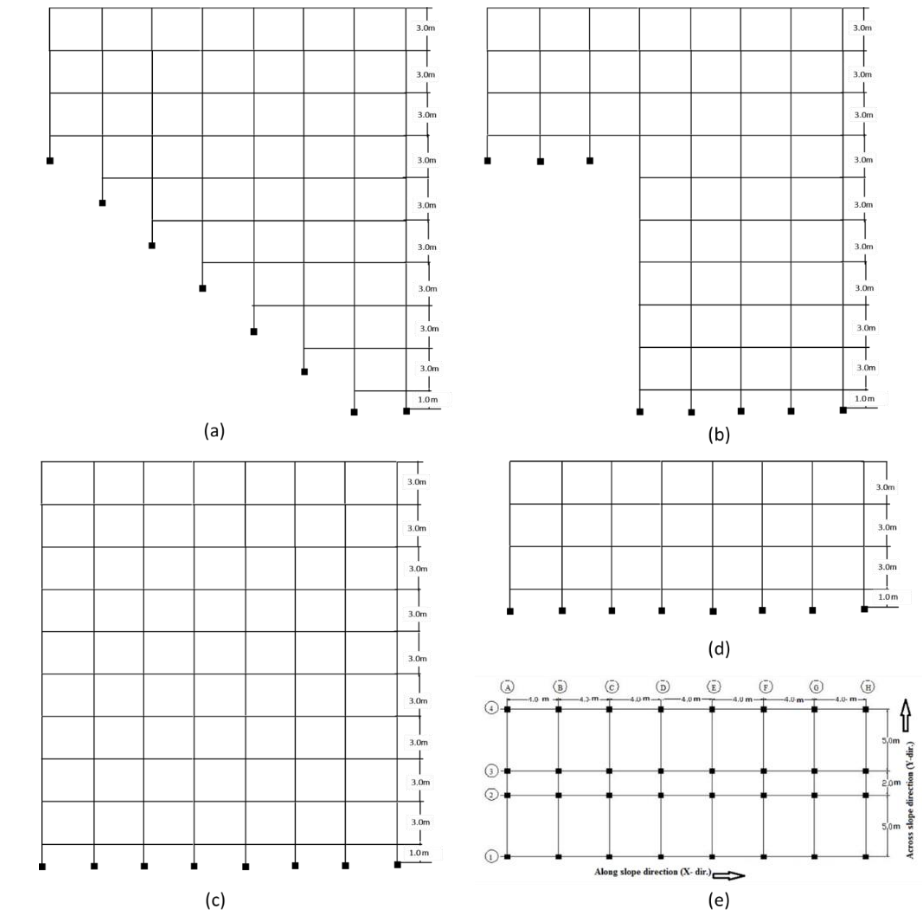


Figure 1.7: Buildings considered with (a) Elevation Type S-1 (b) Elevation Type S-II (c) Elevation Type P-III (d) Elevation Type P-IV (e) Plan of all the models

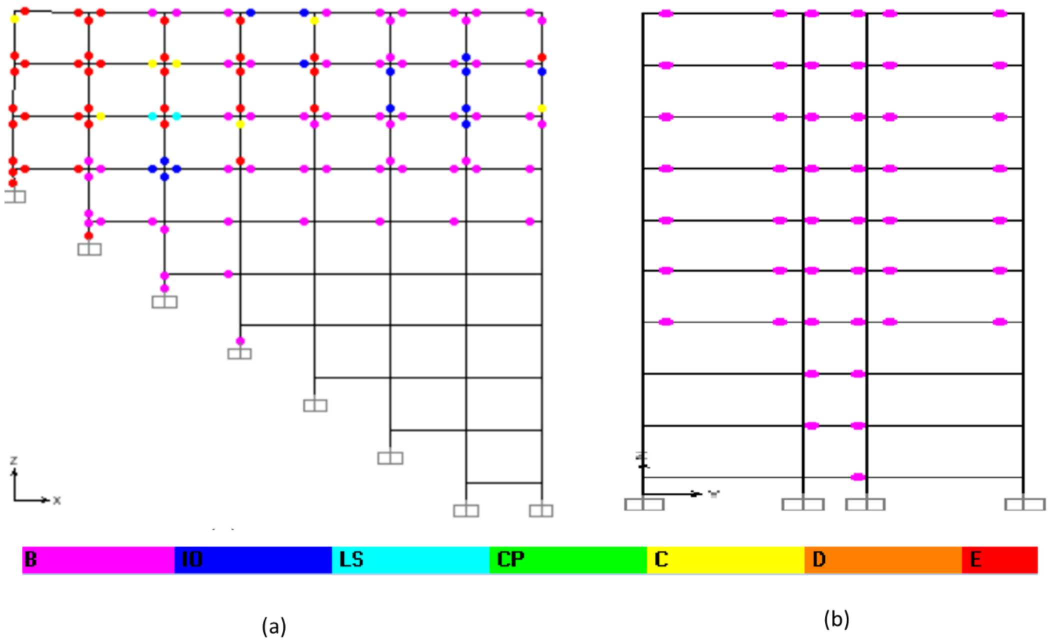
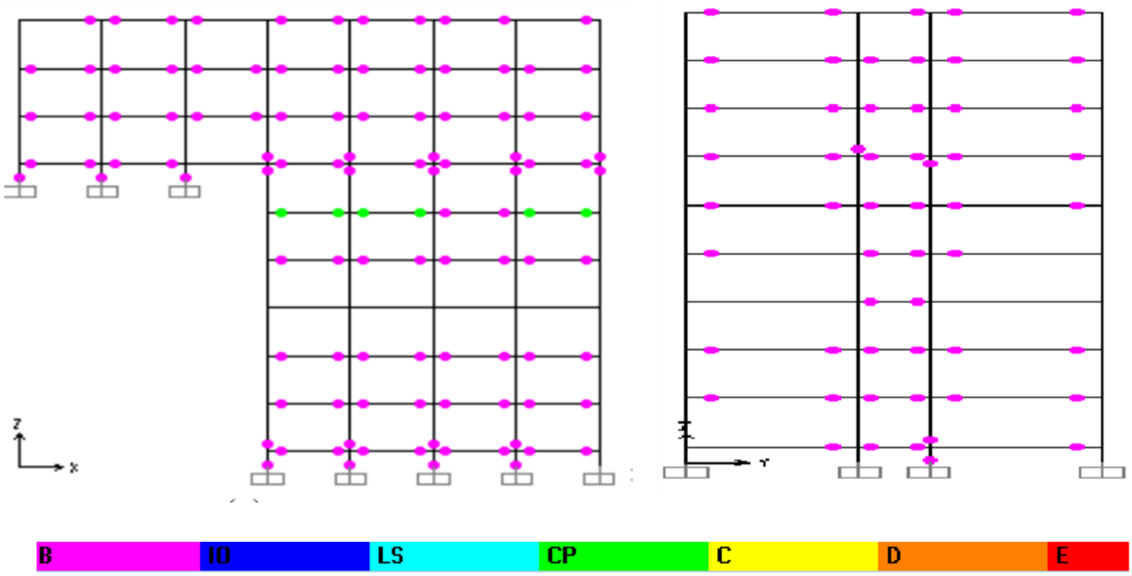


Figure 1.8: Hinge pattern of hill building configuration of Type S-I (a)along slope excitation (b) across slope excitation reported by Singh et al. (2012)



(a) (b)

Figure 1.9: Hinge pattern of hill building configuration of Type S-II (a) along slope excitation (b) across slope excitation reported by Singh et al. (2012)

Narayanan et al. (2012) [22] studied the performance of buildings with varying support conditions and concluded that (a) dynamic properties vary with support conditions, (b) the stability of buildings on slopes depends on the number of stories (c) buildings that have short plan length along slope performed better than that of long length in both valley direction and ridge direction.

Sreerama and Ramancharla (2013) [23] studied the behavior of G+3 story buildings by varying slope angles at 15^0, 30^0, $45^{0,}$ and 60^0 and compared the results with regular buildings. From the study, it is observed that as the slope increases, the building becomes stiffer on one side, and short columns on the uphill side also attract more shear force when compared to downhill columns.

Daniel and Sivakamasundari (2014) [24] studied the dynamic characteristics of buildings on slopes with that of traditional buildings having equal mass in terms of natural period, mode shape, mass participation ratio, deflected shape, and base shear. It is concluded that the sloped buildings have significantly different dynamic characteristics.

1.4.2 VULNERABILITY STUDIES

Surana et al. (2017) [5] studied seismic characterization and vulnerability of building stock in the hilly region. A field study is conducted in two cities, and a building stock inventory is prepared. The field studies identified six-building configurations based on foundation arrangement from the field studies. Fifty-seven building typologies were assigned alpha-numeric strings to generalize the applicability to other hilly regions. Later, fragility curves for common building typologies were developed using nonlinear analysis.

Huggins et al. (2017) [25] studied the performance of stepped buildings in Aizwal using Incremental Dynamic Analysis (IDA). It is observed from their numerical analysis that structural failure begins with axial failure followed by shear failure of base columns at road level followed by failure in down slope columns. Also, increasing the column dimension and transverse reinforcement bars increases the collapse margin ratio.

Surana et al. (2018)[26]studied the fragility analysis of hillside buildings designed according to modern codes. Collapse fragility curves were developed by considering building height, seismic zones, and near-field field ground motions for flat land buildings, split foundations, and step-back buildings. It is concluded that failure is mainly in flexure for flat land buildings. For sloped buildings, collapse occurs as a combination of shear in short columns and flexural in beams and columns in the story above the uppermost foundation level.

Surana et al. (2020)[27] studied the evaluation of damage probability matrices for hillside buildings using three different intensity measures, i.e., peak ground acceleration, spectral acceleration at fundamental building period, and average spectral acceleration over a range of periods. It is concluded that PGA overestimated the mean damage ratio compared to spectral acceleration-based intensity measures.

Agarwal and Saha (2020) studied the seismic loss estimation due to damage to structural components of buildings in hilly regions. A set of 9 RC building models are considered, which are analysed and designed according to the latest Indian codes of practice. The authors considered 3 types of configurations, i.e., (a) Flat land buildings with 2,4 and 8 stories tagged as FL models, (b) Stepped buildings with 2, 4, and 8 stories tagged as SB models, and (c) Split Foundation building models with 2, 4 and 8 stories tagged as SF models. It is observed from the study that seismic loss at the roof level is maximum for SB buildings as compared to SF and FL models. However, at the upper foundation level (UFL), SF models have experienced the highest seismic loss among all the models.

Patil and Raghunandan (2021)[28] investigated the seismic collapse of some prevalent building typologies in the Himalayan region. The authors considered 3 types of configurations, i.e., (a) flat land buildings with 2, 4, and 6 stories tagged as 2P, 4P, and 6P; (b) Stepped buildings with 2, 4, and 6 stories tagged as 2S, 4S and 6S, (c) Split foundation buildings with 2, 4 and 6 stories tagged as 2F,4F and 6F respectively. The ground slope angle is considered between $0-30^{o}$. The collapse metrics are derived by performing incremental dynamic analysis. It is identified that buildings on flat land had collapse capacities of 24-38% and 5-10% higher than stepped and split foundation buildings, respectively. From the multi-linear regression model, it is identified that the building parameters $\frac{N_b}{N_a}$ and ground slope angle (θ) significantly affect the median collapse capacity.

The existing literature can be classified into two groups, i.e., parametric studies for understanding the behaviour of sloped buildings and vulnerability studies. Limitations of the existing literature for understanding the problem of building on hill slopes comprehensively are listed in Table 1.6. Understanding the behaviour of buildings on hill slopes is more complicated when compared to irregular buildings. As mentioned in chapter 1, buildings on slopes can be termed extremely irregular structures as they have the highest probability of possessing both plan and vertical irregularities. IS 1893 does not classify buildings on slopes as a separate category due to limited research in this area.

Most of the research in this area is limited to understanding force and displacement demands on columns on the uphill side by performing static pushover analysis or nonlinear time-history analysis. This may not provide a holistic perspective for buildings located on hill slopes.

Table 1.6: Limitations of past reported studies on sloped buildings

Focus of study	Limitations
Behaviour	Studies focussed on modifying the empirical expressions for calculating the natural period of the sloped buildings. Other force-calculating parameters, i.e., R and Sa, received little or no attention.
Vulnerability assessment	Variation in support conditions is not considered.

It must be noted that building on slopes is a multi-dimensional problem, and each issue may be further divided into multiple issues. Hence, it is essential to disintegrate the problem and classify it into different parameters. In the current work, an attempt was made to identify and classify the critical issues, which would help formulate the guidelines from the perspective of buildings resting on hill slopes. The identified critical issues pertaining to structural issues are narrowed down.

1.5 Critical Issues in Buildings on Slopes

The site condition of the hilly area is the primary cause of irregularity when the building is located on a hill slope. If the natural site condition of the terrain is used and the building is constructed, there will be a compromise in the length of columns on the uphill side. The resulting configuration of the building induces irregularity of building. Sound engineering judgments are required to counter the adverse effects imposed by these structures. Hence, it is essential to have guidelines endorsed by experts in policies and assumptions. Thus, in the current study, critical issues in general for buildings on hill slopes are divided into the following categories:

(A) Site Conditions: The most basic principle in construction is the behaviour of the ground that will impact the structure's performance during an extreme event. The following issues are grouped in this category, and the relevant literature is studied:

i. Landslides
ii. Degree of slope
iii. Slope stability

A landslide is the outward and downward movement of rock mass under gravity. As Earthquake-induced landslides are catastrophic, this area needs necessary attention. National disaster management authority (NDMA) released Landslide management guidelines in 2009 (*NDMA, 2009)* [29]. A lot of instrumentation is installed, and continuous monitoring is done. Using monitored data, NDMA released maps like Landslide Hazard, Landslide susceptibility maps, etc. In this group, the degree of slope, stability, etc., requires more attention. The extent of slope and stability of the slope on which a building can be constructed requires engineering judgment.

(B) Configuration-Related Issues: It has been long acknowledged that robust structural configuration, shape, and symmetry are just as crucial as actual lateral strength[34]. Many failures in past earthquakes remained a direct or indirect consequence of configuration. Configuration-related problems are grouped into the following categories.

i. Plan aspect ratio
ii. Elevation aspect ratio
iii. Distribution and concentration of forces

Deshmukh and Goswamy(2018)[30]suggested using structural walls to control the stiffness irregularity in RC buildings on hill slopes. Based on building typology and ground slope, various wall configurations are examined, and the best configuration is suggested.

(C) Structural Seismic Provisions: For the kind of challenges sloped buildings impose, there is a need for seismic provisions specific to buildings on hill slopes. The following provisions need detailed investigation.

i. Natural period
ii. Force distribution
iii. Torsional coupling and Improvisation of mass participation
iv. Response Reduction factor(R)
v. Torsional sensitivity reduction solutions

vi. Topographic factor

The code emphasizes only natural period calculation from the parameters listed above, while all other parameters received no attention. *Gullapalli and Ramancharla (2019)* [31] proposed a new equation for calculating the natural period of the building. Ambient vibration studies are carried out for a few buildings in Mussoorie and observed that existing code provisions are insufficient for estimating the buildings' natural period, which is resting on hill slopes. An empirical relation was developed by performing regression analysis on 270 buildings.

Along the same lines, *Sreerama et al.(2020)*[32] proposed an empirical expression for calculating the natural period by considering 180 RC moment-resisting frames with varying floor height slopes and analyzing the models using SAP2000. S*ingh et al. (2015)*[33]studied topographic amplification factors using three seismic design codes, i.e., French code(AFPS), Italy code (ICMS), and Eurocode(EC8). They concluded that the Italian code predicts the most significant amplification factor at the ridge. *Deshmukh and Goswamy(2018)*[30]suggested that structural walls can control torsional coupling. Other parameters in this category require detailed investigation.

(D) Modeling: Modelling is an essential criterion for replicating the behaviour of buildings numerically. Various modeling assumptions must be considered for replicating the realistic behaviour of sloped structures. The following are some of the modelling assumptions.

i. Soil-Structure Interaction
ii. Infill modeling
iii. Modeling of support conditions
iv. Rigidity of diaphragm

Huggins et al. (2017)[25]studied soil-structure interaction by considering the beam on the nonlinear wrinkle foundation (BNWF) model. *Narayanan et al. (2012)*[22] studied the effect of fixity conditions on the behaviour of buildings. There is no significant emphasis on the impact of infills and beam-column rigidity on collapse probabilities.

In addition to the above parameters, another critical issue is to identify, define, and limit irregularities. Current practice is to divide the irregularities into plan and elevation irregularities. However, the essential complexity of buildings resting on hill slopes is the co-existence of plan and elevation irregularities.

To summarize, for the degree of complexity posed by buildings on hill slopes, several issues, i.e., site, configuration, seismic provisions, and modeling, need to be addressed. The focus of the current work is specific to the issues enlisted below.

a) **Configuration**: A robust structural configuration results in uniform mass and stiffness distribution in plan and elevation, and such a system possesses the required lateral strength. Such a structural system possessing requisite lateral strength and stiffness exhibits adequate deformation ductility during cycling loading. Achieving such a robust structural configuration is not just a complicated but impossible task, as the buildings resting on hill slopes follow the terrain's natural slope, leading to various irregularities. Thus, the configuration of buildings resting on hill slopes becomes a critical attribute that affects other attributes, i.e., strength, stiffness, and deformations, which need investigation.

b) **Seismic Design Provisions**: Seismic forces acting on the structure depend on the mass of the structure. Seismic design guidelines aid in

    I. Calculation of forces
    II. Distribution of forces
    III. Control of brittle forces

    The presence of irregularities hampers the calculation, distribution, and control of forces. Hence, existing guidelines tend to define, limit, and provide appropriate suggestions for countering the effects of irregularities. The provisions available are not specific to buildings on hill slopes. Hence, existing guidelines are critical in understanding the behaviour of buildings resting on slopes.

c) **Modeling:** The lack of design guidelines prompts the practitioners to apply guidelines applicable to flatland buildings. While using the guidelines, the application of assumptions valid for flatland buildings leads to unanticipated behaviour. The two fundamental assumptions that need thorough investigation are (a) 100% fixity of columns and (b) Rigid diaphragms.

1.6 Literature Review on Elevation Irregular Structures

Irregularities in terms of strength, stiffness, mass, and geometry along building height can be vertical irregular structures. A structure is said to be irregular when there is a discontinuity in the system's stiffness. Open ground story problem or soft story is a classic example of stiffness irregular structure. A structure is said to be irregular in strength when there is a discontinuity in the strength or capacity of the structure. Similarly, a structure is geometrically irregular when there is a discontinuity in the load path or geometry.

Setback buildings can be considered as irregular geometric structures. These irregularities may be present alone or in combination. A lot of research focuses on planning irregular structures compared with vertical irregularities. Even in vertical irregular structures, importance is given to setback irregularities and open-ground story problems. Some of the recent developments in elevation irregular structures are shown in Table 1.7.

Nezhad and Poursha (2015) investigated different types of vertical irregularities using NDA and explored the applicability of consecutive modal analysis. Forty irregular frames with stiffness, strength, and combined stiffness and strength are created by using modification factors. The effects of vertical Irregularity along the height are computed from Nonlinear time history analysis. They concluded that CMP and MPA methods could accurately predict the seismic demands of vertically irregular structures.

Table 1.7: State-of-art literature on elevation irregular buildings

Author	Type of irregularity		
	Mass	**Stiffness**	**Strength**
Nezhad and Poursha (2015)[35]	✓	✓	✓
Trung et al. (2010)[36]	✓	✓	✓

Fragiadakis et al. (2006)[37]	✓	✓	✓

Trung et al. (2010) investigated the seismic behaviour of vertically irregular steel frames. Three types of irregularities are considered, i.e., mass, strength, and stiffness. The effects of different types of irregularities on the seismic behaviour of buildings are investigated: (a)The strength irregularity affected the seismic behaviour of buildings significantly when compared to other irregularities, (b) The Presence of multistorey irregularities is more severe than the presence of Irregularity at the single story, and(c)Vertical irregularities placed at the bottom stories cause more severe effects than their presence at the upper stories.

Fragiadakis et al. (2006) proposed *a* methodology based on IDA for evaluating a structure with the following irregularities: a) Stiffness, b) strength, c) stiffness-strength combination, and d) mass. The authors observed that combined stiffness and strength irregularity have the largest effect among the considered irregularities. Strength irregularities come second, while mass and stiffness irregularities are the least influential.

1.7 Literature Review on Plan Irregular Structures

One of the main problems associated with plan irregular structure is torsion. Though the problem of earthquake-induced torsion in the building received attention as early as 1938, it is still not solved comprehensively. Most of the research on plan-irregular structures is performed either on single-story or multi-story models. Out of these two, single-story models are more predominant due to their simplicity. Also, many code provisions are implemented based on the torsional studies conducted on single-story models. However, in recent times, multi-story plan asymmetric models have gained significance. Therefore, it is essential to understand the existing literature on plan-irregular structures for both single-story plan asymmetric models and multi-story plan asymmetric models. Recent developments in asymmetric plan models are discussed in Table 1.8.

Özbayrak and Altun (2020) Conducted an experimental investigation on the relation between CM, CR, and diaphragm rigidity by creating slab openings. They concluded that even if mass and stiffness overlap, torsional Irregularity can be seen when rigidity is not enough.

Georgoussis and Mamoua (2018) studied the effect of mass eccentricity on the torsional response of a structure is studied. This paper addresses the issue by considering the torsional response of an asymmetric structure in relation to their behaviour when floor masses lie in the same line by adjusting an arbitrary spatial combination of mass eccentricities.

Karimiyan et al. (2013) investigated the margin of safety against the progressive collapse of symmetric and asymmetric structures by considering 5%, 10%, and 15% mass eccentricities. They concluded that the potential of collapse at both stiff and flexible edges increases with an increase in the level of asymmetry.

McCrum and Broderick (2013) conducted experimental (2D) and numerical investigation (3D) of multistorey concentrically braced plan irregular structures. The effectiveness of Eurocode 8 for low levels of mass eccentricity is investigated. They concluded that results from numerical models are valid. Eurocode 8 provisions are sufficient in terms of ductility and drifts, but they are not adequate for floor rotations.

Roy and Chakraborty (2013) studied *d*ifferent strength distribution strategies are studied, i.e., a) CV-M and b) balanced CV-CR, which is the focal point of earlier research. The study highlights the performance of both methods. Design charts for seismic demand are simplified, and a framework was developed for using the design charts.

Aziminezadand Moghadam (2009) investigated asymmetric buildings with different strength distribution strategies. They developed fragility curves with different responses, such as drift, ductility, and plastic hinge rotation models. They concluded that for rigid models, smaller strength eccentricity performs better.

Table 1.8: State-of-art literature on plan irregular structures

Author	**Eccentricity studied**			**Model used**
	(CV)	**(CR)**	**(CM)**	
Özbayrak and Altun(2020) [38]		✓	✓	3D frames
Georgoussis and Mamoua (2018) [39]			✓	3D frames
Karimiyan et al. (2013) [40]			✓	3D frames

Author	Eccentricity studied			Model used
	(CV)	(CR)	(CM)	
McCrum and Broderick (2013) [41]			✓	3D frame
Roy and Chakraborty (2013)[42]	✓	✓	✓	One story model
Aziminezadand Moghadam(2009)[43]	✓			3D frames

1.8 Estimation of the Natural Period

Estimating the fundamental period of vibration is the initial step in the seismic design and analysis of structures. The period of the building mainly depends upon mass, stiffness, story height, number of stories, etc. The building models often encounter different forms of structural irregularity and alter the fundamental time period due to irregularities. However, these aspects have been ignored in code-proposed empirical expressions, and these relations have been idealized for force-based design, yielding conservative results. The code proposed period-height relationship was initially obtained by conducting the regression analysis of experimentally determined building periods. These equations can be readily used to estimate seismic design parameters like base shear without prior knowledge of the cross-sectional dimensions of the structural members. The empirical formulae to estimate the fundamental time period first appeared in US building code ATC 3-06 (ATC 1978) [44] as

$$T = C_t H^{0.75} \tag{1}$$

Where C_t was assumed as 0.03 for reinforced concrete moment resisting frames, and height H is expressed in feet.

FEMA 223 A [45] presented the conservativeness of several equations based on the investigation of the San Fernando earthquake and proposed the equation using the Rayleigh formula mentioned to be effective for computer implementation.

$$T = 2\pi \sqrt{\frac{\sum_{i=1}^{n} w_i \delta_i^2}{g \sum_{i=1}^{n} F_i \delta_i}} \tag{2}$$

Where F_i is the seismic lateral force at level i; w_i is the gravity load assigned at level i; δ_i is the lateral displacement at level i due to forces F_i; g is the acceleration due to gravity.

The European seismic design code (EC8:2004) proposed a period-height relationship as

$$T = 0.075H^{0.75} \tag{3}$$

Similarly, the Indian standard design code (IS 1893: 2002) estimates the fundamental natural period of vibration for a moment resisting frame based on the presence of brick infill walls.

With brick infills

$$T = \frac{0.09H}{\sqrt{D}} \tag{4}$$

Where H is the height of the building, and D is the base dimension of the building in the considered direction

Without brick fills

$$T = 0.075H^{0.75} \tag{5}$$

IS 1893: 2016 underwent significant revisions in the year 2016, and one of the many changes incorporated in the draft is the definitions of height and width of the building in calculating the natural period of the building, as shown in Figure 1.10.

In the seismic design of structures, the preliminary step establishes the base shear calculated with assumed cross-section sizes and empirical natural period (T) calculation. The criteria for earthquake-resistant design of structures IS 1893:2016 recommends short column height in calculating the natural period of building.

However, several studies derived Natural period formulae from regression analysis and ambient vibration studies that consider average height as opposed to short column height suggested by IS 1893 [31], [32]. *Ramya and Ramancharla (2019)*[31] proposed an empirical expression specific to buildings on hill slopes in both along and across the valley based on the regression analysis after conducting ambient vibration studies on representative buildings.

$$T = 0.132{H_{avg}}^{1.4}(1 + \sin\theta)^{-0.63}(D)^{-0.48} \qquad \text{along valley} \tag{6}$$

$$T = 0.015H_{avg}{}^{1.24}(1 + \sin\theta)^{-0.59}(D)^{-0.29} \qquad \text{across valley} \qquad (7)$$

Sreerama et al. (2020) proposed an empirical expression outlined below:

$$T = 0.075H_{avg}{}^{0.75}(1 + \sin\theta)^{\alpha} \qquad (8)$$

Where H_{avg} is the average height of the building; θ is the slope angle of the ground; α is the slope coefficient. Regression analysis is carried out, and coefficients are determined as shown below:

$$\alpha = 3.99\theta^2 - 8.09\theta + 4.73 \qquad (9)$$

Estimating the natural period is a crucial step in the seismic design of structures according to traditional force-based design methods prescribed in codes because the dynamic behaviour of structures is incorporated with natural period T.

1.9 Analysis Methods

Methods to perform structural analysis can be broadly classified into linear and non-linear analysis. Further, both types of analysis can be either static or dynamic loading. Linear static methods ignore redistribution of internal forces, hysteretic effects, etc. The linear dynamic analysis uses an elastic response spectrum or time history of ground motions. The peak responses are obtained from multiple modes by applying a suitable modal combination technique, i.e., SRSS, CQC, etc. In the linear time history method of analysis, the structural response is obtained in the time domain with constant structural properties. In contrast, in nonlinear methods of analysis, the structural responses are obtained by varying structural properties using appropriate material, hysteretic, and degrading effects. Further, the choice of analysis method also depends on the purpose of analysis, i.e., for preliminary design and assessment, the usual practice is to adopt linear static analysis, for detailed design code suggests linear dynamic response history or time history analysis and for detailed assessment nonlinear dynamic analysis is the obvious choice.

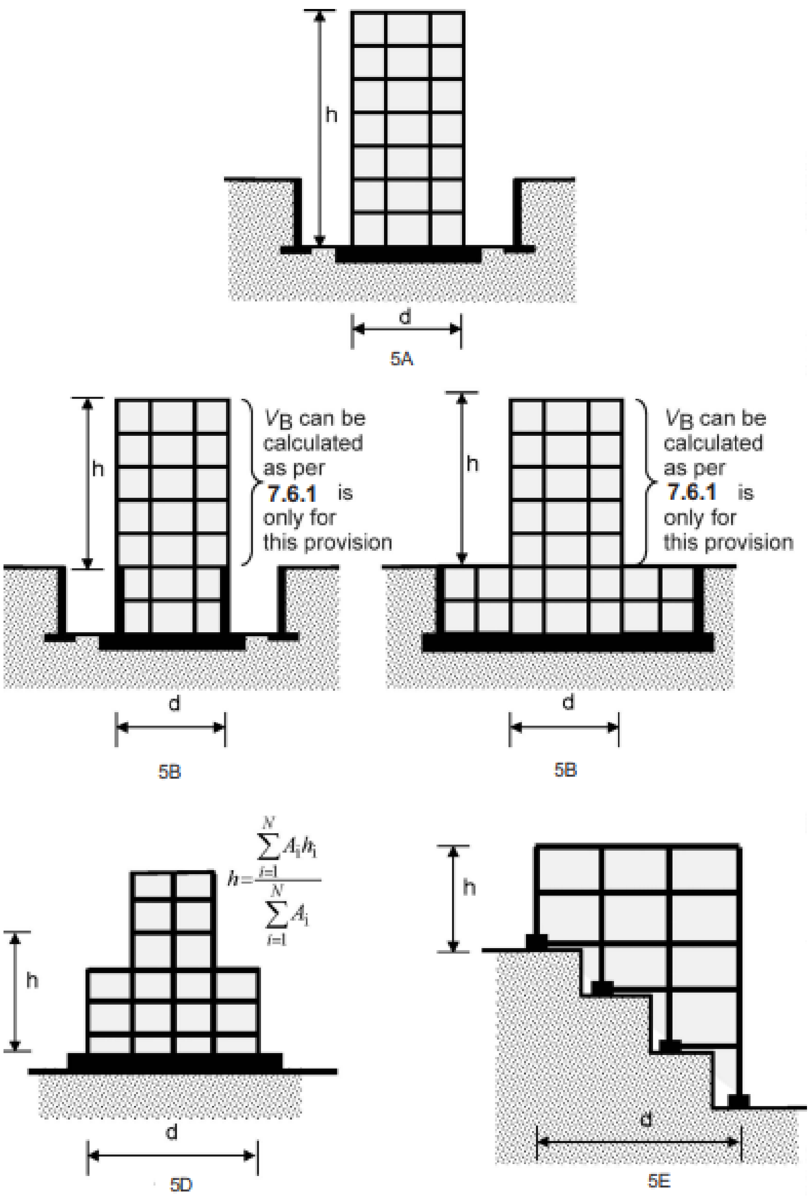


Figure 1.10: The height and width considerations prescribed in IS 1893:2016 for irregular structures.

i. **Equivalent Static Analysis:** In conventional force-based design, equivalent static analysis is a linear static analysis procedure followed for designing low-rise structures. According to IS 1893: 2016, the procedure requires the calculation of base shear by

$$V_b = A_h W = \left[\frac{ZI}{2R}\left(\frac{S_a}{g}\right)\right] W \tag{10}$$

Where A_h is the seismic coefficient, W is the weight of the structure, I is the importance factor, R is the response reduction factor, $\frac{S_a}{g}$ is the design spectrum value depending on the fundamental, translational period of the building, and Z is the seismic zone factor for which a normalized PGA value would be assigned for each zone. The base shear calculated is distributed along the height of the building. However, design codes limit the usage of equivalent static analysis for low-rise structures.

ii. **Linear Dynamic Analysis:** Linear dynamic analysis can use either the elastic response spectrum or the time history of ground motions. In linear elastic response spectrum analysis, each mode's peak responses are combined using an appropriate mode superposition technique, i.e., SRSS, CQC, etc. Similarly, in the linear time history method of analysis, the response history is evaluated in the time domain by subjecting the structural model with constant structural properties to a suite of ground motions.

iii. **Non-Linear Static Analysis:** Non-Linear static analysis requires defining a control node in the structure. The usual practice is to consider the control node at the centre of mass of the roof in the building considered. The mathematical model of the structure is prepared by incorporating nonlinearity, which can be either material or geometric to capture an inelastic response. The modeled structure is subjected to monotonically increasing loads, which can either be force-controlled or displacement-controlled, and an inelastic response is captured. This process is repeated until the structural collapse is noticed. Though the mathematical models incorporate the effects of inelastic material response and help in depicting the inelastic response with reasonable accuracy, there are several disadvantages with nonlinear static procedures. First, nonlinear static procedures do not consider the dynamic response, which varies significantly during inelastic recursions. Furthermore, the presence of structural irregularities hampers the response obtained. More robust analysis procedures are required for complex structural systems like buildings resting on hill slopes.

iv. **Non-Linear Dynamic Analysis:** Like linear dynamic analysis, non-linear dynamic analysis can be carried out using inelastic response spectrum analysis or nonlinear time history analysis. The inelastic response spectrum analysis method evaluates the response from multiple modes by defining an appropriate combination rule. In the nonlinear time history method of analysis, the response is evaluated using step-by-step time history analysis through dynamic analysis by subjecting the building model to a suite of ground motions. The response obtained from the nonlinear dynamic analysis is sensitive to various issues like nonlinear modeling, characteristics of ground motions, etc. Hence, analysis is carried out with a suite of ground motions to overcome the abovementioned uncertainties.

Incremental nonlinear dynamic time history analysis, or simply Incremental Dynamic Analysis (IDA), is another nonlinear dynamic analysis procedure for estimating the nonlinear dynamic response. The method involves subjecting the structure to ground motions scaled to multiple intensities in PGA, PGV, or PGD. Dynamic analysis is carried out with the chosen intensity measure for a predefined damage measure, i.e., for buildings, inter-story drift is a common damage measure.

A review of previous literature highlights that non-linear static procedures are adopted for regular flat land structures due to their simplicity. Most of the research works reported in buildings resting on hill slopes adopted incremental dynamic analysis.

1.10 Problem Statement

The behaviour of sloped buildings is complex because of coupled response in both the principal directions, i.e., along the valley and across the valley. The lack of guidelines for such a complex system forces the practitioners to apply assumptions/design guidelines applicable to buildings resting on flat ground. The outcome of the study proposes a framework specific to the RC MRF buildings on slopes addressing (a) Distribution of Base shear and (b) control of torsion along and across the valley, respectively.

1.11 Scope of Work

Building on slopes is a multi-dimensional problem ranging from plotting stress regimes for seismo-tectonic set up of northeast India to assessing the vulnerability of building inventory. With rapid urbanization happening in hilly terrain and no available design guidelines, there is a pressing need to find out the vulnerability of the buildings in N-E India and provide feasible design solutions that can potentially reduce the collapse of the buildings. Scope regarding buildings on the slope is not limited to vulnerability assessment of buildings, but other areas require equal and immediate attention. Hence, the scope of the study for buildings on the slope is divided into the following categories:

1.11.1 VULNERABILITY STUDIES

Risk is a function of hazard, exposure, and vulnerability. Hazard and exposure are difficult to control, and an engineer's only parameter for reducing the risk is to control vulnerability. The determination of the vulnerability of a building depends on many factors, and one such factor is typology. Various building typologies like brick masonry, stone masonry, reinforced concrete, and traditional buildings exist in northeast India, and there is a need to conduct a vulnerability study on these buildings. To study the vulnerability of the buildings, the appropriate grouping has to be done. Many studies have grouped the building inventory based on the height, age of construction, etc. Also, 2D models are used in generating the fragility curves. For irregular structures, especially buildings on slopes, 2D modeling will not suffice the need, as torsional response occurs when the plan aspect ratio is changed. During the Nepal earthquake, it was noticed that the building attained permanent displacement without cracks in infill walls. This is not the intended behaviour, notifying the importance of infill wall modeling. Though soil-structure interaction studies are essential, they are reserved for future studies. There is also immense scope for proposing quick qualitative vulnerability assessment tools.

1.11.2 *ANALYSIS*

Analysis should be viewed from two perspectives, i.e., i) for the assessment of behaviour and ii) for the incorporation of damage-induced behaviour in design. Published literature has already highlighted the challenges and possible solutions. For the current design, the codes of practice give only analysis suggestions vaguely. For irregular structures, dynamic analysis must be carried out. There is enough scope for improving the analysis suggestions.

1.11.3 *DESIGN GUIDELINES*

Guidelines are required on several critical issues highlighted below for improving the earthquake behaviour of buildings on hill slopes.

a. Natural period
b. Force distribution
c. Torsional coupling and Mass participation improvisation
d. Response Reduction factor(R)
e. Inclusion of Topographic factor

However, the scope of the current work is limited to improving the behaviour of sloped buildings by proposing a suitable framework. Within the framework, the primary focus is on damage distribution and torsion control.

