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Original research article

Impact of Soil-Structure Interaction on Fragility Curves and Ductility Demands in 3D Reinforced Concrete Moment-Resisting Frames

Praveen Oggu[1,*], Gopikrishna K.[2]

[1]*Department of Civil Engineering, Vardhaman College of Engineering, Hyderabad 501218, India*
[2]*Department of Civil Engineering, National Institute of Technology Warangal, Warangal 506004, India*

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ABSTRACT

Earthquakes are one of the most devastative natural hazards that cannot be avoided by mankind. Hence, seismic risk mitigation procedures based on vulnerability assessment of the existing infrastructure is the only alternative to prevent the imperative socio-economic and human loss. The seismic analysis and vulnerability assessment procedures of building structures are usually carried out by ignoring the contribution of soil and foundation characteristics in the seismic response of the superstructure. The present work is primarily focused on investigating the soil structure interaction effects on the performance assessment of 4-story and 12-story building configurations with both fixed base and flexible base conditions designed as per IS 456 and IS 1893. Further, fragility analysis is performed using the Capacity Spectrum method and damage probability matrices are then developed for each damage state to describe the structural behavior. It has been observed from the results that consideration of the SSI effect has significantly altered the response characteristics of the models considered. It can also be observed that the constant 'R' value specified by IS 1893 for a particular RC MRF was found to be significantly less than the 'R' values computed for all the building configurations using NLS analysis. This necessitates the importance of dynamic characteristics of structures in estimating the load-carrying capacity, resulting in an adequate estimation of design forces leading to optimal design configurations.

Keywords: Capacity spectrum method; Ductility demand; Fragility analysis; Pushover analysis; Soil-Structure interaction

***Corresponding author:** praveenoggu@vardhaman.org **doi:** 10.14456/scitechasia.2025.82

1. Introduction

Earthquakes cannot be predicted, but the structure can be designed to resist the earthquake load. Still, the huge structures failed and becoming a reason for human losses and economic losses. One of the reasons is that the structure is designed for the fixed base conditions and the dynamic characteristics of soil are neglected. In the last three decades, the effect of soil-structure interaction has got special attention among researchers and engineers. Most of the researches currently working on theoretical analysis. The interaction among the soil medium, foundation, and structure changes the actual behavior of the structure as compared to fixed base conditions. Due to the movement of soil below the footing decrease the actual stiffness of soil. Due to the movement of soil the natural time period increases. India is an earthquake-prone country. Soil structure interaction is a mutual motion of two systems i.e., foundation structure and supporting soil. The interaction between these two systems results the change in the dynamic response of the structure. In modeling and analysis of dynamic analysis of structure the effect of soil structure interaction is ignored most of the times (Chomchuen and Boonyapinyo 2021, Pitilakis and Petridis 2022).

The effect of soil structure interaction has significant effect when the heavier structure is built on the soft soils. The seismic response of structures depends on both the soil property and structural property. If the structure is supported on the soft soil, then the foundation of the structure shows the inability to deformations of the free field motion, with this effect the base of structure undergoes motion to deviate from free field motion. When analyzing any building it is common to assume it as fixed base. This is realistic assumption when the structure is constructed over the hard rock or when the relative stiffness of foundation soil is higher than the super structure. In other cases, the response of the structure is different (Wolf and Obernhuber 1985; Dutta et al. 2004).

Various seismic codes specify different response reduction factors to scale down the elastic response of a structure. These factors are termed as response modification coefficient, behavior factor, or response reduction factor, generally represented as ‘R’. ASCE 7 classifies RC frame buildings into three ductility classes: Ordinary (OMRF), Intermediate (IMRF), and Special Moment Resisting Frames (SMRF) with corresponding reduction factors as 3, 5 and 8 respectively. European and Mexican codes do not account for reserve strength, only account for ductility. Also, certain codes such as EC 8, ECP-201, and ECP-203 do not differentiate between steel and concrete frames for the assigned ’R’ value. The US codes (NEHRP) have the highest ‘R’ value compared to Indian, Mexico, Japan, and European seismic codes.

