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NONLINEAR SEISMIC RESPONSE AND RESPONSE REDUCTION FACTOR OF BUILDING FRAMES ON SLOPE

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ABSTRACT

The response reduction factor (*R*) is used in seismic design to account for nonlinear response of structures. Significant research efforts have been made to obtain *R* factor for buildings. Various national codes recommend *R* values depending on structural configurations and construction details mainly for buildings on flat grounds. However, studies on *R* factor for buildings on hill slopes are not available. The present study focuses on the nature of *R* factor of building resting on slopes considering the effect of irregularities due to slope. Based on extensive regression study of nonlinear static pushover (NSP) analysis results, an attempt has been made to explore the possibility of obtaining a modification to estimate the *R* factor. The results show a good agreement with the *R* values obtained from the actual NSP analysis results. The various statistical parameters used for validation of a regression model support the prediction capability of the anticipated relation.

KEYWORDS: Hilly Region; Building Frames on Slopes; Nonlinear Static Analysis; Regression; Response Reduction Factor

INTRODUCTION

In the traditional force-based approach of seismic design of structures, the design base shear (V_d) is estimated by reducing the elastic base shear (V_e) obtained from elastic analysis by a reduction factor. Different codes specify this factor by different names e.g., it is referred to as "response reduction factor" in IS 1893 [1], ''response modification coefficient'' in ASCE7 [2] and ''behaviour factor'' in Eurocode 8 [3]. Following IS 1893, it is referred to as 'response reduction factor', *R* in the present study. In design codes, this factor is introduced to account for nonlinear behaviour of structures till failure. Designers prefer to use such simple and less time-consuming method i.e., elastic forced based method for design of structures. The studies on assessment of response reduction factor and its different components for buildings on plain grounds are enormous [4-9]. There is a class of literatures dealing with evaluation of ductility reduction factor for different periods of structures [10-13]. Karavasilis et al. [14] presented simplified expressions for response modification factor for steel moment resisting frames using nonlinear static pushover (NSP) analysis results and nonlinear dynamic analysis results. Mondal et al. [15] obtained the actual values of response reduction factor for reinforced concrete (RC) moment frame buildings designed according to the Indian standards and noted that the values of *R* recommended by the Indian seismic code are higher than the actual values. Mahmoudi and Zaree [16] estimated response modification factors for conventional concentric braced frames and buckling restrained braced frames based on NSP analysis. Ferraioli et al. [17] studied the effects of various parameters e.g. regularity, number of spans and number of storeys influencing response modification factor, and proposed a local ductility criterion to refine the guideline given in the Italian seismic code. Yarahmadi et al. [18] proposed an approach to evaluate response modification factors in a probabilistic performance-based analysis framework to bring economy to seismic design. Malkawi and Al-Shatnawi [19] evaluated response reduction factor in respect of components over strength factor and ductility factor for RC frame building. It is noted that seismic zoning has an impact on ductility reduction factor for different buildings. The overstrength factor varies with seismic zones, number of storeys, and design gravity loads. Borzi and Elnashai [9] used an evenly distributed earthquake dataset to derive the values of force reduction factors for structures such that it does not exceed a presumed level of ductility. Krawinkler and Nassar [20] developed an expression for ductility reduction factor obtained from statistical analyses of fifteen western United States ground motions within a magnitude range between 5.7 to 7.7.

The various studies on response reduction factor as discussed in the above are primarily for regular structures on plane grounds. The various seismic codes recommend the value of *R* to obtain base shear for various regular buildings types on flat ground. However, the recommendations of *R* values are not readily available for irregular buildings. The buildings constructed on slopes in hilly regions are highly irregular in nature as the foundation levels vary according to slopes. Thus, the ground floor column heights are not same due to sloping ground. The most prevalent configurations of buildings on slopes can be broadly identified as step back, and step back-set back configurations (Figure 1). These buildings have large differences in the location of centre of mass and centre of stiffness which also changes along the height of a building, resulting in a complex response behaviour under seismic excitation. These irregularities lead to abrupt reductions of floor area, which in turn results in change of mass and stiffness along the building height. As a consequence, the dynamic characteristics of such buildings as compared with their regular counterparts vary a lot [21]. The performances of such buildings in previous earthquakes reveal that such buildings are more vulnerable than the regular building on plain ground. The *R* values for buildings resting on hill slope which are highly irregular in vertical as well as in horizontal plane are expected to be lesser than the usually recommended value of *R*. Hence, proper understanding of seismic behaviour of irregular buildings is important to structural and earthquake engineering community. It seems to be important to study the nature of variations of *R* values for buildings on different slopes to properly estimate the base shear for safe design of buildings on hill slope.

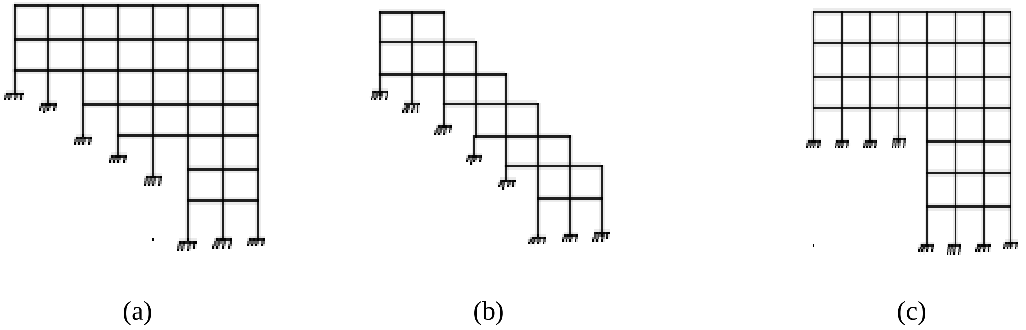
(a) (b) (c)