1.12 Organization of Thesis

Chapter 1 introduces the problem of buildings on hill slopes, and the literature review is carried out. Existing literature is divided into two categories: behaviour studies and vulnerability studies. Existing literature is limited to buildings on slopes, but an overview of other irregularities is discussed. Finally, the scope and problem statement are defined.

Chapter 2 compares existing guidelines, and the definitions outlined in these codes are applied to sloped structures with varying slopes. The existing definitions are also verified on buildings designed according to the latest Indian standards.

In *chapter 3,* a parametric study is formulated by varying critical parameters like building footprint, the height of the building, and the slope. A study is formulated by varying the critical parameters that affect the system's response. Correlation matrices are plotted to understand how the variation of variables affects the response.

In *chapter 4, the* need for proposing a framework is demonstrated first. Four models are created using the proposed framework. Later, the best outcome model using the proposed framework is validated with nonlinear analysis.

In *chapter 5,* summary, observations, and recommendations are outlined.

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Chapter 2 Irregularity in Sloped Buildings: Definition and Limits Imposed by Codes of Practice

2.1 Irregularities

A review of building damages during past earthquakes emphasizes the importance of structural, architectural, and constructional aspects in seismic vulnerability. One of the predominant features of the structural aspects is the irregularity present in the building. Although any structural irregularity must be handled with care, a particular focus is required for buildings on slopes. However, focused literature on such buildings is scarce compared to other irregularities. In addition, the seismic behaviour of buildings on hill slopes, in most cases, is a combination of one or more irregularities, such as plan irregularity and elevation irregularity. Therefore, it is required to understand the seismic behaviour of different irregularities that significantly contribute to the failure of buildings.

Although many irregularities are associated with buildings, they affect the seismic behaviour of buildings because of their presence in lateral force-resisting systems. A structure designed to withstand lateral force is called a lateral force-resisting system. Broadly, there are three lateral force-resisting systems: moment-resisting frames, shear wall systems, and a combination of both. In any of these systems, irregularities are caused due to architectural requirements, aesthetic requirements, functional requirements, and topographical requirements. The behaviour of Irregularity associated with each structural system is unique, and the same has been realized over the past many years. Hence, they are available in the form of guidelines.

2.2 Implications of the Vertical Irregularity Defined in Codes for Buildings Resting on Hill Slopes

To counter the implications posed by vertical irregularities, the Indian standard code of practice (IS 1893:2016) and similar codes define and limit the following irregularities: (a) stiffness irregularity, (b) strength irregularity, (c) Mass irregularity, (d) Irregular modes of oscillation (e) Floating or stub column (f) In-plane discontinuity (g) Vertical geometric irregularity. Applicability of the abovementioned irregularities is generic, i.e., applicable for different irregular structures like step back, setback, etc. Hence, verifying the implications of defined irregularities for buildings resting on hill slopes is essential.

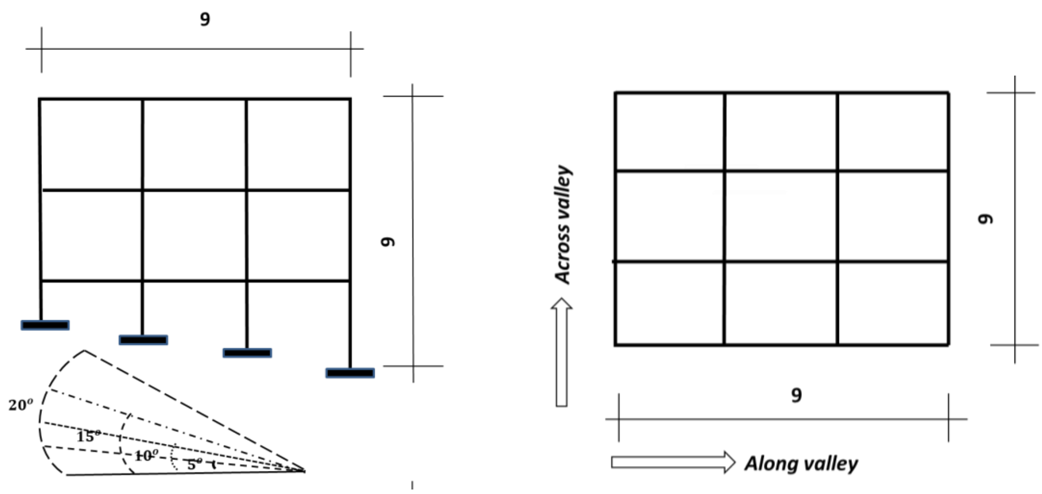


Figure 2.1: Plan and elevation details of the building considered for applying the definitions specified in codes (All dimensions in m).

Table 2.1: Building details for studying definitions outlined in codes.

Assumptions	1. All the supports are assumed to be fixed 2. Flexible diaphragms are assigned according to IS 1893:2016
Loads and load combinations	1. All the loads are assigned according to IS875 2. All the load combinations are assigned according to IS 1893:2016
Design details	**Materials:** 1.M25 grade of concrete and HYSD 415 for rebar **Cross-sections:** 2. Beams: $0.35 \times 0.35\ m$; Columns: $0.4 \times 0.35m$ **Design and Detailing:**

	Design is done according to IS456 and IS13920. However, ductile shear reinforcement is not provided in long columns as shear demands are negligible

The plan and elevation details of the building considered for applying the guidelines mentioned above are shown in Figure 2.1. The plan and elevation details of the building are kept constant, and the slope is varied from 5^o to 20^o at an equal increment of 5^o. The details of the building considered are described in Table 2.1.

2.2.1 *MASS IRREGULARITY*

The IS 1893:2016 defines the presence of mass irregularity if $M_i > 1.5M_{i+1}$ i.e., if the mass of the current story is 150% more than the previous story, mass irregularity ceases to exist. The code suggests performing dynamic analysis in case of irregular mass structures. Like the IS code, the New Zealand code also defines mass irregularity if $M_i > 1.5M_{i+1}$. Other contemporary codes, i.e., ASCE 7 and EC8, do not emphasize mass irregularity. The details of the mass irregularity definitions according to different country codes are presented in Table 2.2. A more detailed comparison of various codes is outlined in Appendix A.

Table 2.2: Mass Irregularity calculations according to standard codes of practice on the models considered.

Model ID	**Story**	M_{alv} **(kg)**	M_{acv} **(kg)**	**IS 1893:2016**		**NZS**	
				$\frac{M_{i,alv}}{M_{i+1,alv}}$	$\frac{M_{i,acv}}{M_{i+1,acv}}$	$\frac{M_{i,alv}}{M_{i+1,alv}}$	$\frac{M_{i,acv}}{M_{i+1,acv}}$
5_9_9_9	1	203576.96	203577				
	2	197538	197538	0.97	0.97	0.97	0.97
	3	109346.41	109346.40	0.55	0.55	0.55	0.55
10_9_9_9	1	200275.91	200275.9				
	2	197538	197538	0.98	0.98	0.98	0.98
	3	109346.41	109346.41	0.55	0.55	0.55	0.55
15_9_9_9	1	174203.04	174203.04				
	2	174869	174869	1.00	1.00	1.00	1.00
	3	90778.7	90778.7	0.51	0.51	0.51	0.51

Model ID	Story	M_{alv} (kg)	M_{acv} (kg)	IS 1893:2016		NZS	
				$\frac{M_{i,alv}}{M_{i+1,alv}}$	$\frac{M_{i,acv}}{M_{i+1,acv}}$	$\frac{M_{i,alv}}{M_{i+1,alv}}$	$\frac{M_{i,acv}}{M_{i+1,acv}}$
20_9_9_9	1	138321.1	138321.1				
	2	200065.3	200065.3	1.44	1.44	1.44	1.44
	3	109346.4	109346.4	0.54	0.54	0.54	0.54

Table 2.2 explains the implications of the existing mass irregularity provisions calculated along the valley and across the valley, according to IS 1893: 2016 and NZS. IS and NZS define mass irregularity when the mass of the story is 150% greater than the story below. Though the mass irregularity for the models considered is within limits, it is important to observe that with increasing slope, mass irregularity increases. The code attempts to address the mass irregularity with analysis suggestions. It suggests that mass irregular buildings located in seismic zones III, IV, and V shall be analysed using dynamic methods. Apart from the analysis mentioned above, which is mandatory for any irregular structures, no specific suggestions could improve the behaviour of irregular mass structures resting on hill slopes.

2.2.2 *STIFFNESS IRREGULARITY*

IS 1893 defines the existence of stiffness irregularity when $K_i < K_{i+1}$ i.e., the stiffness of the current story is less than the stiffness of the upper storey. ASCE 7 defines stiffness irregularity when one of the following two conditions meet.

(a) $K_i < 0.7K_{i+1}$

(b) $K_i < 0.8\left[\frac{K_{i+1}+K_{i+2}+K_{i+3}}{3}\right]$

NZS also defines stiffness irregularity like the ASCE 7 code but requires meeting one of the following three conditions.

(a) $K_i < 0.7K_{i+1}$

(b) $K_i < 0.8\left[\frac{K_{i+1}+K_{i+2}+K_{i+3}}{3}\right]$

(c) $K_i < 0.8\left[\frac{K_{i-1}+K_{i-2}+K_{i-3}}{3}\right]$

The details of the stiffness irregular calculations according to the guidelines on the considered models are outlined below. Story stiffness is directly obtained from ETABS 19 for the applied lateral load.

Table 2.3 explains the implications of the existing stiffness irregularity provisions calculated along the valley and across the valley, according to IS 1893: 2016 and ASCE 7. It is apparent from the results that the bottom story in buildings resting on hill slopes are stiffness irregular structures.

Table 2.3: Stiffness Irregularity calculations according to standard codes of practice on the models considered

Model ID	Story	K_{alv}	K_{acv}	IS 1893:2016		ASCE 7	
				$\frac{K_{i,alv}}{K_{i+1,alv}}$	$\frac{K_{i,acv}}{K_{i+1,acv}}$	$\frac{K_{i,alv}}{K_{i+1,alv}}$	$\frac{K_{i,acv}}{K_{i+1,acv}}$
5_9_9_9	1	24791.8	36512.95	0.25	0.39	0.25	0.39
	2	96776.25	92278.8	1.43	1.42	1.43	1.42
	3	67625	64602.92				
10_9_9_9	1	8349.7	25754.9	0.07	0.26	0.07	0.26
	2	107767	98842.7	1.50	1.48	1.50	1.48
	3	71711.2	66448.7				
15_9_9_9	1	166693.8	36859.87	1.27	0.30	1.27	0.30
	2	130232.2	119221.1	1.71	1.67	1.71	1.67
	3	75927.4	71263.18				
20_9_9_9	1	92327.9	29688.9	1.04	0.32	1.04	0.32
	2	88057.2	90449.2	1.08	1.24	1.08	1.24
	3	81418.6	72533				

It is also important to observe that for lower slope angles, i.e., 5^o and 10^o.The structure displays a high degree of irregularity both along the valley and across the valley directions calculated according to IS 1893 and ASCE 7. As the slope angle increases, 15^o and 20^o The stiffness irregularity is visible across the valley direction according to IS and ASCE 7 definitions. Hence, it cannot be generalized that as slope increases, stiffness irregularity increases. It depends on factors like the length of the short column, building length along the slope, first-story height, etc.

2.2.3 *STRENGTH IRREGULARITY*

IS codes define the existence of strength irregularity when $V_i < V_{i+1}$ i.e., the strength of the current story is less than the strength of the upper stories. ASCE 7 defines strength irregularity exists when $V_i < 0.8V_{i+1}$.NZS defines the strength irregularity like ASCE 7 code, i.e., $V_i < 0.8V_{i+1}$.

Table 2.4: Strength Irregularity calculations according to standard codes of practice on the models considered

Model ID	Story	V_{alv} (kN)	V_{acv} (kN)	IS 1893:2016		ASCE 7	
				$\frac{V_{i,alv}}{V_{i+1,alv}}$	$\frac{V_{i,acv}}{V_{i+1,acv}}$	$\frac{V_{i,alv}}{V_{i+1,alv}}$	$\frac{V_{i,acv}}{V_{i+1,acv}}$
5_9_9_9	0	49.87	101.42	0.11	0.22	0.11	0.22
	1	450.38	450.38	1.21	1.22	1.21	1.22
	2	370.19	370.19	2.10	2.11	2.10	2.11
	3	176.22	176.22				
10_9_9_9	0	9.29	55.33	0.02	0.12	0.02	0.12
	1	446.61	446.61	1.15	1.17	1.15	1.17
	2	385.79	385.79	2.02	2.00	2.02	2.00
	3	190.37	190.37				
15_9_9_9		40.08	25.31	0.10	0.06	0.10	0.06
	1	382.49	382.49	1.09	1.10	1.09	1.10
	2	348.96	348.96	2.01	2.03	2.01	2.03
	3	173.40	173.40				
20_9_9_9	0	28.04	23.39	0.11	0.07	0.11	0.07
	1	235.20	235.20	0.63	0.82	0.63	0.82
	2	369.18	369.18	1.90	1.93	1.90	1.93
	3	193.50	193.50				

In addition, NZS also defines strength irregularity to be extreme when$V_i < 0.68V_{i+1}$. The strength irregular calculations are obtained from ETABS 19, which are derived based on maximum storey forces. Table 2.4 explains the implications of the existing strength irregularity provisions calculated along the valley and across the valley according to IS 1893: 2016 and ASCE 7. The bottom stories, irrespective of the slope angle and definitions outlined by codes, are becoming strength irregular. For lower slope angles, i.e., 5^o and 10^o The bottom most story tends to strengthen irregularly, whereas when the slope angle increases, i.e., 20^o Strength deficiency is redistributed to upper stories. Hence, it is essential to notice that buildings on slopes are not just stiffness irregular structures.

2.2.4 GEOMETRIC IRREGULARITY

IS 1893:2016 defines the existence of geometric irregularity when $dimension_i > 1.2dimension_{i-1}$. Similarly, NZS defines geometric irregularity. $dimension_i > 1.3dimension_{i-1}$. The calculations according to both the design guidelines are outlined in Table 2.5. Table 2.4 explains the implications of the existing strength irregularity provisions calculated along the valley and across the valley according to IS 1893: 2016 and ASCE 7. With increasing slope, geometric irregularity increases due to reducing the length of columns.

2.2.5 IRREGULAR MODES OF VIBRATION

IS 1893: 2016 defines the existence of irregular modes of vibration when the following conditions are met:

a. The first three modes contribute $M_p < 0.65$ in each principal direction.
b. The building's fundamental natural period (T) is closer to each other by 10% of larger value.

Table 2.6 explains the implications of the irregular modes of vibration calculated along the valley and across the valley direction according to IS 1893: 2016. Firstly, with an increasing slope, mass participation along the valley decreases drastically, whereas the decrease is much more gradual across the valley direction. In addition, the first three modes of mass participation in each principal direction do not fall under 65% for the models considered. All the models considered exhibit closely spaced modes.

This study indicates that the variation in stiffness proportionality defined by current guidelines does not consider the slope angle and length of the short column at the first storey, due to which definitions do not capture the irregularity. For example, a 15^0 slope angle with a shorter column at the first storey may be more vulnerable than 20^0 slope angle with a relatively larger column height.

Table 2.5: Geometric Irregularity calculations according to standard codes of practice on the models considered.

Model ID	Story	D_{alv}	D_{acv}	IS 1893:2016		NZS	
				$\frac{D_{i,alv}}{D_{i-1,alv}}$	$\frac{D_{i,acv}}{D_{i-1,acv}}$	$\frac{D_{i,alv}}{D_{i-1,alv}}$	$\frac{D_{i,acv}}{D_{i-1,acv}}$
5_9_9_9	1	9	9	1	1	1	1
	2	9	9	1	1	1	1
	3	9	9	1	1	1	1
10_9_9_9	1	9	9	1	1	1	1
	2	9	9	1	1	1	1
	3	9	9	1	1	1	1
15_9_9_9	1	9	9	1	1	1	1
	2	9	9	1	1	1	1
	3	9	9	1	1	1	1
20_9_9_9	1	6	9	1.5	1	1.5	1
	2	9	9	1	1	1	1
	3	9	9	1	1	1	1

2.3 Implication of Plan Irregularity Defined in Codes for Buildings Resting on Hill Slopes

To counter the implications posed by plan irregularities, the Indian standard code of practice (IS 1893:2016) and similar codes define and limit the following irregularities: (a) Torsional irregularity, (b) Re-entrant corners, (c) Floor slabs having excessive openings or cut-outs (d) Out of plane offsets in vertical elements (e) non-parallel lateral force system. Of all the parameters listed above, implications of the torsional irregularity need to be verified for buildings resting on hill slopes.

The plan and elevation details of the building considered for applying the codal provisions are shown in Figure 2.1. The plan does not contain re-entrant corners, floor slabs having excessive openings or cut-outs, out-of-plane offsets, or non-parallel lateral force resistance systems and hence omitted from comparison.

Table 2.6: Irregular modes of vibration calculations according to standard codes of practice

Model ID	**Mode**	**Direction**	$M_{p,i}$	$\sum_{i=1}^{3} M_{p,i}$	T_i	**IS 1893:2016**	
						M_p	$M_{spacing}$
5_9_9_9	1	Along valley	0.75	0.99	0.46	0.99	1.29
	2	Across valley	0.78	0.99	0.45	0.99	
10_9_9_9	1	Along valley	0.66	0.99	0.42	0.99	4.22
	2	Across valley	0.71	0.99	0.40	0.99	
15_9_9_9	1	Along valley	0.58	0.99	0.33	0.99	4.42
	2	Across valley	0.60	0.99	0.32	0.99	
20_9_9_9	1	Along valley	0.57	0.99	0.35	0.99	6.46
	2	Across valley	0.67	0.91	0.33	0.91	

2.3.1 *TORSIONAL STRENGTH IRREGULARITY*

IS 1893:2016 Code defines the existence of torsion when the following conditions occur:

a. $\Delta_{max} < 1.4\,\Delta_{avg}$
b. $\Delta_{max} > 1.2\,\Delta_{avg}$
c. $\Delta_{max} > 1.4\,\Delta_{avg}$

ASCE 7 defines torsional irregularity when

a. $\Delta_{ends} > 1.2\Delta_{avg}$

b. $\Delta_{ends} > 1.4\Delta_{avg}, Torsion\ is\ extreme$

The details of the Torsional irregularity calculations according to the codal provision on the considered models are shown in Table 2.7.

(a) Across the valley:

Table 2.7 explains the implications of the torsional strength irregularity provisions calculated along and across the valley direction according to IS 1893: 2016 and ASCE 7. Except for the low slope angle of 5^o, all other slope angles, i.e., 10^o, 15^o and 20^o Torsional irregularity is predominant in the valley direction.

Table 2.7: Torsional irregularity calculations according to standard codes of practice on the models considered across the valley.

Model ID	Nodes	U_x	U_y	Δ_{avg}	Δ_{max}	Δ_{min}	IS1893	ASCE7
5_9_9_9	27	1.272	3.249	3.905	4.561	3.249	1.40	1.16
	38	1.272	3.249					
	75	1.272	4.561					
	86	1.272	4.561					
10_9_9_9	27	1.356	2.411	3.199	3.988	2.411	1.65	1.24
	38	1.356	2.411					
	75	1.356	3.988					
	86	1.356	3.988					
15_9_9_9	27	0.803	1.645	2.089	2.534	1.645	1.54	1.21
	38	0.803	1.645					
	75	0.803	2.534					
	86	0.803	2.534					
20_9_9_9	27	1.117	1.398	2.083	2.768	1.398	1.97	1.32
	38	1.117	1.398					
	75	1.117	2.768					
	86	1.117	2.768					

(b) Along the valley:

Torsion irregularity is not significant along the valley, as shown in Table 2.8.

Table 2.8: Torsional irregularity calculations according to standard codes of practice on the models considered along the valley.

Model ID	Nodes	U_x	U_y	Δ_{avg}	Δ_{max}	Δ_{min}	IS1893	ASCE7
5_9_9_9	27	3.94	0.328	0.354	0.381	0.328	1.16	1.07
	38	3.94	0.328					
	75	3.94	0.381					
	86	3.94	0.381					
10_9_9_9	27	3.167	0.248	0.287	0.326	0.248	1.31	1.13
	38	3.167	0.248					
	75	3.167	0.326					
	86	3.167	0.326					
15_9_9_9	27	2.003	0.15	0.169	0.188	0.150	1.25	1.11
	38	2.003	0.15					
	75	2.003	0.188					
	86	2.003	0.188					
20_9_9_9	27	1.117	0.109	0.131	0.153	0.109	1.40	1.16
	38	1.117	0.109					
	75	1.117	0.153					
	86	1.117	0.153					

Force calculation problem
1. Time period
2. R factor
3. Amplification factor

Damage distribution problem
1. Distribution of lateral forces
2. Localized damages

Behavior control problem
1. Torsional control
2. Torsional sensitivity reduction solutions

Figure 2.2: Categorizing the problem of buildings on hill slopes.

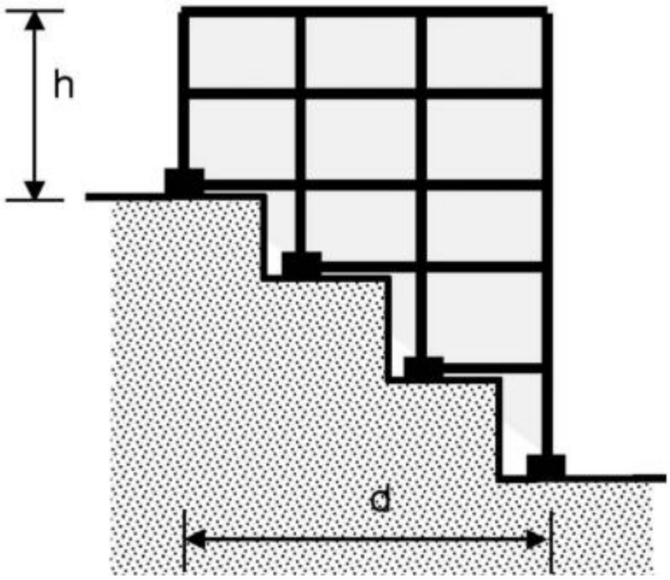


Figure 2.3: Definition of height and width for Natural period calculation according to IS 1893:2016

2.4 Discussion on Design Parameters That Enhance the Behaviour of Buildings Resting on Hill Slopes

Irregularity is a complex phenomenon that modifies the natural properties of structures substantially. Hence, Irregularity is simplified by design guidelines into two types, i.e., plan irregularity and elevation irregularity. These irregularities are well documented and explained for irregular flat land buildings. The nature of buildings on slopes is the interaction of irregularities simultaneously so that the adverse effects get pronounced. To improve the behaviour of these structures, it is critical not just to identify the issues but classify them to solve the problem. As listed in section 4.2, the critical issues for improving the guidelines are:

a. Natural period
b. Force distribution
c. Torsional coupling
d. Response Reduction factor(R)
e. Torsional sensitivity reduction solutions
f. Inclusion of Topographic factor

It also emphasizes the need to understand force flow during the design stage. Hence, the parameters identified are classified into the following problem categories.

(a) Force calculation problem

(b) Damage distribution problem

(c) Behaviour control problem

The approach of code is predominantly a force calculation problem. Hence, it suggests considering the shorter column height of the building for the calculation of base shear, as shown in Figure 2.3.

The research progressed in the same direction, and *Gullapalli and Ramancharla (2019)* [31] proposed a new equation based on regression analysis for calculating the natural period of the building. Later, *Sreerama et al. (2020)* [43] proposed another equation based on regression analysis. A comparison of the Time period and base shear is shown in Table 2.9.

Table 2.9: Natural Period and Base Shear Calculation

Model_ID	T sec Code	T sec (*Gullapalli and Ramancharla (2019)*	T sec (*Sreerama et al* (2020)	Base Shear(kN) (code)	Base shear(kN) (*Gullapalli and Ramancharla (2019)*	Base shear(kN) *Sreerama et al (2020)*
5_9_9_9	0.2463	0.5385	0.5951	450.53	450.53	411.83
10_9_9_9	0.2223	0.3559	0.6137	447.61	447.61	396.78
15_9_9_9	0.1975	0.2751	0.6343	388.21	388.21	332.94
20_9_9_9	0.1717	0.2208	0.6487	395.16	395.11	331.06

Table 2.10: Natural period calculation along and across the slope

Model_ID	T sec Code	T sec (*Gullapalli and Ramancharla*		T sec *Sreerama et al (2020)*	Mode_Tx (Along valley)	Mode_Ty (Along ridge)
		Along valley	Along ridge			
5_9_9_9	0.2463	0.5383	0.6377	0.5288	0.456	0.462
10_9_9_9	0.2223	0.3559	0.4369	0.6307	0.409	0.427
15_9_9_9	0.1975	0.2751	0.3456	0.6797	0.327	0.348
20_9_9_9	0.1717	0.2142	0.2756	0.6602	0.334	0.359

Table 2.11: % difference in modal periods calculated after design using various equations

Model ID	**Code** (% diff from Tmode)	**(*Gullapalli and Ramancharla* (% diff from Tmode)**		***Sreerama et al. (2020)* (% diff from Tmode)**
5_9_9_9	45.98(-)	16.6 (+)	34.45(+)	30.5(+)
10_9_9_9	45.64(-)	12.98(-)	0.07(+)	50.04(+)
15_9_9_9	39.60(-)	15.84(-)	2.64(-)	93.97(+)
20_9_9_9	48.59(-)	33.89(-)	18.9(-)	93.95(+)

The natural period proposed by (Gullapalli and Ramancharla (2019)[31] matched the modal periods. The percentage difference with modal periods calculated after the design is listed in Table 2.11.

The natural period proposed by *Gullapalli and Ramancharla (2019)* [31] matches with numerical periods by approximately 15% conservatively for mild slopes, whereas for higher slopes, the difference increases by about 33%. Another advantage of the model proposed by *Gullapalli and Ramancharla (2019)* is that it can capture the variation efficiently across the valley direction without losing the conservativeness in calculating the base shear.

2.5 Proposal for Problem Classification

(a) From Table 2.9 and Table 2.11, though the % difference in the calculation of the natural period according to the equations proposed are as high as 93.97%, the corresponding % difference in estimated base shear is only 14.23%.

(b) The ratios of the base shear of the structure estimated according to the code suggested equation to the base shear attracted under short columns outlined in Table 2.12.

Table 2.12: % of base shear attracted by short and long column

Model ID	$\boldsymbol{V_B}$ (kN)	$\boldsymbol{\frac{V_{S,col}}{V_B}}$	$\boldsymbol{\frac{V_{l,col}}{V_B}}$

5_9_9_9	450.53	36.5%	10.50%
10_9_9_9	447.61	65.67%	1.20%
15_9_9_9	388.21	115.18%	9.13%
20_9_9_9	395.16	74.79%	7.13%

It is evident that concentrated shear demands are occurring in shorter columns and the same need to redistribute to downhill columns. Current codes do not focus on the distribution of forces. Similarly, from Table 2.7, it is apparent that torsional response is significant when the slope of the ground reduces. This, in turn, reduces the length of the column. Hence, the problem is more of shear distribution and torsion control rather than force calculation alone. Therefore, the current work classifies the problem of buildings on slopes, as shown in Figure 2..

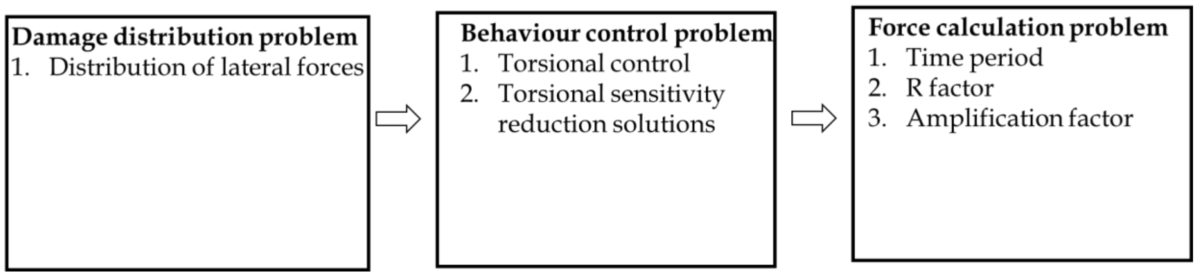


Figure 2.4: Proposed problem classification

2.6 Summary

1. Existing guidelines tend to define and limit irregularities from various parameters, *viz.*, site, architectural, structural, and construction practices. Guidelines specific to irregularities outlined in 4 national codes, i.e., IS 1893, ASCE7, NZS, EC8, are applied on a reference structure assumed to be resting on four slope angles, i.e., 5^o, 10^o, 15^o and 20^o. The complexity of buildings resting on a hill slope is such that a simple plan with a lesser slope angle, i.e.,$5-10^o$ violates the existing irregular definitions. Even before emphasizing the need for imposing limits, it is also important to provide specific definitions, i.e., in terms of (a) forces, (b) deformations, and (c) Modal

properties are needed for highly irregular structures like buildings resting on hill slopes.

2. Though definitions are violated, limits imposed by existing guidelines are not stringent. In this context, there is a need to identify parameters through which limits can be imposed.
3. The existing guidelines tend to treat the problem of building on slopes as a force calculation problem, hence the suggestion of considering the total height of the building as a short column height in calculating the natural period of the building. The actual problem of building on slopes is the concentration of shear force near the uphill column due to the short column effect, for which there are no suggestions. Hence, more emphasis is needed on damage distribution along the valley and behaviour control across the valley.

…

Chapter 3 Identification and Correlation of Parameters Influencing the Behaviour of Sloped Buildings

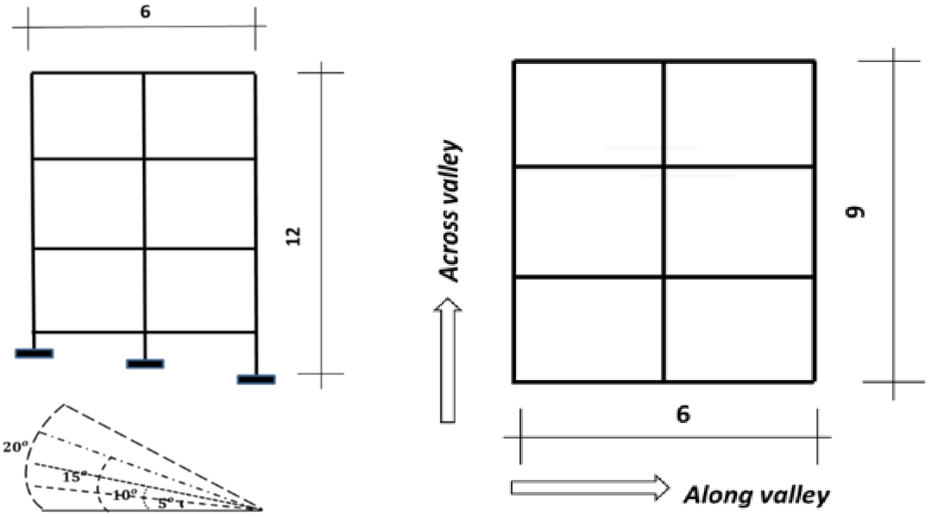
3.1 Identification of Parameters

Two parameters are considered critical in understanding the behaviour of buildings on slopes: (a) Aspect Ratio (AR), defined as the ratio of the length of the building (along the slope) to the width of the building (across the slope), and (b) Slenderness Ratio (SR), defined as the ratio of the height of the building to the width of the building along the valley. In the current study, AR is varied by modifying the building length along the valley for a constant width of 9m. This is reasonable considering the practical situation of land scarcity across the valley. Similarly, SR is varied by modifying the height of the building for a constant plan area.

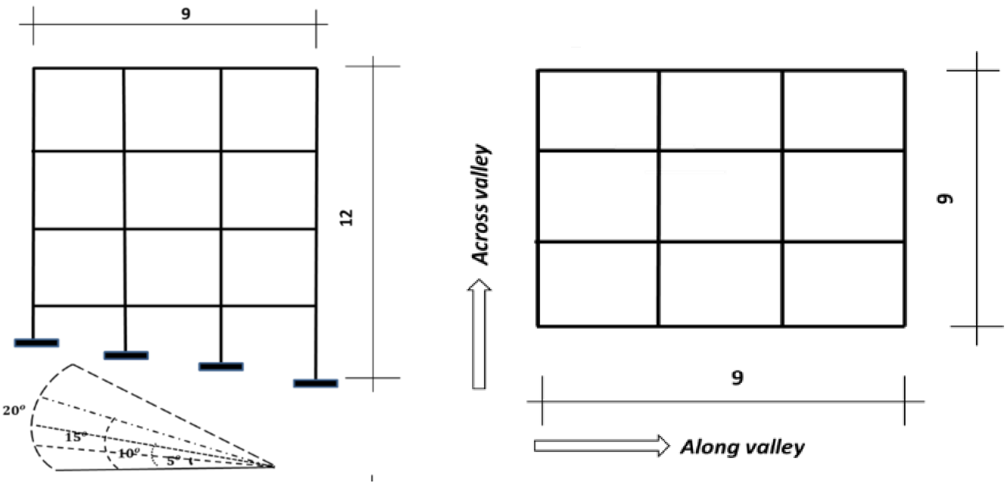
The objective of the parametric study is to understand how varying slopes with the AR and SR interact with (a) Design forces, i.e., P, V, M, T, (b) Dynamic response, i.e., drifts, and (c) Dynamic characteristics, i.e., Modal properties of buildings and develop correlation matrices.

3.2 Building Models

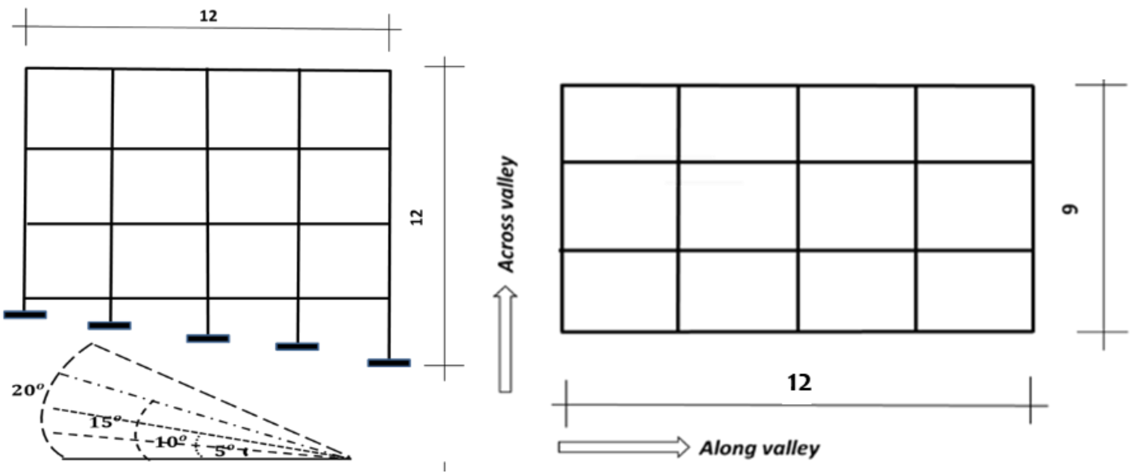
A total of 40 structural models are created with different combinations of slope, AR, and SR. Of these, 20 models are related to 5 different aspect ratios, i.e., 0.66, 1, 1.33, 1.66, and 2; and 20 models are related to 5 varying slenderness ratios, i.e., 0.66, 1.00, 1.33, 1.66 and 2, present on four different slope angles, i.e., 5^0 , 10^0, 15^0, and 20^0, respectively. The details of the building models are outlined in Figure 3.1 and Figure 3.2, and the dynamic properties of AR and SR models are shown in Table 3.1 and Table 3.2, respectively.


