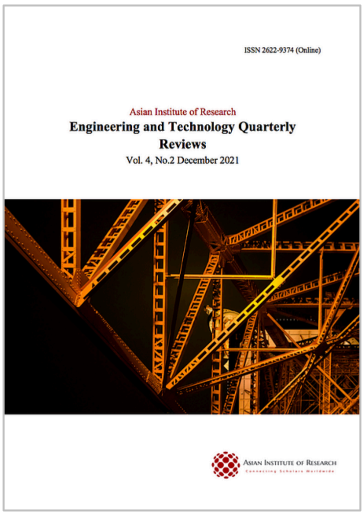

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Effect of Building Configuration on Overstrength Factor and Ductility Factor

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Abstract

It is important for the structure to be economical and still have a high level of life safety. The lateral force sustained by the structures during a large earthquake would be several times larger than the lateral force for which the structures are designed. This is opposite to the fact that design loads such as gravity in codes are usually higher than the actual anticipated load. It is based on the probability that the occurrence of large earthquakes is quite rare and the capacity of the structure to absorb energy. The co-factors of response reduction factor which is the overstrength factor and ductility factor reduce the design horizontal base shear coefficient. A total of 36 low-rise residential buildings having different storey, bay and bay lengths are selected and analysed in this paper. NBC 105: 2020 is selected for the seismic design of RC buildings while provision provided in FEMA 356:2000 is used to carry out non-linear pushover analysis. The results indicated that between the different structures, the value of overstrength factor and ductility factor has a high deviation.

Keywords: Overstrength Factor, Ductility Factor, Response Reduction Factor, Pushover Analysis

1. Introduction

1.1 Background

Reinforced concrete (RC), is a composite building material where compressive strength is provided by concrete and tensile strength is provided by rebars. RC buildings are the ones in which members resisting lateral and gravity loads are made up of reinforced concrete. A total 54,23,297 buildings exist in Nepal out of which 5,39,004 of them are RC structures which tallies to around 9.94%. Post 2015 Gorkha earthquake, RC structures in Nepal is increasing rapidly and are replacing the load-bearing masonry structure.

The structure should be designed in a way that it is economical without compromising the safety and serviceability. The design philosophy followed by most of the design code states that absolute safety and no damage, even in an earthquake with a reasonable probability of occurrence, cannot be achieved. Utilization of inelastic responses of the structure helps in reducing the member sizes making the structure economical while still maintaining a high level of life safety. A term referred to as force reduction factor or response reduction factor is used in most of the seismic design codes to reduce the elastic lateral force to a lateral design force.

1.2 Need for Research

The force reduction factor in most of the seismic design codes makes use of a constant pre-determined value mostly depending just upon structural type and the detailing procedure for seismic analysis and design. The NBC 105:2020 has a single value for overstrength factor and ductility factor on behalf of response reduction, both of which are governed by the type of structural system. This may not be justified as it has been found that it depends upon various parameters such as building configuration. Very few researches account for their effect on overstrength factor and ductility factor in RC buildings. Therefore, it is essential to study the real behavior of RC buildings through non-linear analysis to assess the value of these factors considering different geometry of the structure in the context of Nepal.

1.3 Literature Review

Literature review is carried out as a part of this study to gain insight into the works to be carried out and to act as a guide for the successful completion of this paper. The problems related to this work are identified and necessary references was taken from the literatures shown below:

H. Chaulagain et al. (2015) published a paper on seismic response of RC buildings in Kathmandu valley. The house survey was done in 10 districts and a total of 300 houses were surveyed out of which 200 houses were taken for the study and were classified under various topics. Some of the conclusions were that engineered structures have higher strength and lower deformation whereas non-engineered buildings in Nepal exhibited high vulnerability with low ductility.

Barakat et al. (1997) carried out seismic nonlinear time-history analysis three-dimensional G+3, G+5, and G+7 storey RC buildings. These buildings had shear walls in both orthogonal directions. The number of bays and bay sizes also differed in both orthogonal directions. The code of practice for design was the Jordanian Seismic Code and seismic zones were varied from zones 4, 3, 2, to 1. The El Centro (N-S) earthquake record of May 1940 as an actual earthquake excitation was used for time-history analysis. It was observed that the seismic zoning has a slight effect on the ductility reduction factor for different buildings and the value of the ductility reduction factor was almost the same as the displacement ductility ratio. The overstrength factor was found to vary with number of stories, seismic zones, and design gravity loads. However, seismic zones affected the overstrength most. The overstrength decreased as the number of storeys increased. The variation in response reduction factor has a significant implications for the seismic design codes which currently does not account for it.