Fig. 1 (a) Step back (b) step back-set back (c) building on steep slope configuration

In the present study, a comprehensive study on nonlinear seismic response of building frames resting on hill slope is taken up. Specifically, the study focuses on the nature of variation of response reduction factor of building resting on hill slope with due consideration to the effect of irregularities due to slope. In doing so, the variation of different seismic response parameters e.g., base shear and roof displacement, ductility (μ) and different components of response reduction factor i.e., over strength factor (V_s), ductility reduction factor (R_μ) for varying slopes are studied first. Based on extensive regression study of the NSP analysis results, an attempt has been made to explore the possibility of obtaining an empirical relation to estimate the *R* factor of RC irregular building frames of stepback configuration resting on sloping grounds. The results of preliminary study show good agreement with the R values obtained from NSP analysis results for various combinations of slope and framing configurations. The various statistical parameters used for validation of a regression model support the prediction capability of the proposed relation.

RESPONSE REDUCTION FACTOR OF BUILDINGS ON SLOPE

1. Fundamentals of Response Reduction Factor

The response reduction factor, first introduced in ATC [22] is used to reduce elastic base shear, V_u obtained by elastic analysis to obtain design base shear, V_d. It is based on the basic concept that a seismically designed and detailed framing systems can withstand large inelastic deformations without collapse (ductile behaviour) and can develop sufficient lateral strengths in addition to their design strength commonly known as reserve strength. Thus, the response reduction factor, *R* reflects the capability of a structure to dissipate energy through inelastic behaviour. The response reduction factor, *R* is expressed as,

$$R = R_s \times R_\mu \times R_r \tag{1}$$

where, R_s is the overstrength factor, R_μ is the ductility reduction factor and R_r is the redundancy factor.

Overstrength is a parameter that is used to measure the difference between the required design strength and the actual strength of the material of a member. The overstrength primarily stems from various sources such as the load factor applied based on code prescribed design seismic forces, lower gravity load applied at the time of earthquake excitation, partial safety factor for materials, actually provided higher strength value of materials and member dimensions, special ductility requirement, effect of non-structural elements etc. The overstrength factor, R_s that reflect the reserve strength of structure can be obtained as [6],

$$R_s = \frac{V_y}{V_d} \tag{2}$$

where, V_y is the yield strength and V_d is the design strength. The overstrength factor in a structural system depends on the prescribed safety margins by the seismic guidelines used for seismic design. Moreover, based on the choice of designer, R_s may vary even if the same seismic code is used for design as the geometric design may not exactly match the design requirement.

Ductility reduction factor (R_μ), also known as the yield strength reduction factor is defined as the ratio between the maximum elastic base shear (V_e) and the yield base shear (V_y) i.e.

$$R_\mu = \frac{V_e}{V_y} \tag{3}$$

In which, the elastic base shear (V_e) is defined as the maximum demand force for the structure to remain elastic during an earthquake [6]. The ductility reduction factor depends on the ductility demand of a structure. The ductility demand (μ) is obtained as the ratio of the maximum absolute inelastic deformation (Δ_u) to deformation at first yield (Δ_y) experienced by the structure subjected to a given ground motion. The value of R_μ is basically a function of ductility demand (μ) and time period (T) of a structure. There are numerous studies on evaluation of R_μ. The ductility reduction factor is obtained in the present study following Miranda's approach [13] which is given as,

$$R_\mu = \frac{(\mu - 1)}{\phi} + 1 \tag{4}$$

where, ϕ is a function of μ and T that has different values for rock, alluvium and soft soil sites which can be obtained from the following,

$$\phi = 1 + \frac{1}{10T - \mu T} - \frac{1}{2T}\exp(-1.5(\log(T) - 0.6)^2) \text{ for rock site} \tag{5a}$$

$$\phi = 1 + \frac{1}{12T - \mu T} - \frac{2}{5T}\exp(-2(\ln(T) - 0.20)^2) \text{ for alluvial soil} \quad (5b)$$

$$\phi = 1 + \frac{3}{T_1} - \frac{3T_1}{4T}\exp(-3(\ln 9\frac{T}{T_1} - 0.25)^2) \text{ for soft soil} \quad (5c)$$

The redundancy factor, R_r is introduced on the basis of redundancy of a structure. For a redundant structure, after failure of one member, the load is redistributed i.e., failure of one-member does not indicate global failure. Hence, the redundancy of a system provides additional strength to carry the load. Thus, the reliability of a system depends on the system of redundancy or non-redundancy. In this study, the value of redundancy factor is taken as 1.0 [23].

2 Nonlinear Response of Buildings on Sloping Ground

A rigorous parametric analysis is performed by considering one hundred and eighty stepback type RC frame buildings resting on varying slopes and with different geometrical configurations to study the nature of variations of various nonlinear seismic responses parameters and associated response reduction factor. Typical RC frame having three to seven storey configurations resting on plane as well as on sloping ground (2.5^0 to 30^0, @ 2.5^0) located in the hilly region of Northeast India, one of the most seismically vulnerable regions, are considered. These structures are designed and detailed for the seismic demands as obtained following IS 1893 for seismic zone V. The structural design is based on the provisions of IS456 [24] and IS13920 [25]. All the building frames have the same plan arrangement with five numbers of bays of 5m length in both the directions. The floor-to-floor height is 3.2 m for all the storeys. The details of the considered buildings are as following: plan dimension: 25 m x 25 m, live load on floor: 3 kN/m^2, live load on roof: 1.5 kN/m^2, water proof: 3 kN/m^2, slab thickness: 150 mm, external wall thickness: 230 mm, internal wall thickness: 125 mm, parapet height: 900 mm, parapet thickness: 100 mm, floor finish thickness: 25 mm. All the selected models are designed with M30 grade of concrete and Fe415 grade reinforcing steel as per Indian Standard. The different geometric configurations considered are described in Table 1. It may be noted from the table that for different slopes, frame configurations of all storeys and bay numbers may not be feasible. The tick marks (✓) in the Table 1 indicate the different configurations of frames (number of storeys and number of bays) considered at different slopes. The details of the beams and columns for different stories are depicted in Table 2.