According to seismic provisions specified by IS 1893, moment-resisting frames are grouped into two types: ordinary and special moment-resisting frames with corresponding response reduction factors as 3 and 5 respectively. However, these constant values do not address the influence of the changes in structural configuration, viz., building height, number of bays present, bay width, irregularities arising out of mass and stiffness, etc. which has a significant effect on the dynamic characteristics of the structure. This implies, adopting a constant ‘R’ value cannot ensure adequate design demand for all the structural configurations (Oggu et al. 2021). Analytically ‘R’ value can be computed using non-linear static analysis (NLS) and non-linear dynamic analysis

(NLD). Nevertheless, NLS is more widely adopted owing to its simplicity in implementation. Further, it has been reported in the literature that response reduction factors computed from pushover analysis were found to be smaller than the values given in the respective design codes. Besides, an investigation presented in the literature by Mondal et al. (2013) on the estimation of actual 'R' value for an Indian code designed SMRF using pushover analysis, has been compared with the corresponding 'R' value suggested by the code. It was concluded that the 'R' value suggested by Indian code has been considerably higher than computed from pushover analysis and was reported to be potentially dangerous. This holds with most of the existing seismic codes around the world. Owing to this, several investigations have been reported in this direction using non-linear static analysis for the estimation of the response reduction factor (Mondal et al. 2013; Chaulagain et al. 2014; Abdi et al. 2016; Abou-Elfath and Elhout 2018; Sharifi and Toopchi-Nezhad 2018). Soil–structure interaction (SSI) significantly influences R by increasing the fundamental period of vibration through foundation flexibility and enhancing energy dissipation via damping into the soil. Hence in this research, it is attempted to assess the adequacy of 'R' value considered in IS 1893 (2016) using NLS analysis considering six different RC models.

The objectives for the present study are to understand the influence of soil on seismic behavior and vulnerability characteristics of RC building and to understand the variation of damage distribution due to soil structure interaction by developing fragility curves. This paper aims at studying the seismic behavior of RC buildings (with a fixed base and a flexible base). Flexible support conditions are provided as equivalent spring supports to the structures considered. The non-linear static analyses are also performed on these structures to determine the capacity of the structure in various damage states with the help of pushover analysis. Fragility curves for the buildings (with a fixed base and a flexible base) are developed for spectral displacement with the help of capacity curves. The soil structure interaction of building is studied considering the flexible base for hard soil and soft soil.

2. Modelling and Analysis

2.1 Description of the analytical model

The building is modeled in SAP2000. The buildings considered for the study were designed as ductile frames corresponding to Special Moment Resisting Frames (SMRF). The properties of the building model considered in present work are summarized in Table 1 and Fig. 1.

Table 1. Specifications of model.

Property	Data
Height of each floor	3 m
Plan dimension	12 × 12 m
Floor thickness	0.15 m
Wall thickness	230 mm
Parapet wall thickness	200 mm
Compressive strength of concrete	30 N/mm^2
Grade of steel	Fe 415
Damping coefficient	0.05
Size of column	300 × 300mm
Size of beam	300 × 400mm
Seismic zone	III
Depth of footing (D)	1.5 m
Depth to centroid of effective side wall contact (h)	1.25 m

2.2 Details of soil parameters considered

The shear modulus of soil is calculated using below relation

$$G = \rho v^2, \tag{2.1}$$

where G is shear modulus of soil in N/m2, ρ = density of soil in kg/m^3, v is shear wave velocity in m/s. The details of soil

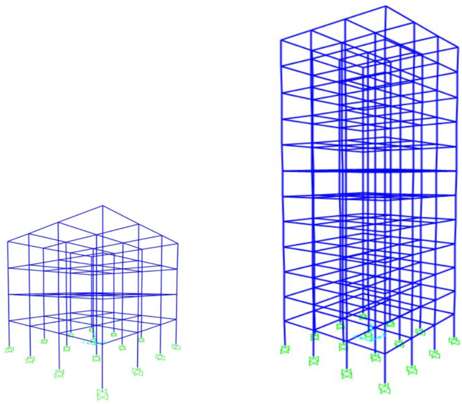

Fig. 1. Three-dimensional geometrical representation of frames.

parameters are shown in Table 2. In modelling the flexible base, the soil was idealized using equivalent springs and dashpots to simulate its stiffness and damping properties. The soil was assumed to be homogeneous, isotropic, and linearly elastic, ensuring a simplified yet reliable representation of soil–structure interaction (SSI). Damping was approximated, which accounts for energy dissipation into the soil, while non-linear soil effects were not explicitly modelled. These assumptions allowed an appropriate analysis of SSI influence on dynamic response.