Elnashai et al. (2002) published a paper to address the issue of overstrength in modern code-designed RC buildings. The nonlinear static pushover analysis and time history analysis for twelve buildings of various characteristics were carried out. He concluded that the nonstructural elements contribute to producing higher overstrength in the building. He also stated that pushover analysis is more appropriate for low-rise and short period structures to predict the responses.

Humar and Rahgozar (1996) published a paper to establish a concept of overstrength in seismic design. A static nonlinear pushover analysis was carried at the moment resisting steel building frames from G+1 to G+29. The result concluded that the building designed using a current seismic code possesses considerable reserve strength. He also highlighted the sources contributing to the reserve strength in the buildings i.e., serviceability criteria, actual vs nominal material strength, discrete member sizes, code-based strength, presence of non-structural members etc.

$$\Omega = \frac{V_y}{V_d} \tag{1}$$

Miranda (1993) evaluated the site-dependent strength reduction factor that is used to reduce elastic design spectra to account for the hysteretic energy dissipation of the structure. A total of 124 earthquake ground motions which were recorded on 3 different soil conditions i.e., rock soil sites (38 records), alluvium soil sites (62 records), and soft soil sites (24 records). After carrying out the regression analysis the equation for ductility reduction factor (R_μ) was computed assuming 5% critical damping. The findings also included that soil condition also greatly affects the mean strength reduction factor. The total of 3 different equations was proposed for 3 different soil conditions which depend upon displacement ductility ratio (μ) and period of vibration (T).

The equations are as follow: -

$$R_\mu = \frac{\mu - 1}{\phi} + 1 \geq 1 \tag{2}$$

For rock soil sites

$$\phi = 1 + \frac{1}{10T - \mu T} - \frac{1}{2T} \exp\left[-\frac{3}{2}\left(\ln T - \frac{3}{5}\right)^2\right] \tag{3}$$

For alluvium soil sites

$$\phi = 1 + \frac{1}{12T - \mu T} - \frac{2}{5T} \exp\left[-2\left(\ln T - \frac{1}{5}\right)^2\right] \tag{4}$$

For soft soil sites

$$\phi = 1 + \frac{T_g}{3T} - \frac{3T_g}{4T} \exp\left[-3\left(\ln\frac{T}{T_g} - \frac{1}{4}\right)^2\right] \tag{5}$$

Where,

ϕ	=	Function necessary to compute approximate strength reduction factor
T	=	Period of vibration
Tg	=	Predominant period of ground motion

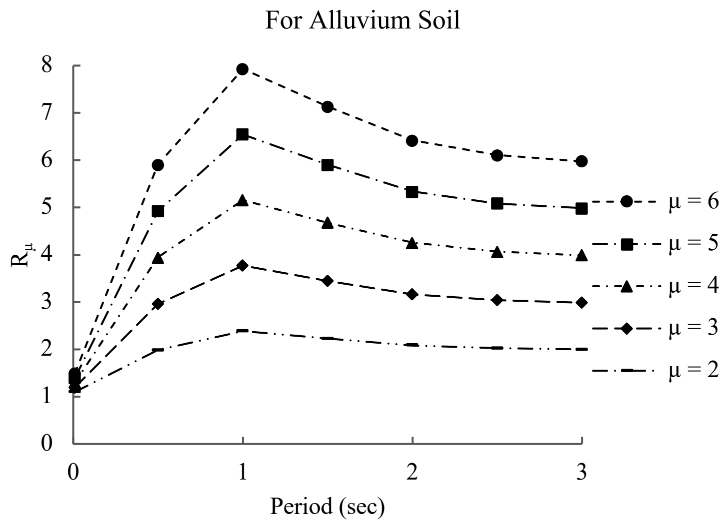

Figure 1: Ductility factor for alluvium soil as per Miranda

2. Building Description and Modelling

For modeling, analyzing (linear and non-linear), and designing of all the models, finite element analysis software ETABS v19.0.0 is used. The mathematical model is created in the software which closely represents the real model. 3D models of the structures are created where beams and columns are modelled as the frame elements and slabs as shell elements that are interconnected at nodes.

