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COMPARATIVE STUDY ON THE SEISMIC PERFORMANCES OF TYPICAL RC RESIDENTIAL BUILDINGS DESIGNED WITH OLD AND NEW INDIAN CODES IN BHUTAN

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ABSTRACT

In Bhutan, the Indian seismic codes are referred for the design of reinforced concrete (RC) buildings. These seismic codes were revised in 2016 as a result of earthquakes that took place after their publication. The frequent and recent earthquakes in and around the Himalayan region have caused substantial damage to RC buildings necessitating for study of seismic performance. This paper presents a comparative study on the seismic performances of typical five-story RC residential buildings designed with old and new Indian codes. Accordingly, two three-dimensional models of a building designed with the old and new Indian codes are developed using STERA_3D software. The seismic performances are evaluated using the Capacity Spectrum Method (CSM) and non-linear Response History Analysis (RHA) for three input ground motions. In addition, the structural damage estimates given by damage indices are compared under scaled ground motions. Results show that the building designed with the new Indian codes provide reduced structural responses when compared to that designed with old Indian codes. Furthermore, the assessment of the damage indices for the building designed using the new codes shows that the building has more even damage dispersion over the floors and prevents collapse-level structural damage under the considered maximum scaled ground motion.

Keywords: RHA, CSM, Input Ground Motions, Damage Indices.

1. INTRODUCTION

Considering the frequent seismicity in and around the Himalayan region, in their paleoseismic research, Le Roux - Mallouf et al., (2016) reported the occurrence of an earthquake event with a magnitude 8 ± 0.5. In another study, by Roux-Mallouf et al., (2020) stated that at least five of such earthquakes struck the country over the last 2600 years. In 2015, Nepal suffered from a catastrophic Mw7.8 magnitude earthquake. Bhutan was spared from this disaster despite its proximity to Nepal and its comparable geography. However, by the 2009 Mongar Earthquake (Mw6.1), which struck 180 kilometers east of Thimphu, destroyed several houses and took many lives. Similarly, the 2011 Sikkim earthquake (Mw6.9), with an epicenter near the borders of India and Nepal, also caused substantial structural damage to the country. The study conducted by Bhagat et al., (2015), which examined the seismic

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performance of RC building designed with Indian codes in Nepal offers valuable insight and importance to evaluate seismic performance of buildings in Bhutan as well.

In Bhutan, two important seismic codes are generally referred for design of RC framed buildings; Indian Standard IS 1893:2002 and IS 13920:1993. In 2016, both the codes were revised as a result of earthquakes that took place after their initial publication. Therefore, this study primarily aims to investigate and compare the seismic performances of a typical RC residential buildings designed using the old Indian codes and new Indian codes. For this purpose, two models were developed using different member sections from the design output of the former and recent Indian codes. In addition, the damage estimate on the target models is evaluated using the Damage Indices under scaled real-ground motions.

2. TARGET BUILDINGS AND MODELS

Figure 1 illustrates a typical existing five-story residential RC framed building that exists in Bhutan. The model designed using old Indian codes is hereafter referred to as "OLD Model" while that designed using new codes is referred to as "NEW Model". Both the models were designed as fixed-base special moment-resisting RC famed buildings without brick infill for 500 MPa yield strength and 25 MPa concrete compressive strength. Seismic zone V and medium soil site condition were considered for both models. To evaluate the Seismic Performance of the target models, Software STERA_3D developed by another author is used. The first mode of vibration for both the models is observed in the Y-direction. The fundamental natural period, T_1 of OLD and NEW Models are 0.897 s and 0.785 s, respectively.

Figure 1. Target Building: (a) Floor Plan; (b) Sectional Elevation.