6
12
20°
15°
10°
5°
Across valley
9
6
Along valley

(a)


9
12
20°
15°
10°
5°
Across valley
9
9
Along valley

(b)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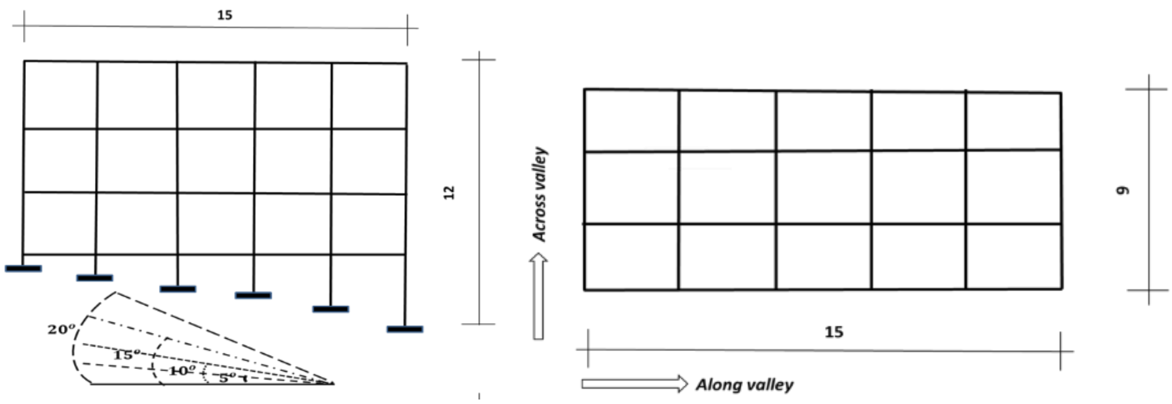
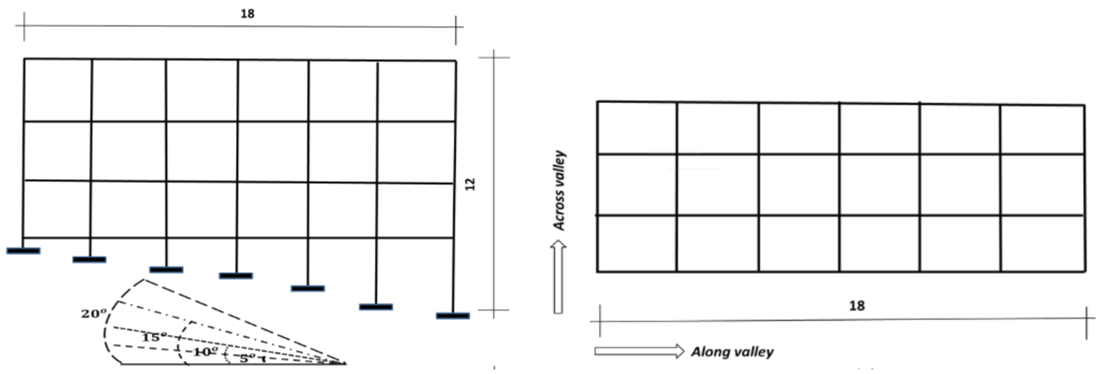
20°
15°
10°
5°
Across valley
9
12
Along valley

(c)



(d)



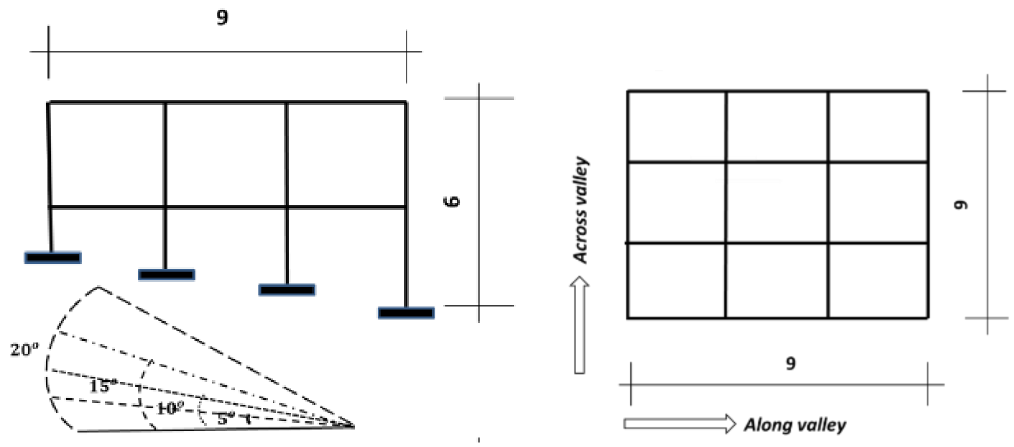
(e)

Figure 3.1 Plan and elevation details of buildings for studying the effect of AR (All dimensions in m)

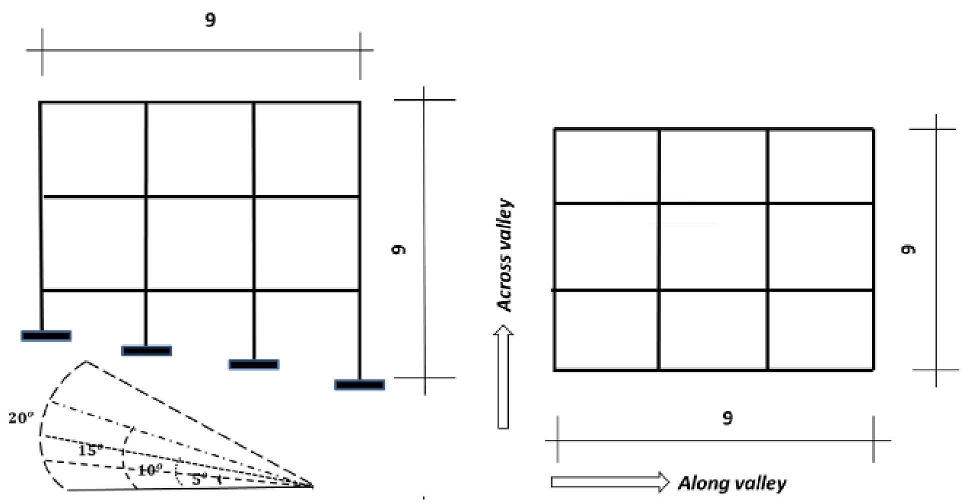
Table 3.1: Dynamic properties of AR models

Model_ID	$T_{1,x}$	$T_{2,y}$	$T_{3,\theta}$	$T_{spacing}$	$\sum_{i=1}^{3} M_x$	$\sum_{i=1}^{3} M_y$	$\sum_{i=1}^{3} M_\theta$
5_6_9_12	0.640	0.626	0.542	2.188	0.789	0.793	0.799
5_9_9_12	0.664	0.659	0.571	0.759	0.780	0.783	0.786
5_12_9_12	0.763	0.711	0.627	6.815	0.759	0.773	0.770
5_15_9_12	0.656	0.624	0.572	4.878	0.754	0.761	0.759
5_18_9_12	0.652	0.608	0.568	6.748	0.739	0.751	0.745
10_6_9_12	0.589	0.583	0.489	1.019	0.770	0.773	0.780
10_9_9_12	0.626	0.610	0.529	2.623	0.729	0.741	0.740
10_12_9_12	0.609	0.574	0.516	6.098	0.692	0.715	0.707
10_15_9_12	0.591	0.541	0.501	9.242	0.655	0.689	0.673
10_18_9_12	0.587	0.527	0.486	10.221	0.675	0.705	0.722
15_6_9_12	0.560	0.551	0.466	1.607	0.731	0.739	0.744

Model_ID	$T_{1,x}$	$T_{2,y}$	$T_{3,\theta}$	$T_{spacing}$	$\sum_{i=1}^{3} M_x$	$\sum_{i=1}^{3} M_y$	$\sum_{i=1}^{3} M_\theta$
15_9_9_12	0.573	0.553	0.484	3.617	0.661	0.683	0.678
15_12_9_12	0.564	0.529	0.465	6.206	0.688	0.706	0.726
15_15_9_12	0.549	0.479	0.420	12.750	0.678	0.705	0.705
15_18_9_12	0.521	0.414	0.373	20.537	0.621	0.685	0.644
20_6_9_12	0.526	0.516	0.437	1.901	0.681	0.695	0.697
20_9_9_12	0.552	0.529	0.451	4.348	0.703	0.714	0.733
20_12_9_12	0.518	0.480	0.690	7.336	0.661	0.696	0.677
20_15_9_12	0.437	0.358	0.330	18.078	0.534	0.626	0.574
20_18_9_12	0.428	0.330	0.286	22.897	0.555	0.641	0.624



(a)



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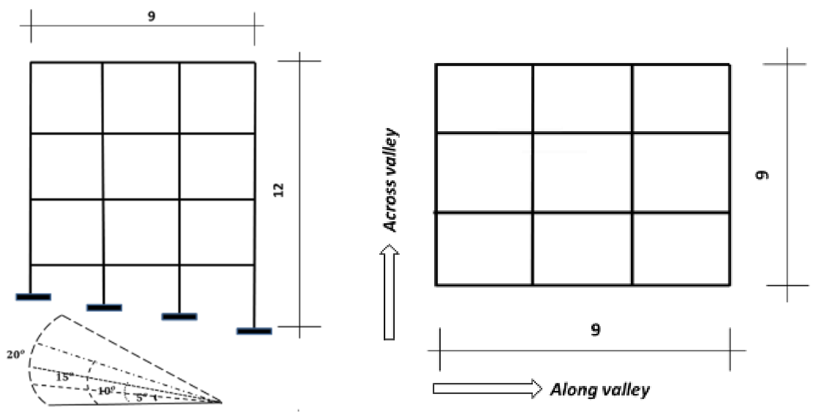

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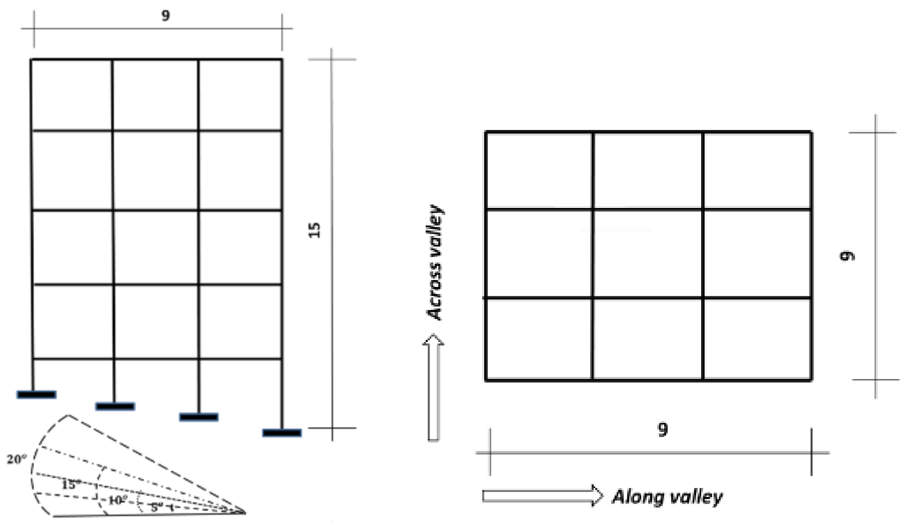

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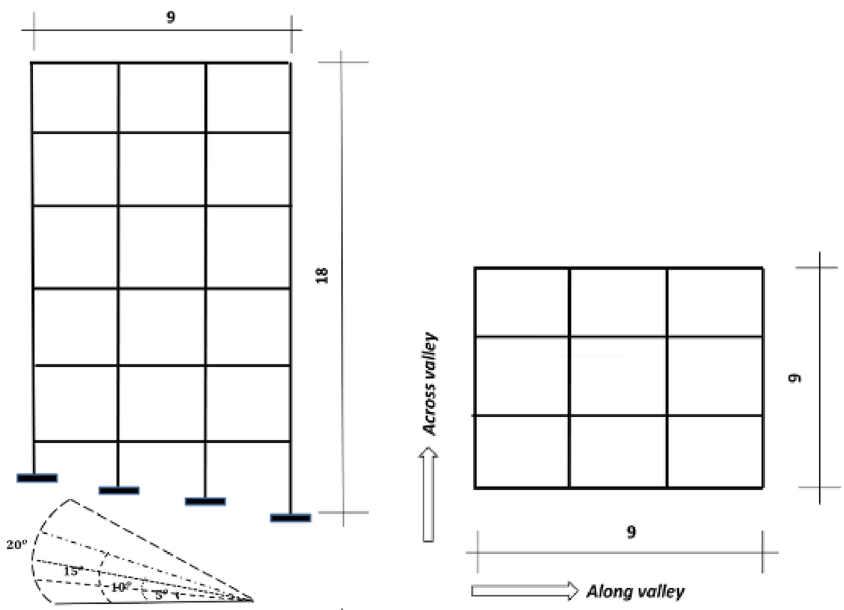


(e)

Figure 3.2 Plan and elevation details of buildings for studying the effect of SR (All dimensions in m)

Table 3.2: Dynamic properties of SR models

Model_ID	$T_{1,x}$	$T_{2,y}$	$T_{3,\theta}$	$T_{spacing}$	$\sum_{i=1}^{3} M_x$	$\sum_{i=1}^{3} M_y$	$\sum_{i=1}^{3} M_\theta$
5_9_9_6	0.275	0.267	0.236	2.996	0.807	0.809	0.808
5_9_9_9	0.462	0.456	0.397	1.316	0.786	0.788	0.792
5_9_9_12	0.664	0.659	0.571	0.759	0.780	0.783	0.786
5_9_9_15	0.872	0.868	0.746	0.461	0.779	0.781	0.786
5_9_9_18	1.084	1.080	0.922	0.370	0.780	0.781	0.786
10_9_9_6	0.245	0.225	0.196	8.889	0.689	0.711	0.700
10_9_9_9	0.426	0.408	0.355	4.412	0.712	0.728	0.724
10_9_9_12	0.626	0.610	0.529	2.623	0.729	0.741	0.740
10_9_9_15	0.833	0.818	0.705	1.834	0.739	0.748	0.750
10_9_9_18	1.044	1.030	0.882	1.359	0.747	0.754	0.757
15_9_9_6	0.200	0.180	0.161	11.111	0.488	0.559	0.537
15_9_9_9	0.348	0.327	0.296	6.422	0.609	0.644	0.635
15_9_9_12	0.573	0.553	0.484	3.617	0.661	0.683	0.678
15_9_9_15	0.778	0.760	0.659	2.368	0.688	0.704	0.703
15_9_9_18	0.989	0.971	0.835	1.854	0.706	0.718	0.719
20_9_9_6	0.191	0.170	0.140	12.353	0.599	0.627	0.701
20_9_9_9	0.356	0.333	0.281	6.907	0.670	0.683	0.715
20_9_9_12	0.552	0.529	0.451	4.348	0.703	0.714	0.733
20_9_9_15	0.775	0.734	0.626	5.586	0.719	0.728	0.742
20_9_9_18	0.963	0.945	0.803	1.905	0.731	0.738	0.750

3.3 Stress Resultants

3.3.1 CORRELATION OF AR WITH STRESS RESULTANTS

The aspect ratio plays a crucial role in understanding the behaviour of sloped buildings. An increased aspect ratio makes the longer frame more flexible, whereas the shorter frame remains rigid. This will create torsion across the slope direction. This phenomenon can be explained with the help of the modal properties. Along with modal properties, an increase in aspect ratio also affects the distribution of forces, i.e., shear force in columns and axial forces in beams. The critical location, i.e., ridge (uphill) column and beam, where localized shear and axial forces are noticed in columns and beams, is chosen for deriving the stress ratios. The design force is obtained at the critical location and converted into stress ratios, as explained in Table 3.3, and the ratios derived are shown in Table 3.4.

Once required parameters are derived, to understand how variation in one variable is affecting the other variable, Pearson coefficient r is calculated and plotted in the form of a matrix using origin software. The value of r varies from 0 to1, with 0 indicating no correlation and 1 indicating a perfect correlation. The data points are fitted by an ellipse with 95% confidence interval. In the case of a perfect correlation, the ellipse becomes a straight line. To understand the significance of correlation P-values are analysed. The P stands for probability and measures how likely that any observed difference between groups is due to chance. Figure 3.3 shows the variation stress ratio with modification of AR and slope. Flexural stresses. The correlation among variables is discussed below:

Table 3.3: Stress ratios at a critical location for understanding the effect of AR

ID	Slope	AR	Stress ratio at the ridge beam				Stress ratio at the ridge column			
			BAS	BSR	BFS	BTS	CAS	CSR	CFS_M3	CFS_M2
5_6_9_12	5	0.66	0.010	0.023	0.015	0.049	0.106	0.016	0.076	0.086
5_9_9_12	5	1	0.012	0.024	0.014	0.058	0.116	0.022	0.085	0.114
5_12_9_12	5	1.33	0.018	0.024	0.014	0.058	0.115	0.028	0.087	0.126
5_15_9_12	5	1.66	0.024	0.024	0.013	0.056	0.114	0.034	0.087	0.136
5_18_9_12	5	2.00	0.033	0.024	0.013	0.055	0.114	0.042	0.092	0.148
10_6_9_12	10	0.66	0.018	0.023	0.015	0.049	0.108	0.024	0.087	0.107
10_9_9_12	10	1	0.036	0.023	0.012	0.055	0.114	0.043	0.090	0.152
10_12_9_12	10	1.33	0.080	0.021	0.011	0.048	0.111	0.081	0.095	0.184
10_15_9_12	10	1.66	0.161	0.017	0.009	0.033	0.107	0.162	0.104	0.190
10_18_9_12	10	2.00	0.161	0.016	0.009	0.031	0.127	0.162	0.091	0.192
15_6_9_12	15	0.66	0.031	0.021	0.013	0.045	0.107	0.036	0.091	0.123

ID	Slope	AR	Stress ratio at the ridge beam				Stress ratio at the ridge column			
			BAS	BSR	BFS	BTS	CAS	CSR	CFS_M3	CFS_M2
15_9_9_12	15	1	0.104	0.018	0.009	0.038	0.110	0.106	0.101	0.178
15_12_9_12	15	1.33	0.105	0.017	0.009	0.034	0.126	0.108	0.086	0.179
15_15_9_12	15	1.66	0.101	0.017	0.009	0.031	0.125	0.102	0.086	0.166
15_18_9_12	15	2.00	0.102	0.023	0.011	0.053	0.076	0.092	0.085	0.233
20_6_9_12	20	0.66	0.048	0.018	0.010	0.035	0.104	0.053	0.094	0.130
20_9_9_12	20	1	0.059	0.017	0.009	0.035	0.127	0.063	0.080	0.139
20_12_9_12	20	1.33	0.063	0.023	0.011	0.057	0.078	0.057	0.085	0.192
20_15_9_12	20	1.66	0.199	0.018	0.008	0.039	0.072	0.189	0.085	0.267
20_18_9_12	20	2.00	0.192	0.018	0.008	0.036	0.081	0.181	0.073	0.255

1. **Beam Axial Stress (BAS):** Axial stresses in beams correlated well with AR, having a partial correlation factor of 0.61. The ratio indicates that with increasing AR, there is an increase in axial stresses in beams. In general, beam axial stresses are insignificant, and hence, beams are usually designed for moments, shear, and torsional stress. In the case of buildings on hill slopes, the large, unbalanced shear forces in columns are transferred to beams as axial forces. This is particularly true in beams connecting the short column at road level. Hence, when the length of the column is reduced, shear forces in columns, along with axial forces in beams, are increasing.

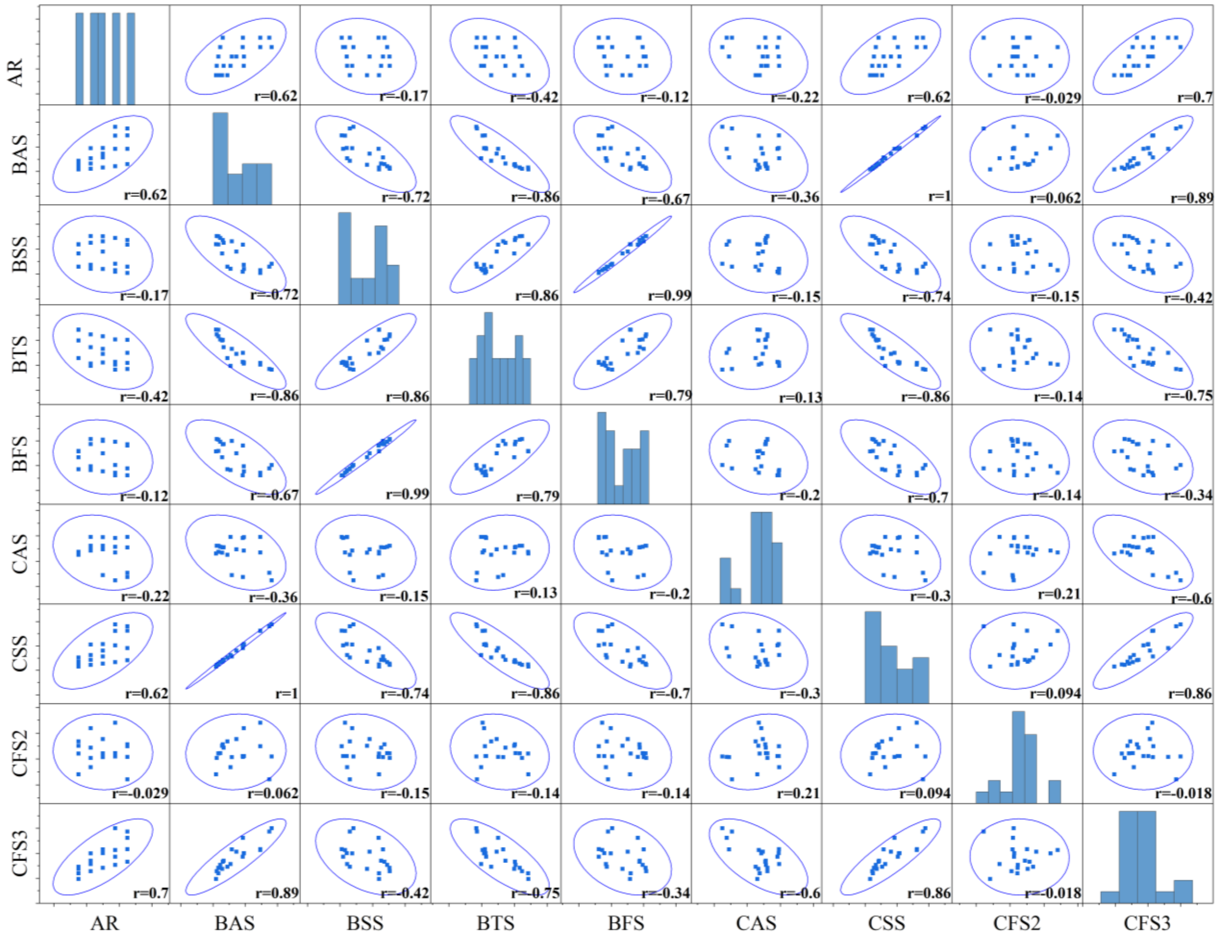


Figure 3.3: Correlation of Stress ratios with AR

Table 3.4: P-value analysis at 0.05 level for correlating stress ratios with AR

	AR	BAS	BSS	BTS	BFS	CAS	CSS	CFS_2	CFS_3
AR	1	0.00	0.48	0.06	0.60	0.35	0.00	0.90	0.00
BAS	0.00	1	0.00	0.00	0.00	0.11	0.00	0.79	0.00
BSS	0.48	0.00	1	0.00	0.00	0.52	0.00	0.51	0.06
BTS	0.06	0.00	0.00	1	0.00	0.58	0.00	0.56	0.00
BFS	0.60	0.00	0.00	0.00	1	0.39	0.00	0.56	0.13
CAS	0.35	0.11	0.52	0.58	0.39	1	0.19	0.36	0.00
CSS	0.00	0.00	0.00	0.00	0.00	0.19	1	0.69	0.00
CFS_2	0.90	0.79	0.51	0.56	0.56	0.36	0.69	1	0.94
CFS_3	0.00	0.00	0.06	0.00	0.13	0.00	0.00	0.94	1

2. **Beam shear stress (BSR):** Shear stresses in beams correlated with a partial correlation factor of -0.16. Though the partial correlation is insignificant, an inference can be drawn that beam shear stresses decrease with increasing AR.
3. **Beam Flexural Stress (BFS):** Beam flexural stresses correlate with a partial correlation factor of -0.12. This indicates that with increasing AR, beam flexural stresses decrease.
4. **Beam Torsional stress (BTS):** Torsional stresses in beams correlate with a partial correlation factor of -0.42, indicating a decrease in torsional stresses with an increase in AR.
5. **Column Axial stress (CAS):** Axial stresses in columns correlate with a partial correlation factor of -0.21, which indicates that an increase in AR decreases the axial stresses in columns.
6. **Column Shear stress (CSR):** Shear stresses in columns correlated well with AR, having a partial correlation factor of 0.61. The ratio indicates that with increasing AR, there is an increase in shear stresses in columns.
7. **Column Flexural stress (CFS_M3 and CFS_M2):** Flexural stresses in columns (CFS_M3) correlate well with AR, having a partial correlation factor of 0.70. The ratio indicates that with an increase in AR, flexural stresses in columns increase.

It must be noted that the design emphasizes beam and column flexural forces. In general, IS 13920-2016 [50] mentions that beam axial stress can be ignored up to 0.08 f_{ck}. If it exceeds, then it needs to be designed as a beam-column element. On top of this clause, there is an assumption of the rigid diaphragm concept, which will suppress the axial forces in beams. It can be observed from the correlation that with an aspect ratio increase, the design parameters, such as moments and shear, decrease. However, the usually ignored forces increase, such as beam axial force. For columns, though moments are increasing, shear stresses are also increasing. From this study of stress ratios, all the forces not significant in flexural design are becoming prominent with an increase in slope and aspect ratio.

Similarly, a rigid diaphragm is an essential assumption for modeling the effect of slabs. The assumption needs to be used with care for buildings on slopes. Building with a rigid diaphragm assumes that the diaphragms are completely rigid; hence, no axial stresses are developed in beams at road level. However, building with semi-rigid diaphragms allows flexibility to the diaphragms, and hence the beams deform axially. On the other hand, the presence of a slab does not allow for completely neglecting the diaphragms. Hence, all the models are assigned semi-rigid diaphragms for the design of buildings on slopes.

From analysis BAS, CSS and CFS_3 are identified as variables having significant correlation

3.3.2 CORRELATION OF SR WITH STRESS RESULTANTS

The slenderness ratio (SR) is another parameter that helps understand a building's behaviour on slopes. An increase in the slenderness ratio increases the deformations. There are limits on the slenderness ratio to prevent excessive deformations. Buildings on slopes undergo differential deformations, and hence, it is vital to verify whether the variation of SR has a positive or negative effect on the force distribution.

Figure 3.4 shows the variation stress ratio varies with modification of SR and slope and flexural stresses. The correlation among variables is discussed below:

1. **Beam Axial Stress (BAS):** Axial stresses in beams correlated well with SR, having a partial correlation factor of 0.50. The ratio indicates that with increasing SR, there is an increase in axial stresses in beams.
2. **Beam Shear Stress (BSS):** Shear stresses in beams correlated well with SR, having a partial correlation factor of 0.80. The ratio indicates that with increasing SR, there is an increase in the shear stresses of beams.
3. **Beam Flexural Stress (BFS):** Flexural stresses in beams correlated well with SR, having a partial correlation factor of 0.82. The ratio indicates that with increasing SR, there is an increase in beam flexural stresses.
4. **Column Axial Stress (CAS):** Axial stresses in columns correlated well with SR, having a partial correlation factor of 0.98. The ratio indicates that with increasing SR, there is an increase in column axial stresses.

5. **Column Shear Stress (CSR):** Shear stresses in columns correlated well with SR, having a partial correlation factor of 0.57. The ratio indicates that with increasing SR, there is an increase in column shear stresses.
6. **Column Flexural Stress (CFS_M3 and CFS_M2):** Flexural stresses in columns correlated well with SR, having a partial correlation factor of 0.97 and 0.92, respectively. The ratio indicates that with increasing SR, there is an increase in column flexural stresses.

With an increase in the Slenderness ratio significance of design forces like moments, shear demands increase in beams. For columns, moments are increasing, and shear stresses are increasing but rather gradually compared to AR. From the study, it can be concluded that SR is better at retaining the flexural stresses in beams and columns. Hence, it would be an essential design parameter in imparting conservativeness in flexural stresses.

Table 3.5 Stress ratios at a critical location for understanding the effect of SR

ID	Slope	SR	Stress ratio at the ridge beam				Stress ratio at the ridge column			
			BAS	BSR	BFS	BTS	CAS	CSR	CFS_M3	CFS_M2
5_9_9_6	5	0.66	0.006	0.015	0.005	0.026	0.042	0.010	0.031	0.046
5_9_9_9	5	1	0.009	0.020	0.009	0.042	0.075	0.016	0.057	0.080
5_9_9_12	5	1.33	0.012	0.024	0.014	0.058	0.116	0.022	0.085	0.114
5_9_9_15	5	1.67	0.015	0.028	0.018	0.072	0.162	0.027	0.108	0.143
5_9_9_18	5	2	0.019	0.032	0.021	0.086	0.215	0.033	0.132	0.173
10_9_9_6	10	0.66	0.016	0.015	0.005	0.025	0.049	0.019	0.032	0.057
10_9_9_9	10	1	0.026	0.019	0.008	0.040	0.074	0.031	0.060	0.105
10_9_9_12	10	1.33	0.036	0.023	0.012	0.055	0.114	0.043	0.090	0.152
10_9_9_15	10	1.67	0.044	0.026	0.015	0.067	0.160	0.054	0.117	0.190
10_9_9_18	10	2	0.053	0.030	0.018	0.079	0.213	0.064	0.144	0.230
15_9_9_6	15	0.66	0.044	0.013	0.004	0.019	0.038	0.046	0.031	0.073
15_9_9_9	15	1	0.058	0.015	0.006	0.024	0.068	0.060	0.052	0.097
15_9_9_12	15	1.33	0.104	0.018	0.009	0.038	0.110	0.106	0.101	0.178
15_9_9_15	15	1.67	0.128	0.020	0.011	0.045	0.155	0.129	0.133	0.222
15_9_9_18	15	2	0.152	0.022	0.013	0.052	0.207	0.154	0.164	0.266

ID	Slope	SR	Stress ratio at the ridge beam				Stress ratio at the ridge column			
			BAS	BSR	BFS	BTS	CAS	CSR	CFS_M3	CFS_M2
20_9_9_6	20	0.66	0.025	0.013	0.004	0.018	0.042	0.027	0.028	0.061
20_9_9_9	20	1	0.042	0.015	0.006	0.026	0.081	0.045	0.053	0.100
20_9_9_12	20	1.33	0.059	0.017	0.009	0.035	0.127	0.063	0.080	0.139
20_9_9_15	20	1.67	0.073	0.019	0.010	0.043	0.179	0.078	0.104	0.170
20_9_9_18	20	2	0.087	0.022	0.012	0.050	0.235	0.092	0.128	0.201

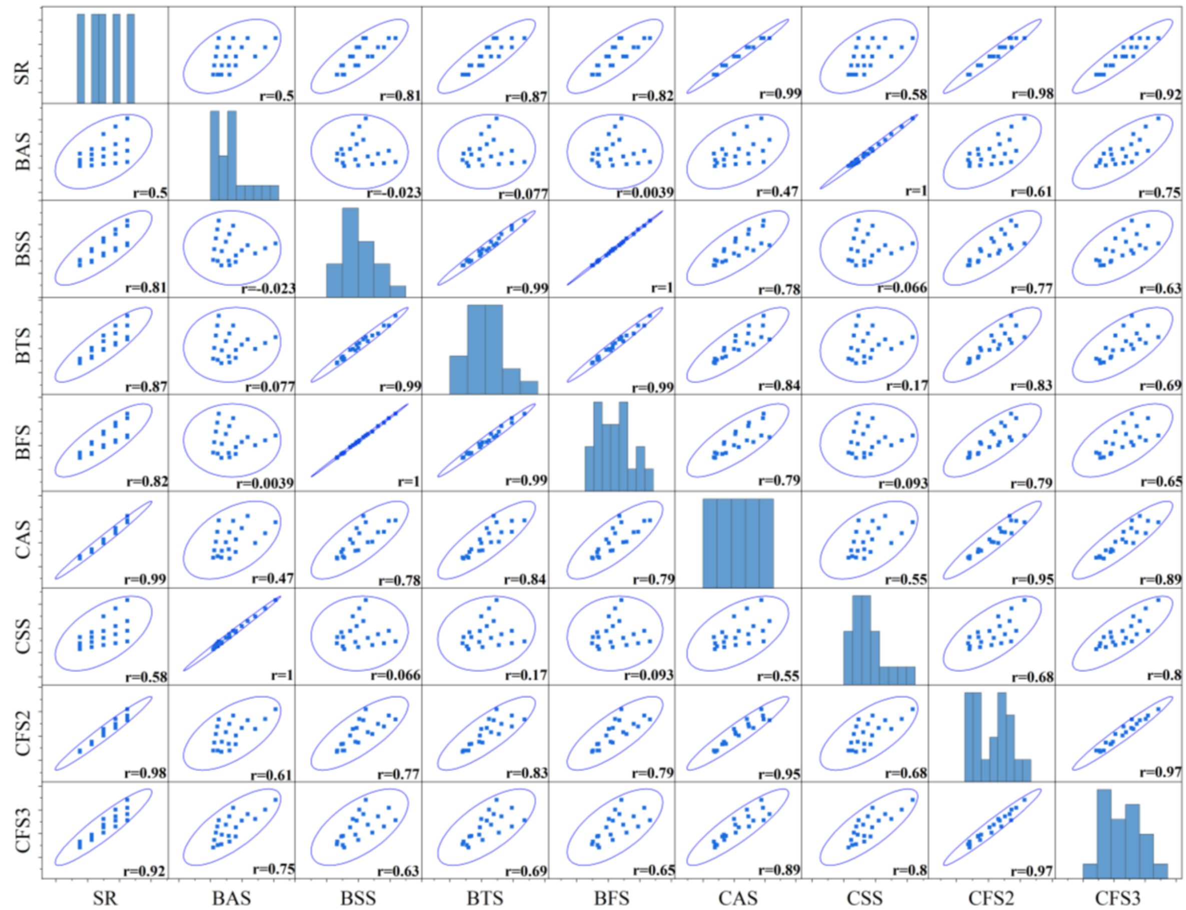


Figure 3.4: Correlation of stress ratios with SR

Table 3.6 P-value analysis at 0.05 level for correlating stress ratios with SR

	SR	BAS	BSS	BTS	BFS	CAS	CSS	CFS_2	CFS_3
SR	1	0.02	0.00	0.00	0.00	0.00	0.00	0.00	0.00
BAS	0.02	1	0.92	0.74	0.98	0.03	0.00	0.00	0.00
BSS	0.00	0.92	1	0.00	0.00	0.00	0.78	0.00	0.00

BTS	0.00	0.74	0.00	1	0.00	0.00	0.48	0.00	0.00
BFS	0.00	0.98	0.00	0.00	1	0.00	0.69	0.00	0.00
CAS	0.00	0.03	0.00	0.00	0.00	1	0.01	0.00	0.00
CSS	0.00	0.00	0.78	0.48	0.69	0.01	1	0.00	0.00
CFS_2	0.00	0.00	0.00	0.00	0.00	0.00	0.00	1	0.00
CFS_3	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	1

From analysis all the variables are having significant correlation

3.4 Strength

The lateral strength of the building is one of the key virtues that help analyze the structure's behaviour. The preliminary step in earthquake-resistant design is determining the base shear attracted by the structure. Once base shear is obtained, the next step involves the distribution of the base shear throughout the height. Cross-sections are decided predominantly based on the story shear calibrated based on the distribution adopted. It must be noted that in force-based design philosophy, strength and stiffness are interdependent, and stiffness is assumed to be known prior. Buildings on slopes are a stiffness irregular system, and since both strength and stiffness are interlinked, it is essential to understand the effect of AR and SR on strength distribution.

3.4.1 CORRELATION OF AR WITH STRENGTH

To study the effect of increasing AR and slope angles on the lateral strength of the structure, the building on the slope is divided into parts, i.e., part of the building above the road and part of the building below the road level thus making the road level as the reference line as shown in figure 3.5.

The story shear forces are obtained and normalized with the base shear value. Normalized shear at the story below and above the road level is shown in Figure 3.5. Abbreviations used are defined below:

a. **V_A:** Normalized story shear calculated ***below*** the road level when the structure is excited ***along the valley*** direction.

b. **V_B:** Normalized story shear calculated ***above*** the road level when the structure is excited ***along the valley*** direction.

c. **V_C:** Normalized story shear calculated ***below*** the road level when the structure is excited ***across the valley*** direction.

d. **V_D:** Normalized story shear calculated ***above*** the road level when the structure is excited ***across the valley*** direction.