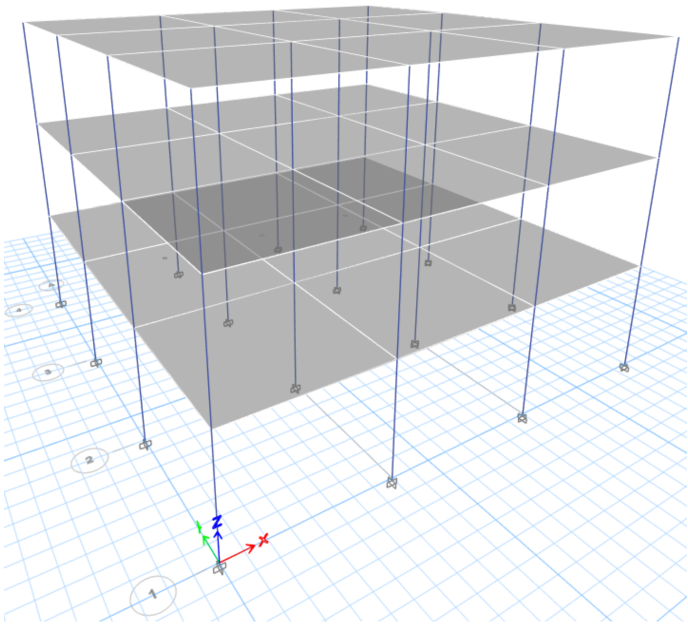

Figure 2: Finite element modelling of 3 storey 3 bay 4m bay length model

2.1 Structural Modelling Parameters

To incorporate the maximum number of different configurations of RC buildings of Nepal, following modeling parameters are selected.

1) Low-rise buildings having 2, 3, 4, and 5 number of storeys with a regular storey height of 2.9m are modeled.
2) The number of bays is taken as 2, 3, and 4 having bay lengths of 3m, 3.5m, and 4m which make these buildings have the plinth area from 36m2 to 256m2.
3) The buildings having regular plan and elevation with an equal number of bays in both the horizontal directions are considered in this study.

Table 1: Material properties

Properties	Reinforcement	Concrete
Grade	HYSD 500	M25
Unit Weight	76.97 kN/m^3	25 kN/m^3
Modulus of Elasticity	200 Gpa	25 Gpa
Poisson's Ratio	0.3	0.2

Table 2: Structure details

Building and Seismic Parameters	
Storey Height	2.9 m
Slab Thickness	125 mm
Soil Type	Medium Soil
Seismic Zoning Factor	0.4

Importance Factor	1
Damping in Structure	5%

Table 3: Loads on structures

Applied Loads	
Floor Finish	1 kN/m^2
Roof Live Load	1.5 kN/m^2
General Floor Live Load	2 kN/m^2
Outer Wall Load	7.5 kN/m
Partition Wall Load	4 kN/m
Lateral Load	NBC 105: 2020

2.2 Structural Members

The total of 36 unique configuration of buildings are modelled with smallest size of beam and column possible to satisfy the necessary design and serviceability check following NBC 105:2020 in which sizes of column and beam differ according to number of storeys only i.e., within a particular storey for different number of bay and bay length, the sizes of beam and column does not differ. However, all the frame elements (column, beam) in a particular building model are of the same size.

For Storey	Beam Dimension (D X B)	Column Dimension (D X B)
2	14“ X 9“	12“ X 12“
3	14“ X 9“	13“ X 13“
4	14“ X 10“	14“ X 14“
5	16“ X 10“	15“ X 15“

3. Pushover Analysis

The non-linear analysis is also known as pushover analysis is carried out to obtain the base shear vs top floor displacement curve. ASCE 7-16 is used to assign default hinges based on ASCE 41-13 at beam column joint where the members are expected to fail. This captures the material non-linearities. A non-linear gravity case is applied which incorporates total dead load plus 30% of live load which is a force-controlled load. The pushover load case is continued at the end of the gravity case until the displacement reaches an assigned value or the structure becomes unstable due to the formation of a plastic hinges. The base shear vs top floor displacement curve also known as capacity curve is obtained which is then idealized based on the provision provided in FEMA 356:2000 to obtain yield displacement (dy), ultimate displacement (du), significant yield strength (Vy). The bilinearization based on equal energy concept.

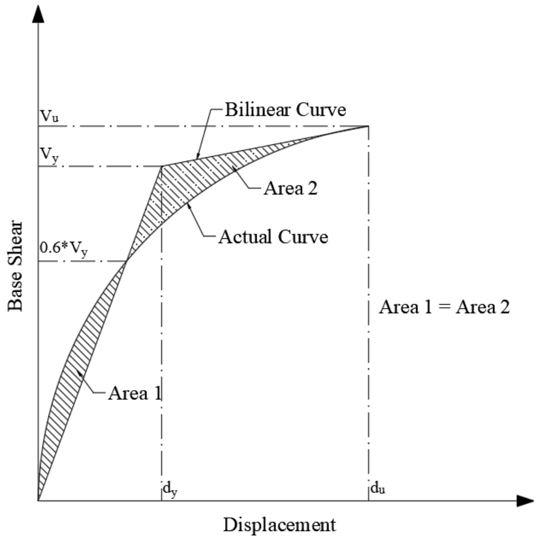

Figure 3: Bilinearization of pushover curve

4. Results and Discussions

This section presents the results obtained from the non-linear pushover analysis of the building models. The results are evaluated and compared to find the influence of different parameters on overstrength factor (Ω) and ductility factor (R_μ).