Table 2. Details of soil parameters considered.

Type of soil	N SPT	V (m/sec)	ρ (kg/m^3)	G (MPa)	Bearing capacity (kN/m^2)
Hard soil	52	564	2082	662.27 $\times 10^6$	956
Soft soil	13	152	1682	38.86 $\times 10^6$	407

2.3 Calculation of equivalent stiffness of footing

To perform the soil structure interaction, it required to calculate the equivalent stiffness of the spring. In this study the expressions given by FEMA356 was used to calculate the stiffness of spring. The expressions given by FEMA356 are shown below in Table 3.

The dimensions of footing were calculated from the allowable bearing capacity of soil and the load on footing. From these two parameters the area of footing is calculated. After getting the area of footing the dimensions of cross-section were calculated. For each soil structure interaction model three different sizes cross-section were obtained. So, for each footing different stiffness was obtained. If the maximum value of load coming on to the footing is considered then, each model is modeled using single value of stiffness value. If single value of stiffness is used for a model, then that model acts as fixed base modal. So, there is no variation in the results of pushover analysis. So, to get variation in analysis of pushover, stiffness of each footing was calculated. The dimensions of the different footing are shown in Table 4.

2.4 Models of soil structure interaction

The models for four-story and twelve-story buildings are modeled in SAP2000. The models were modeled by giving the stiffness which was calculated in above Table 5. Due to these springs with different spring constants, the footing of building will act as flexible foundation as shown in Fig. 2.

2.5 Non-linear static analysis

Non-linear static analysis, also known as pushover analysis is the most widely used analysis procedure for design and seismic performance evaluation of structures, given its simplicity and ease of implementation. In this analysis, a predefined lateral load pattern as per Indian Standard is applied onto the gravity-loaded structure distributed throughout the height of the building (Oggu et al. 2019;

Table 3. Embedment Soil Foundation Stiffness in six directions.

Degrees of freedom	Stiffness of foundation at surface level	Correction factor for embankment
Translational along X-axis	$Kx = \frac{GB}{2-\mu}[3.4(\frac{L}{B})^{0.68} + 1.2]$	$\beta x = (1 + 0.21\sqrt{\frac{D}{B}})[1 + 1.6(\frac{hd(B+L)}{BL^2})^{0.4}]$
Translational along Y-axis	$Ky = \frac{GB}{2-\mu}[3.4(\frac{L}{B})^{0.68} + 0.4(\frac{L}{B}) + 0.8]$	$\beta y = (1 + 0.21\sqrt{\frac{D}{B}})[1 + 1.6(\frac{hd(B+L)}{BL^2})^{0.4}]$
Translational along Z-axis	$Kz = \frac{GB}{1-\mu}[1.55(\frac{L}{B})^{0.75} + 0.8]$	$\beta z = [\frac{D}{21B}(2 + 2.6\frac{B}{L})][1 + 0.32(\frac{d(B+L)}{BL})^{\frac{2}{3}}]$
Rocking about X-X axis	$Kxx = \frac{GB^3}{1-\mu}[0.4(\frac{L}{B}) + 0.1]$	$\beta xx = 1 + 2.5\frac{d}{B}[1 + \frac{2d}{B}(\frac{d}{D})^{-0.2}\sqrt{\frac{B}{L}}]$
Rocking about Y-Y axis	$Kyy = \frac{GB^3}{1-\mu}[0.47(\frac{L}{B})^{2.4} + 0.034]$	$\beta yy = 1 + 1.4(\frac{d}{B})^{0.6}[1.5 + 3.7(\frac{d}{L})^{1.9}(\frac{d}{D})^{-0.6}]$
Rocking about Z-Z axis	$Kzz = GB^3[0.53(\frac{L}{B})^{2.45} + 0.51]$	$\beta zz = 1 + 2.6(1 + \frac{B}{L})(\frac{d}{B})^{0.9}]$

Table 4. Dimensions of footing.