Table 1: The details of the building configurations on different ground slopes

	7 storey					6 storey				
	7 bay	6 bay	5 bay	4 bay	3 bay	7 bay	6 bay	5 bay	4 bay	3 bay
2.5^o	✓	✓	✓	✓	✓		✓	✓	✓	✓
5^o	✓	✓	✓	✓	✓		✓	✓	✓	✓
7.5^o	✓	✓	✓	✓		✓	✓	✓	✓	✓
10^o	✓		✓			✓	✓	✓	✓	✓
12.5^o	✓	✓	✓	✓		✓	✓	✓	✓	✓
15^o	✓	✓	✓	✓			✓	✓	✓	✓
17.5^o	✓	✓	✓	✓			✓	✓	✓	✓
20^o	✓	✓	✓	✓	✓		✓	✓	✓	✓
22.5^o		✓	✓	✓			✓	✓	✓	✓
25^o	✓	✓	✓	✓			✓		✓	✓
27.5^o			✓	✓						
	5 storey					4 storey				
2.5^o				✓	✓	✓	✓		✓	✓
5^o	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓
7.5^o	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓
10^o		✓	✓	✓	✓	✓	✓	✓	✓	✓
12.5^o	✓	✓	✓	✓	✓	✓	✓	✓	✓	
15^o			✓	✓	✓	✓	✓	✓	✓	✓
17.5^o	✓		✓	✓	✓		✓	✓	✓	
20^o		✓		✓	✓				✓	

22.5°			✓	✓	✓			✓	✓	
25°					✓			✓	✓	
27.5°					✓			✓	✓	
	3 storey									
2.5°	✓	✓	✓	✓	✓					
5°	✓	✓	✓	✓	✓					
7.5°		✓	✓	✓	✓					
10°				✓	✓					
12.5°		✓	✓	✓						
15°	✓			✓						
17.5°				✓						

Table 2: The details of the beams and columns of various frames

Frame	Members	Floors	Depth	Width	Reinforcement Details
3 Storey	Beams	1-3	450	300	3-20Ø+2-16 Ø (top) + 3-20 Ø (bottom)
	Columns	1-3	450	450	8-20 Ø (equally distributed on four sides)
4 Storey	Beams	1-4	600	300	6-20 Ø (top) +3-20 Ø (bottom)
	Columns	1-4	600	600	8-25 Ø (equally distributed on four sides)
5 Storey	Beams	1-3	600	300	6-20Ø(top)+3-20Ø (bottom)
	Columns	1-3	600	600	12-20 Ø (equally distributed on four sides)
	Columns	4-5	500	500	8-20 Ø (equally distributed on four sides)
6 Storey	Beams	1-6	600	300	6-20 Ø(top)+3-20 Ø (bottom)
	Columns	1-4	600	600	4-25 Ø (corners)+8-20Ø (2 intermediate bars at each side)
	Columns	5-6	500	500	8-20 Ø (equally distributed on four sides)
7 Storey	Beams	1-7	600	300	6-20 Ø (top)+3-20 Ø(bottom)
	Columns	1-4	600	600	12-25Ø (equally distributed on four sides)
	Columns	5-7	500	500	8-25Ø (equally distributed on four sides)

3 Nonlinear Seismic Responses

The various key components required to obtain response reduction factor i.e., the strength and ductility factors as discussed in the previous section are usually evaluated from a pushover curve obtained by NSP analysis procedure. The method furnishes nonlinear load-deformation characteristics of individual components and elements of a building under monotonically increasing lateral loads and allows studying the sequences of plastic hinge formation and failure of structural components throughout the procedure. The applied lateral load pattern significantly affects the pushover analysis results. The lateral load patterns should approximate the actual lateral forces that the building will be subjected to during an earthquake. Several investigations [26, 27] suggest that a triangular or trapezoidal lateral load pattern matches better with dynamic analysis results at the elastic range but at larger deformations, the dynamic envelops are unable to capture such variations under earthquake loading. To evaluate the inelastic response of structures more accurately, the load profile should be capable of describing the actual dynamic force profiles. It is very difficult to predict such load profiles as different kind of load profiles can be generated during non-linear time history response analysis of any structure. Thus, it will be more appropriate to consider adaptive load pattern for pushover analysis of buildings. But such approach will be computationally demanding and will be time consuming for extensive parametric study planned in the present study. Thus, the lumped plasticity model and the modal pushover analysis (MPA) based load pattern comparatively simpler but found to yield reasonable accuracy is adopted [28]. The uniaxial moment plastic hinges and the P-M-M interaction hinges are assigned at both ends of the beams and the columns, respectively. The idealized force-deformation curves (deformation control) for the plastic hinges for the beams and the columns have been considered as per ASCE [2]. The shear failure has been modelled for all the columns as force-controlled action [2]. The shear capacity has been evaluated as per IS 456 [24]. The analysis is performed by using SAP 2000 v 16.0.

The various response parameters such as ultimate base shear, ultimate and yield displacements, ductility factor etc. are obtained from the pushover curve for each of the building frame configurations. The design base shear is obtained following IS 1893, correspond to the fundamental time period of each building frame obtained from the free vibration analysis of the frames. The overall response reduction factor R, the strength reduction factor R_s and the ductility reduction factor R_μ are evaluated using Equations (1), (2) and (4), respectively. The typical results for four storey three bay and six storey seven bay configurations for different slopes are shown in the Table 3. From the table, it can be observed that the base shear for the four storey building decreases for increase in slopes beyond 10^{o} and for the six-storey building the base shear decreases for all increasing values of slopes. This occurs due to removal of some spans of the buildings to cope with increasing slopes which results in reduction of mass. Overall, the response reduction factor and the ductility of the building decrease with increasing slopes.