4.1 Effect on Overstrength Factor

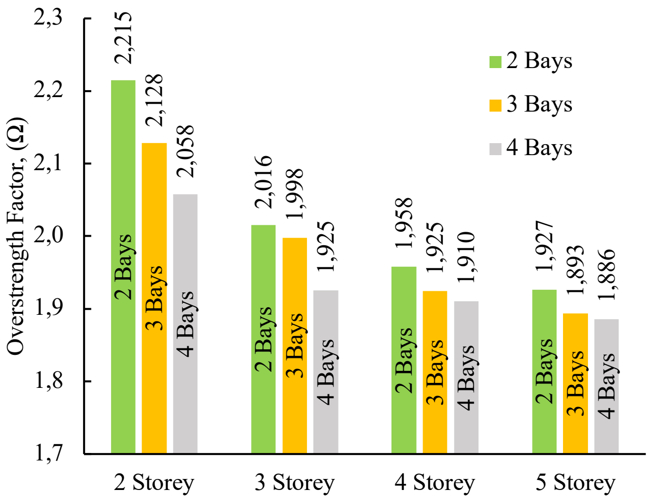

Figure 4: Overstrength factor for 4m bay length model varying number of storey and bay

The overstrength factor decreases while increasing the number of storey. While increasing the number of storey, both the design base shear and the yield strength increase but the yield strength increases at a lower rate than the design base shear which eventually decreases the overstrength factor. While increasing the number of bay does not affect the overstrength factor, as both the design base shear and the yield strength increases at almost the same rate. So, the overstrength factor varies only slightly. Also, its effect further decreases with an increase in the number of storey.

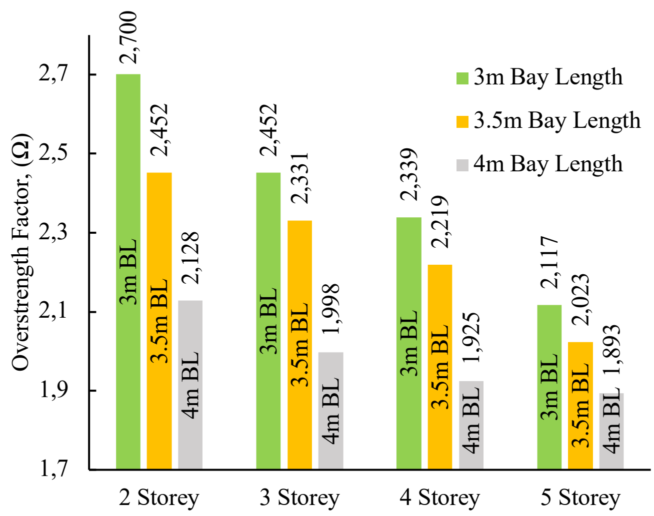

Figure 5: Overstrength factor for 3 bay model varying number of storey and bay length

In the case of bay length, increasing it decreases the overstrength factor. This can be explained as increasing the bay length only increases the seismic weight / design base shear but does not increase the lateral stiffness.
The overstrength factor varied from the highest 2.873 for the smallest model having 2 storey, 2 bays and 3m bay span to the lowest 1.886 for the largest model having 5 storey, 4 bays and 4m bay span.

4.2 Effect on Ductility Factor

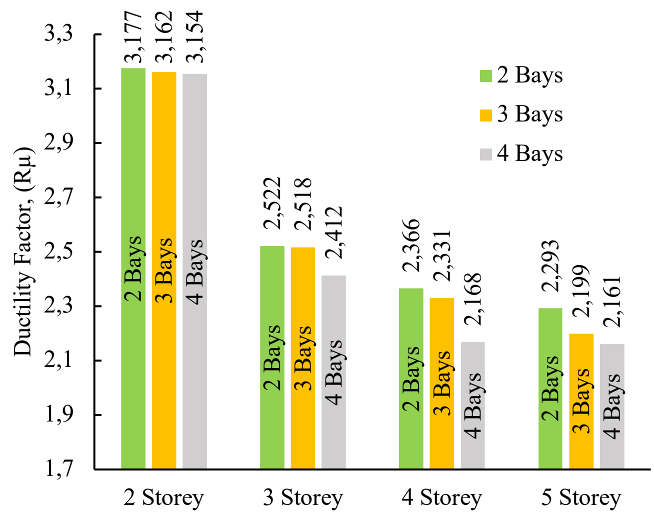

Figure 6: Ductility factor for 4m bay length model varying number of storey and bay