Footing location	Size of footing in Hard soil	Size of footing in soft soil
Corner footing	0.8 × 0.8m	1.2 × 1.2m
Side footing	1.06 × 1.06m	1.6 × 1.6m
Inner footings	1.33 × 1.33m	2 × 2m

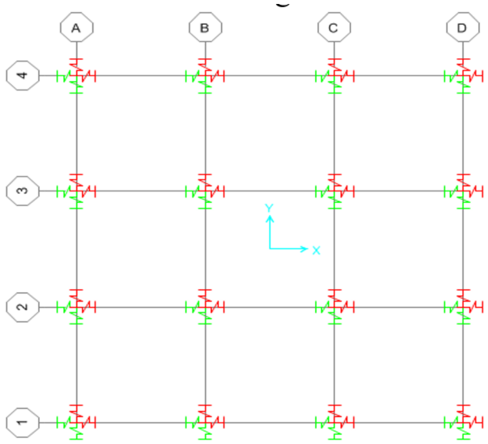

Fig. 2. Plan of building with springs (flexible footing).

Oggu et al. 2020). The lateral loads are monotonously increased until the structure reaches its target displacement or its ultimate capacity. The outcome of pushover analysis is the generation of the capacity curve as per FEMA356 guidelines. The yield strength, yield displacement, ultimate strength, and ultimate displacement are evaluated for the structural model from the capacity curve. These parameters aid in the estimation of various constituent components of 'R' value. The modal properties such as fundamental time periods of the models with different bases obtained from performing modal analysis are shown in Table 6.

3. Development of Fragility Curve

Fragility curves represent the probability of exceeding a damage limit state for a given structure type subjected to a seismic excitation. Fragility curves involve uncertainties associated with structural capacity, damage limit state definition and records of ground motion accelerations. Mainly two methods are used to develop fragility curves for a given building type. The methods are nonlinear static analysis and nonlinear dynamic analysis. In this study the fragility curves are developed using the nonlinear static analysis (pushover analysis). From pushover analysis capacity curve is obtained as shown in Fig. 3; by converting this capacity curve into bilinear curve the yield point and ultimate point of displacement and acceleration is obtained. The fragility parameters were calculated by using the equation 1 given by Barbat (2008). The Capacity Spectrum Method (CSM) was employed by first obtaining a pushover curve through nonlinear static analysis, representing base shear versus roof displacement. This curve was transformed into

Table 5. Equivalent soil stiffness values for isolated footing.

Direction	Units	Soil stiffness ($\times 10^3$)					
		Corner		Side footing		Inner footing	
		Hard soil	Soft soil	Hard soil	Soft soil	Hard soil	Soft soil
K_x	kN/m	5853.7	403.62	4921.5	462.79	6422.05	519.29
K_y	kN/m	5853.7	403.62	4921.5	462.79	6422.05	519.29
K_z	kN/m	3753.93	278.80	4381.03	617.25	5076.61	398.7
K_{xx}	kN-m/rad	1056.74	161.88	2033.09	158.175	3431.88	175.76
K_{yy}	kN-m/rad	913.03	175.39	1987.28	180.69	3716.99	196.64
K_{zz}	kN-m/rad	1551.26	231.63	2985.24	264.83	5103.03	292.45

Table 6. Fundamental time periods of the models with different bases.

Model	Fixed base	Hard soil	Soft soil
4-story model	0.396 s	0.399 s	0.422 s
12-story model	1.242 s	1.212 s	1.209 s

the Acceleration–Displacement Response Spectrum (ADRS) format to align structural capacity with seismic demand. The performance point was identified at the intersection of the capacity curve and the demand spectrum modified by effective damping. Displacement values at this point were then compared with predefined limit states for different damage levels.

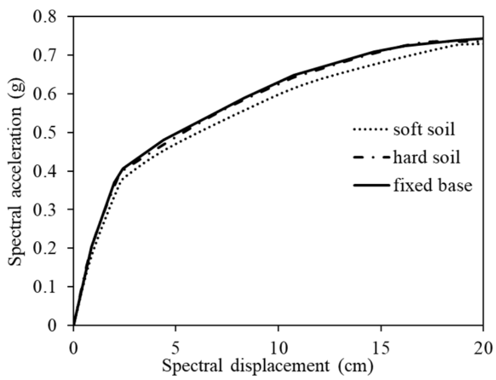

Fig. 3. Capacity Curves of 4-story models with Fixed and Flexible Bases.