Table 3: Results of NSP analysis of a four storey three bay and six storey seven bay frames

Slope	V_u	Δ_u	Δ_y	V_d	R_s	R_μ	R
Four storey three bay frame (4S3B*)							
2.5^{o}	829	0.034	0.0188	317.89	2.61	1.77	4.63
5^{o}	835	0.031	0.0175	316.82	2.63	1.72	4.53
7.5^{o}	833	0.028	0.0173	315.76	2.64	1.56	4.13
10^{o}	841	0.027	0.0171	314.96	2.67	1.52	4.06
15^{o}	723	0.02	0.0114	288.5	2.51	1.64	4.10
17.5^{o}	686	0.016	0.009	287.7	2.38	1.63	3.89
22.5^{o}	611	0.013	0.01	266.1	2.3	1.24	3.00
Six storey six bay frame (6S6B)							
2.5^{o}	1566	0.073	0.0369	637	2.46	2.208	5.43
5^{o}	1529	0.066	0.0338	669	2.29	2.146	4.91
7.5^{o}	1520	0.059	0.0303	691	2.20	2.099	4.62
10^{o}	1450	0.053	0.0270	704	2.06	2.054	4.23
12.5^{o}	1371	0.045	0.0222	688	1.99	2.064	4.11
15^{o}	1390	0.038	0.0196	719	1.93	1.921	3.71
17.5^{o}	1359	0.033	0.016	717	1.90	1.999	3.79
20^{o}	1160	0.027	0.014	651	1.78	1.812	3.23
22.5^{o}	982	0.019	0.008	636	1.54	2.128	3.29
25^{o}	894	0.016	0.007	589	1.52	1.993	3.03
*S represents storey and B represents bay							

Further, the variations of ultimate displacement, ductility factor, ductility reduction factor and response reduction factor with varying slopes are studied. Figure 2 shows the variation of ultimate displacement with varying slopes for different building configurations. It can be observed that the ultimate displacement decreases with increase in slope. This is due to the fact that the structures become stiffer as the ground storey column height reduces due to increase in slopes.

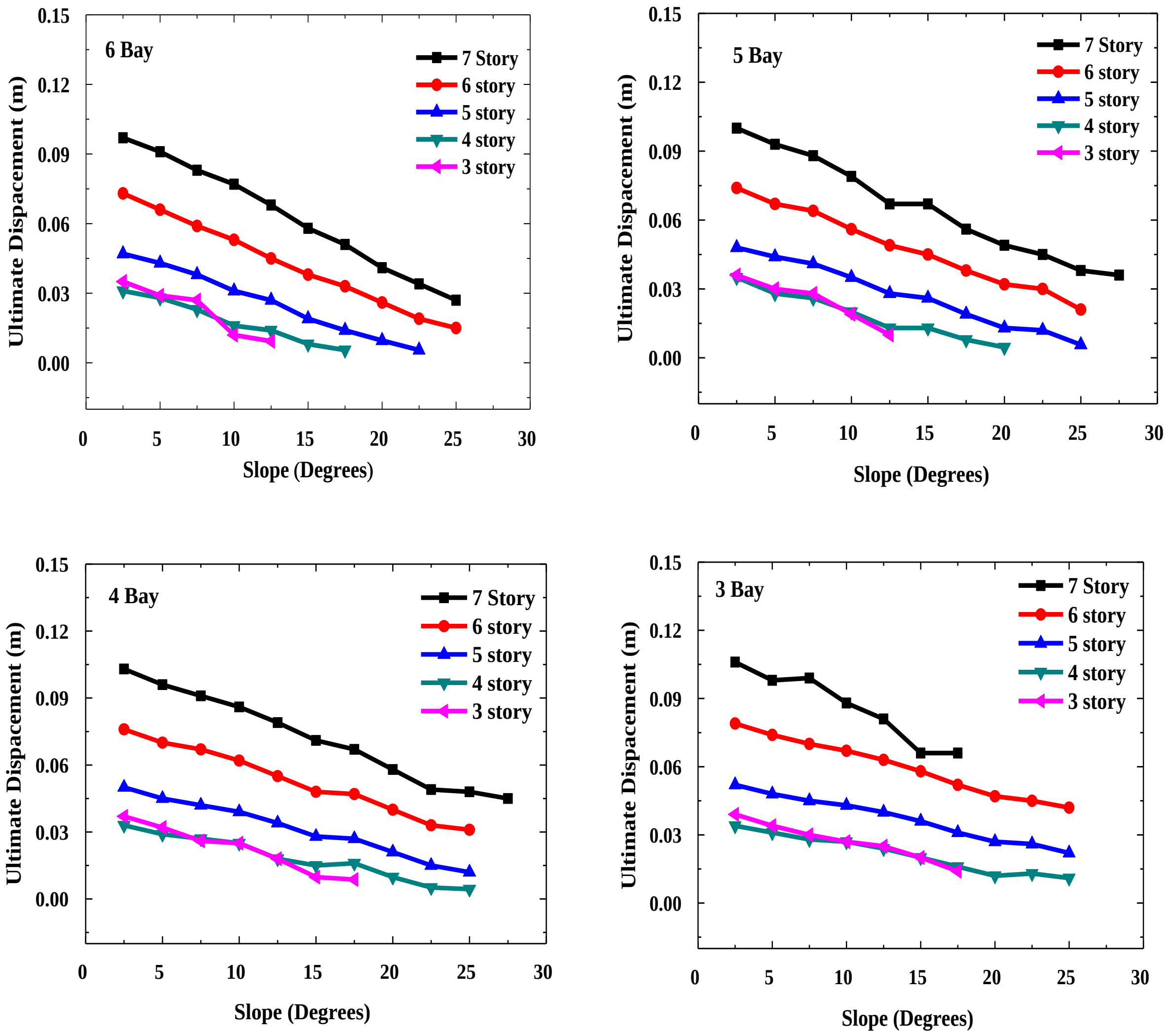

Fig. 2 The variation of ultimate displacement with varying slope for different building configurations