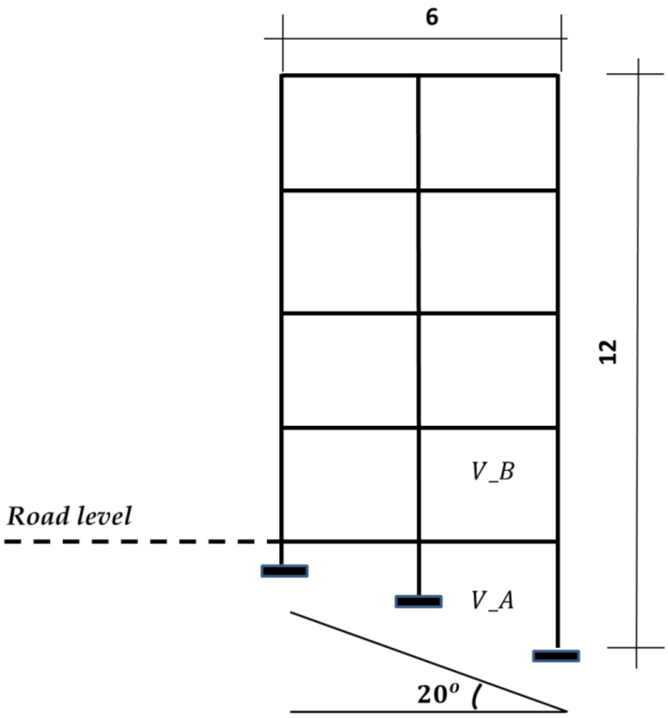


Figure 3.5: Story strength calibration for models considered.

Table 3.7: Normalised story strength ratios above and below the road level when excited along and across the valley for varying AR

ID	Slope	AR	Story Strength			
			V_A	V_B	V_C	V_D
5_6_9_12	5	0.66	1.000	0.893	1.000	0.894
5_9_9_12	5	1	1.000	0.900	1.000	0.898
5_12_9_12	5	1.33	1.000	0.907	1.000	0.903
5_15_9_12	5	1.66	1.000	0.915	1.000	0.908
5_18_9_12	5	2.00	1.000	0.924	1.000	0.913
10_6_9_12	10	0.66	1.000	0.911	1.000	0.911

ID	Slope	AR	Story Strength			
			V_A	V_B	V_C	V_D
10_9_9_12	10	1	1.000	0.929	1.000	0.921
10_12_9_12	10	1.33	1.000	0.946	1.000	0.933
10_15_9_12	10	1.66	1.000	0.963	1.000	0.947
10_18_9_12	10	2.00	0.811	1.000	0.975	1.000
15_6_9_12	15	0.66	1.000	0.933	1.000	0.932
15_9_9_12	15	1	1.000	0.959	1.000	0.955
15_12_9_12	15	1.33	0.702	1.000	0.861	1.000
15_15_9_12	15	1.66	0.356	1.000	0.739	1.000
15_18_9_12	15	2.00	0.149	1.000	0.569	1.000
20_6_9_12	20	0.66	1.000	0.954	1.000	0.956
20_9_9_12	20	1	0.586	1.000	0.756	1.000
20_12_9_12	20	1.33	0.190	1.000	0.545	1.000
20_15_9_12	20	1.66	0.170	1.000	0.213	1.000
20_18_9_12	20	2.00	0.154	1.000	0.205	1.000

Figure 3.6 shows the variation strength varies with the modification of AR and slope. The correlation among variables is discussed below:

1. **V_A: The s**trength of the building in the lower story below the road level when force is applied along the valley correlated negatively with AR having a partial correlation factor of -0.50. The ratio indicates that with increasing AR, strength in the story below road level decreases (i.e., the lower story becomes weak)

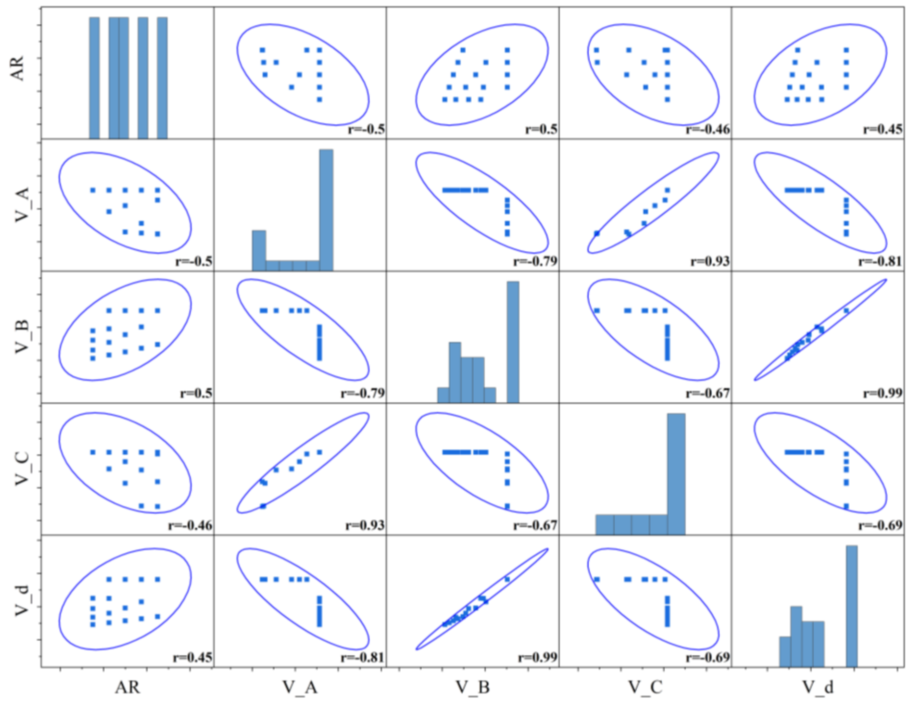


Figure 3.6: Correlation of strength ratios with AR

Table 3.8: P-value analysis at 0.05 level for correlating strength ratios with AR

	AR	V_A	V_B	V_C	V_d
AR	1	0.02	0.02	0.03	0.03
V_A	0.02	1	0.00	0.00	0.00
V_B	0.02	0.00	1	0.00	0.00
V_C	0.03	0.00	0.00	1	0.00
V_d	0.03	0.00	0.00	0.00	1

2. **V_B:** The strength of the building in the story above the road level when force is applied along the valley correlated positively with AR, having a partial correlation factor of 0.49. The ratio indicates that with increasing AR, strength in the story above road level increases (i.e., upper stories become stronger than lower story). It is also important to note that the increase in the strength of the upper stories is almost proportional to the decrease in the strength of the lower stories.
3. **V_C:** The strength of the building in the lower story below the road level when force is applied across the valley correlated negatively with AR having a partial correlation factor of -0.45. The ratio indicates that with increasing AR,

strength in the story below road level decreases (i.e., the lower story becomes weak)

4. **V_D: The s**trength of the building in the story above the road level when force is applied across the valley correlated positively with AR having a partial correlation factor of 0.44. The ratio indicates that with increasing AR, strength in the story below road level increases (i.e., upper stories become stronger than the lower story), and similar observation is seen across the valley direction.

From analysis all the variables are having significant correlation with AR

3.4.2 CORRELATION OF SR WITH STRENGTH

Like the study correlating AR with story strength, the effect of increasing SR on strength is conducted, and the results are tabulated in Table 3.7.

Figure 3.7 shows the variation strength varies with the modification of SR and slope. The correlation among variables is discussed below:

1. **V_A:** Though the Strength of the building in the lower story below the road level when force is applied along the valley correlated negatively, the correlation is insignificant with a factor of -0.17.
2. **V_B:** The strength of the building in the story above the road level when force is applied along the valley correlated positively with SR having a partial correlation factor of 0.62. The ratio indicates that with increasing SR, strength in the story above road level increases.
3. **V_C:** The **s**trength of the building in the story below the road level when force is applied across the valley correlated negatively with SR having a partial correlation factor of -0.24, which is insignificant.
4. **V_D:** The **s**trength of the building in the story above the road level when force is applied across the valley correlated positively with SR having a partial correlation factor of 0.65. The ratio indicates that with increasing SR, strength in the story above road level increases.

Table 3.9: Normalised story strength ratios above and below the road level when excited along and across the valley for varying SR.

ID	Slope	AR	Story Strength			
			V_A	V_B	V_C	V_D
5_9_9_6	5	0.67	1.000	0.583	1.000	0.904
5_9_9_9	5	1.00	1.000	0.822	1.000	0.819
5_9_9_12	5	1.33	1.000	0.900	1.000	0.898
5_9_9_15	5	1.67	1.000	0.928	1.000	0.927
5_9_9_18	5	2.00	1.000	0.942	1.000	0.941
10_9_9_6	10	0.67	1.000	0.641	1.000	0.619
10_9_9_9	10	1.00	1.000	0.864	1.000	0.851
10_9_9_12	10	1.33	1.000	0.929	1.000	0.921
10_9_9_15	10	1.67	1.000	0.948	1.000	0.943
10_9_9_18	10	2.00	1.000	0.957	1.000	0.954
15_9_9_6	15	0.67	1.000	0.707	1.000	0.695
15_9_9_9	15	1.00	1.000	0.912	1.000	0.908
15_9_9_12	15	1.33	1.000	0.959	1.000	0.955
15_9_9_15	15	1.67	1.000	0.966	1.000	0.969
15_9_9_18	15	2.00	1.000	0.968	1.000	0.975
20_9_9_6	20	0.67	0.944	1.000	1.000	0.873
20_9_9_9	20	1.00	0.637	1.000	0.824	1.000
20_9_9_12	20	1.33	0.586	1.000	0.756	1.000
20_9_9_15	20	1.67	0.573	1.000	0.716	1.000
20_9_9_18	20	2.00	0.570	1.000	0.697	1.000

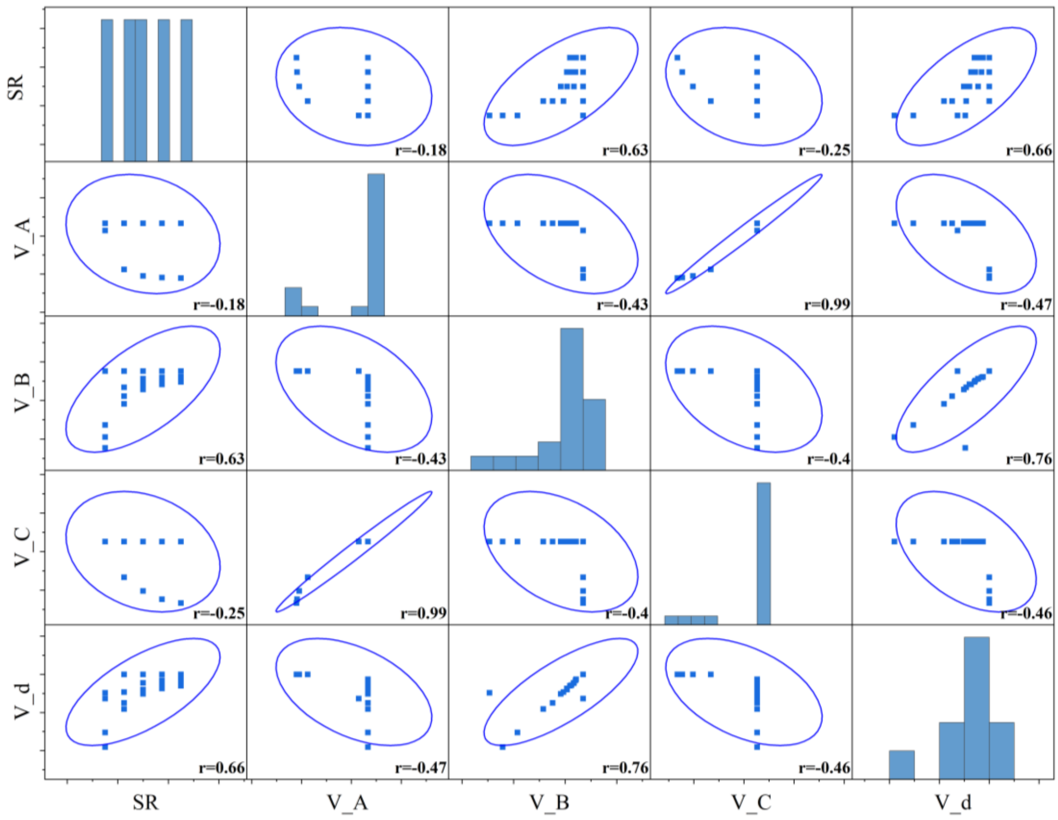


Figure 3.7: Correlation of Strength ratios with SR

Table 3.10 Significance analysis (0.05 level) of strength ratios with SR

	SR	V_A	V_B	V_C	V_d
SR	1	0.18	0.00	0.10	0.00
V_A	0.18	1	0.05	0.00	0.05
V_B	0.00	0.05	1	0.07	0.00
V_C	0.19	0.00	0.07	1	0.07
V_d	0.00	0.05	0.00	0.07	1

From analysis V_B and V_d are identified as variables having significant correlation with SR

3.5 Stiffness

The stiffness of the building is one of the key virtues that help analyze the structure's behaviour. In the traditional design, method stiffness is known before the final design based on assumed cross-section sizes. Based on the assumed cross-section sizes, the base shear calculated is distributed depending on the stiffness of the frames.

Interdependency of strength and stiffness with treating buildings on slopes as essentially stiffness irregular makes stiffness variation an important parameter for correlating AR and SR.

3.5.1 CORRELATION OF AR WITH STIFFNESS

To study the effect of increasing AR and slope angles on the lateral strength of the structure, the buildings on the slope are divided into parts, i.e., part of the building above the road and part of the building below the road level thus making road level as the reference line as shown in Figure 3.8.

The story stiffnesses are obtained and normalized to the values with the maximum stiffness value. Normalized stiffness below and above the road level is shown in Figure 3.8. Abbreviations used are defined below:

1. **K_A:** Normalized story stiffness calculated ***below*** the road level when the structure is excited ***along the valley*** direction.
2. **K_B:** Normalized story stiffness calculated ***above*** the road level when the structure is excited ***along the valley*** direction.
3. **K_C:** Normalized story stiffness calculated ***below*** the road level when the structure is excited ***across the valley*** direction.
4. **K_D:** Normalized story stiffness calculated ***above*** the road level when the structure is excited ***across the valley*** direction.

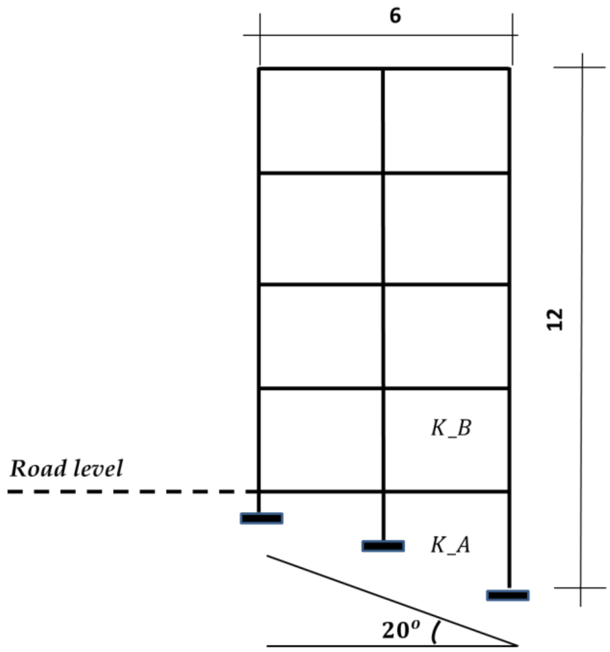


Figure 3.8: Storey strength calibration for models considered.

Figure 3.9 shows the variation stiffness with modification of AR. The correlation among variables is discussed below:

1. **K_A:** Though the stiffness of the building models in the story below the road level when force is applied along the valley correlated negatively, the correlation is insignificant with a factor of -0.04.
2. **K_B:** Though the stiffness of the structural models in the story above the road level when force is applied along the valley correlated positively, the correlation is insignificant with a factor of 0.16.
3. **K_C:** The **s**tiffness of the structural models in the story below the road level when force is applied across the valley correlated negatively with AR having a partial correlation factor of -0.64. The ratio indicates that with increasing AR, stiffness in stories below the road level across the valley decreases.
4. **K_D:** Normalized stiffness across the valley in all the cases is 1. Hence, partial correlation cannot be defined.

Table 3.11: Normalized story stiffness ratios above and below the road level when exited along and across the valley for varying AR

ID	Slope	AR	Story Strength			
			K_A	K_B	K_C	K_D
5_6_9_12	5	0.66	0.470	1.000	0.717	1.000
5_9_9_12	5	1	0.249	1.000	0.394	1.000
5_12_9_12	5	1.33	0.148	1.000	0.313	1.000
5_15_9_12	5	1.66	0.079	1.000	0.259	1.000
5_18_9_12	5	2.00	0.035	1.000	0.219	1.000
10_6_9_12	10	0.66	0.339	1.000	0.627	1.000
10_9_9_12	10	1	0.067	1.000	0.262	1.000
10_12_9_12	10	1.33	0.304	1.000	0.146	1.000
10_15_9_12	10	1.66	0.632	1.000	0.066	1.000
10_18_9_12	10	2.00	0.633	1.000	0.061	1.000
15_6_9_12	15	0.66	0.103	1.000	0.487	1.000
15_9_9_12	15	1	1.000	0.981	0.136	1.000
15_12_9_12	15	1.33	0.999	1.000	0.133	1.000
15_15_9_12	15	1.66	0.799	1.000	0.138	1.000
15_18_9_12	15	2.00	0.669	1.000	0.130	1.000
20_6_9_12	20	0.66	0.651	1.000	0.191	1.000
20_9_9_12	20	1	0.923	1.000	0.249	1.000
20_12_9_12	20	1.33	0.708	1.000	0.228	1.000
20_15_9_12	20	1.66	0.239	1.000	0.261	1.000
20_18_9_12	20	2.00	0.249	1.000	0.219	1.000

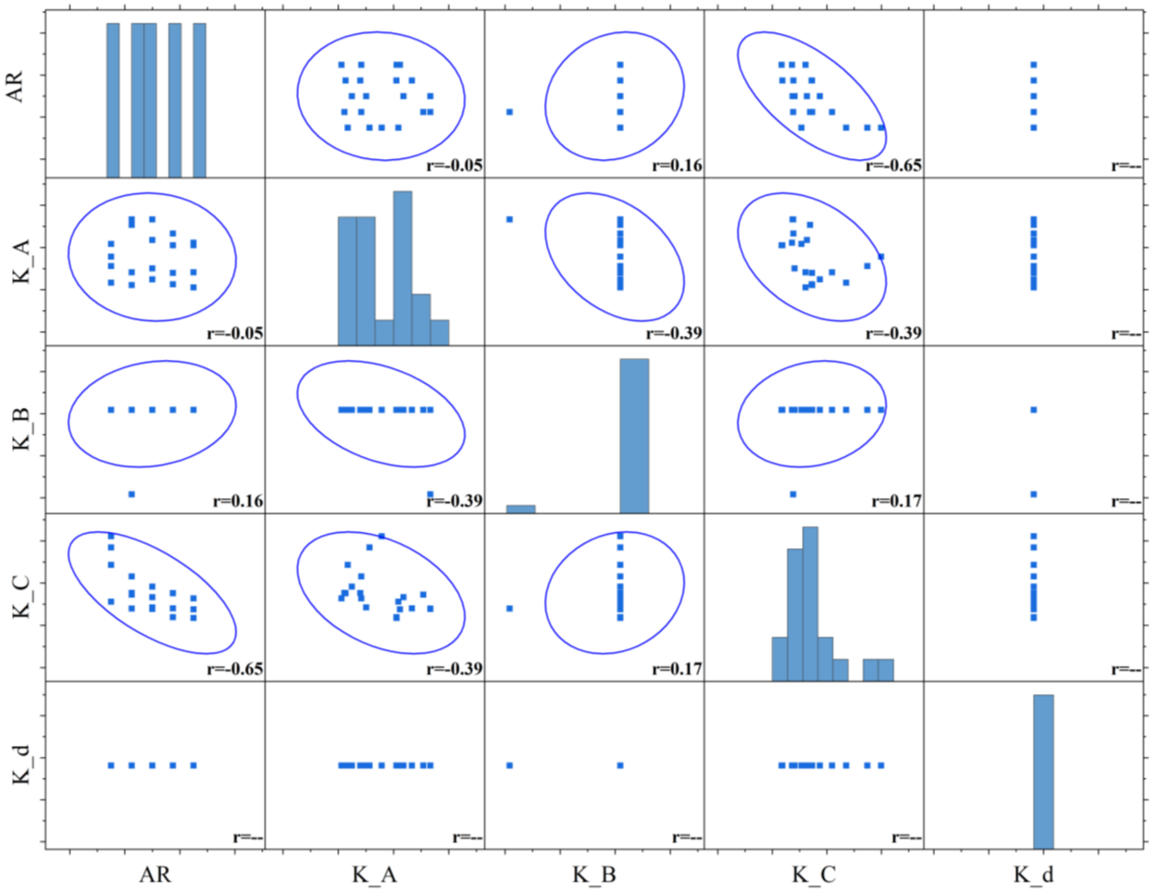


Figure 3.9: Correlation matrices for stiffness ratio with varying AR

Table 3.12 Significance analysis (0.05 level) of stiffness ratios with AR

	AR	K_A	K_B	K_C	K_d
AR	1	0.27	0.07	0.32	0.07
K_A	0.27	1	0.20	0.20	0.25
K_B	0.07	0.18	1	0.45	0.20
K_C	0.00	0.20	0.45	1	0.00
K_d	0.07	0.25	0.20	0.00	1

From analysis K_C is identified as variables having significant correlation with AR

3.5.2 CORRELATION OF SR WITH STIFFNESS

1. K_A: Though the stiffness of the building models in the story below the road level when force is applied along the valley correlated positively with increasing SR, the correlation is insignificant with a factor of 0.10.

2. **K_B:** Though the stiffness of the structural models in the story above the road level when force is applied along the valley correlated negatively with increasing SR, the correlation is insignificant with a factor of -0.04.
3. **K_C:** The **s**tiffness of the structural models in the story below the road level when force is applied across the valley correlated negatively with SR having a partial correlation factor of -0.17.
4. **K_D:** Normalized stiffness across the valley in all the cases is 1 with increasing SR. Hence, partial correlation cannot be defined.

Table 3.13: Normalized story stiffness ratios above and below the road level when exited along and across the valley for varying SR

ID	**Slope**	**SR**	**Storey stiffness**			
			K_A	K_b	K_c	K_d
5_9_9_6	5	0.67	0.350	1.000	0.482	1.000
5_9_9_9	5	1.00	0.256	1.000	0.396	1.000
5_9_9_12	5	1.33	0.249	1.000	0.394	1.000
5_9_9_15	5	1.67	0.246	1.000	0.387	1.000
5_9_9_18	5	2.00	0.244	1.000	0.383	1.000
10_9_9_6	10	0.67	0.159	1.000	0.365	1.000
10_9_9_9	10	1.00	0.077	1.000	0.261	1.000
10_9_9_12	10	1.33	0.067	1.000	0.262	1.000
10_9_9_15	10	1.67	0.079	1.000	0.253	1.000
10_9_9_18	10	2.00	0.086	1.000	0.247	1.000
15_9_9_6	15	0.67	0.687	1.000	0.171	1.000
15_9_9_9	15	1.00	1.000	0.781	0.309	1.000
15_9_9_12	15	1.33	1.000	0.981	0.136	1.000
15_9_9_15	15	1.67	1.000	0.908	0.159	1.000
15_9_9_18	15	2.00	1.000	0.875	0.170	1.000
20_9_9_6	20	0.67	0.563	1.000	0.207	1.000
20_9_9_9	20	1.00	1.000	0.882	0.328	1.000
20_9_9_12	20	1.33	0.923	1.000	0.249	1.000

ID	Slope	SR	Storey stiffness			
			K_A	K_b	K_c	K_d
20_9_9_15	20	1.67	0.994	1.000	0.281	1.000
20_9_9_18	20	2.00	1.000	0.961	0.300	1.000

Figure 3.10 explains the anticipated deformations of buildings resting on hill slopes. With increasing slope, the stiffness centre shifts towards a shorter frame, thereby creating (I) a flexible longer frame creating torsion and (II) Significantly fewer deformations in lower stories than roof deformations, forcing the system to act as two independent units.

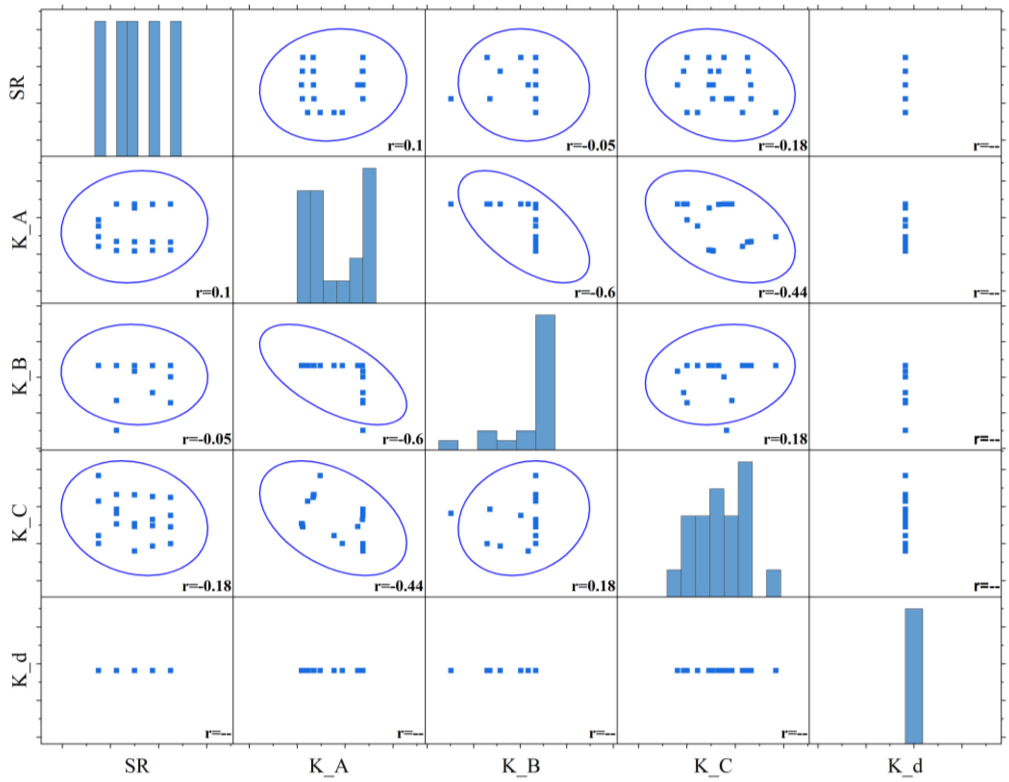


Figure 3.10: Correlation matrices for stiffness ratio with varying SR

Table 3.14 Significance analysis (0.05 level) of stiffness ratios with SR

	SR	K_A	K_B	K_C	K_d
SR	1	0.27	0.07	0.32	0.07
K_A	0.27	1	0.00	0.20	0.25
K_B	0.07	0.00	1	0.45	0.00
K_C	0.32	0.20	0.45	1	0.00

K_d	0.07	0.25	0.00	0.00	1

From analysis no variable is having significant correlation with SR

3.6 Drifts

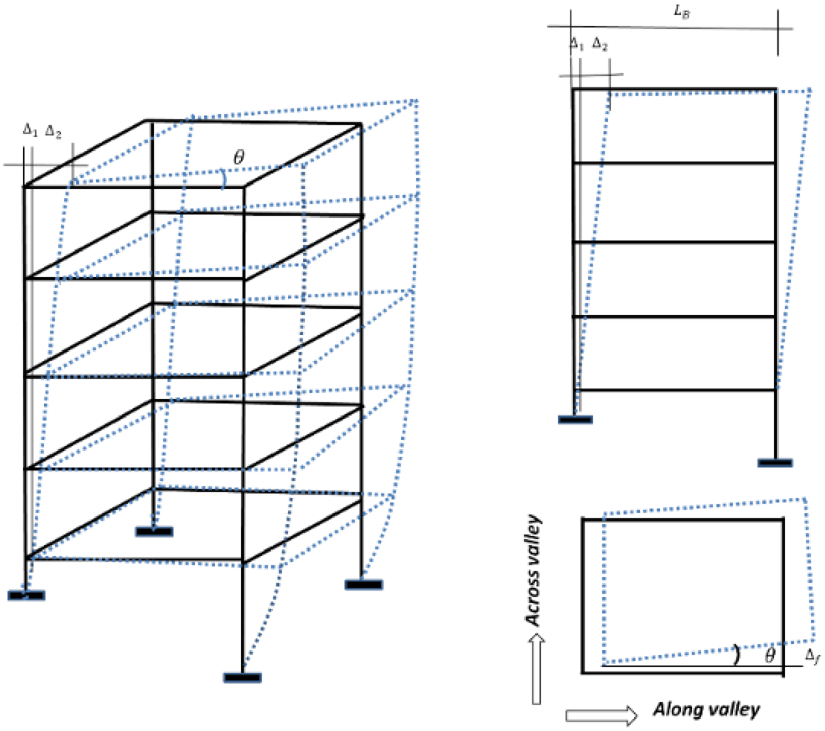


Figure 3.11: Deformation pattern of buildings resting on hill slopes

The following parameters, defined by equations (11) and (12), correlate deformations with increasing slope, AR, and SR.

a) The parameter D is defined as

$$D = \frac{\Delta_1}{\Delta_2} \tag{11}$$

b) The parameter T_t defined as

$$T_t = \theta = \tan^{-1}\left(\frac{\Delta_f}{L_B}\right) \tag{12}$$

c) The parameter Tb is defined as

$$Tb = \theta = \tan^{-1}\left(\frac{\Delta_{f,Storey\ 1}}{L_B}\right) \tag{13}$$

Ta and Tb are calculated for all the models and further analysed for correlation among variables.

3.6.1 CORRELATION OF AR WITH DRIFTS

With increasing AR, the lower stories of building models become rigid, and deformation in the lower stories reduces. Hence, the ratio of rigid deformations to flexible deformations in plan and elevation is critical, and the calculations according to equations are outlined in Table 3.9.

Figure 3.12 shows the variation of deformations with modification of AR and slope. The correlation among variables is discussed below:

1. **D:** The ratio of deformation in the lower story to the deformation at the rooftop correlated negatively with AR having a partial correlation factor of -0.47. The ratio indicates that with increasing AR, D reduces, i.e., bottom storeys behave like a rigid block, and upper storeys behave like a flexible block.
2. **Tb:** There is no significant correlation in twist measured at road level with AR.
3. **Tt:** Twist measured at the roof correlated positively with AR, having a partial correlation factor of 0.70. The ratio indicated that twist increases with increasing AR.

Table 3.15: Deformation ratios with varying AR

ID	Slope	AR	Drift Parameters		
			D	T_b	T_t
5_6_9_12	5	0.66	0.215	0.143	7.566
5_9_9_12	5	1	0.193	4.846	12.395
5_12_9_12	5	1.33	0.175	5.285	14.068
5_15_9_12	5	1.66	0.154	5.200	14.388
5_18_9_12	5	2.00	0.132	4.751	13.676
10_6_9_12	10	0.66	0.176	2.748	1.690
10_9_9_12	10	1	0.125	5.427	15.984
10_12_9_12	10	1.33	0.078	4.156	14.467
10_15_9_12	10	1.66	0.035	1.997	10.104

ID	Slope	AR	Drift Parameters		
			D	T_b	T_t
10_18_9_12	10	2.00	0.031	3.102	15.589
15_6_9_12	15	0.66	0.125	3.310	1.203
15_9_9_12	15	1	0.043	2.361	10.641
15_12_9_12	15	1.33	0.048	2.659	17.496
15_15_9_12	15	1.66	0.050	1.928	16.813
15_18_9_12	15	2.00	0.049	2.005	19.381
20_6_9_12	20	0.66	0.061	1.938	1.213
20_9_9_12	20	1	0.056	3.110	12.517
20_12_9_12	20	1.33	0.056	2.577	23.316
20_15_9_12	20	1.66	0.034	0.716	13.649
20_18_9_12	20	2.00	0.030	0.821	21.231

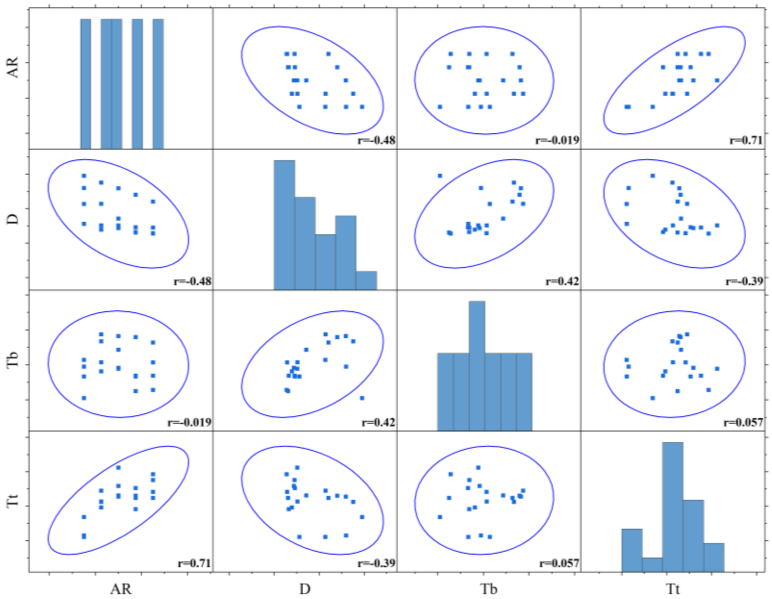


Figure 3.12: Correlation matrices for deformation ratios with varying AR

Table 3.16 Significance analysis (0.05 level) of Drift ratios with AR

	AR	D	Tb	Tt
AR	1	0.03	0.93	0.00
D	0.03	1	0.06	0.08
Tb	0.93	0.06	1	0.81

Tt	0.00	0.08	0.81	1

From analysis D, Tt are identified as variables having significant correlation

3.6.2 *CORRELATION OF SR WITH DRIFTS*

With increasing SR, the lower stories of building models become flexible, but deformation in the lower stories increases. Hence, the ratio of rigid deformations to flexible deformations in plan and elevation is critical, and the calculated according to equations are outlined in Table 3.10.

Figure 3.12 shows the variation of deformations with modification of SR and slope. The correlation among variables is discussed below:

1. **D:** The ratio of deformation in the lower story to the deformation at the rooftop correlated negatively with SR having a partial correlation factor of -0.45. The ratio indicates that with increasing SR, D reduces.
2. **Tb:** Twist measured at road level correlated positively with SR having a partial correlation factor of 0.63. The ratio indicated that twist increases with increasing SR.
3. **Tt:** Twist measured at the roof correlated positively with AR, having a partial correlation factor of 0.81. The ratio indicated that twist increases with increasing SR.

Table 3.17: Drift ratios with varying SR

ID	Slope	SR	Drift Parameters		
			D	T_b	T_t
5_9_9_6	5	0.667	0.420	2.348	4.011
5_9_9_9	5	1.000	0.267	3.738	8.394
5_9_9_12	5	1.333	0.193	4.846	12.395
5_9_9_15	5	1.667	0.150	4.890	13.315
5_9_9_18	5	2.000	0.122	4.346	12.335
10_9_9_6	10	0.667	0.308	2.240	4.276
10_9_9_9	10	1.000	0.180	3.934	10.081

ID	Slope	SR	Drift Parameters		
			D	$\boldsymbol{T_b}$	$\boldsymbol{T_t}$
10_9_9_12	10	1.333	0.125	5.427	15.984
10_9_9_15	10	1.667	0.095	5.975	18.715
10_9_9_18	10	2.000	0.077	5.503	17.768
15_9_9_6	15	0.667	0.150	0.834	2.462
15_9_9_9	15	1.000	0.070	1.407	5.711
15_9_9_12	15	1.333	0.043	2.361	10.641
15_9_9_15	15	1.667	0.032	6.249	13.676
15_9_9_18	15	2.000	0.025	5.983	13.110
20_9_9_6	20	0.667	0.217	1.178	2.767
20_9_9_9	20	1.000	0.565	6.353	8.736
20_9_9_12	20	1.333	0.391	9.976	16.763
20_9_9_15	20	1.667	0.297	11.244	23.835
20_9_9_18	20	2.000	0.236	11.689	23.920

3.7 Modal Properties

Modal analysis helps determine a system's characteristics without external loads, as shown in Figure 3.13. Modal properties help analyze the system's irregularity, i.e., increasing irregularity decreases mass participation. Hence, the correlation of modal properties with system properties is essential. Mass participation is identified as a critical parameter, and its probable correlation with other building parameters is analysed.