Repeating this across intensities enabled development of fragility curves, which quantify probabilities of exceeding specific damage states under varying earthquake demands.

$$P\left[\frac{ds_i}{S_d}\right] = \Phi\left[\frac{1}{\beta_{ds_i}} \ln\left(\frac{S_d}{\bar{S}_{d,ds_i}}\right)\right], \quad (3.1)$$

where P is the conditional probability that the component will be damaged to damage state, i or a more severe damage state as a function of demand parameter, S_d; Φ denotes the standard normal cumulative distribution function; S_{d,ds_i} denotes the threshold spectral displacement at which the probability of the damage state d_{s_i} is 50%; and β_{ds_i} denotes the standard deviation of the natural logarithm of this spectral displacement. The fragility parameters were calculated from the bilinear curve of capacity curve. To calculate fragility parameters the yield point and ultimate points are needed. This yield point and the ultimate point were calculated from the bilinear curves. The threshold values of different damage states are shown in Table 7. The value of lognormal standard deviation depends on various uncertainties involved in the analysis. These values are directly available in HAZUS MH 2.1. The steepness of fragility curve depends on lognormal standard deviation. If the value increases, then the fragility curve becomes flatter. In present work the value of lognormal standard is taken as 0.7. The variation of lognormal distribution is not that much significant beyond 0.7. The effect of steepness of the fragility will not show that much variation beyond 0.7. The fragility curves of models with different base conditions are shown in Fig. 4.

Table 7. Threshold damage states.

Threshold Spectral Displacement	Damage state
$\bar{Sd}_1 = 0.7\ D_y$	Slight damage
$\bar{Sd}_2 = D_y$	Moderate damage
$\bar{Sd}_3 = D_y + 0.25(D_u - D_y)$	Extensive damage
$\bar{Sd}_4 = D_u$	Complete damage

3.1 Comparison of damaged probability matrices

The damaged probability matrices give idea about the variation in spectral displacement and damaged probability in each state. From damage probability matrixes the percentage of probability of exceedance can be calculated from each damage state. Tables 8-9 given below shows the variation of damaged probability for fixed base and flexible base models. The results are more adverse for soft soil as compared to hard soil and fixed base. This adverse effect is showing up to moderate damaged state, due to the lesser value of spectral displacement. For extensive and collapsed damaged state the probability of exceedance is more for fixed base, hard soil as compared to the soft soil, due to less value of spectral displacement. The probability of exceedance almost same in extensive damaged state for fixed base and that of hard soil.

4. Response Reduction Factors (R) for Different Structural Models

Response reduction factor generally designated as 'R' in most of the seismic codes. It is specified to account for nonlinear behavior and deformation characteristics in a linear elastic design. Further, the computation of 'R' value provides a qualitative understanding of seismic response and expected behavior of a code-compliant building for a design earthquake. Hence, an accurate estimation of 'R' is imminent in understanding the seismic behavior of a building. It has been reported in the literature that 'R' does not get affected by the number of bays and spans of the bays in a building frame. 'R' is a measure of the ability of a structure to dissipate energy and sustain inelastic deformations without collapse, allowing reduced seismic design forces compared to an elastic system. It encapsulates ductility, overstrength, and redundancy. Soil–structure interaction (SSI) significantly influences R by increasing the fundamental period of vibration through foundation flexibility and enhancing energy dissipation via damping into the soil. These effects often reduce seismic demand, leading to larger analytically derived R values compared to fixed-base assumptions.

Table 8. Damage probability matrix for 4-story model with different base conditions.

Spectral Displacement (cm)	Type of base	Probability of damage state			
		Slight	Moderate	Extensive	Complete
3	Fixed base	0.165	0.485	0.155	0.006
	Hard soil	0.143	0.557	0.155	0.005
	Soft soil	0.125	0.637	0.127	0.002
5	Fixed base	0.080	0.466	0.359	0.040
	Hard soil	0.060	0.507	0.363	0.033
	Soft soil	0.047	0.581	0.325	0.020

Table 9. Damage probability matrix for 12-story model with different base conditions.