The variation of ductility and ductility response reduction factor with varying slopes for different building configurations are shown in Figures 3 and 4, respectively. Gradual reduction in ductility and ductility reduction factor is observed in Figure 3 and 4 with increase in slope. However, for some cases at higher slopes, these values increase. This may be due to the fact that at high slope, the intermediate columns are shorter than uphill side column. Thus, after failure of intermediate short columns, the structure carries further load until uphill side column fails. Figure 5 depicts the variation of overstrength response reduction factor. The overstrength reduction factor also decreases with increasing slope.

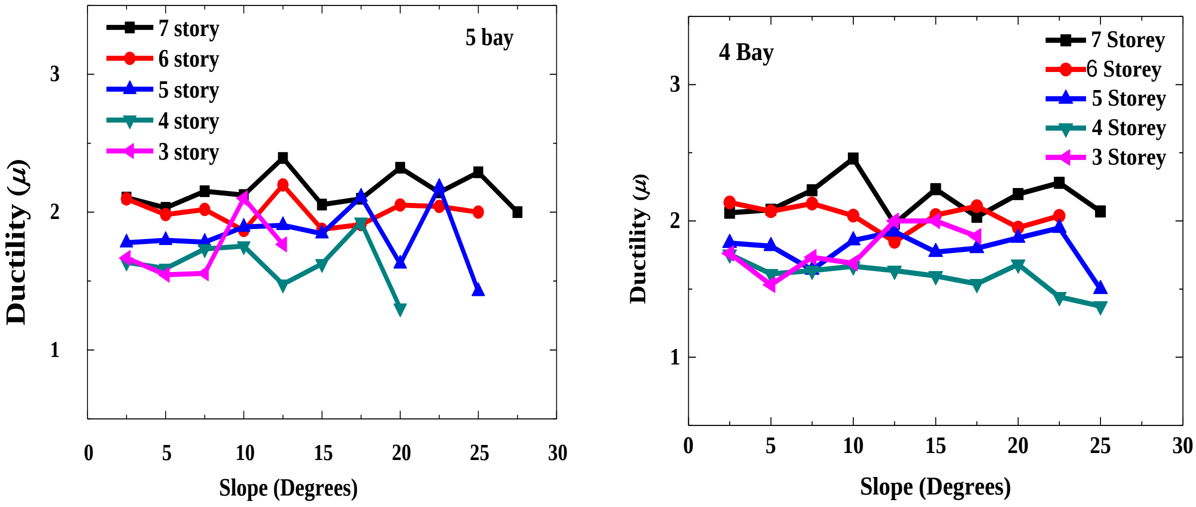

Fig. 3 The variation of ductility with varying slope for different building configurations

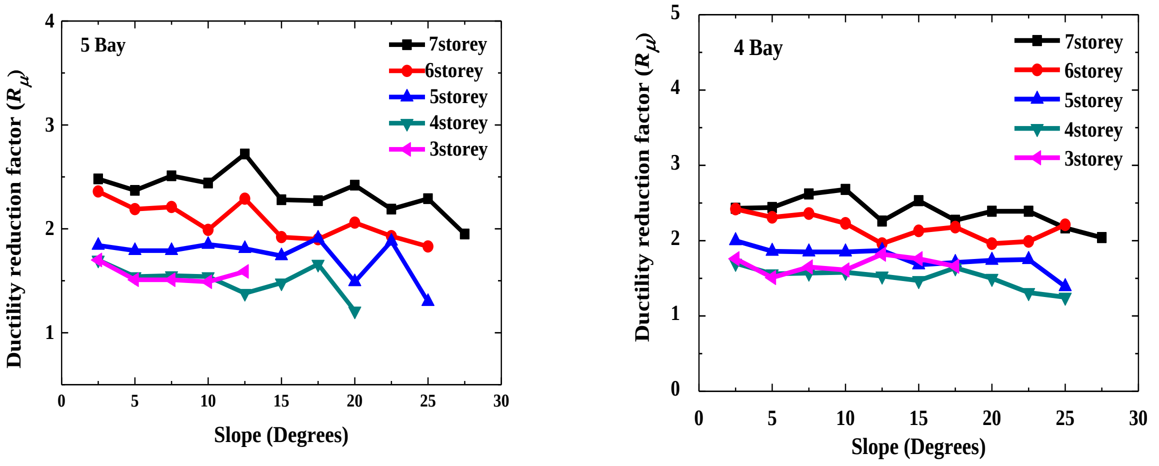

Fig. 4 The variation of ductility response reduction factor with varying slope for different building configurations