3.7.1 CORRELATION OF AR WITH MODAL PROPERTIES

With increasing irregularities, mass participation reduces. A description of modal response parameters considered for plotting the correlation matrices is shown in Table 3.11, and variation in modal response with increasing AR is shown in Figure 3.14.

Figure 3.14 shows the variation of deformations with modification of AR and slope. The correlation among variables is discussed below:

1. **R1$_Z$:** Contribution of rotational mass participation about a vertical axis in the first mode correlated significantly with AR with a partial correlation factor of 0.90, i.e., first mode rotational contribution is increasing with AR.

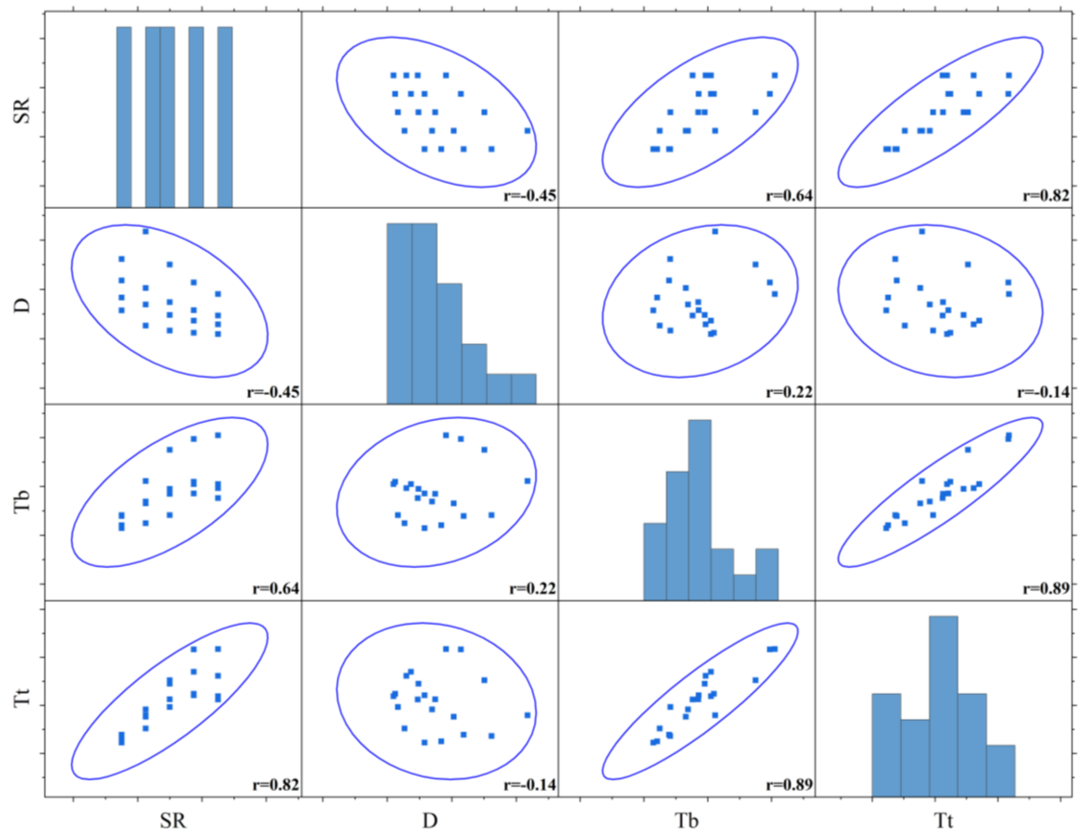


Figure 3.13: Correlation matrices for Drift ratios with varying SR

Table 3.18 Significance analysis (0.05 level) of Drift ratios with SR

	SR	D	Tb	Tt
SR	1	0.04	0.00	0.00
D	0.04	1	0.35	0.55
Tb	0.00	0.35	1	0.00
Tt	0.00	0.55	0.00	1

From analysis all the Drift variables are having significant correlation with SR

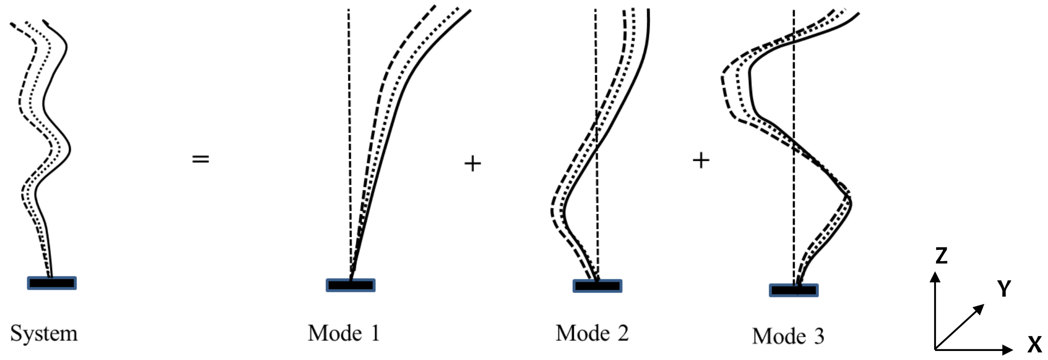


Figure 3.14: Anticipated variation of modal response

Table 3.19: Description of Modal properties

Modal property	Representation
The sum of Mass participation in the X-direction considered from the first three modes	SU_x
The Sum of Mass participation in the Y-direction considered from the first three modes	SU_y
Sum of Mass participation in rotation about X	SR_x
Sum of Mass participation in rotation about Y	SR_y
Sum of Mass participation in rotation about Z	SR_z
Mass participation in rotation about Z-axis in the first mode	$R1_z$

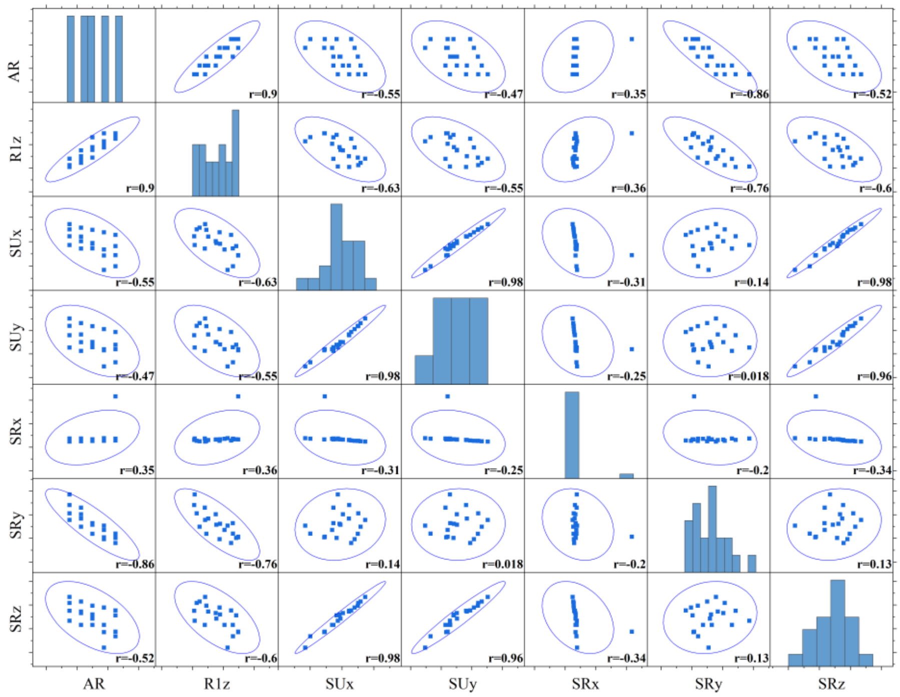


Figure 3.15: Correlation matrices for mass participation ratios with increasing AR

Table 3.20 Significance analysis (0.05 level) of mass participation ratios with AR

	AR	R1z	SUX	SUY	SRX	SRY	SRZ
AR	1	0.00	0.01	0.03	0.12	0.00	0.01
R1z	0.00	1	0.00	0.01	0.12	0.00	0.00
SUX	0.01	0.00	1	0.00	0.17	0.55	0.00
SUY	0.03	0.01	0.00	1	0.29	0.93	0.00
SRX	0.12	0.12	0.17	0.29	1	0.39	0.14
SRY	0.00	0.00	0.55	0.93	0.39	1	0.59
SRZ	0.01	0.00	0.00	0.00	0.14	0.59	1

From analysis it is identified that all the modal response variables are having significant correlation with AR

Table 3.21: Variation in modal response with increasing AR

ID	**Slope**	**AR**	**Modal Properties**					
			R1Z	**SUX**	**SUY**	**SRX**	**SRY**	**SRZ**
5_6_9_12	5	0.66	0.038	0.801	0.800	0.108	0.139	0.808

ID	Slope	AR	Modal Properties					
			R1Z	SUX	SUY	SRX	SRY	SRZ
5_9_9_12	5	1	0.021	0.781	0.783	0.116	0.117	0.787
5_12_9_12	5	1.33	0.042	0.769	0.772	0.120	0.093	0.768
5_15_9_12	5	1.66	0.082	0.755	0.762	0.125	0.076	0.759
5_18_9_12	5	2.00	0.117	0.740	0.751	0.130	0.063	0.746
10_6_9_12	10	0.66	0.013	0.771	0.773	0.123	0.161	0.780
10_9_9_12	10	1	0.048	0.730	0.742	0.138	0.143	0.741
10_12_9_12	10	1.33	0.072	0.692	0.715	0.150	0.124	0.707
10_15_9_12	10	1.66	0.096	0.656	0.689	0.162	0.106	0.678
10_18_9_12	10	2.00	0.132	0.675	0.705	0.152	0.080	0.723
15_6_9_12	15	0.66	0.005	0.731	0.740	0.141	0.188	0.744
15_9_9_12	15	1	0.037	0.661	0.683	0.167	0.179	0.679
15_12_9_12	15	1.33	0.083	0.688	0.707	0.152	0.126	0.727
15_15_9_12	15	1.66	0.138	0.621	0.685	0.151	0.082	0.645
15_18_9_12	15	2.00	0.138	0.623	0.689	0.974	0.084	0.647
20_6_9_12	20	0.66	0.007	0.681	0.696	0.168	0.223	0.698
20_9_9_12	20	1	0.070	0.703	0.715	0.148	0.158	0.733
20_12_9_12	20	1.33	0.122	0.661	0.696	0.151	0.128	0.678
20_15_9_12	20	1.66	0.106	0.538	0.628	0.172	0.120	0.573
20_18_9_12	20	2.00	0.122	0.560	0.642	0.165	0.094	0.626

1. $\boldsymbol{SU_x}$: Sum of mass participation in the first three modes along the valley direction correlates negatively with AR with a partial correlation factor of 0.55. This indicates increasing irregularity.
2. $\boldsymbol{SU_y}$: Sum of mass participation in the first three modes across the valley direction also correlates negatively with a partial correlation factor of 0.47.
3. $\boldsymbol{SR_x}$: Sum of mass participation in rotation about the valley correlates positively with AR having a partial correlation factor of 0.35.

4. SR_y: Sum of mass participation in rotation across the valley correlates negatively with AR having a partial correlation factor of -0.85.
5. SR_z: Sum of participation of mass in rotation about vertical direction correlates negatively with AR having a partial correlation factor of 0.52.

3.7.2 CORRELATION OF SR WITH MODAL PROPERTIES

Figure 3.15 shows the variation of deformations with modification of SR and slope. The correlation among variables is discussed below:

1. $R1_z$**:** Contribution of rotational mass participation about the vertical axis in the first mode correlated significantly with SR with a partial correlation factor of -0.88, i.e., first mode rotational contribution decreases with SR.
2. SU_x: Sum of participation of mass in the first three modes along the valley direction correlates positively with SR with a partial correlation factor of 0.41
3. SU_y: Sum of participation of mass in the first three modes across the valley direction also correlates positively with a partial correlation factor of 0.39
4. SR_x: Sum of participation of mass in rotation about valley correlates positively with SR having a partial correlation factor of 0.05
5. SR_y: Sum of participation of mass in rotation across the valley correlates positively with SR having a partial correlation factor of 0.33
6. SR_z: Sum of participation of mass in rotation about vertical direction correlates positively with AR having a partial correlation factor of 0.35.

Table 3.22: Variation in modal response with increasing SR

ID	Slope	SR	Modal Properties					
			R1Z	**SUX**	**SUY**	**SRX**	**SRY**	**SRZ**
5_9_9_6	5	0.66	0.110	0.807	0.809	0.039	0.039	0.809
5_9_9_9	5	1	0.045	0.787	0.789	0.081	0.081	0.791
5_9_9_12	5	1.33	0.021	0.781	0.783	0.116	0.117	0.787
5_9_9_15	5	1.67	0.012	0.780	0.782	0.141	0.142	0.786

ID	Slope	SR	Modal Properties					
			R1Z	SUX	SUY	SRX	SRY	SRZ
5_9_9_18	5	2	0.007	0.780	0.781	0.159	0.160	0.787
10_9_9_6	10	0.66	0.178	0.689	0.712	0.058	0.061	0.700
10_9_9_9	10	1	0.091	0.715	0.730	0.103	0.108	0.742
10_9_9_12	10	1.33	0.048	0.730	0.742	0.138	0.143	0.741
10_9_9_15	10	1.67	0.027	0.740	0.749	0.162	0.168	0.750
10_9_9_18	10	2	0.017	0.747	0.755	0.178	0.183	0.757
15_9_9_6	15	0.66	0.137	0.489	0.560	0.085	0.097	0.537
15_9_9_9	15	1	0.098	0.624	0.658	0.354	0.388	0.647
15_9_9_12	15	1.33	0.092	0.610	0.645	0.371	0.406	0.635
15_9_9_15	15	1.67	0.021	0.688	0.705	0.191	0.201	0.703
15_9_9_18	15	2	0.013	0.706	0.719	0.204	0.213	0.719
20_9_9_6	20	0.66	0.155	0.599	0.627	0.452	0.096	0.701
20_9_9_9	20	1	0.111	0.674	0.690	0.116	0.172	0.719
20_9_9_12	20	1.33	0.070	0.703	0.715	0.148	0.158	0.733
20_9_9_15	20	1.67	0.043	0.720	0.728	0.172	0.180	0.743
20_9_9_18	20	2	0.028	0.731	0.738	0.188	0.194	0.751

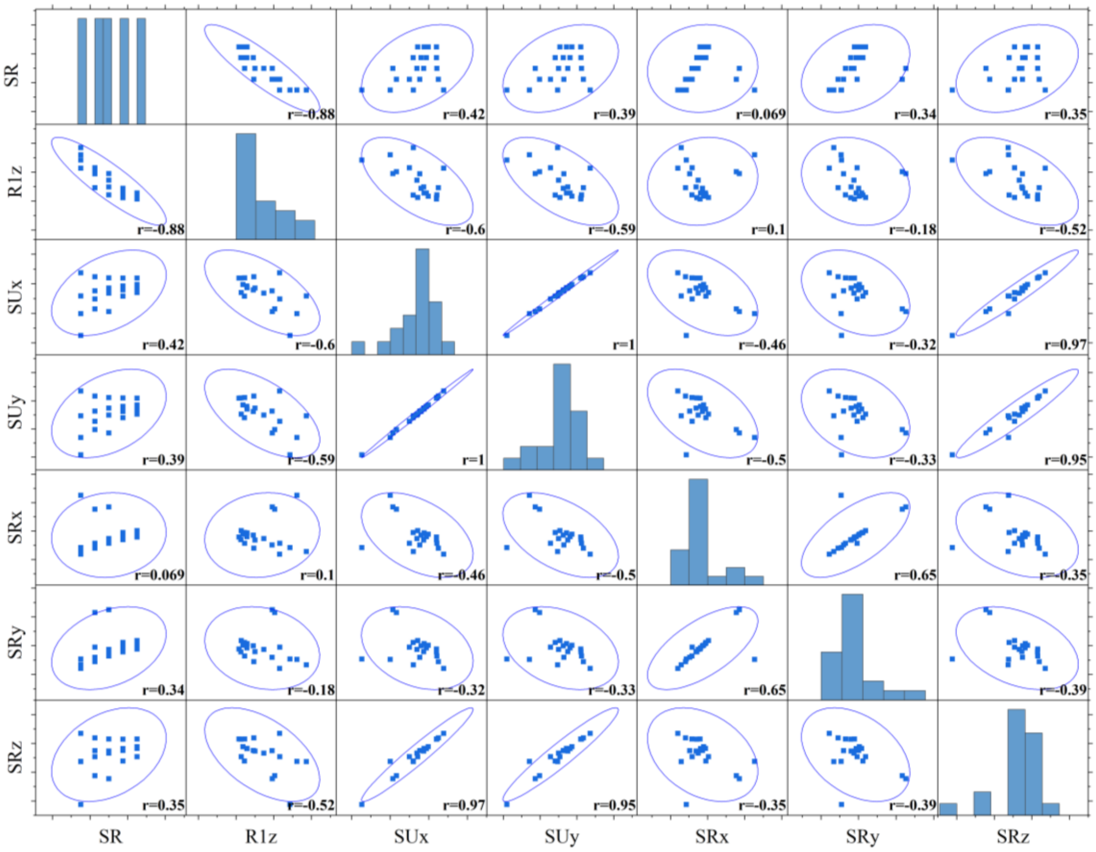


Figure 3.16: Correlation matrices for mass participation ratios with increasing SR

Table 3.23 Significance analysis (0.05 level) of mass participation ratios with SR

	SR	R1z	SUX	SUY	SRX	SRY	SRZ
SR	1	0.00	0.06	0.08	0.77	0.14	0.12
R1z	0.00	1	0.00	0.00	0.66	0.44	0.01
SUX	0.06	0.00	1	0.00	0.04	0.17	0.00
SUY	0.08	0.00	0.00	1	0.02	0.15	0.00
SRX	0.77	0.66	0.04	0.02	1	0.00	0.13
SRY	0.14	0.44	0.17	0.15	0.00	1	0.08
SRZ	0.12	0.01	0.00	0.00	0.13	0.08	1

From analysis R1z is identified as variable having significant correlation

3.8 Summary

Two parameters, i.e., Aspect Ratio (AR) and Slenderness Ratio (SR), are analyzed to identify the design and limiting variable. Stress ratios, strength, stiffness, drifts, and modal properties on a set of 40 models having varied AR and SR are analysed, and correlation matrices are plotted.

1. Evaluation of data on design forces reveals that beam axial stresses (BAS) and column shear stresses (CSS) increase with increasing AR, thus making AR a parameter to limit. Similarly, all the flexural stresses increase with SR, making an ideal design variable.
2. Strength and stiffness, which are interdependent, are evaluated in stories below and above the road level. It is usual practice to treat buildings on the slope as stiffness irregular. Still, Initial stiffness is assumed to be a known parameter in conjunction with assumed c/s in the traditional design method. Based on the data, building on slopes also has irregular strength distribution, making the lower stories weak. With increasing AR, the story's strength below the road level decreases.
3. Analysing the dynamic characteristics of structures revealed that rotation response in the first mode strongly correlates positively with AR. This would reduce the mass participation in translational modes, which is confirmed by correlation matrices. Similarly, with increasing SR, the rotation responses correlated negatively, i.e., increasing height reduces the first mode rotational response.

In summary, SR is a better design variable since it has a better correlation with flexural forces, first-mode rotational response, etc. Similarly, AR has a significant correlation with factors triggering negative effects, i.e., beam axial stresses and first-mode rotational response. Hence, during the design stage, the plan of the building represented by AR should be limited, and the height of the building, particularly the height of the story above and below the road level, could be increased to improve flexural stresses, modal response, and lower story drifts.

Chapter 4 Framework for Improving Seismic Behaviour of RC Buildings Resting on Hill Slopes

4.1 Need for Framework

Based on topography, principal components of buildings resting on hill slopes can be classified into two categories, i.e., the direction along the slope (valley direction) and the direction across the slope (ridge direction). Along the slope, with an increase in slope, a reduction in the column length will occur. This, in turn, makes the column and complete frame present on the uphill side shear predominant. The columns and frames present on the downhill side will be flexible. Further, because of the shift in the centre of stiffness due to the reduced length of columns, lower stories become stiffer in elevation, and upper stories become flexible, creating differential deformations, i.e., rigid deformations in lower stories and flexible deformations in upper stories.

Codes of practice tend to define and limit the irregularities in plan and elevation to counter the negative effects of buildings resting on hill slopes. Though the definitions, i.e., strength irregularity and stiffness irregularity, etc., hold good for buildings resting on hill slopes, the limits imposed, or analysis suggestions should be more precise for buildings resting on hill slopes.

In addition to the definitions, codes of practice also suggest height considerations in calculating the natural period of buildings resting on hill slopes. This is the preliminary step in the seismic design of structures. To be on the conservative side, considering short column height is suggested in calculating the base shear attracted to the structure. Few codes, like Eurocode 8 and ASCE 7, suggest site amplification factors for slope terrain. Both natural period calculation and site amplification affect the base shear calculation, but localized shear demand in short columns cannot be addressed with force calculation alone. The localized shear demand distribution is equally important, which is not specifically addressed in current design guidelines.

Further, it is usual practice to conduct analysis and design structures with assumptions like 100% fixity of columns to the ground and 100% rigid diaphragm. Though these assumptions are valid for buildings resting on flat ground, they will severely hamper the design and analysis outcomes for buildings resting on hill slopes.

To demonstrate the ill effects of the issues outlined above, nonlinear dynamic analysis is performed on the structure analysed and designed according to IS 1893:2016.

4.1.1 NONLINEAR MODELLING OF REFERENCE BUILDING

A reference building with a slope15^{o}and AR and SR of 1 are chosen from the above building catalogue. The nonlinear stress-strain behaviour of concrete is determined using Mander's model [51] and the cyclic rules proposed by Martinez-Rueda and Elnashai [52]. The stress-strain rules proposed by Menegotto and Pinto and the isotropic hardening rules proposed by Filippou et al. are used for rebar material [53], [54], as shown in Figure 4.1.

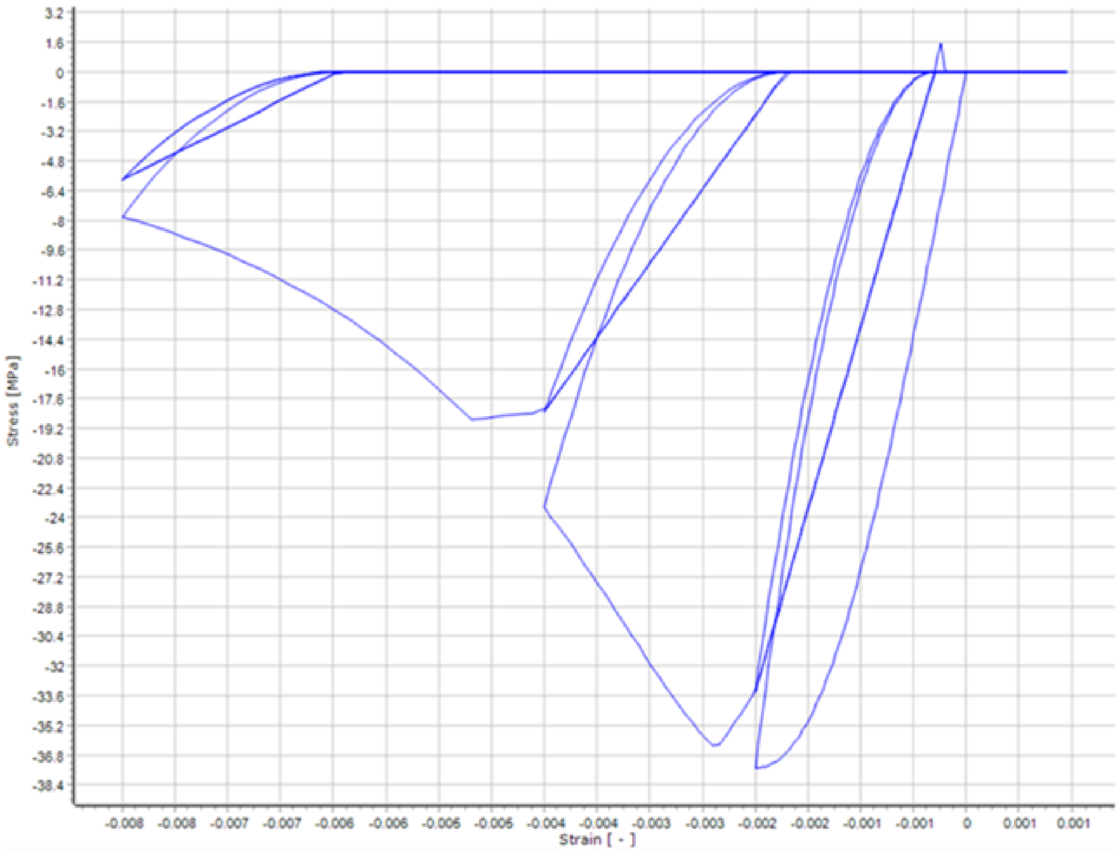
Beam and columns are modelled in the fiber approach to represent the cross-sectional behaviour where each fiber is associated with the stress-strain property described above. Distributed inelasticity can be implemented in two methods, i.e., force-based and displacement-based approaches. In the displacement approach, the displacement shape function corresponds to the linear variation of strain, whereas in the force approach, linear variation of the moment is assumed [55].

In non-linear dynamic analysis, hysteretic damping, which is usually responsible for the dissipation of most of the energy introduced by the earthquake energy, is already implicitly included in the nonlinear fiber model formulation. There is, however, a small quantity of the non-hysteretic type of damping that is also mobilised during the dynamic response of structures through phenomena such as friction between structural and non-structural members, friction in opened concrete cracks, energy radiation through the foundation, etc., that might not have been modeled in the analysis. Traditionally, such modest energy dissipation sources have been considered using Rayleigh damping with equivalent viscous damping values varying from 1% to 8%, depending on the structural type. In seismostruct, several options are available to model damping: (i) not to use any viscous damping, (ii) to employ stiffness-proportional damping, (iii) to introduce mass-proportional damping, or (iv) to utilise Rayleigh damping. In the current work, Rayleigh damping is implemented with an equivalent viscous damping value of 5%.

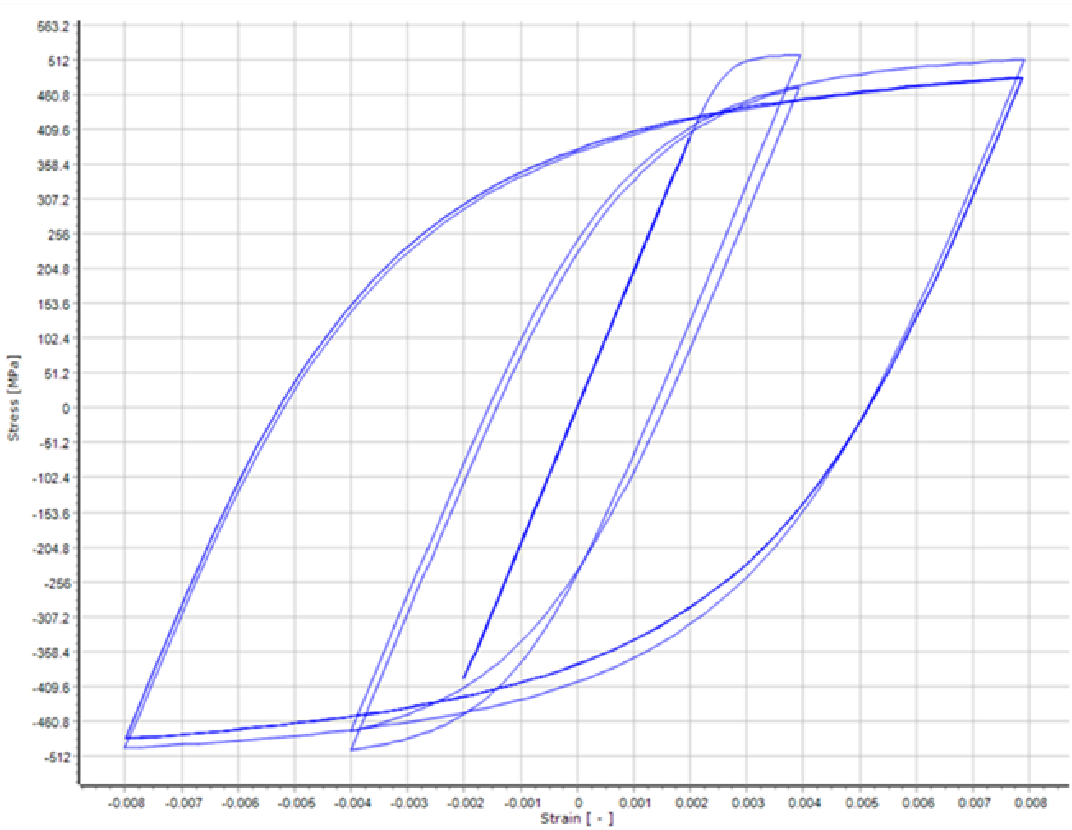
Damages in various elements are captured using limit state definitions. The process is straightforward for plastic hinge modelling. In the current work, distributed inelasticity is implemented, and hence, the following strain limits are monitored to observe the damage patterns. ASCE 41-17 [25] equations implemented in seismostruct [58] are monitored for shear and chord rotations. Table 4.1 enlists the damage stages and corresponding strain values.

4.1.2 GROUND MOTION SELECTION AND SCALING

Dynamic analysis is performed by selecting a suite of ground motions and scaling the ground motion suite by choosing an appropriate Intensity parameter. The building of interest is subjected to the scaled ground motions, and damage is monitored for a demand parameter of interest.



(a)



(b)

Figure 4.1: (a) Cyclic Mander concrete Stress-Strain model (b) Menegotto-Pinto stress-strain model

Table 4.1: Damage characteristics monitored during Nonlinear analysis.

Damage type	Monitoring value
Crushing of unconfined concrete	0.0035(-)
Yielding of reinforcement	0.0043(+)
Crushing of confined concrete	0.008(-)
Usable strain limit	0.05(+)
Shear capacity	ASCE41-17 (col) and ACI 318 (beams)
Chord rotation capacity	ASCE 41-17

Given the inherent variability in ground motion records, design standards require multiple ground motions to measure reliable demand parameters. Currently, there is no consensus on the suite of ground motions. For design verification, ASCE 7-16 [47] specifies a minimum of 11 ground motions to determine the mean value of demand parameters for design purposes.

In the absence of site-specific ground motions, the usual approach is scaling a suite of ground motions. The ground's spectral shape significantly affects the structure's collapse capacity. There are three primary scaling methods: PGA scaling, spectral acceleration at the fundamental building period, and spectral acceleration over a range of periods. Several studies identified the efficient intensity measure to capture the structural response [27].

The peak ground acceleration overestimates the mean damage compared with spectral acceleration-based intensity measures. Further, it is also shown that spectral acceleration at a building's fundamental period does not capture information about spectral ordinates at higher modes or elongated periods, whereas spectral acceleration values over a range of periods $s_a(0.2T - 3T, 5)$ carry spectral shape effects [27]. Katsanos et al. [56] investigated the elongation of structural periods to refine the process of selection and scale of ground motion, i.e., $s_a(0.2T - 2T, 5)$ suggested in lierature. Based on observations, the authors proposed that the factor of 2.0 is highly conservative and should be revised to $s_a(0.2T - 1.5T, 5)$ at least for new buildings.

Seismo-select [59] is used to select and scale a suite of 11 ground motions to verify the design of sloped buildings. The selected ground motions are downloaded from the PEER database, and scaling factors for $s_a(0.2T - 1.5T, 5\%)$ are applied to the ground motion suite. Details of ground motions selected for spectral scaling are outlined in Table 4.2.

4.1.3 DAMAGE PATTERN

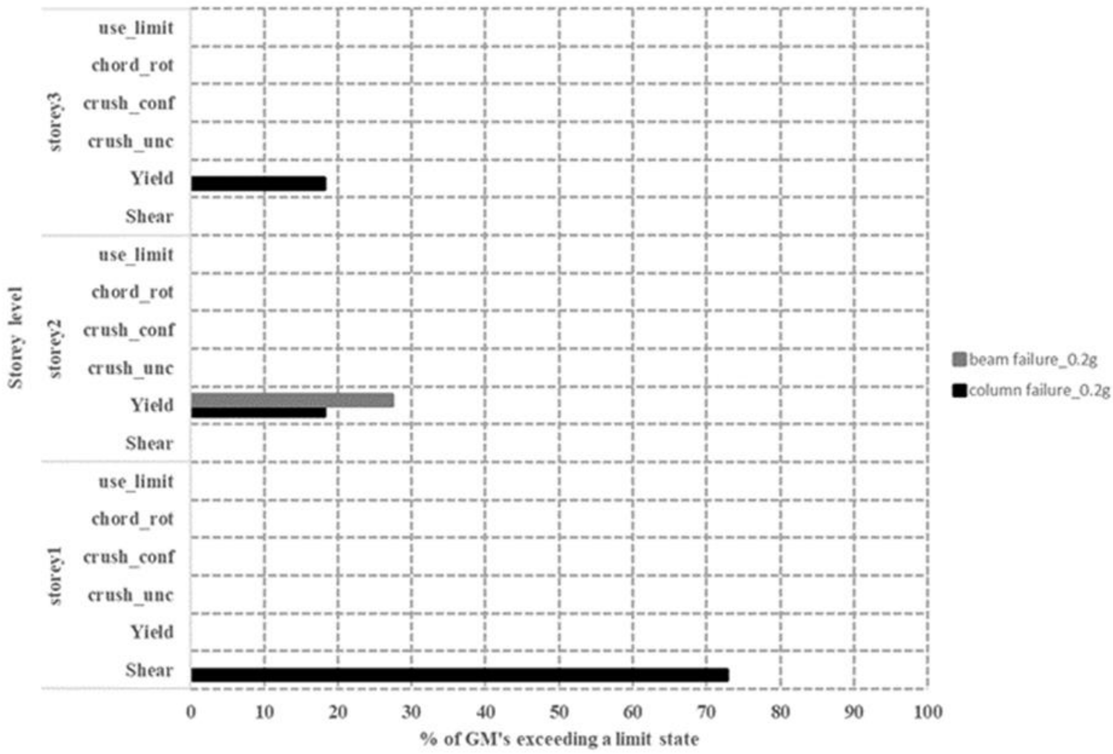
Nonlinear dynamic analysis is performed on the structure in two directions, i.e., along the valley and across the valley, by incrementing the ground motions from a spectral acceleration of 0.2g to observe the pattern of damage. The damage pattern is reported in the form of 2D histograms in which the Y-axis represents the limit states, and the X-axis represents the percentage of ground motions exceeding a particular limit state.