The value for ductility factor is higher for 2 storey buildings but reduces with the increase in number of storey from 2 to 5. Even though the time period increases as the number of storey increases but the value of displacement ductility ratio (μ) decreases significantly which then reduces the ductility factor. The effect of number of bays on the ductility factor is very less showing slight decrease with an increase in number of bay. This can be explained as increasing the number of bay makes the building stiff. But contrary to overstrength factor, the effect of number of bay further demises as the storey decreases.

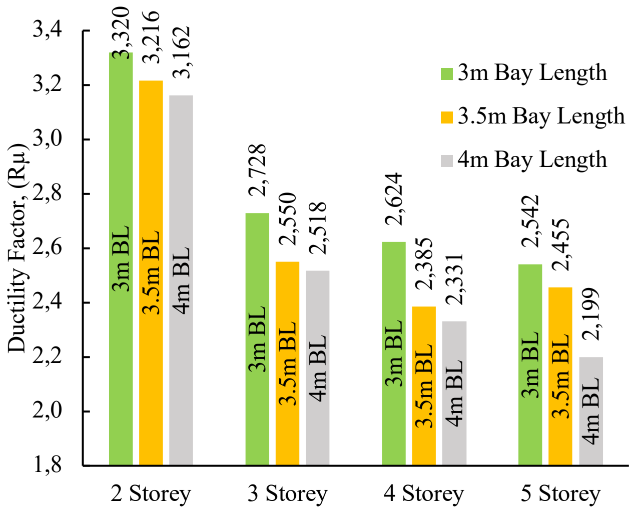

Figure 7: Ductility factor for 3 bay model varying number of storey and bay length

Similar to the overstrength factor, increasing the bay length decreases the ductility factor. This can be justified cause increasing the bay length decreases the displacement ductility ratio (μ), decreasing the ductility factor.

4.3 Generalized Equation for Overstrength Factor and Ductility Factor

A generalized equation has been proposed to calculate the overstrength factor and ductility factor by carrying out the regression analysis. The factors considered are number of storey, number of bay and bay length.

$$\Omega_u = 4.4668 - 0.1233S - 0.0601B - 0.4627BL \qquad (6)$$
$$R_\mu = 4.6839 - 0.2603S - 0.0502B - 0.2719BL \qquad (7)$$

Where,

Ω_u	=	Overstrength Factor
R_μ	=	Ductility Factor
S	=	Number of Storey
B	=	Number of Bays
BL	=	Bay Length

5. Conclusions

A total of 36 buildings models were analysed to obtain the overstrength factor (Ω_u) and ductility factor (R_μ). The force vs displacement curves were obtained by non-linear static pushover analysis. Using extensive statistical tools, an empirical equation has been proposed for overstrength factor and ductility factor.

$$\Omega_u = 4.4668 - 0.1233S - 0.0601B - 0.4627BL \qquad (8)$$
$$R_\mu = 4.6839 - 0.2603S - 0.0502B - 0.2719BL \qquad (9)$$

In addition to the formulation of empirical relation, the following conclusions has been made from the analytical study carried out by varying the building configurations.

- Both the overstrength factor (Ωu) and ductility factor ($R\mu$) is dependent upon many parameters such as building configurations. Using a single value for them will introduce the unwanted uncertainty in the building.
- The dependency on bay length and number of storey is more than the number of bays for both the overstrength factor and ductility factor. For the overstrength factor the effect of bay length and number of

bays reduced as the number of storey increased but it is opposite in the case of ductility factor.

- According to NBC 105: 2020 for ultimate limit state, the value of overstrength factor and ductility factor are 1.5 and 4 respectively for RC moment resisting frame. The value obtained from the analysis showed the higher value (>1.5) for overstrength factor ranging from 1.886 to 2.873 while for the ductility factor the value fluctuated from 2.161 to 3.283 which were less than (<4) that specified in the code.

These conclusions are limited to the scope of the work carried out in this research. More wider parameters need to be included to reduce the limitations of this research in order to accurately predict the overstrength factor and ductility factor.

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