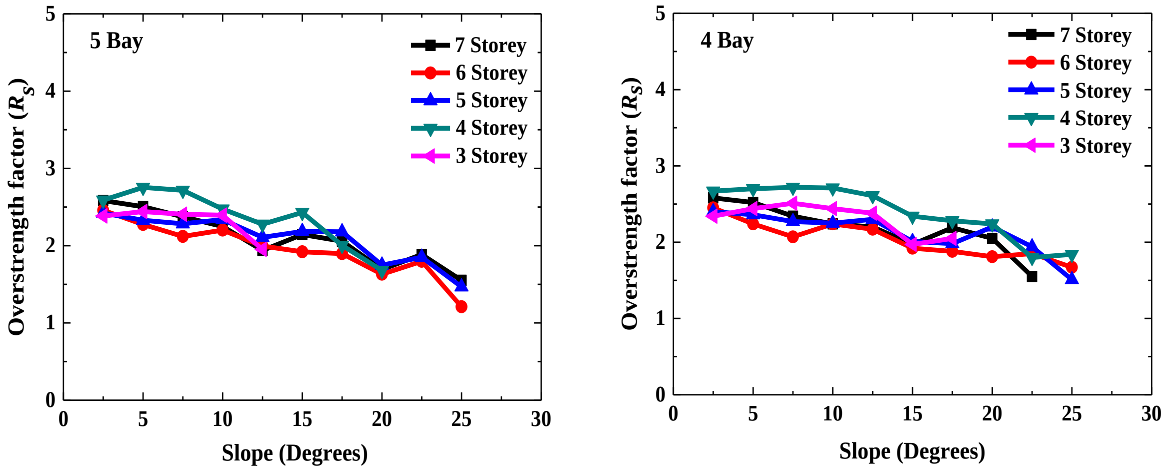

Fig. 5 The variation of overstrength response reduction factor with varying slope for different building configurations

Finally, Figure 6 shows the variation of response reduction factor with varying slopes for different building configurations. The overall response reduction factor decreases with increasing slope. The various numerical results presented clearly indicate that the slope of the ground has considerable impact on nonlinear performance of building. Thereby, affects the various response parameters that decide the value of response reduction factor of a building frame. Hence, it is felt important to explore to evaluate the value of *R* with due consideration to the effect of ground slope and associated changes in building configurations.

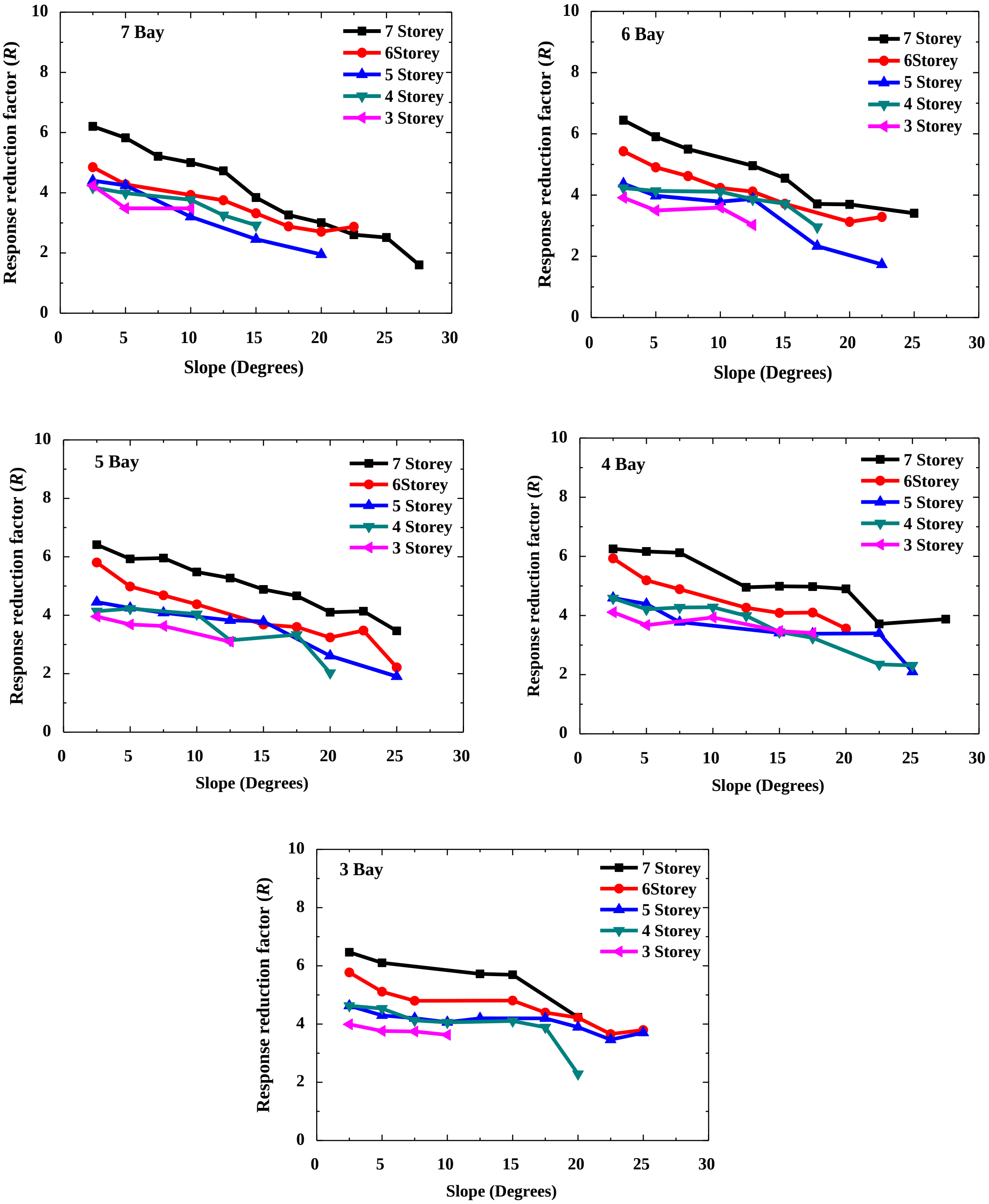

Fig. 6 The variation of response reduction factor with varying slope for different building configurations