Table 4.2: Ground motion records for spectral scaling

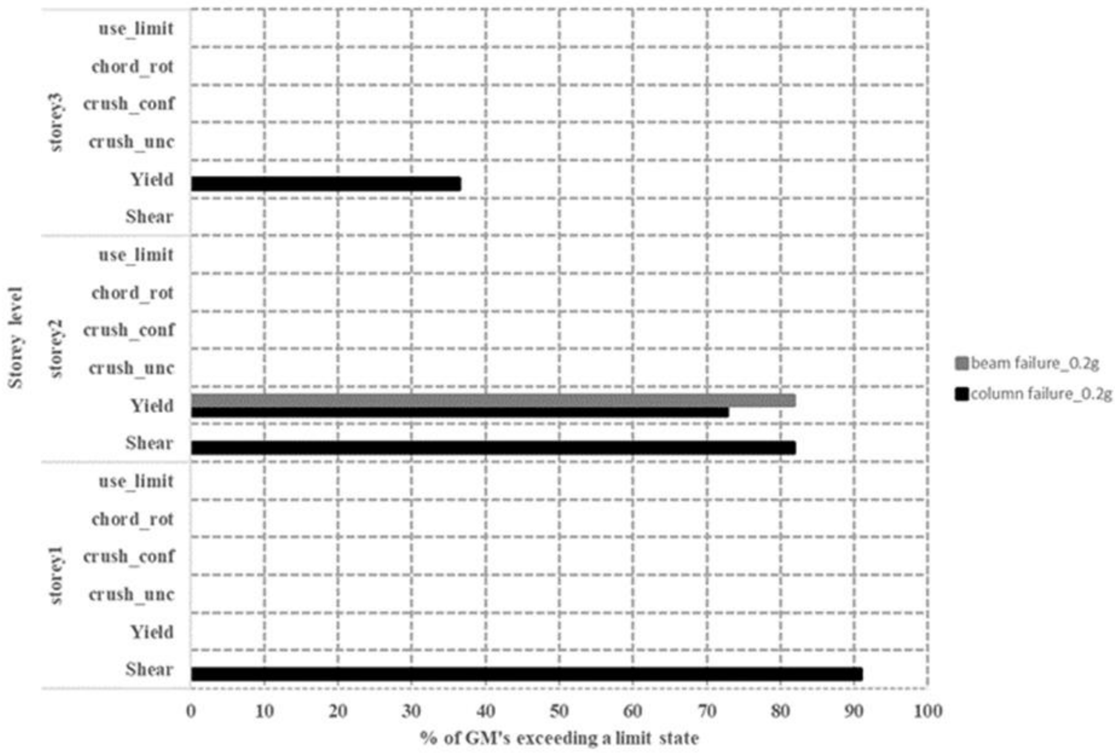
S.No.	Station	Earthquake	Station	Year	Mag.	Mechanism
1	RSN15	Kern Country	Taft Lincoln School	1952	7.36	Reverse
2	RSN 66	San Fernando	Hemet fire station	1971	6.61	Reverse
3	RSN464	Morgan Hill	Hollister Array#3	1984	6.19	Strike-slip
4	RSN797	Loma Prieta	SF-Rincon Hill	1989	6.93	Reverse oblique
5	RSN840	Landers	BigTujunga_Angeles	1992	7.29	Strike-slip
6	RSN929	Big Bear-01	Salton city	1992	6.46	Strike-slip
7	RSN1029	Northridge	Leona Valley#3	1994	6.69	Reverse
8	RSN3457	Chi-Chi	TCU050	1999	6.3	Reverse
9	RSN4076	Parkfield-02	San Luis Obispo	2004	6.0	Strike-slip
10	RSN4466	L'Aquila_Italy	Carsoli 1	2009	6.3	Normal
11	RSN8102	Christchurch	LINC	2011	6.2	Reverse oblique

From Figure 4.2 (a), the predominant failure when the structure is subjected to ground motion along the valley direction is the shear failure of columns in story 1 followed by the yielding of columns in story 2. and story 3. The shear failure of columns in story 1 occurred in ~75% of considered ground motions. Similarly, the yielding of columns in story 2 and story 3 occurred in ~20% of ground motions. Failure of beams is insignificant in story 1 and story 3, whereas beams in story 2 yielded ~30% of ground motions.

The failure mechanism across the valley observed in Figure 4.2 (b) further reinforces the shear failure of columns in story 1 occurred in ~90% of considered ground motions. In addition, shear failure of columns in story 2 occurred in ~80% of ground motions. Similarly, the yielding of columns in story 2 and story 3 occurred in ~20% of ground motions. Failure of beams is insignificant in story 1 and story 3, whereas beams in story 2 yielded in ~80% of ground motions.



(a)



(b)

Figure 4.2: Pattern of damage observed when ground motions are scaled to 0.2g in reference model (a) along the valley (b) across the valley.

4.2 Components of Framework

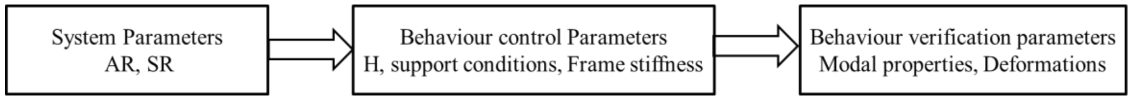


Figure 4.3: Components of the framework

From correlation matrices, response spectrum, and nonlinear analysis, it is apparent that the problem of buildings on slopes is multifaceted along and across the slope. Some of the significant issues identified are outlined in Figure 4.3, which form the framework's basis.

a) System Parameter:

System parameters are variables whose influence on the design of the whole structure is negative. In the current methodology, system variables are determined to impose limits. To determine the system parameter, it is important to determine the positive and negative impact of the parameter on the behavior.

Correlation matrices are plotted for the following parameters to identify the system variable, as shown in Figure 4.4. The criteria set for choosing a system variable is that all the parameters triggering negative features in (i) design forces, i.e., BAS, CSS, (ii) dynamic characteristics, i.e., R1z, and (iii) dynamic response, i.e., D and Tt, are correlated with anticipated negative system properties AR and eccentricity (ex).

Figure 4.4 clearly explains that all the parameters which are triggering the adverse effects in the behaviour of buildings resting on hill slopes correlated well with, ex, AR making them ideal system parameters to impose limits.

b) Behaviour Control Parameter:

Stiffness irregularity forms the crux of the problem of buildings resting on hill slopes. The degree of slope and building length along the slopes enhances the rate at which stiffness irregularity increases. Higher slope angles and longer buildings would comprise the length of the columns and increase the stiffness. The end moments and forces in columns used in deriving the stiffness matrix are shown in Figure 4.5.

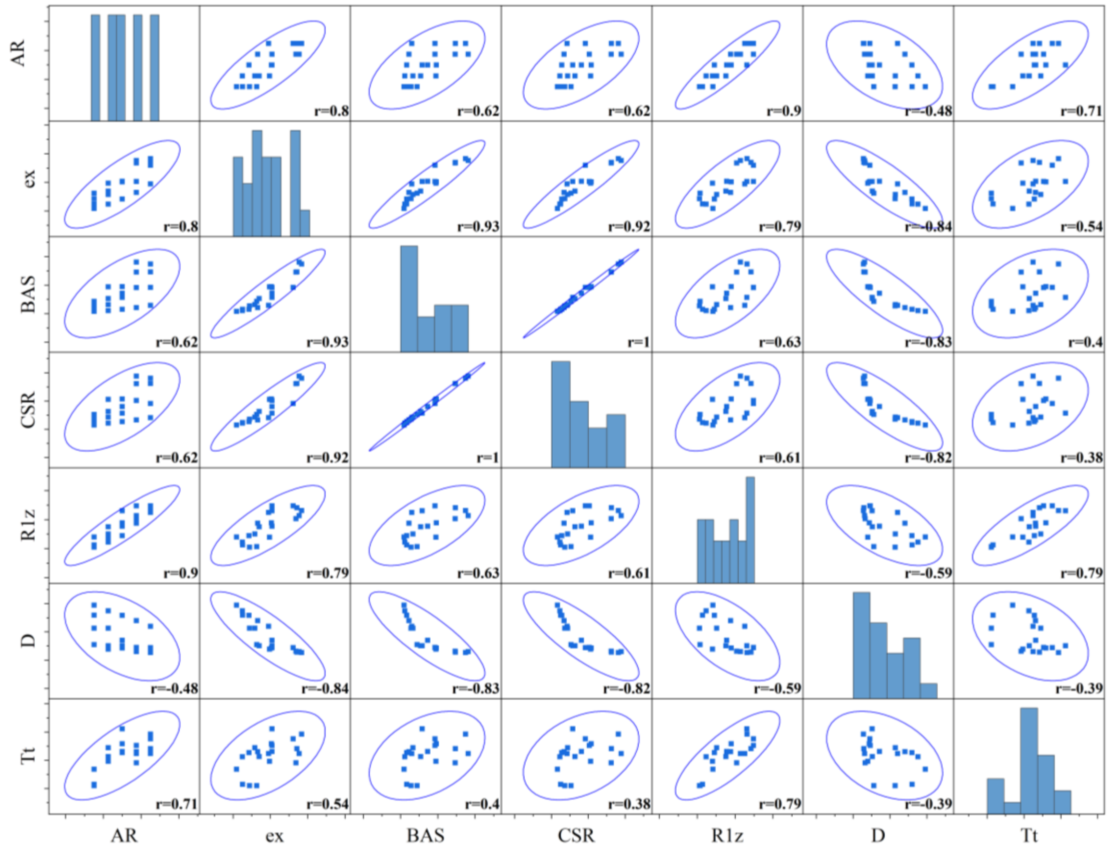


Figure 4.4: correlation among various parameters triggering adverse effects.

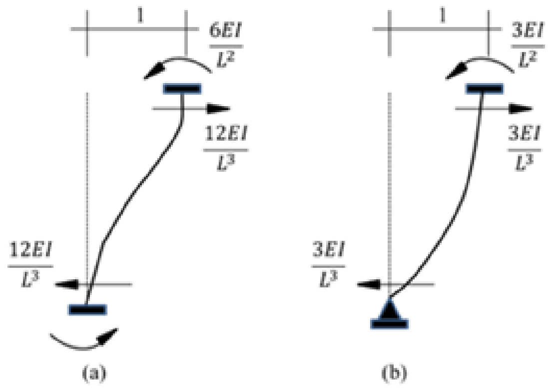


Figure 4.5: End moments and forces in column (a) Fixed at the bottom (b) Hinged at the bottom

The end moments and forces for a column when the column is fixed and hinged at the bottom. From the forces shown, stiffness defined as force per unit deflection k for the condition of fixed and hinged at the base are given by equations ((14) and ((15)

$$M == \frac{6EI}{h^2}\ (fixed\ at\ base) \quad ((14)$$

$$M = \frac{3EI}{h^2}\ (hinged\ at\ base) \quad ((15)$$

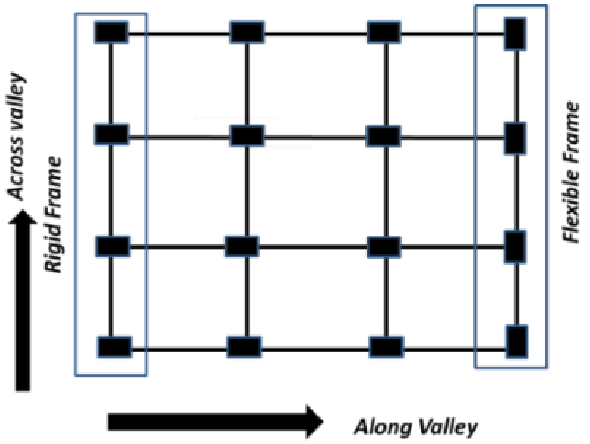


Figure 4.6: Behaviour of building across the valley

The reduction in moments because of end conditions reduces the shear force on the column. Similarly, an increase in the height of the column reduces the moment; thereby, shear demand reduces.

The problem of twisting across the valley can be countered by making the flexible frame stiff, and the same can be achieved in many ways, i.e., placing a structural wall down the hill or by reorienting valley frame columns, etc., as shown in Figure 4.6.

c) Behaviour Verification Parameters:

Irregularity in sloped buildings affects the following properties. Hence, parameters that need to be verified after modifying the suggestions mentioned in (a) and (b), i.e., (i) Design forces, (ii) deformations at the lower story, twist, and (iii) Modal properties.

4.3 Proposal of Framework

The improvement in the behaviour of buildings can be achieved by imposing limits on AR. The optimization should be verified by checking design forces, dynamic characteristics, and dynamic response. If the envisioned optimization is not achieved, behaviour control parameters along and across the valley should be modified. The iteration can be terminated once the intended optimization is achieved. The pictorial representation of the proposed framework is shown in Figure 4.7.

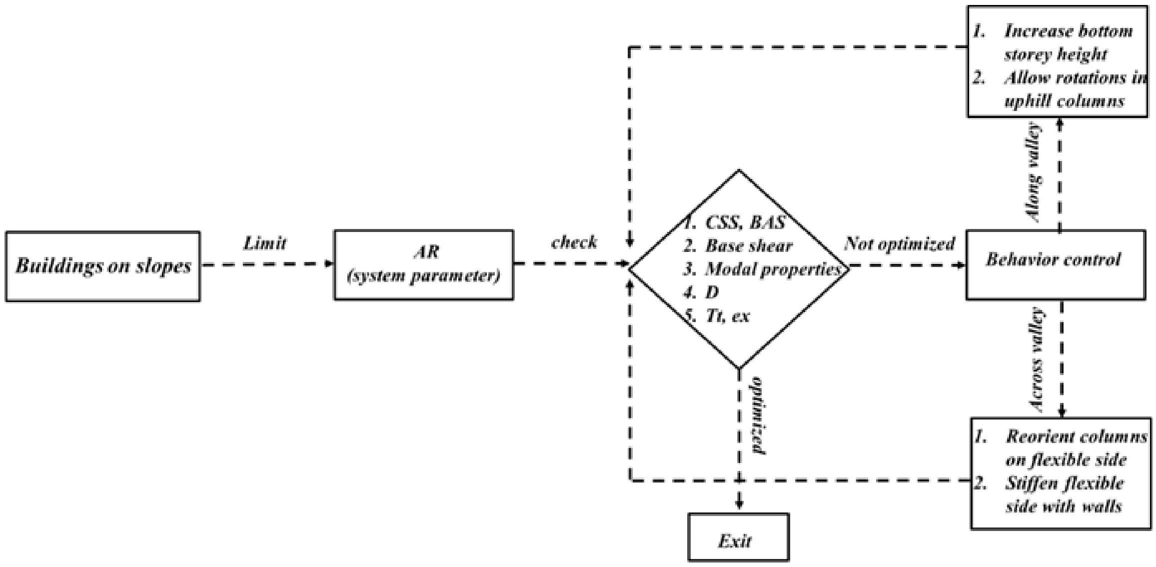


Figure 4.7: Proposed Framework

4.4 Validation of Proposal

Three models are created, along with a reference model named model 0. The reference model has a plan dimension of $9 \times 9m$, and the overall height of the building is 12m resting on 15^0 slope. Model 1 is created and redesigned by modifying the first story height, thereby changing SR. In addition to the first story height, longer frame stiffness is adjusted by reorienting the columns with larger dimensions across the valley in Model 2. Support conditions for 50% of the column lines, i.e., two rows of uphill columns, are modified from fixed support to hinged supports in Model 3 to allow rotations in uphill columns.

Shear forces obtained by performing Response spectrum analysis are normalized w.r.t shear forces obtained in uphill columns. The per-modification model has $\boldsymbol{S}_{r,\boldsymbol{avg}}$ i.e., SR is calculated based on the average height of the column ($\boldsymbol{H}_{\boldsymbol{avg}}$) whereas $\boldsymbol{S}_{r,\boldsymbol{v}}$ is the SR calculated based on the height of the valley column. Other parameters in the pre-modification model are support conditions (Fixed supports for all columns, hinge supports for uphill columns) and valley frame flexibility, which is vital in coupling rotational mass participation with translational mass participation.

Figure 4.9 shows that the deformations in lower stories are negligible both along and across the valley. With proposed modifications on Model 3, the deformations in the lower storey improved drastically, i.e., with improved deformations in lower stories, shear failure can be countered. IDR alone cannot give the complete picture of improved behaviour, and it is also essential to check the story shear ratios.

Table 4.3 explains the improvement of shear distribution ratios by modifying different parameters. For Model 1, the change in SR improved the shear distribution in uphill columns, whereas improvement is not observed in downhill columns. Model 2, where flexible frames are made rigid by reorienting, improves mass participation. Model 3 is the study's outcome in which relative shear ratios and modal participation in two translations, one rotation component, drastically improved. Model 3 is subjected to nonlinear analysis and compared with the reference model in terms of IDR, shear force distribution, and base shear distribution.

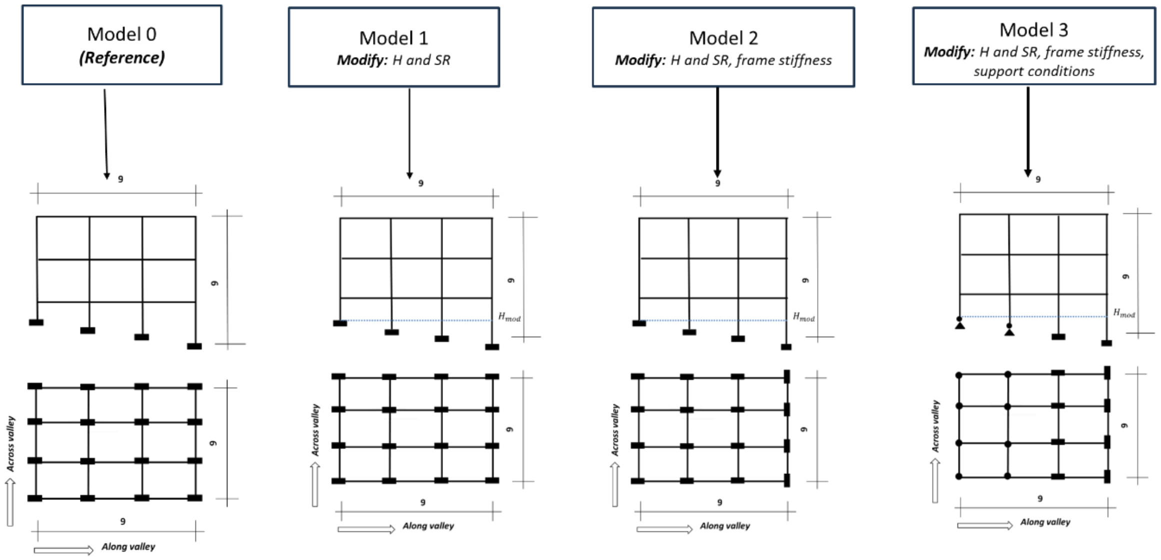
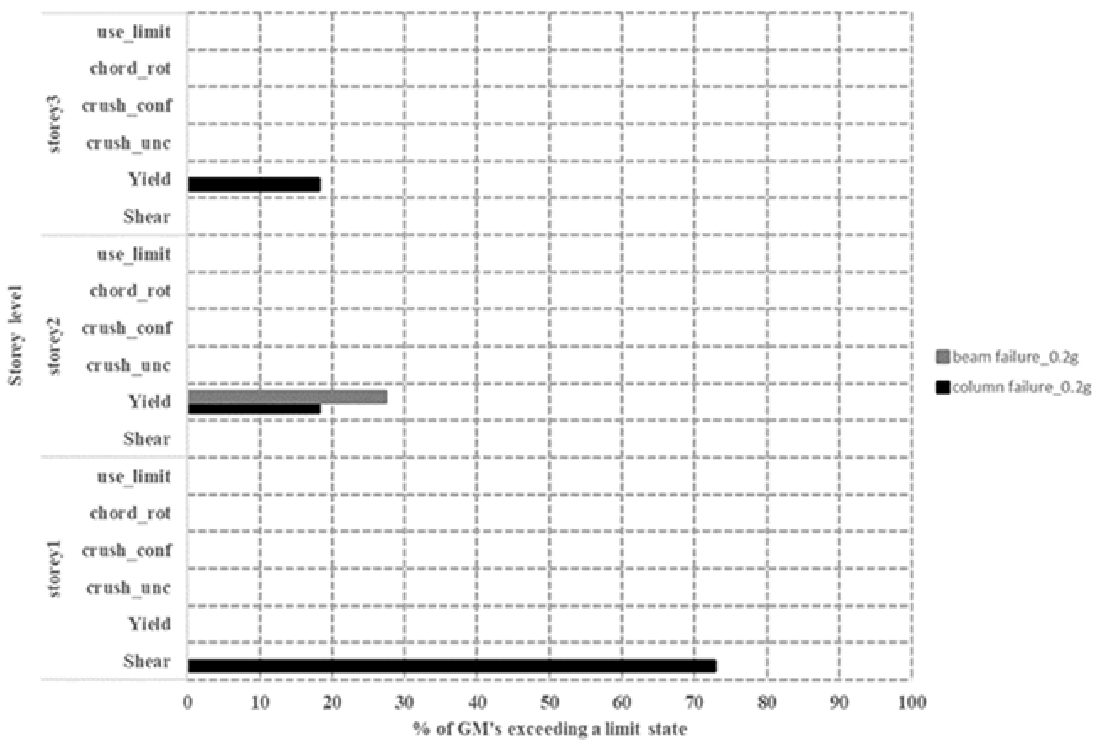


Figure 4.8: Validation models for verifying proposed methodology.

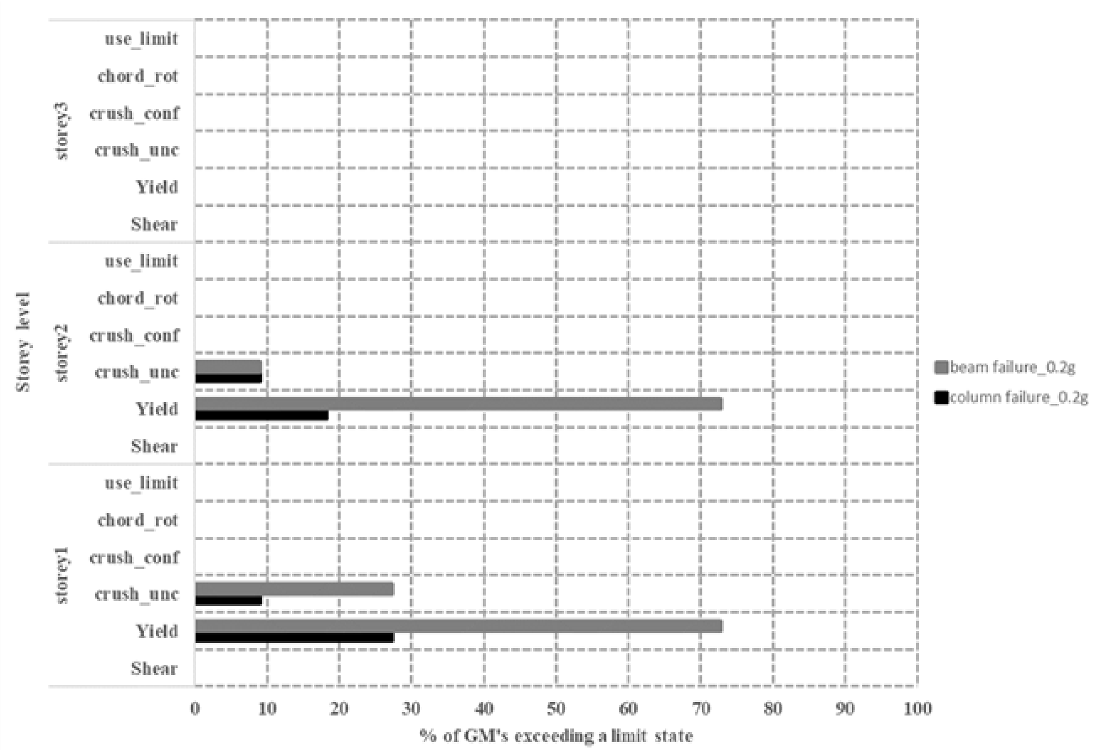
Table 4.3: Normalized shear forces w.r.t to uphill columns and participating mass ratios

Model	Column shear ratio	$\boldsymbol{M_{k,x}}$	$\boldsymbol{M_{k,y}}$	$\boldsymbol{M_{k,\theta_z}}$
Reference Model (I) Slenderness ratio: $\boldsymbol{S_{r,avg} = 0.8660}$; $\boldsymbol{S_{r,v} = 1}$ $\boldsymbol{h_{1,avg} = 1.7942}$; $\boldsymbol{H_{avg} = 7.7942}$ (b) Fixity: All supports are fixed (c) Frame flexibility: Not modified	Col 1=1.00 Col 2= 0.05 Col 3=0.08 Col 4=0.08	$M_{1,x} = 0.00$ $M_{2,x} = 0.60$ $M_{3,x} = 0.00$	$M_{1,y} = 0.57$ $M_{2,y} = 0.00$ $M_{3,y} = 0.06$	$M_{1,\theta_z} = 0.09$ $M_{2,\theta_z} = 0.00$ $M_{3,\theta_z} = 0.54$

Model	Column shear ratio	$\boldsymbol{M_{k,x}}$	$\boldsymbol{M_{k,y}}$	$\boldsymbol{M_{k,\theta_z}}$
(II) Model 1 (a) Slenderness ratio: $\boldsymbol{S_{avg} = 1.1142}$; $\boldsymbol{S_{r,v}} = \boldsymbol{1.25}$ $\boldsymbol{h_{1,avg}} = \boldsymbol{2.5283}$;$\boldsymbol{H_{avg}} = \boldsymbol{10.0283}$ (b) Fixity: All supports are fixed (c) Frame flexibility: Not modified	Col 1=1.00 Col 2= 0.35 Col 3=0.09 Col 4=0.03	$M_{1,x} = 0.00$ $M_{2,x} = 0.70$ $M_{3,x} = 0.00$	$M_{1,x} = 0.71$ $M_{2,x} = 0.00$ $M_{3,x} = 0.04$	$M_{1,\theta_z} = 0.07$ $M_{2,\theta_z} = 0.00$ $M_{3,\theta_z} = 0.66$
(III) Model 2 (a) Slenderness ratio: $\boldsymbol{S_{avg} = 1.1142}$; $\boldsymbol{S_{r,v}} = \boldsymbol{1.25}$ $\boldsymbol{h_{1,avg}} = \boldsymbol{2.5283}$;$\boldsymbol{H_{avg}} = \boldsymbol{10.0283}$ (b) Fixity: All supports are fixed (c) Frame flexibility: Column orientation in the valley frame modified	Col 1=1.00 Col 2= 0.35 Col 3=0.09 Col 4=0.01	$M_{1,x} = 0.00$ $M_{2,x} = 0.70$ $M_{3,x} = 0.00$	$M_{1,y} = 0.74$ $M_{2,y} = 0.00$ $M_{3,y} = 0.02$	$M_{1,\theta_z} = 0.04$ $M_{2,\theta_z} = 0.00$ $M_{3,\theta_z} = 0.69$
(III)Model 3 (a) Slenderness ratio: $\boldsymbol{S_{avg} = 1.1142}$; $\boldsymbol{S_{r,v}} = \boldsymbol{1.25}$ $\boldsymbol{h_{1,avg}} = \boldsymbol{2.5283}$;$\boldsymbol{H_{avg}} = \boldsymbol{10.0283}$ (b) Fixity: 50% fixed (c) Frame flexibility: Column orientation in the valley frame modified	Col 1=1.00 Col 2= 0.45 Col 3=0.87 Col 4=0.22	$M_{1,x} = 0.00$ $M_{2,x} = 0.81$ $M_{3,x} = 0.00$	$M_{1,y} = 0.82$ $M_{2,y} = 0.00$ $M_{3,y} = 0.00$	$M_{1,\theta_z} = 0.00$ $M_{2,\theta_z} = 0.00$ $M_{3,\theta_z} = 0.82$



(a)



(b)

Figure 4.9: Damage pattern in (a) Model 0(reference), (b) Model 3 (proposed) when the structure is subjected to a ground motion along the valley

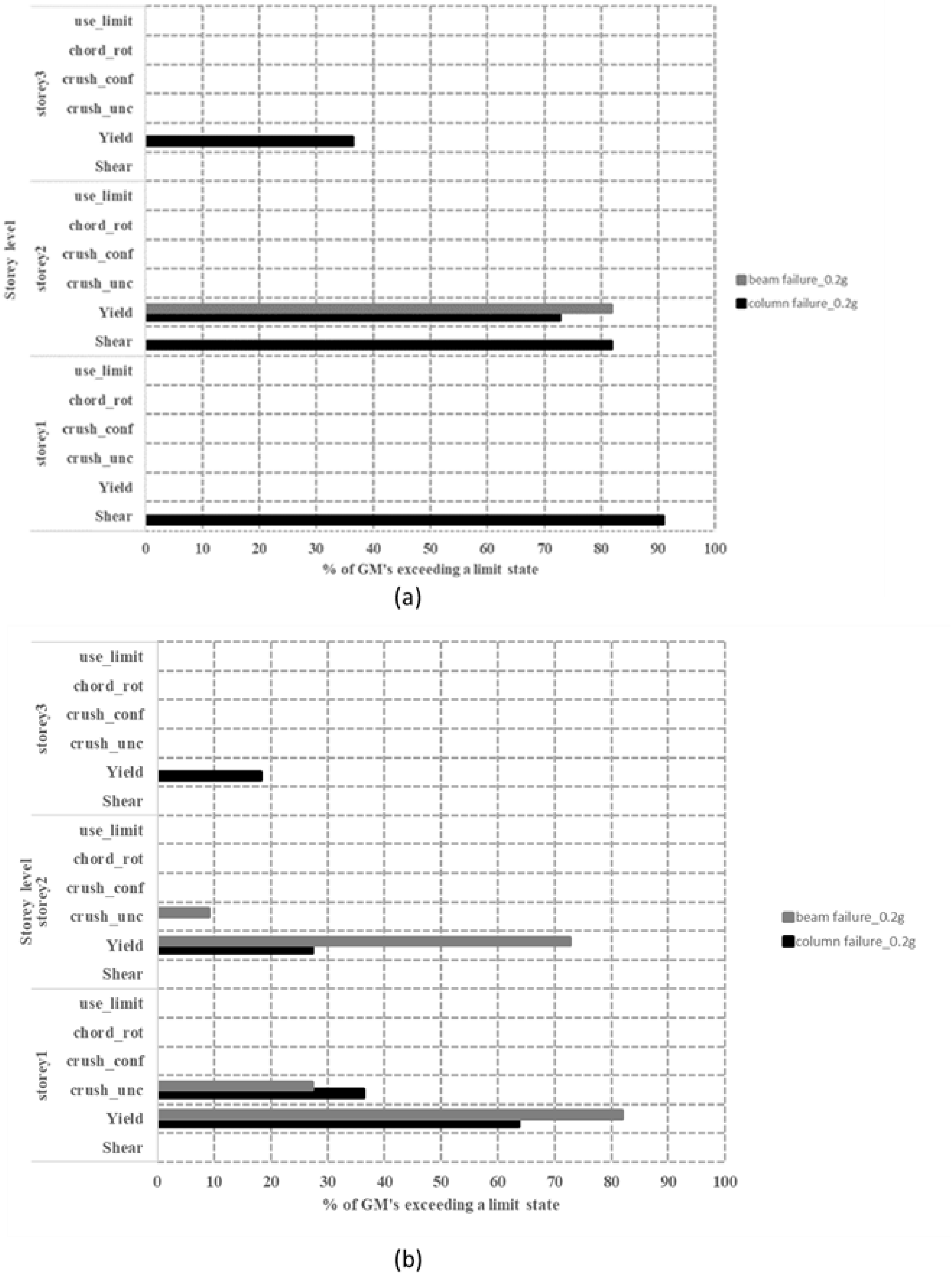
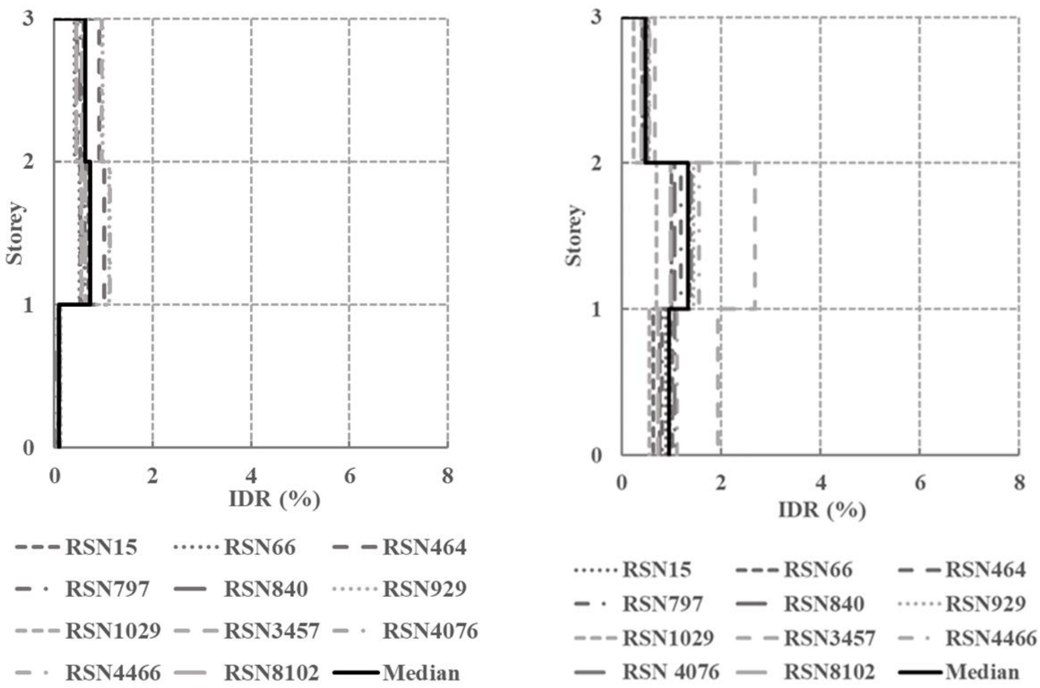
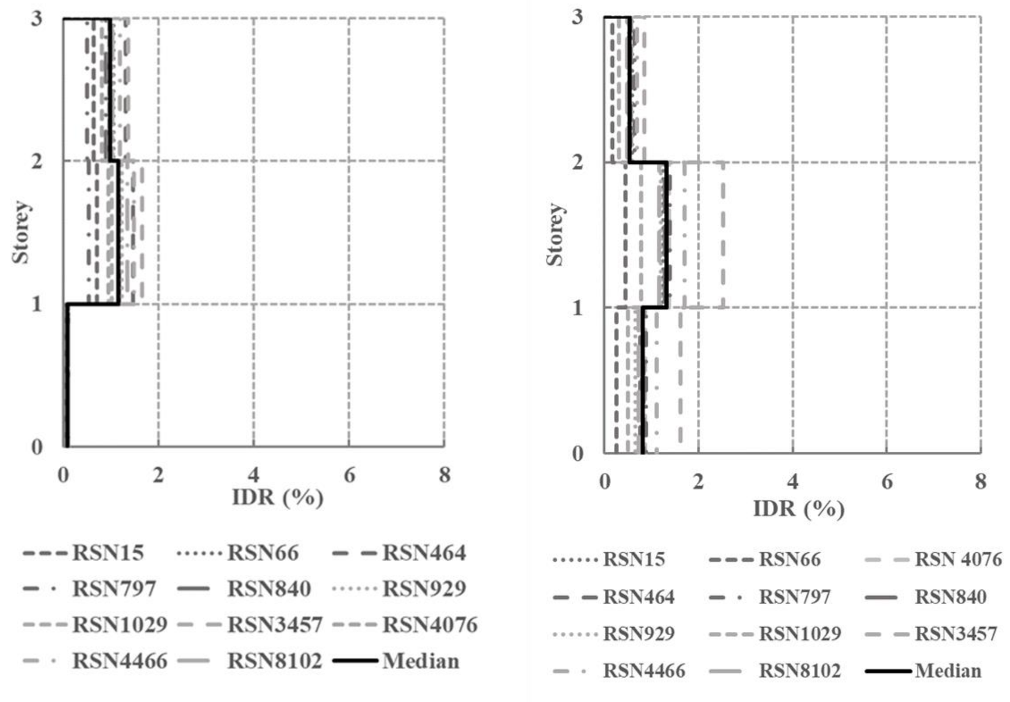


Figure 4.10: Damage pattern in (a) Model 0(reference), (b) Model 3 (proposed) when the structure is subjected to a ground motion across the valley



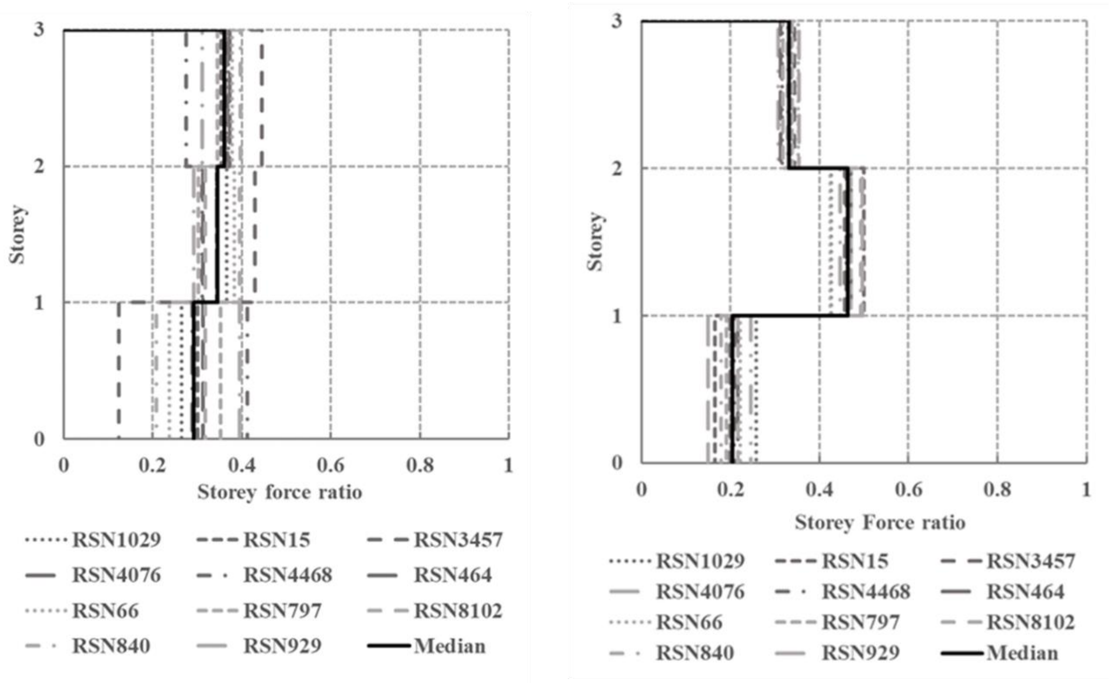
(a) (b)