In general, the response reduction factor is estimated as the product of over strength factor (R_s), ductility factor (R_μ), damping factor (R_ξ), and a redundancy factor (R_R). Since the structural models con-

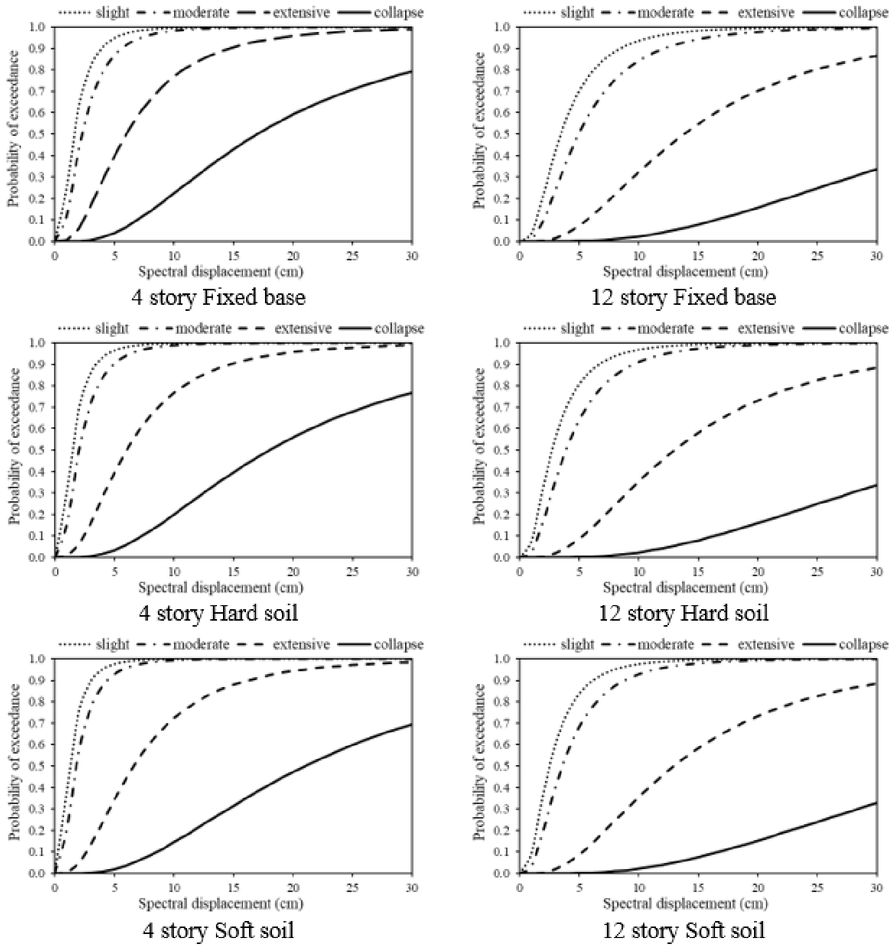

Fig. 4. Fragility curves for fixed and flexible base models.

sidered here do not have any damping energy dissipation devices, the damping factor is considered to be equal to 1. Similarly, the redundancy factor is considered to be 1. Hence, the critical factors for the estimation of 'R' boil down to Rs and R_μ. The parameters to be considered in Fig. 5 are as follows: design base shear (V_d), yield base shear (V_y), roof displacement at yield point (Δ_y), maximum elastic base shear (V_e), displacement at elastic base shear (Δ_e), and maximum displacement ($\Delta_{\max}$). From these parameters, R_μ. is estimated using the relationship proposed by Newmark and Hall (1982) shown in Eqs. (3.2)-(3.5).

$$R_\mu = 1 \quad \text{for } T < 0.2\ s, \tag{4.1}$$

$$R_\mu = \sqrt{2\mu - 1} \quad \text{for } 0.2\ s < T < 0.5\ s, \tag{4.2}$$

$$R_\mu = \mu \quad \text{for } T > 0.5\ s, \tag{4.3}$$

$$\mu = \frac{\Delta_{\max}}{\Delta_y}. \tag{4.4}$$

From Fig. 6, it can be observed that the ductility factors of both 4-story and 12-story models computed from NLS analysis were much higher than that recommended by IS 1893. Moreover, the ductility factors

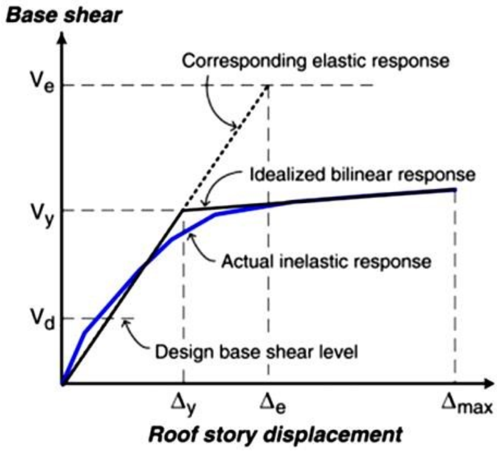

Fig. 5. A typical capacity curve for a structure.

computed for structural models with SSI contribution is found to be higher than corresponding fixed base models. The influence of SSI on seismic response of frames is evident from these results. This change in R values further affects the design base shear values, and might lead to unsafe or uneconomical design.