4 Modified Response Reduction Factor

The nature of variations of different response parameters and overall response reduction factor with slope reveals that the code recommended values cannot be applied to obtain response reduction factor for building on hill slope. It is evident from the variation of overall response reduction factor *R* with slope of ground as noted in the Figure 6 that the overall response reduction factor *R* reduces when slope increases. The results of parametric study clearly indicate that the actual response reduction factors of buildings on slope are less than the code recommended values; particularly for buildings resting on higher slope. Thus, the code recommended *R* value is expected to underestimate the base shear and could be a potential problem with regard to safety of the buildings on slope. To address this issue, an attempt has been made in the present study to explore an empirical relation to obtain the *R* value with due consideration to the effect of ground slope. In this regard, it is important to note that the height of building is not same in all the bays of a building on hill slope and it has been also realized from the detailed parametric studies that the value of *R* factor varies with effective height of a building. Thus, a relation is investigated in terms of effective height of a building and slope of ground on which building is resting. The effective height is defined as the average height of all the columns of a building [29]. Following Figure 7, the effective height can be obtained as,

$$h_{eff} = \sum_{i=1}^{n} \frac{h_i}{n} \tag{6}$$

where, n is the number of storeys, h_i is the height of the i-th column.

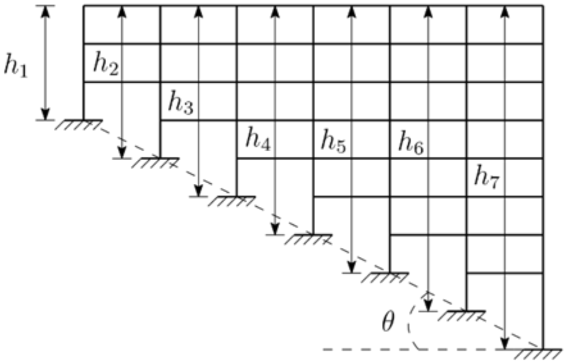

Fig. 7 The effective height of building on slope

Total one hundred eighty numbers of frames resting on varying slopes as detailed in the previous section are analysed by NSP procedure and the *R* values are obtained accordingly. The regression analysis is performed to obtain the *R* value in terms of equivalent height and slope. Based on the regression analysis, the modified relation to approximate the *R* value of such frames resting on hill slope in terms of effective height of building and slope of the ground is proposed as,

$$R_m = 0.4 R_c h_{eff}^{0.37} \theta^{-0.17} \tag{7}$$

where, R_c is the IS code recommended value of the reduction factor for special moment resisting frames considered in the present regression study, θ is the slope of the ground on which the frame is resting and h_{eff} is the average height of the frame as explained in Figure 7.

The response reduction factors are obtained based on the approximate relation described by the Equation (6) and also by the direct NSP analysis for all the 180 building frame configurations. The correlation between the predicted and the response reduction factor as obtained from the NSP for all the frames considered for regression studies are plotted in Figure 8 which shows a good correlation. It may be noted that the predicted *R* values obtained from the proposed relation are purposefully kept biased on the lower side. This will ensure conservative estimate of base shear which is also followed in the codal practice of seismic design.

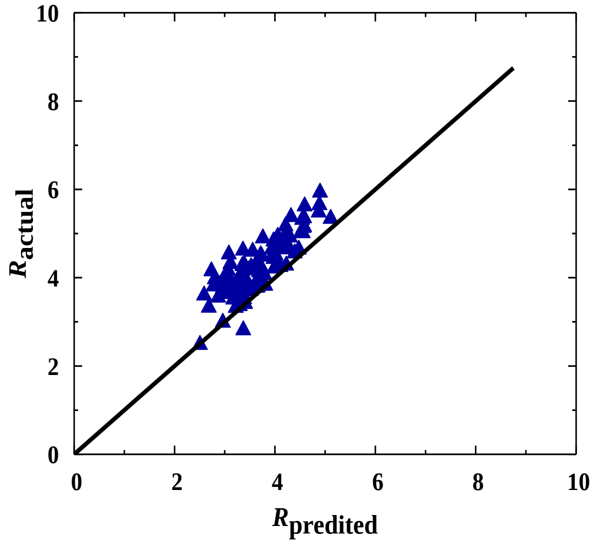

Fig. 8 Comparison of *R* values obtained by the present modification and by actual NSP analysis procedure

Further, the root mean square error (RMSE), the coefficient of determination (R^2) and the standard average error (ε_m) are computed to judge the predictability of the proposed relation. The statistical norms are computed using the following relations [30],

$$RMSE=\sqrt{\frac{1}{m}\sum_{1}^{m}\left(R_{\theta i}-\hat{R}_{\theta i}\right)^{2}},\quad R^{2}=1-\frac{\sum_{1}^{m}\left(R_{\theta i}-\hat{R}_{\theta i}\right)^{2}}{\sum_{1}^{m}\left(R_{\theta i}-\bar{R}_{\theta}\right)^{2}}\quad \text{and}\quad \varepsilon_{m}=\frac{1}{m}\sum_{1}^{m}\left(100\frac{\left|R_{\theta i}-\hat{R}_{\theta i}\right|}{R_{\theta i}}\right) \qquad (8)$$

where, $R_{\theta i}$ is the response reduction factor obtained directly from the pushover analysis, $\hat{R}_{\theta i}$ is the predicted response reduction factor as obtained from the proposed relation for the '*i*'th frame and $\bar{R}_{\theta}$ is the mean value of the response reduction factor and *m* is the total numbers of sample frames considered for statistical study. The values of the statistical norms are obtained as: $RMSE$ =0.7363, R^2 =0.827, ε_m =14.15%. The statistical norms obtained are quite satisfactory considering the fact that the response reduction factor predicted by the proposed formula is intentionally kept biased on the lower side to ensure that the obtained base shear remains on the higher side to ensure desired safety in seismic design.