3. SEISMIC PERFORMANCES OF THE TARGET BUILDINGS

3.1. Input Ground Motions

For Response History Analysis, three Ground Motions are considered. Two input ground motions of duration 120 s are simulated using algorithm developed by another author. The first simulated ground motion is scaled to match the target design spectrum (i.e., acceleration response spectrum, h =5% of IS 1893:2002), with random phases and max. acceleration of 3.47 m/s^2 is obtained. The second artificial ground motion is simulated by scaling to the target design spectrum with the retained phases of the Kobe 1995 ground motion recorded at Totoro Station, Japan and max. acceleration of 3.52 m/s^2 is obtained. The third input ground motion considered is real Nepal 2015 earthquake record with duration of 300 s and max. acceleration of 1.6 m/s^2,

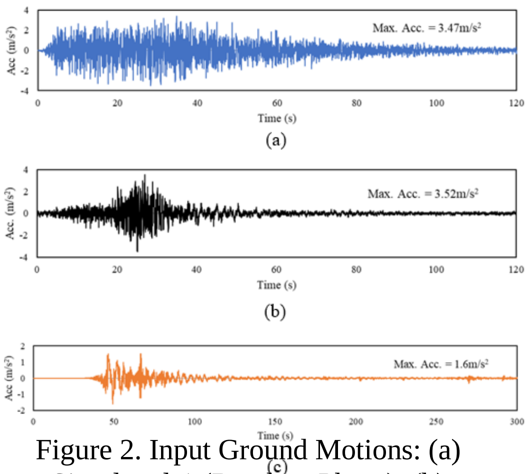


Figure 2. Input Ground Motions: (a) Simulated-1 (Random Phase); (b) Simulated-2 (Kobe_TOT90 Phase); (c) KATNP_360.

KATNP_360 measured at Kathmandu station. Figure 2 shows the time history of input ground motions.

3.2. RHA Results for Input Ground Motions

Prior to seismic performance evaluation, the comparison between the ultimate strengths (base shear) of the target models is first evaluated from the pushover analysis. Pushover analysis results showed a difference of 37% in strength. The NEW model has a strength of 3405 kN whereas the OLD model has strength of 2487 kN which is significantly larger when compared to design base shear.

The direct numerical integration of the differential equations of the motion of target models using the input ground motions with the aid of software like STERA_3D, RHA is performed. The maximum responses of the RHA for input ground motions obtained for both the models are as presented in Table 1. The responses over the height of the structures are presented in Figure 8.

Table 1. Maximum Response Results for Input Ground Motions in the Y-direction.

Maximum Response		Story Drift Angle, R_i	
		OLD Model	*NEW Model*
Simulated-1	Second Floor	**1/74**	1/90
Simulated-2	Second Floor	1/81	**1/86**
KATNP_360	Second Floor	1/80	1/143

Maximum response of Story Drift Angle, 1/74 is obtained under Simulated-1 ground motion for OLD model whereas for NEW model 1/86 is obtained under Simulated-2 ground motion. A distinct difference in the responses of the OLD and NEW models is also observed under real ground motion. Based on the response results obtained from the RHA under the selected input ground motions, the NEW model yields a better performance than the OLD model.

3.3. CSM Procedure and Results for the Design Code Demand Spectrum

The CSM, a nonlinear static analysis developed by Freeman, (1998) is performed for design code demand spectrum to obtain performance points which represent the actual response of the structure under seismic loading.