Figure 4.11: Inter-storey drift ratios when ground motions are scaled to 0.2g for (a) Model 0 along the valley, (b) Model 3 along the valley



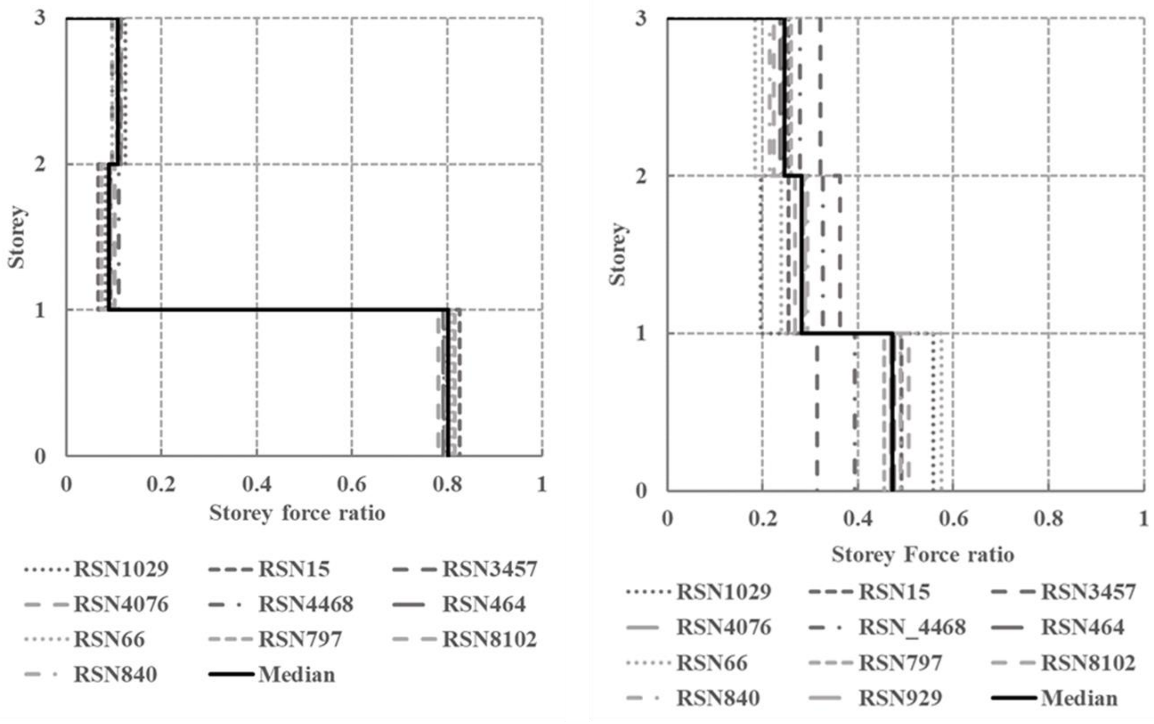
(a) (b)

Figure 4.12: Inter-storey drift ratios when ground motions are scaled to 0.2g for (a) Model 0 across the valley, (b) Model 3 across the valley



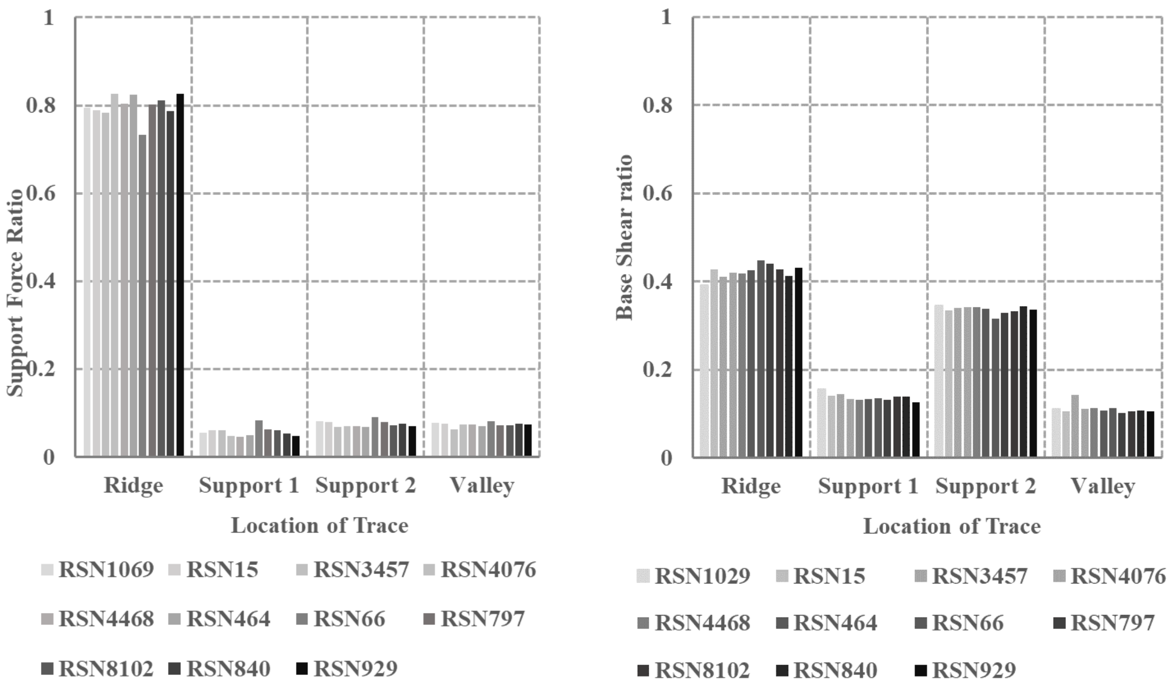
(a) (b)

Figure 4.13: Story force ratio when ground motions are scaled to 0.2g for (a) Model 0 (reference) along the valley, (b) Model 3 (proposed) along the valley



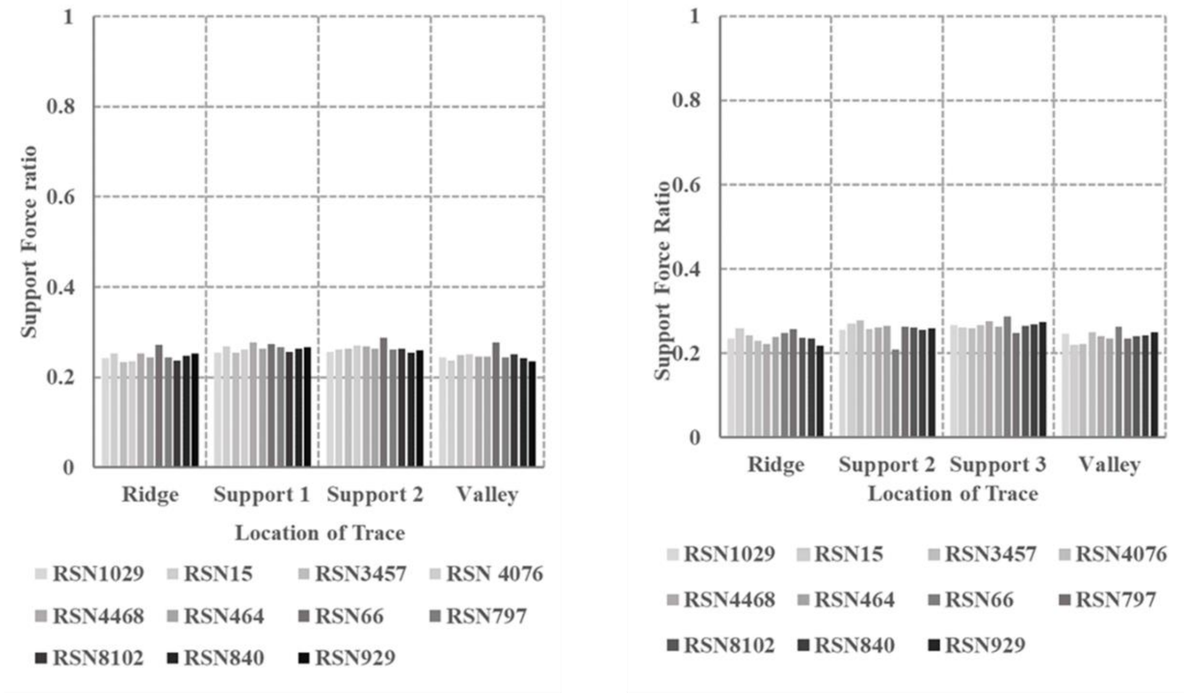
(a) (b)

Figure 4.14: Story force ratio when ground motions are scaled to 0.2g for (a) Model 0 (reference) across the valley, (b) Model 3 (proposed) across the valley



(a) (b)

Figure 4.15: Base shear ratio when ground motions are scaled to 0.2g for (a) Model 0 (reference) along the valley, (b) Model 3 (proposed) along the valley



(a) (b)

Figure 4.16: Base shear ratio when ground motions are scaled to 0.2g for (a) Model 0 (reference) across the valley, (b) Model 3 (proposed) across the valley

Figure 4.9 (a) and (b) explain the pattern of damage noticed in beams along and across the valley, respectively. Beams in lower stories yield first, which is a significant improvement in the proposed methodology. Figure 4.9 (a) and (b) explain the failure pattern observed when the structure is subjected to a scaled acceleration of 0.2g. Ground story columns yielded in 3 ground motions; in any case, shear failure of columns is not noticed, rendering the proposed method viable for implementation. The reason for such change can be explained in terms of inter-story drift ratio (IDR), storey force ratio calculated for a frame, and normalized base shear ratios.

Figure 4.13 (a) and (b) show the story force ratios for the reference model and proposed model along the valley for a scaled motion intensity of 0.2g. Figure 4.14 (a) and (b) show the story force ratios for (a) the reference model and (b) the proposed model across the valley. Short Columns along the valley in the reference model attract shear force as high as 80% of the total base shear. The proposed changes implemented in Model 3 improved the shear force distribution in columns along the valley and reduced the shear force demand on an uphill column by ~50%.

Figure 4.15 shows the column support shear distribution normalized with base shear. The critical issue of buildings on slopes is shear distribution in columns. The sequence of proposals listed above yields significant improvement in base shear distribution. Post-modification, the median shear distribution in columns is 0.40:0.15:0.35:0.10, which is way better than 0.78:0.079:0.063:0.069. More importantly, shear failure is controlled.

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Chapter 5 Conclusions and Future Work

5.1 Summary

The thesis highlights the impediments of the lack of special code provisions for buildings resting on hill slopes. The lack of guidelines prompts the practitioners to apply the assumptions valid for flat land buildings. In addition, the thesis also highlights immediate attention to the estimation and distribution of design base shear and to limit torsional irregularities for buildings resting on hill slopes. Correlation matrices are plotted on the results from linear elastic analysis of 40 study buildings. The necessity to develop a design framework is demonstrated by studying design forces, dynamic characteristics, and dynamic response. The performance of a reference building designed as per current IS design code provisions is assessed using nonlinear time history analysis. Due to the poor performance of the above reference building, its design is modified as per the proposed framework, and its seismic performance is investigated using nonlinear time history analysis. Finally, a comparison is made between the seismic performance of reference buildings and modified building to demonstrate the efficiency of the proposed framework.

5.2 Observations

1. Two parameters, i.e., AR and SR, are evaluated by plotting the correlation matrices for the following variables. From the correlation plots, SR turned out to be the design variable and AR to be the limiting variable. The reasons for SR becoming a design variable are essentially based on the correlation matrices plotted on (a) Design forces, (b) Dynamic characteristics, and (c) Dynamic response varying SR and AR.
2. The design forces are evaluated at a critical location, i.e., the joint connecting the uphill column and corresponding beam. The forces from the elements connecting the critical location are extracted and converted into stress factors. Correlation matrices plotted on the results of AR models show that there is a partial correlation between Beam Axial stress (BAS), Column shear stress (CSS), column flexural stress (CFS_M3), and AR with ratios of 0.61,0.61 and 0.70, respectively. The correlation

matrices of AR reveal that an increase in beam flexural stresses and shear stresses in the beam reduces the axial stress in beams and shear stresses in columns, as they are negatively correlated with values of 0.73 and 0.72, respectively. Similarly, the correlation data of design forces extracted from SR models indicate a strong correlation between flexural design stresses and SR. With SR, the design flexural stresses BFS, CFS_2, and CFS_3 correlated with ratios of 0.82. 0.975 and 0.92. Unlike AR models, in SR models, the stresses that induce brittle forces, i.e., BAS and CSS, correlated less significantly with factors 0.57 and 0.51, respectively. The ratios infer that modifying the height of the building (SR) helps retain the flexural forces. Increasing the length of the building along the slope induces brittle forces.

3. Analyzing the dynamic characteristics of structures revealed that rotation response in the first mode has a strong positive correlation ratio of 0.90 with AR. This would reduce the mass participation in translational modes, which is confirmed by correlation matrices. With increasing AR, mass participation along and across the valley correlated negatively with ratios of -0.55 and -0.47, respectively. Similarly, with increasing SR, the rotation responses correlated negatively with a ratio of -0.88, i.e., increasing height reduces the first mode rotational response. Also, increasing SR correlated positively with mass participation along and across the valley with ratios of 0.41 and 0.39, respectively. The results establish that increasing the height of the building, i.e., SR, is a better parameter for improving dynamic characteristics.

4. Dynamic responses of the models are evaluated for all the models. The results highlight that the relative deformations of lower stories w.r.t roof displacements are drastically decreasing with increasing AR. Correlation matrices plotted on the results of AR models show that there is a negative correlation with a ratio of -0.47 between D, i.e., relative deformation and AR. They are further increasing AR correlates with a ratio of 0.70 with Tt, i.e., twist angle. Similarly, correlation matrices plotted on SR models show a negative correlation with a ratio of -0.47 between D and SR. Increasing SR correlates with a ratio of 0.81 with Tt. The results conclude that neither the length of the building along the slope nor the height of the building are effective in controlling deformation and twist. Hence, additional treatments for improving the deformation response are required.

5. Nonlinear analysis is performed on a reference model to identify the damage pattern in a reference model. The damage results reinforced that the predominant failure is the shear failure of all the uphill columns (shorter columns) followed by the yielding of the immediate story column for a spectral acceleration value of 0.2g.

6. From the insights gained from the analysis performed on 40 models and damage patterns from nonlinear analysis, 3 models are created in addition to the reference model. The following modifications are performed on the structure (a) Model 1: SR increased (b) Model 2: SR increased; uphill columns are hinged to allow rotations in columns (c) Model 3: SR increased; uphill columns are hinged, and valley frame stiffened by reorienting columns. Mass participation and column shear distribution drastically improved for model 3. Model 3 has mass participation of 0.82, 0.81, and 0.82, respectively, in the first three modes. Model 0 (reference) model has mass participation of 0.57, 0.60, and 0.54 in the first three modes.

7. Nonlinear analysis performed on the proposed model gave promising results of completely restricting the shear failure of uphill columns. The deformation in the lower stories increased considerably. The median column shear distribution from 11 ground motions improved from 0.78(col 1):0.079 (col2):0.063(col3):0.069(col4) noticed in Model 0(reference) to 0.40:0.15:0.35:0.10 (proposed) model without shear failure tested till 0.8g.

5.3 Recommendations

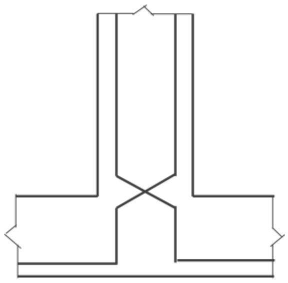
I. *Include Dimensions of the building in* design

The study demonstrates the importance of including dimensions of the building as controlling parameters. AR should be used as limiting parameter and SR as design parameter. For buildings on hill slopes, it is recommended to limit AR as close to 1 as possible. However, the suggestion does not recommend increasing number of stories while using SR as design parameter.

II. *Short column shear demand reduction*

The study highlights the significance of reducing the shear demands in short columns and to achieve the same, the study recommends two ways:

1. **Hinge supported short columns**: As shear is directly proportional to moment demands in column, providing hinge supports instead of fixed support condition significantly reduces the moments thereby shear demands will also be reduced. To achieve the objective hinging at site, necessary detailing measures should be taken care. A typical reinforcement detailing is shown below.



2. **Increase Bottom storey height:** Another way to reduce shear demand is by increasing the bottom storey height thereby increasing the SR. The increase in bottom storey should be small increments ($h_1 + 0.1h_1$) till half of the length of the valley column ($h_1 + 0.5h_1$)where point of contraflexure lies. Increment should be terminated once required shear demand is achieved.

III. Stiffening of valley frame

The study highlights the necessity of stiffening the valley frame. Depending on the complexity of flexible edge deformations, several possibilities exist. They are:

1. Reorienting the column with larger dimension across the valley
2. Increasing the size of valley frame columns
3. Provide bracings.
4. Provide structural walls.

It is recommended to terminate the stiffening of valley when one of the following conditions meet:

a. $\Delta_{flexible,frame} \cong \Delta_{rigid,frame}$
b. $M_{x|y,\theta_z} \cong 0$ (Mass participation of rotational modes in translational modes (x or y) is approximately 0.

5.4 Future work

1. The buildings on hill slopes can further be classified as

Item	Low Slope $<20^{\circ}$	Medium Slope $20^{\circ}-40^{\circ}$	Large Slope $>40^{\circ}$
Low AR(<2)			
Medium AR (2-4)			
Large AR (>4)			

The focus of the current work is limited to building models having low AR and slope angles. The application of the framework for different AR and slope angles mentioned in the above table needs to be verified.

2. Several configurations are essentially used in hilly regions, i.e., (a) slopped buildings, (b) Step back-set back buildings, (c) Split buildings, and (d) Stepped buildings. The proposed framework is for slopped buildings. The applicability and deviations in behaviour for other configurations need to be quantified.
3. RC moment resting frames are used in the current work. The presence of infills will significantly alter the stiffness and, thereby, deformations in the lower stories of RC MRF buildings resting on hill slopes and, hence, should be validated.
4. Soil-structure interaction is not considered in the present study. The proposed framework needs to be validated by incorporating interaction studies.
5. The proposed framework should be validated with experimental studies.

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Appendix A: Comparison of Irregularity Defined According to Various Country Codes

Along with the Indian standard code on earthquake-resistant design (IS 1893:2016), some widely used codes include ASCE 7-16, Eurocode EC8-2014, and New Zealand code NZS-2004. Table A.1 compares the irregularities emphasized in the codes.

Table A.1: Comparison of various national codes

Type	Parameter	**IS 1893**[46]	**ASCE7-16**[47]	**EC8:2011**[48]	**NZS:2004**[49]
Plan Irregular: **Torsion**	**Definition**	a. $\Delta_{max} < 1.4\ \Delta_{avg}$ b. $\Delta_{max} > 1.2\ \Delta_{avg}$ c. $\Delta_{max} > 1.4\ \Delta_{avg}$ (Amendment 2)	$drift_{ends} > 1.2 drift_{Avg}$ including accidental torsion $A_x = 1$	At each floor level, the structural eccentricity e and the torsional radius shall be in accordance with the two conditions: $e < 0.3r$ $r > l$	Horizontal Irregularity resulting from torsional sensitivity exists when γ exceeds 1.4. γ_i is $\frac{d_{max}}{d_{avg}}$ γ is the max of all values of γ_i In both orthogonal directions
	Limits	a. If $1.2 < \Delta_{max} < 1.4$ i. Revise configuration ii. Ensure fundamental Torsion mode is	If $drift_{ends} > 1.4 drift_{Avg}$ including accidental torsion, $A_x = 1$ then torsion is extreme	The slenderness ratio shall not be greater than 4	-

Type	Parameter	IS 1893[46]	ASCE7-16[47]	EC8:2011[48]	NZS:2004[49]
		less than translational mode. b. If $\Delta_{max} > 1.4$ Building configuration must be revised			
	Analysis suggestion	.	Structures having torsional Irregularity should be analyzed using a 3D representation. b. Equivalent static analysis is not permitted	-	-
Un-balanced lateral strength	**Definition**	-	-	-	Potential increase in lateral displacements because of unbalanced lateral strength is calculated using the ratcheting Index, which is calculated as: a) $r_i = r_{i,1} + r_{i,2}$ $r_{i,1} = \frac{s_f}{s_r} > 1$ and $r_{i,2} = \frac{s_g}{s_r}$ where

Type	Parameter	IS 1893[46]	ASCE7-16[47]	EC8:2011[48]	NZS:2004[49]
					s_f and s_r are lateral strength in forward and reverse direction s_g is the change in lateral strength due to a portion of the eccentric gravity load
	Limitation	-	-	-	If r_i<1.5 effects of ratcheting can be neglected
	Analysis suggestion				If r > 1.5_i then time history analysis shall be used
Re-entrant corner	**Definition**	$A_{proj} > 0.15A_p$	$A_{proj} > 0.15A_p$	a. Each floor shall be delimited by a convex polygon line. Even if re-entrant corners exist, regularity of plan may be assumed provided re-entrant corners and the area between the outline of the floor and	-

Type	Parameter	IS 1893[46]	ASCE7-16[47]	EC8:2011[48]	NZS:2004[49]
				convex polygon line does not exceed 5% of the floor area	
	Limitation	-	-	-	-
	Analysis suggestion	Three-dimensional Dynamic analysis should be adopted for buildings with re-entrant corners.	All analysis procedures are permitted for structures with irregularities for heights less than 48.8m (160 ft).	a. If a building is irregular in plan irrespective of elevation irregularity, then There is no need to decrease behavioral factor value. b. If a building is irregular in plan and elevation, modal analysis is preferred. The behavioral factor value should be decreased by 20%.	
Floor slabs having excessive openings.	**Definition**	$A_{open} > 0.5A_p$	$A_{open} > 0.5A_{gross}$ or $K_{sd,i} > 0.5K_{sd,i+1}$	-	-
	Limitation	a. If $A_{open} < 0.5A_p$, then the floor slab shall be taken as	-	-	-

Type	Parameter	IS 1893[46]	ASCE7-16[47]	EC8:2011[48]	NZS:2004[49]
		Rigid or flexible depending on the location. b. If $A_{open} > 0.5A_p$, then the floor slab shall be taken as flexible.			
	Analysis suggestions	-	All analysis procedures are permitted for structures with irregularities for heights less than 48.8m (160 ft).	-	-
Out-of-plane offsets in vertical elements	**Definition**	A building is said to be out of plane offset when structural walls are moved out of the plane in any story.	Out-of-plane offsets are defined to exist when there is a discontinuity in the lateral force-resisting path of at least one of the vertical elements.	-	Horizontal offsets of the column shall be considered to exist when The average of the absolute values of the tangent of the offset angle a. $\frac{\Sigma_{N_c}\left\lvert\frac{a_j}{b_j}\right\rvert}{N_c}>0.1$ or b. For a single column, the tangent of the offset angle

Type	Parameter	IS 1893[46]	ASCE7-16[47]	EC8:2011[48]	NZS:2004[49]
					$\frac{a_j}{b_j}$>0.4 Where, a_j is horizontal offset in the column b_j is the vertical distance between
	Limitation	If a building is in seismic zones III, IV, and V, then Lateral drift shall be less than 0.2% in the story having offset and in stories below	-	-	-
	Analysis suggestion	Suggests specialist literature	a. Structures having out-of-plane offsets should be analyzed using a 3D representation. b. All analysis procedures are permitted for structures with irregularities for height less than 48.8m (160 ft)	-	-

Type	Parameter	IS 1893[46]	ASCE7-16[47]	EC8:2011[48]	NZS:2004[49]
Nonparallel lateral force system	**Definition**		Nonparallel Irregularity exists where vertical force-resisting elements are not parallel to the major orthogonal axis of the seismic force-resisting elements.	-	-
	Limitation	-		-	-
	Analysis suggestion	Buildings with nonparallel lateral force-resisting systems shall be analyzed for a 30% load combinations rule	a. Structures having torsional Irregularity should be analyzed using a 3D representation. b. All analysis procedures are permitted for structures with irregularities for heights less than 48.8m (160 ft).	-	-
Horizontal offsets in column	**Definition**	-	-	-	Horizontal offsets of the column shall be considered to exist when The average of the absolute values of the tangent of the offset angle

Type	Parameter	IS 1893[46]	ASCE7-16[47]	EC8:2011[48]	NZS:2004[49]
					a. $\frac{\sum_{N_c}\left\lvert\frac{a_j}{b_j}\right\rvert}{N_c}>0.1$ or b. For a single column, the tangent of the offset angle $\frac{a_j}{b_j}>0.4$ Where, a_j is horizontal offset in the column b_j is the vertical distance between columns
	Limitation	-	-	-	-
	Analysis suggestion	-	-	-	
Vertical Irregularity: **Stiffness Irregularity**	**Definition**	$K_i < K_{i+1}$	$K_i < 0.7K_{i+1}$ $K_i < 0.8\left[\frac{K_{i+1}+K_{i+2}+K_{i+3}}{3}\right]$	a. Set-back shall not be greater than 20% of the previous floor b. For a single setback with a lower 15% of the total height of the building, the	$K_i < 0.7K_{i+1}$ $K_i < 0.8\left[\frac{K_{i+1}+K_{i+2}+K_{i+3}}{3}\right]$ or $K_i < 0.8\left[\frac{K_{i-1}+K_{i-2}+K_{i-3}}{3}\right]$

Type	Parameter	IS 1893[46]	ASCE7-16[47]	EC8:2011[48]	NZS:2004[49]
				setback shall not be greater than 50% of the previous floor. c. If setbacks are not preserving symmetry, the sum of setbacks at all stories shall not be greater than 30% of the plan dimension at the ground floor.	
	Limitation	In buildings designed considering URM infills, drift should be limited to 0.2% in the story with stiffening.	A story is considered to be extremely soft if $K_i < 0.6K_{i+1}$ $K_i < 0.7\left[\frac{K_{i+1}+K_{i+2}+K_{i+3}}{3}\right]$	-	-
	Analysis suggestions	a. If SPD> 20%, URM infills shall be considered by explicitly modeling the same in structural analysis. b. The design forces for RC members shall be larger than:	-	a. If a building is irregular in elevation irrespective of elevation irregularity, then a planar model can be created; analysis should be Modal.	-

Type	Parameter	**IS 1893**[46]	**ASCE7-16**[47]	**EC8:2011**[48]	**NZS:2004**[49]
		i. Analysis from bare frame ii. Frames with URM infills, using 3D modeling of structure		The behavioral factor value should be decreased. b. If a building is irregular in plan and elevation, then a spatial model should be created, and the modal analysis should be performed. The behavioral factor value should be decreased by 20%.	
Mass Irregularity	**Definition**	$M_i > 1.5 M_{i-1}$			$M_i > 1.5 M_{i-1}$
	Limitation	-	-	-	-
	Analysis suggestion	In buildings with mass Irregularity and located in seismic zones III, IV, and V, the dynamic analysis shall be performed	-	-	-
Vertical Geometric Irregularity	**Definition**	If $(\text{dimension})_i > 1.2\ (\text{dimension})_{i-1}$	-	-	If $(\text{dimension})_i > 1.3\ (\text{dimension})_{i-1}$

Type	Parameter	IS 1893[46]	ASCE7-16[47]	EC8:2011[48]	NZS:2004[49]
	Limitation	-	-	-	-
	Analysis suggestion	For buildings in zones III, IV, and V, dynamic analysis shall consider earthquake effects.	-	-	-
In-plane discontinuity	**Definition**	(IPO)>1.2(PL)	-	-	-
	Limitation	a. In zone II, the lateral drift of the building should be limited to 0.2% of the building height. b. For zones III, IV, and V, In-plane discontinuity shall not be permitted	-	-	-
	Analysis suggestion	-	a. Structures having torsional Irregularity should be analyzed using a 3D representation. b. All analysis procedures are permitted for structures with irregular heights less than 48.8m (160 ft).	-	-

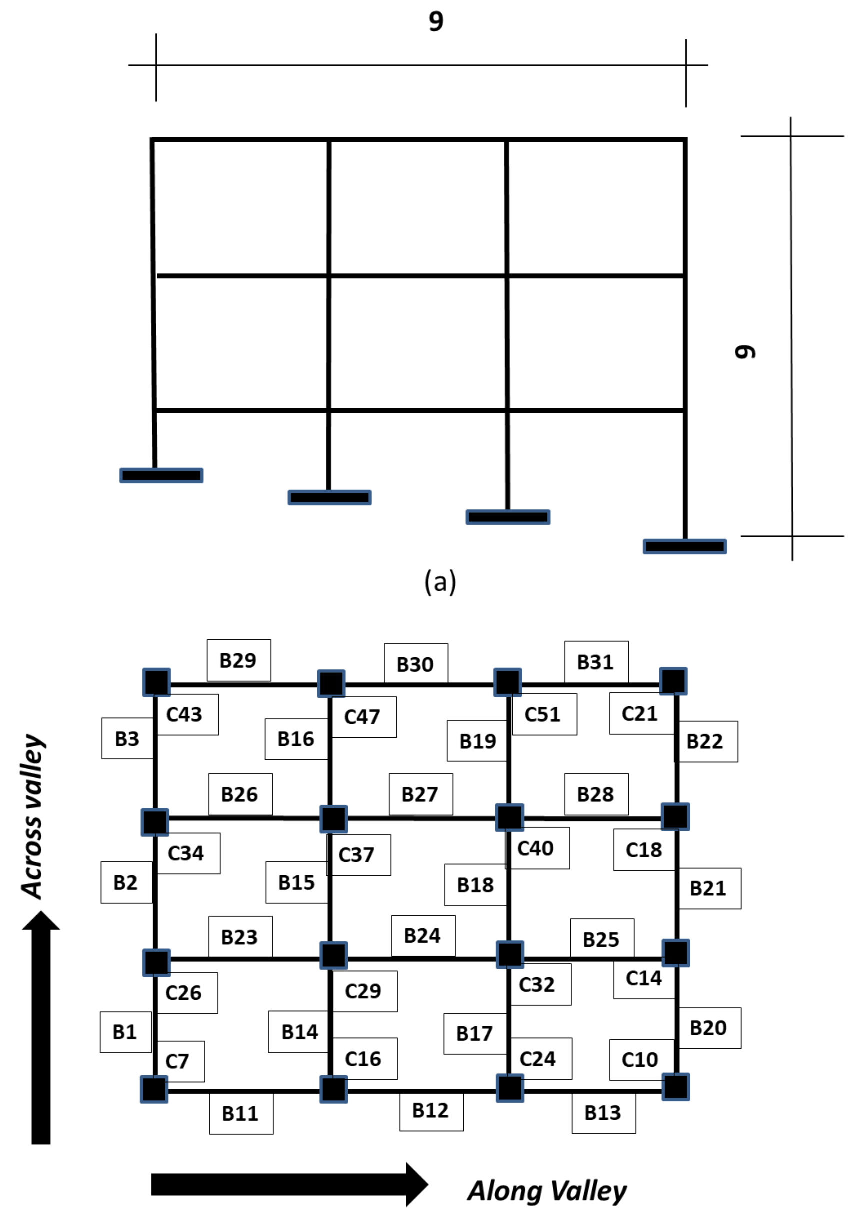
Type	Parameter	IS 1893[46]	ASCE7-16[47]	EC8:2011[48]	NZS:2004[49]
Strength Irregularity	**Definition**	$V_i < V_{i+1}$	$V_i < 0.8V_{i+1}$ Irregularity is extreme if $V_i < 0.65V_{i+1}$	-	$V_i < 0.8V_{i+1}$
	Limitation	Buildings in zones III, IV, and v should be carefully designed.	-	-	-
	Analysis suggestion	-	a. Structures having torsional Irregularity should be analyzed using a 3D representation. b. All analysis procedures are permitted for structures with irregular heights less than 48.8m (160 ft).	-	-
Floating or stub column	**Definition**	-	-	-	-
	Limitation	This feature is not desirable and, hence, prohibited	-	-	-
	Analysis suggestion		-	-	-
Irregular Modes of Vibration	**Definition**	a. First, three modes contribute	-	-	-

Type	Parameter	IS 1893[46]	ASCE7-16[47]	EC8:2011[48]	NZS:2004[49]
		$M_p < 0.65$ to each principal plan b. The fundamental natural period (T) of the building are closer to each other by 10% of larger values			
	Limitation	a. Buildings located in seismic zone II and III, $M_p > 0.65$ (b) For buildings located in seismic zone IV and V, (i) First 3 modes, $M_p > 0.65$ (ii) Fundamental natural period of the building is away from each other by at least 10%	-	-	-
	Analysis suggestion	-	-	-	-

Appendix B: Design Details of the Reference Building Used for Non-Linear Analysis

The design details of the buildings designed according to the IS 1893 and IS 13920 are shown below:

a. **Reference Model: 15_9_9_9**



(b)

Figure B.1 (a) Elevation and (b) Plan of the building model considered

Table B.1 Cross section and reinforcement details of beam

Story	Label	c/s size	Top reinforcement	Bottom reinforcement	Shear reinforcement
Story3	B1	0.3 × 0.3	3#12	3#12	2legged#8@150
Story3	B2	0.3 × 0.3	3#12	3#12	2legged#8@150
Story3	B3	0.3 × 0.3	3#12	3#12	2legged#8@150
Story3	B11	0.3 × 0.3	3#12	3#12	2legged#8@150
Story3	B12	0.3 × 0.3	3#12	3#12	2legged#8@150
Story3	B13	0.3 × 0.3	3#12	3#12	2legged#8@150
Story3	B14	0.3 × 0.3	3#12	3#12	2legged#8@175
Story3	B15	0.3 × 0.3	3#12	3#12	2legged#8@200
Story3	B16	0.3 × 0.3	3#12	3#12	2legged#8@175
Story3	B17	0.3 × 0.3	3#12	3#12	2legged#8@175
Story3	B18	0.3 × 0.3	3#12	3#12	2legged#8@200
Story3	B19	0.3 × 0.3	3#12	3#12	2legged#8@175
Story3	B20	0.3 × 0.3	3#12	3#12	2legged#8@150
Story3	B21	0.3 × 0.3	3#12	3#12	2legged#8@150
Story3	B22	0.3 × 0.3	3#12	3#12	2legged#8@200
Story3	B23	0.3 × 0.3	3#12	3#12	2legged#8@200
Story3	B24	0.3 × 0.3	3#12	3#12	2legged#8@200
Story3	B25	0.3 × 0.3	3#12	3#12	2legged#8@200
Story3	B26	0.3 × 0.3	3#12	3#12	2legged#8@200
Story3	B27	0.3 × 0.3	3#12	3#12	2legged#8@200
Story3	B28	0.3 × 0.3	3#12	3#12	2legged#8@150
Story3	B29	0.3 × 0.3	3#12	3#12	2legged#8@150
Story3	B30	0.3 × 0.3	3#12	3#12	2legged#8@150
Story3	B31	0.3 × 0.3	3#12	3#12	2legged#8@150
Story2	B1	0.3 × 0.3	3#16	2#16	2legged#10@150
Story2	B2	0.3 × 0.3	3#16	2#16	2legged#10@175
Story2	B3	0.3 × 0.3	3#16	2#16	2legged#10@150
Story2	B11	0.3 × 0.3	3#16	2#16	2legged#10@150
Story2	B12	0.3 × 0.3	3#16	2#16	2legged#10@150
Story2	B13	0.3 × 0.3	3#16	2#16	2legged#10@175
Story2	B14	0.3 × 0.3	3#16	2#16	2legged#10@175
Story2	B15	0.3 × 0.3	3#16	2#16	2legged#10@150
Story2	B16	0.3 × 0.3	3#16	2#16	2legged#10@150
Story2	B17	0.3 × 0.3	4#16	2#16	2legged#10@150
Story2	B18	0.3 × 0.3	3#16	2#16	2legged#10@150
Story2	B19	0.3 × 0.3	4#16	2#16	2legged#10@150