Further, to study the capability of the proposed relation, the *R* factors obtained by the proposed relation for ninety-two numbers of new frame configurations are compared with those obtained by the NSP procedure. The slopes of 4, 8, 12, 16, 21 and 26 degrees and bay width of 4.0 m and 3.0 m are considered. The floor-to-floor height is 3.5 m and the ground floor height is 4.2 m. The values of the statistical norms are obtained as: $RMSE$ =.6463, R^2 =0.882, ε_m =13.46%. The ratios of the *R* values obtained from the NSP analysis approach with varying slope of the ground to that of obtained from the present relation are shown in Figure 9. It may be noted that these ratios are marginally higher than unity as the prediction formula is conservatively kept biased for marginally lower prediction of the R values to ensure desired safety.

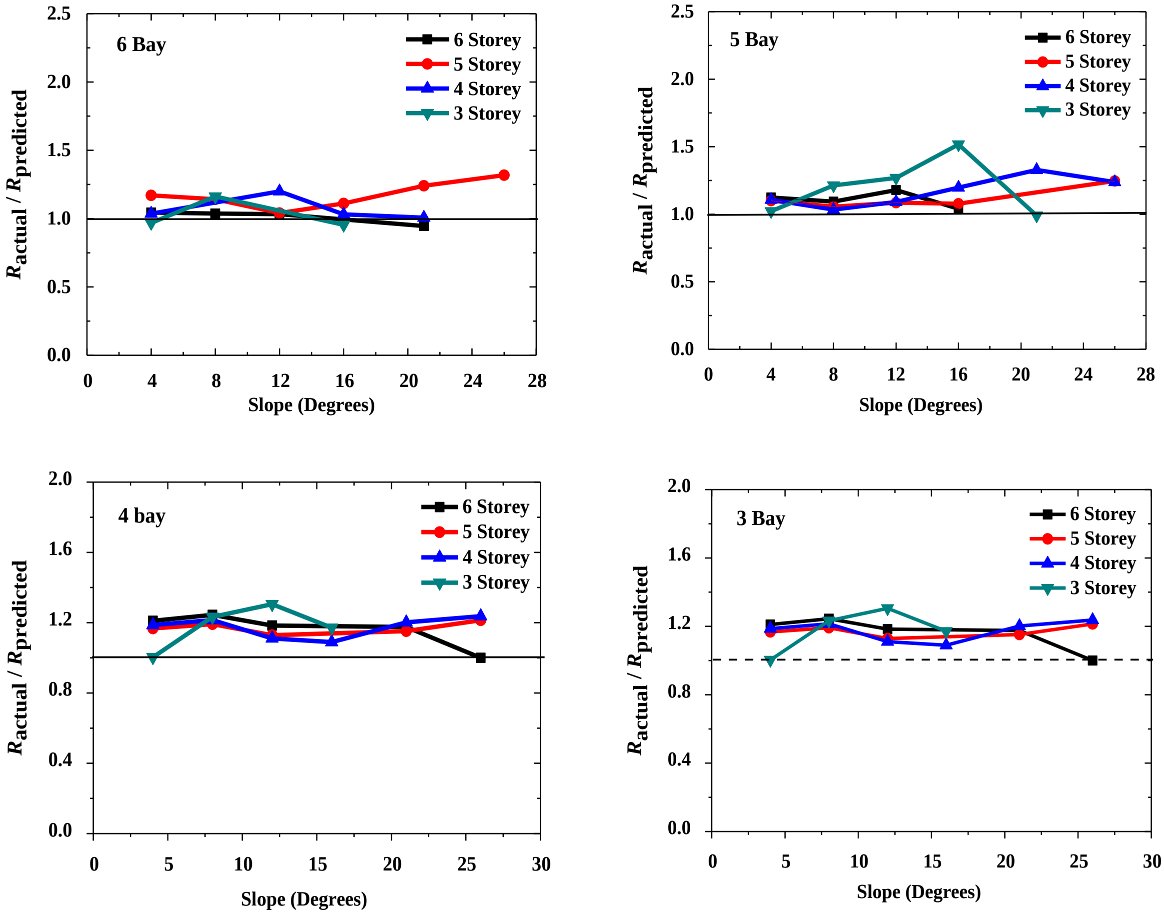

Fig. 9 The comparison of *R* values obtained by the proposed relation with *R* values obtained by NSP analysis for varying slope of the ground

SUMMARY AND CONCLUSION

The nonlinear seismic behaviour of RC building frame resting on sloping ground typically observed in hilly region is studied. The detailed investigation on the nature of variation of nonlinear seismic response of such building frames revealed that the ultimate displacement and ductility decrease with increase of slope. It indicates that the building frames on sloping ground become stiffer. In general, the response reduction factor decreases when slope increases. It is important to note that the *R* values obtained by the NSP are lesser than the codal *R* values. Thereby, the codal *R* values underestimate the base shear used to estimate the design base shear for seismic design of structures by force-based design philosophy. Based on the nature of variations of nonlinear seismic response parameters, a regression analysis is performed to obtain an empirical relation in terms of effective height of a building and slope of the ground to estimate response reduction factor for RC buildings resting on sloping ground. The study is restricted to stepback buildings. However, it can be readily extended to setback stepback buildings on hill slopes. This is expected to supplement the preliminary analysis and design of building on slopes. The results of numerical study with different bay width and slopes show that the developed empirical expression is capable of estimating response reduction factor of stepback building frames resting on sloping ground with reasonable accuracy. The correlation between the predicted response reduction factor and that of obtained from the NSP analysis procedure shows good correlation considering the fact that the prediction formula is conservatively kept biased for marginally lower prediction of R values. The various statistical parameters generally used for validation of such regression-based relations also ensure the prediction capability of the proposed relation with good accuracy. The present preliminary study considers stepback type plane frames on sloping ground and seems to be an important first step towards assessing seismic response of buildings on sloping ground. However, for useful final recommendations in the form of response modification factor needs more detailed analysis of three-dimensional model considering bidirectional seismic loading.

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