5. Conclusion

The present study is primarily focused on assessing the seismic behavior and checking the adequacy of code-based 'R' to appropriately represent the non-linear seismic demand of the structure. Soil-Structure Interaction (SSI) effects have been investigated to understand the seismic vulnerability of the building structures for varying soil profiles by modelling the foundation as spring models. Fragility curves have been developed as per the guidelines specified in ATC 40 for the four-story and twelve-story buildings. To study the soil-structure interaction effect, two different soils were considered (hard and soft). Nonlinear static analysis is performed to develop the capacity curve.

The damage probability matrices illustrate the variation in damage probability for both fixed base and flexible base models. The results indicate more severe effects for soft soil compared to hard soil and the fixed base condition. This adverse influence is particularly evident up to the moderate damage state, owing to the lower spectral displacement values. However, for extensive and collapse damage states, the probability of exceedance is higher in the fixed base and hard soil cases than in soft soil, again due to the relatively lower spectral displacement. Notably, in the extensive damage state, the probability of exceedance for the fixed-base model is nearly the same as that observed for hard soil.

The 'R' value is specified to reduce the actual base shear to design base shear, considering the inelastic behavior and deformation limits of the structures. Further, the changes in base conditions, which significantly alter the dynamic behavior of the structure, envisioned in terms of ductility demands needs to be considered. Moreover, analysis of RC buildings for estimation of seismic design forces is usually carried out only on the moment-resisting frames (MRF), ignoring the influence of SSI. This results in the erroneous estimation of the seismic behavior of the structure. Hence, NLS analyses are carried out on six hypothetical structural models, to estimate the adequacy of code specified constant 'R' value in estimating the seismic demand and the seismic behavior of the building.

Moreover, the ductility factors computed for structural models with SSI contribution is found to be higher than corresponding fixed base models. However, caution must be exercised because overly high R values may reduce conservatism, potentially increasing vulnerability if soil conditions or nonlinear behaviors are not properly captured. Hence, balance between

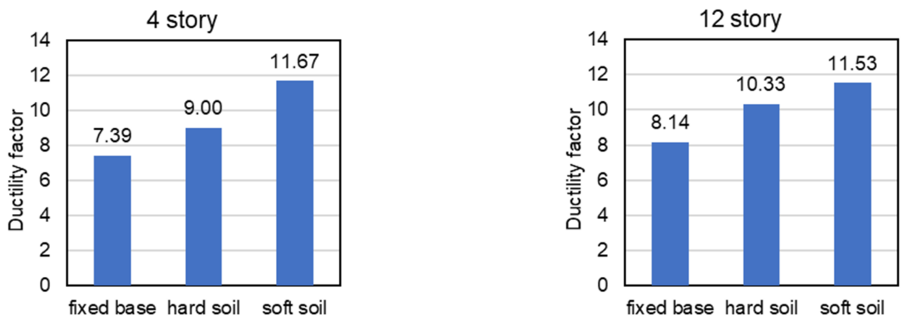

Fig. 6. Ductility factors of 4-story and 12-story models for different base conditions.

economy and safety is crucial. This signifies the dependence of '*R*' value on the structural configuration emphasizing an imminent need to consider the influence of SSI in the estimation of '*R*' value for a building structure. It can be observed that the '*R*' value specified by IS 1893 for a particular RC MRF is constant and is found to be significantly less than the '*R*' values computed for all the building configurations using NLS analysis. This necessitates the importance of dynamic characteristics of structures in estimating the load-carrying capacity, resulting in an adequate estimation of design forces leading to optimal design configurations. This discrepancy highlights the need for more refined guidelines that explicitly incorporate soil flexibility in determining design *R* values. Therefore, it can be concluded that the estimation of '*R*' values should encompass the dynamic characteristics of building configurations.

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