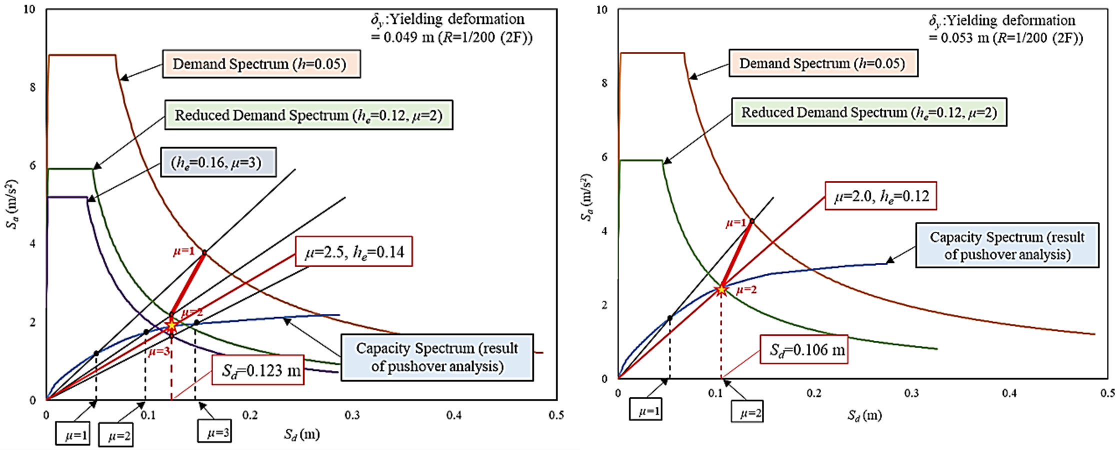
$$S_d = \frac{S_a}{\omega^2} = S_a \left(\frac{T}{2\pi}\right)^2 \quad (1)$$

$$\mu = \frac{S_d}{\delta_y} \quad (2)$$

$$F_h = \frac{1.5}{1 + 10h_e} \quad (3)$$

$$h_e = 0.25\left(1 - \frac{1}{\sqrt{\mu}}\right) + 0.05 \quad (4)$$

At first, the Spectral Displacement, S_d is calculated using Eq. (1) where S_a, is spectral acceleration values of Design Acceleration Spectrum for 5% damping and medium soil site (IS 1893). ω is the natural frequency. From pushover analysis, the Capacity Spectrum is obtained using nonlinear distribution of deformation along the height of the structure (Kuramoto et al., 2000). The yield displacement, δ_y is then decided for inter story drift angle of 1/200. Then the reduction of structural response due to increasing level of damage is applied to Demand Spectrum by multiplying both values of S_a and S_d by a reduction factor, F_h. The reduction factor for each equivalent damping ratio, h_e corresponding to ductility factor, μ =2 and μ =3 is calculated using Eq. (3). For the equivalent damping ratio, h_e Eq. (4) and for ductility factor, μ Eq. (2) is used. Finally, the performance points obtained by CSM are presented in Figure 3 and corresponding responses are plotted in Figure 4.



(a) (b)

Figure 3. Performance Points for Design Code Demand Spectrum in Y-direction:
(a) OLD Model; (b) NEW Model.

The response results by CSM for Design Code Demand Spectrum shows that the second floor of OLD model will sustain more damage with story drift angle 1/78 when compared to the NEW model with less story drift angle of 1/98.

3.4. CSM Results for the KATNP_360 Demand Spectrum and Comparison with the RHA Results

Similarly, CSM is carried out for KATNP_ 360 record. The CSM response results indicated that the NEW model with maximum story drift angle of 1/128 will sustain less damage when compared to the OLD model with the higher story drift angle of 1/86.

The results are then compared with RHA results, and it shows good agreement in both values and pattern as presented in Figure 5. The CSM requires less computational effort and time, making it convenient and applicable when a large number of buildings has to be studied.

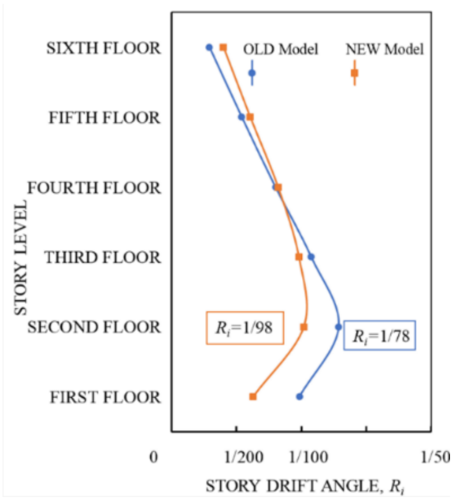


Figure 4. Response of Target Models by CSM for Design Code Demand Spectrum.