Story	Label	c/s size	Top reinforcement	Bottom reinforcement	Shear reinforcement
Story2	B20	0.3 × 0.3	4#16	2#16	2legged#10@150
Story2	B21	0.3 × 0.3	3#16	2#16	2legged#10@150
Story2	B22	0.3 × 0.3	4#16	2#16	2legged#10@150
Story2	B23	0.3 × 0.3	3#16	2#16	2legged#10@150
Story2	B24	0.3 × 0.3	3#16	2#16	2legged#10@150
Story2	B25	0.3 × 0.3	3#16	2#16	2legged#10@150
Story2	B26	0.3 × 0.3	3#16	2#16	2legged#10@175
Story2	B27	0.3 × 0.3	3#16	2#16	2legged#10@175
Story2	B28	0.3 × 0.3	3#16	2#16	2legged#10@150
Story2	B29	0.3 × 0.3	3#16	2#16	2legged#10@175
Story2	B30	0.3 × 0.3	3#16	2#16	2legged#10@175
Story2	B31	0.3 × 0.3	3#16	2#16	2legged#10@150
Story1	B1	0.3 × 0.3	2#16	2#16	2legged#10@175
Story1	B2	0.3 × 0.3	2#16	2#16	2legged#10@175
Story1	B3	0.3 × 0.3	2#16	2#16	2legged#10@175
Story1	B11	0.3 × 0.3	4#16	3#16	2legged#10@200
Story1	B12	0.3 × 0.3	3#16	2#16	2legged#10@175
Story1	B13	0.3 × 0.3	3#16	2#16	2legged#10@125
Story1	B14	0.3 × 0.3	3#16	2#16	2legged#10@150
Story1	B15	0.3 × 0.3	3#16	2#16	2legged#10@175
Story1	B16	0.3 × 0.3	3#16	2#16	2legged#10@175
Story1	B17	0.3 × 0.3	4#16	2#16	2legged#10@175
Story1	B18	0.3 × 0.3	4#16	2#16	2legged#10@175
Story1	B19	0.3 × 0.3	4#16	2#16	2legged#10@150
Story1	B20	0.3 × 0.3	4#16	3#16	2legged#10@175
Story1	B21	0.3 × 0.3	4#16	2#16	2legged#10@150
Story1	B22	0.3 × 0.3	4#16	3#16	2legged#10@175
Story1	B23	0.3 × 0.3	4#16	3#16	2legged#10@150
Story1	B24	0.3 × 0.3	3#16	2#16	2legged#10@125
Story1	B25	0.3 × 0.3	3#16	2#16	2legged#10@175
Story1	B26	0.3 × 0.3	4#16	3#16	2legged#10@125
Story1	B27	0.3 × 0.3	3#16	2#16	2legged#10@150
Story1	B28	0.3 × 0.3	3#16	2#16	2legged#10@175
Story1	B29	0.3 × 0.3	4#16	3#16	2legged#10@125
Story1	B30	0.3 × 0.3	3#16	2#16	2legged#10@150
Story1	B31	0.3 × 0.3	3#16	2#16	2legged#10@175

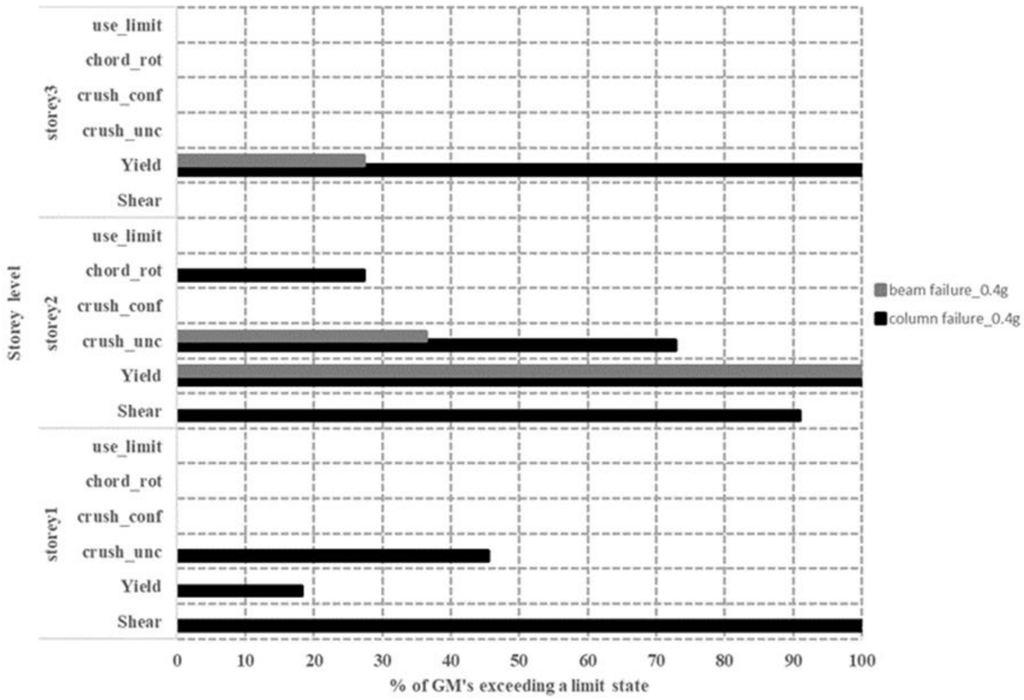
Table B.2 Cross section and reinforcement details of columns

Story	Label	c/s size	Reinforcement	Shear reinforcement
Story3	C1	0.4 × 0.3	4#18	2legged#8@225
Story3	C2	0.4 × 0.3	4#18	2legged#8@225
Story3	C3	0.4 × 0.3	4#18	2legged#8@225
Story3	C4	0.4 × 0.3	4#18	2legged#8@225
Story3	C6	0.4 × 0.3	4#18	2legged#8@225
Story3	C9	0.4 × 0.3	4#18	2legged#8@225
Story3	C10	0.4 × 0.3	4#18	2legged#8@225
Story3	C11	0.4 × 0.3	4#18	2legged#8@225
Story3	C13	0.4 × 0.3	4#18	2legged#8@225
Story3	C14	0.4 × 0.3	4#18	2legged#8@225
Story3	C15	0.4 × 0.3	4#18	2legged#8@225
Story3	C17	0.4 × 0.3	4#18	2legged#8@225
Story3	C18	0.4 × 0.3	4#18	2legged#8@225
Story3	C19	0.4 × 0.3	4#18	2legged#8@225
Story3	C20	0.4 × 0.3	4#18	2legged#8@225
Story3	C21	0.4 × 0.3	4#18	2legged#8@225
Story2	C1	0.4 × 0.3	8#18	2legged#8@225
Story2	C2	0.4 × 0.3	8#18	2legged#8@225
Story2	C3	0.4 × 0.3	8#18	2legged#8@225
Story2	C4	0.4 × 0.3	8#18	2legged#8@225
Story2	C6	0.4 × 0.3	8#18	2legged#8@225
Story2	C9	0.4 × 0.3	8#18	2legged#8@225
Story2	C10	0.4 × 0.3	8#18	2legged#8@225
Story2	C11	0.4 × 0.3	8#18	2legged#8@225
Story2	C13	0.4 × 0.3	8#18	2legged#8@225
Story2	C14	0.4 × 0.3	8#18	2legged#8@225
Story2	C15	0.4 × 0.3	8#18	2legged#8@225
Story2	C17	0.4 × 0.3	8#18	2legged#8@225
Story2	C18	0.4 × 0.3	8#18	2legged#8@225
Story2	C19	0.4 × 0.3	8#18	2legged#8@225
Story2	C20	0.4 × 0.3	8#18	2legged#8@225
Story2	C21	0.4 × 0.3	8#18	2legged#8@225
Story1	C10	0.4 × 0.3	8#18	2legged#8@225
Story1	C14	0.4 × 0.3	8#18	2legged#8@225
Story1	C18	0.4 × 0.3	8#18	2legged#8@225
Story1	C21	0.4 × 0.3	8#18	2legged#8@225
Story1	C42	0.4 × 0.3	6#18	2legged#8@225

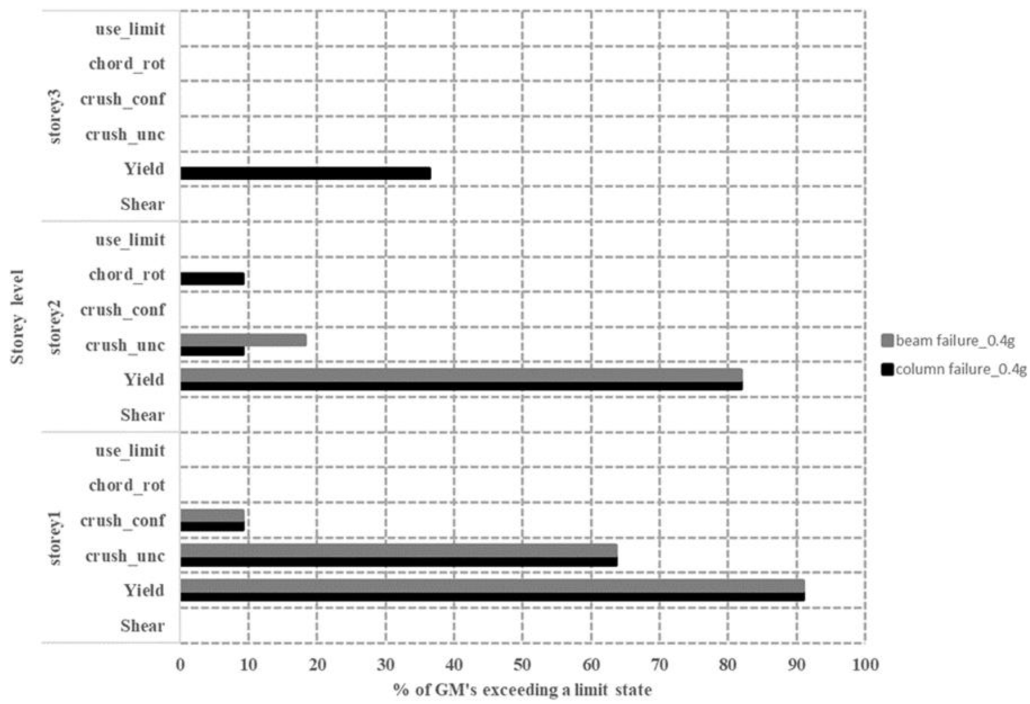
Story	Label	c/s size	Reinforcement	Shear reinforcement
Story1	C44	0.4×0.3	6#18	2legged#8@225
Story1	C46	0.4×0.3	6#18	2legged#8@225
Story1	C48	0.4×0.3	6#18	2legged#8@225
Story1	C50	0.4×0.3	6#18	2legged#8@225
Story1	C52	0.4×0.3	4#18	2legged#8@225
Story1	C54	0.4×0.3	4#18	2legged#8@225
Story1	C56	0.4×0.3	6#18	2legged#8@225
Story1	C58	0.4×0.3	12#18	2legged#10@150
Story1	C60	0.4×0.3	12#18	2legged#10@175
Story1	C62	0.4×0.3	12#18	2legged#10@175
Story1	C64	0.4×0.3	12#18	2legged#10@150

Appendix C: Additional Non-Linear Analysis

C1. Damage state of Reference and Proposed models for ground motions scaled to 0.4g along the valley.

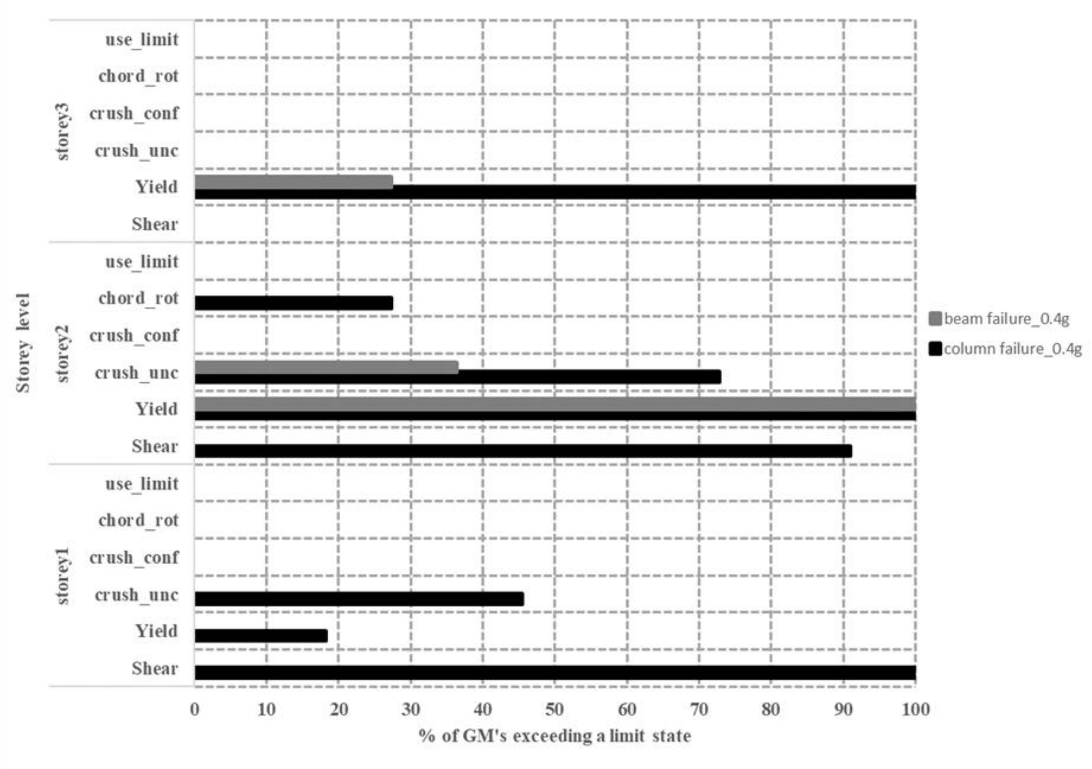


(a)

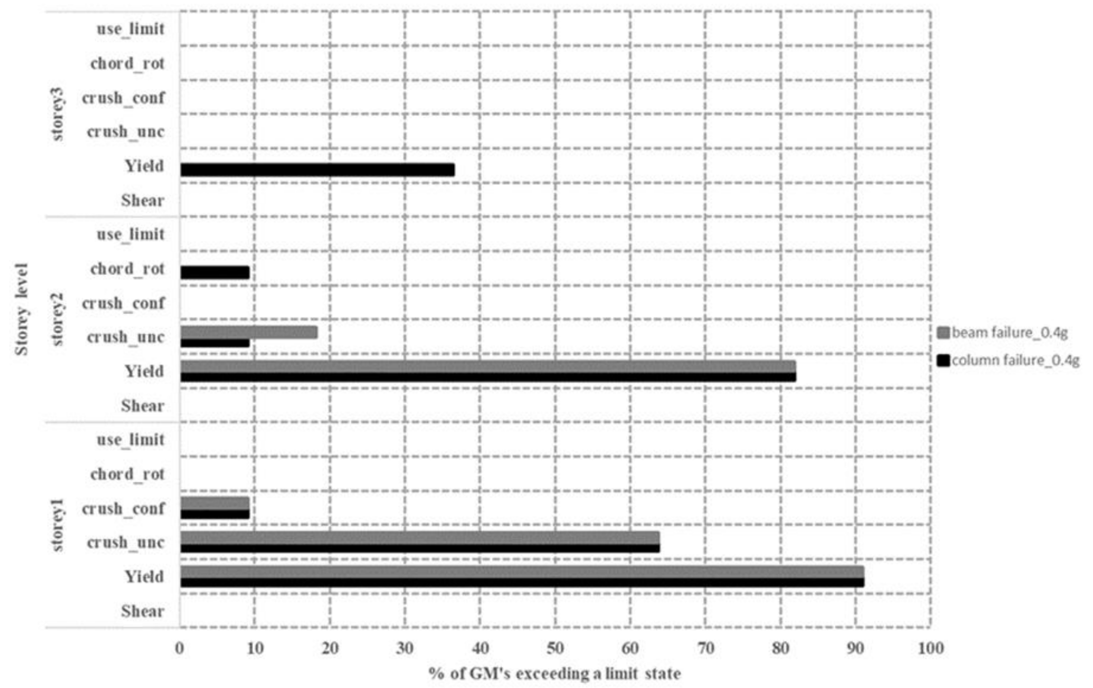


(b)

C2. Damage state of Reference and Proposed models for ground motions scaled to 0.4g across the valley.

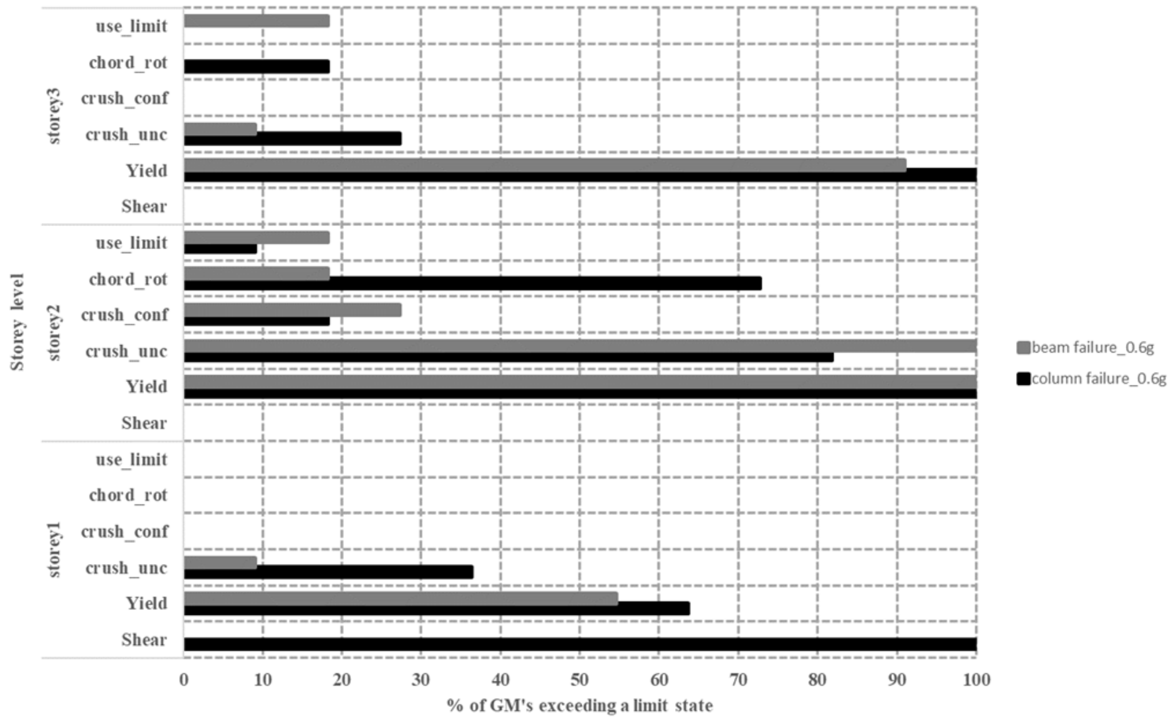


(a)

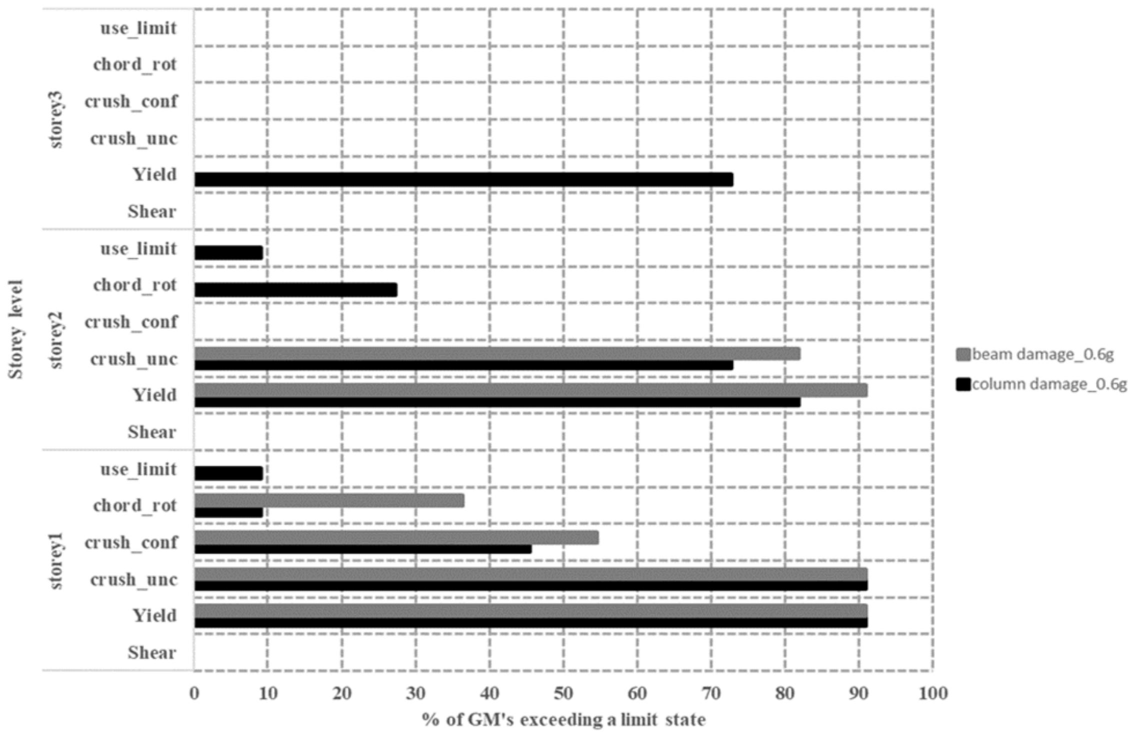


(b)

C3. Damage state of Reference and Proposed models for ground motions scaled to 0.6g along the valley.

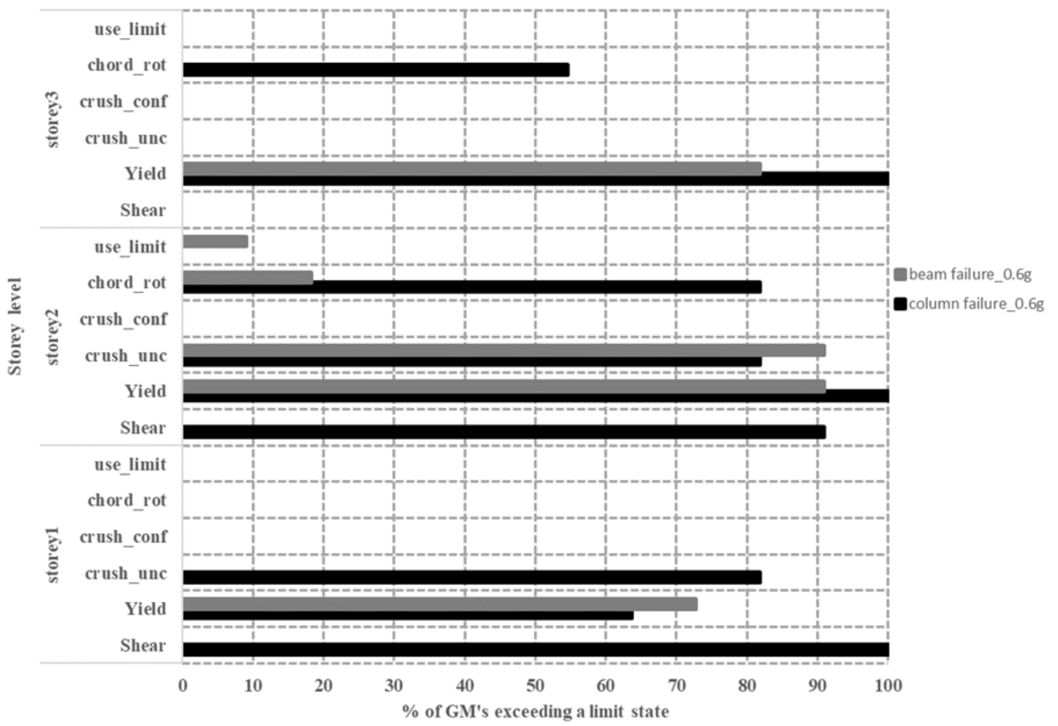


(a)

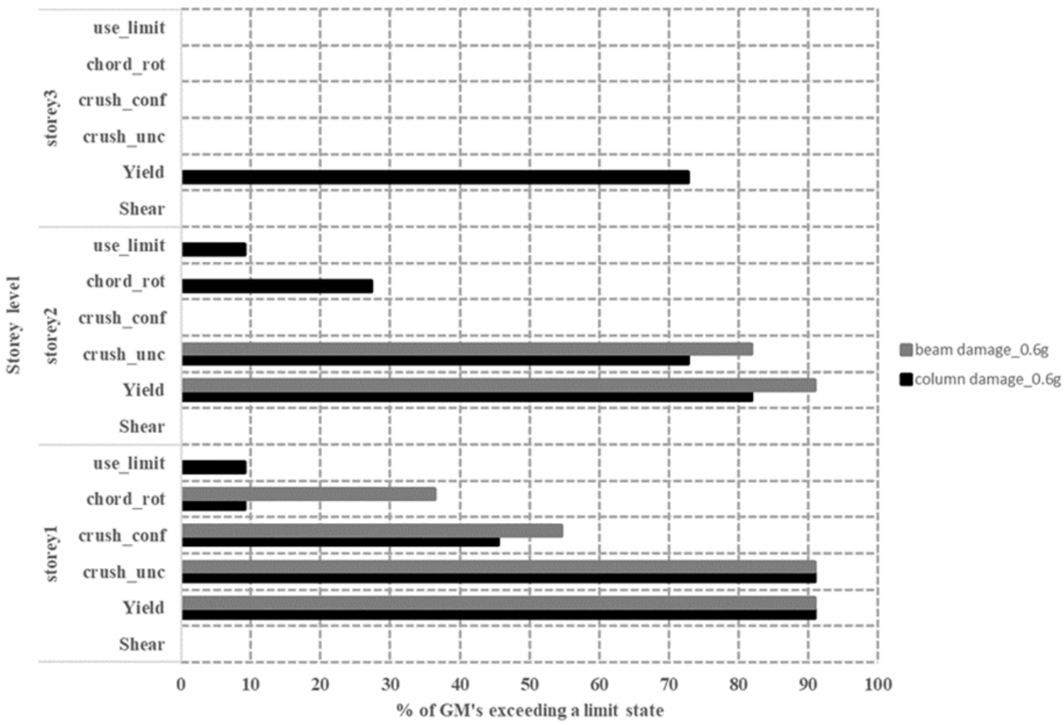


(b)

C4. Damage state of Reference and Proposed models for ground motions scaled to 0.6g across the valley

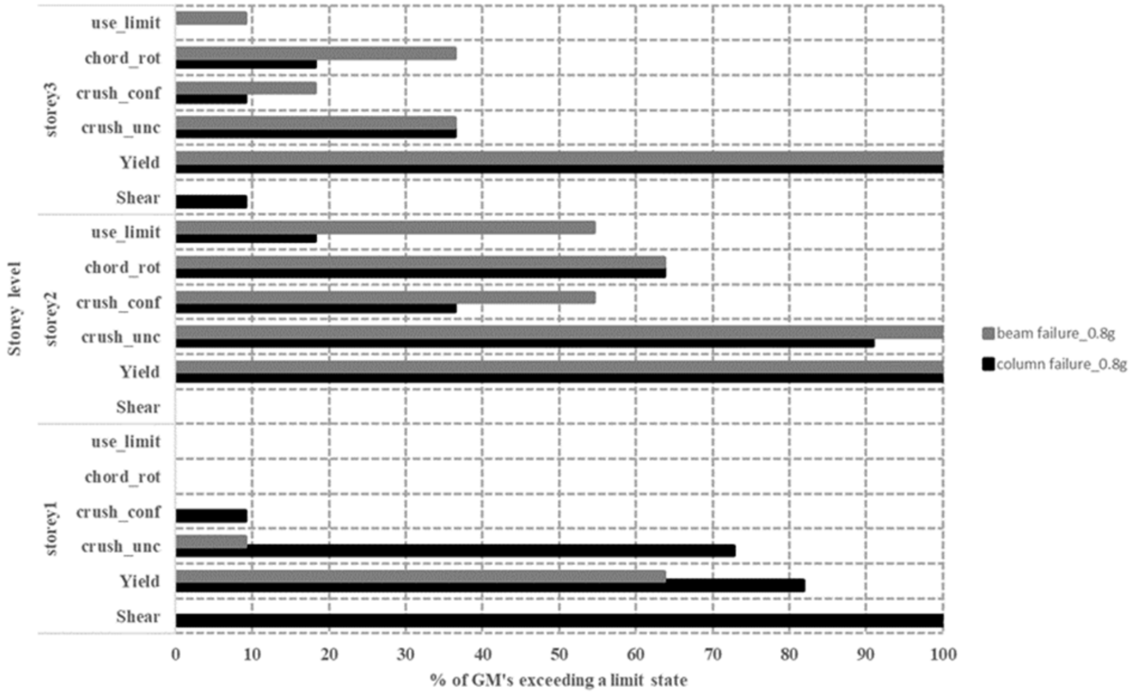


(a)

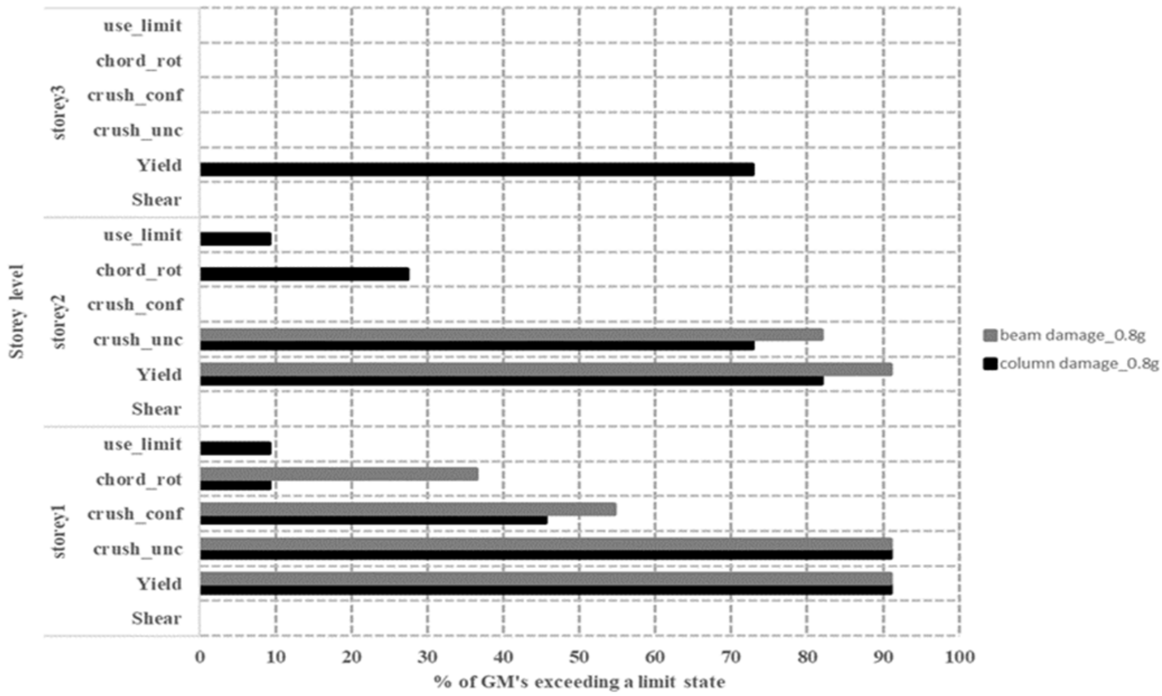


(b)

C5. Damage state of Reference and Proposed models for ground motions scaled to 0.8g along the valley

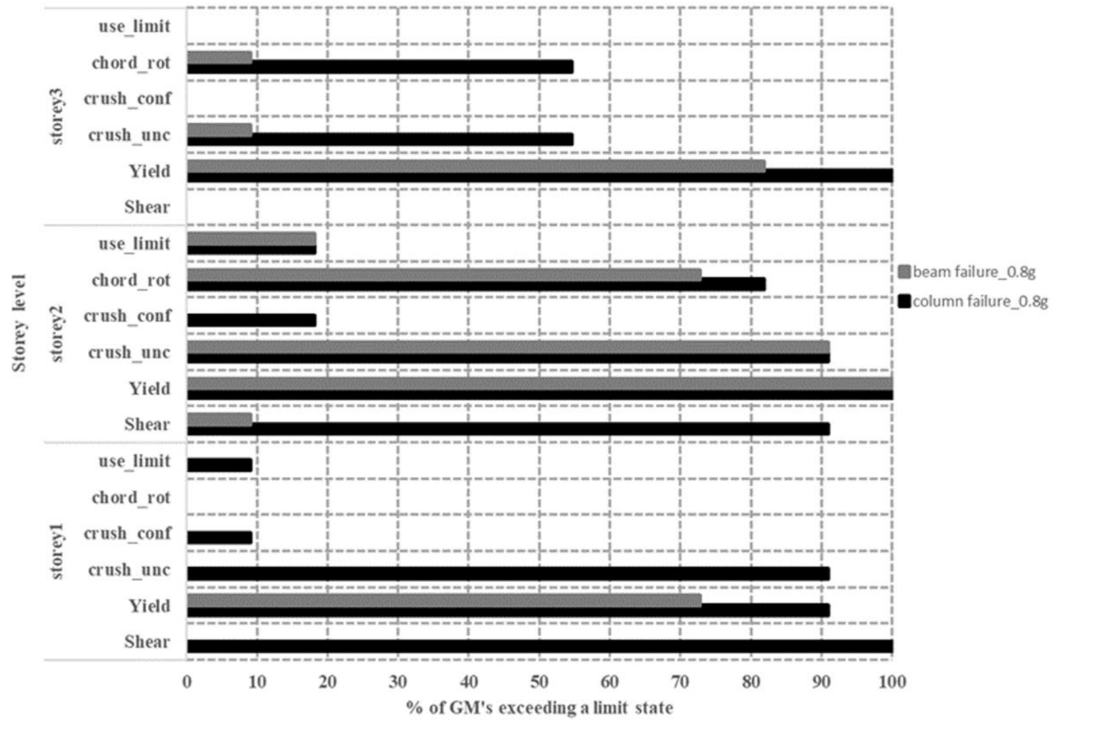


(a)

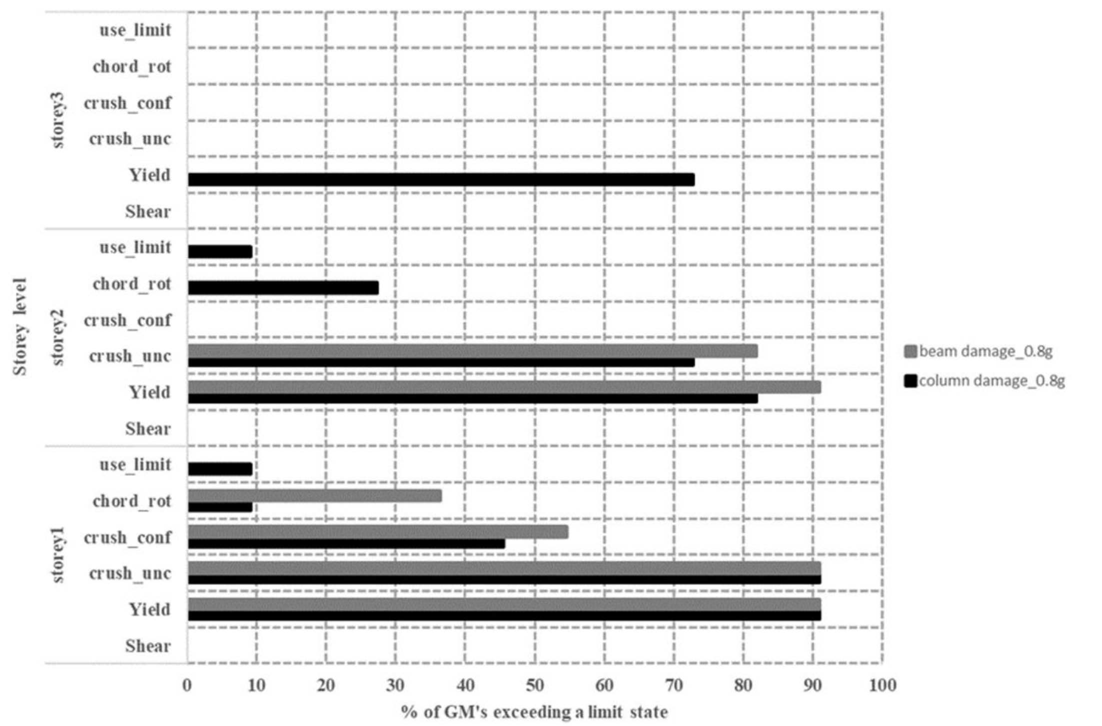


(b)

C6. Damage state of Reference and Proposed models for ground motions scaled to 0.8g across the valley.



(a)



(b)