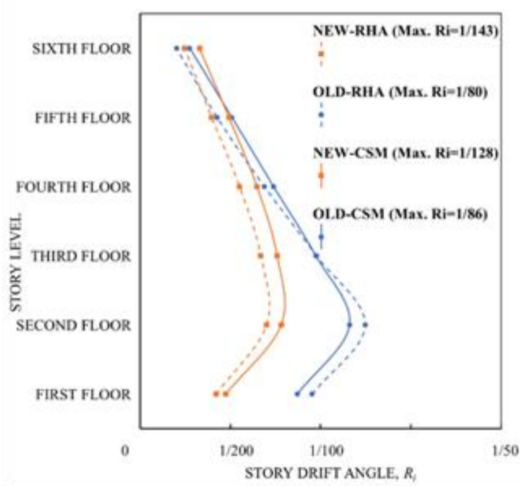


Figure 5. Comparison of CSM and RHA response results (KATNP_360)

4. DAMAGE INDEX

The basic model developed by Park et al., (1984) to represent the structural damage on the RC elements such as beams and columns is given by Eq. (5) and it is expressed simply by Figure 6.

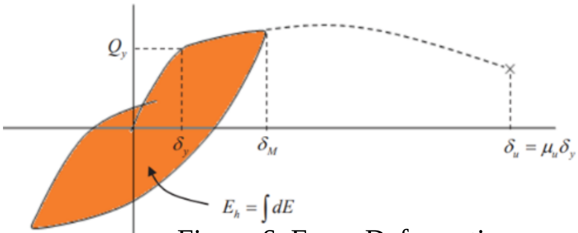


Figure 6. Force Deformation Relationship of an RC Element.

$$D_i = \frac{\delta_m}{\delta_u} + \beta \frac{E_h}{Q_y \delta_u} = \frac{\mu_m}{\mu_u} + \beta \frac{E_h}{Q_y \delta_u} \qquad (5)$$

where, $\delta_m = \mu_m \delta_y$, is the maximum deformation under an earthquake; $\delta_u = \mu_u \delta_y$, is the ultimate deformation under a monotonic loading; μ_m, is the maximum ductility factor under an earthquake; μ_u, is the ultimate ductility factor under a monotonic loading (STERA_3D adopts μ_u =15); δ_y, is the yield deformation; Q_y, is the yield strength; β, is the parameter related to the cumulative loading effect given by Eq. (6) and; $E_h = \int dE$, is the dissipated hysteretic energy.

$$\beta = \left(-0.447 + 0.073 \frac{l}{d} + 0.24 n_0 + 0.314\, p_t\right) \times 0.7^{\rho_w} \qquad (6)$$

where, $\frac{l}{d}$, is the shear span ratio (replaced by 1.7 if $\frac{l}{d}$<1.7); n_0, is the normalized axial stress, axial force by compressive strength (replaced by 0.2 if n_0<0.2); p_t, is the longitudinal steel ratio as a percentage (replaced by 0.75% if p_t<0.75%); ρ_w, is the confinement ratio.

Further the RC damage index is modified to the story-level damage index given by Eq. (7), and it is calibrated into the categories of physical structural damage. The story-level damage index, $D_{story} \leq 0.4$ is considered to be Reparable, whereas $D_{story} > 0.4$ represents Damage Beyond Repair, and $D_{story} \geq 1.0$ represents Total Collapse.

$$D_{story} = \sum_i^n w_i D_i \qquad (7)$$

$$w_i = \frac{E_{h,i}}{\sum_i^n E_{h,i}} \qquad (8)$$

where, D_{story}, is the story-level damage index of structure; n, is the number of elements in the story or whole structure; D_i, is the damage index of the i-th element; w_i, is the weighting factor of the i-th element given in Eq. (8).

Under different intensities of KATNP_360 ground motion, the weighted maximum story-level damage indices of the target models are evaluated as shown in Figure 9. The results indicates that under the 2.2KATNP ground motion, the level between base to first floor of the NEW model will sustain damage beyond repair with a maximum story damage index of 0.497 whereas OLD model will collapse with maximum story damage index of 1.053 which is not desirable.

5. SEISMIC PERFORMANCE OF THE RETROFITTED OLD BUILDING

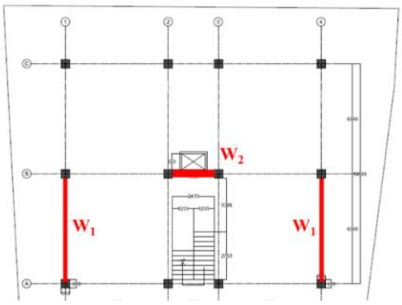


Figure 7. Layout of Shear Walls.

Throughout the height of the building, designed with old Indian codes, three shear walls are proposed, two in weaker direction and one in stronger direction as illustrated in Figure 7. The walls are designed for 500 MPa yield strength and 25 MPa concrete compressive strength and the reduction of strength for openings is applied from Seismic Retrofit of Existing Reinforced Concrete Buildings, 2001 (JBDPA). For the Retrofitted model the fundamental natural period is 0.596 s and the first mode of vibration is in X-direction.

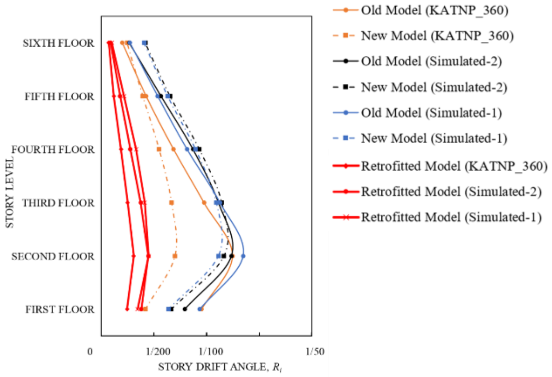


Figure 8. RHA Results in Y-direction.

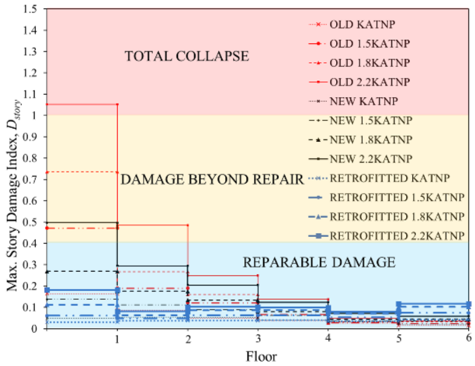


Figure 9. Damage Index Results

From Figure 8, the results show that under the considered input ground motions the Retrofitted Model will demonstrate improved performance by limiting responses within 1/200 story drift. In addition, the maximum story-level damage indices for all the models plotted in Figure 9 indicate that the Retrofitted model will exhibit only repairable damage over all floors even under maximum considered intensity.

6. CONCLUSIONS

The following conclusions are drawn based on the evaluation results. Although the base shear capacities of the target models are significantly larger than the design values, the earthquake-induced shear force from the input ground motions was large enough to cause damage to both target models. The responses showed that the structure designed with new Indian codes perform better. The results from CSM for Design Code Demand Spectrum also indicated that the structure designed using the new codes provided improved performance compared to that designed with old codes. The CSM provided reliable results when compared to RHA results. Under maximum intensity of real-ground motion, the damage indices indicate collapse-level damage for the building designed with old Indian codes, which is not desirable. However, adding shear walls as retrofitting measure not only prevents collapse level damage but also reduces the response of the structure by large.

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