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# Evaluation of Seismic Response of Reinforced Concrete Building Designed as per as NBC 105:2019 (Draft)

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**Abstract**

This study presents the evaluation of the seismic performance of low rise reinforced concrete building design as per NBC 105:2015 draft using linear analysis.Nonlinear analysis was use to develop capacity curve of the buildings.NBC 105:2019 is the revised document of seismic code in Nepal published as"Nepal National Building Code for Seismic Design of Buildings in Nepal, NBC 105:2019'. This is the first revision of the seismic code in Nepal which was developed in the Nineties as "Nepal National Building Code for Seismic Design of Buildings in Nepal, NBC 105:1994". However, to this date, the document NBC 105:2019 is not yet approved only the draft version is published. In this study, three regular building models were selected. It was assumed that the selected building lies on seismic zone four with zone factor 0.4 and site class B. Seismic load was calculated as stipulate in NBC105:2019. Linear analysis of the building were made using the appropriate load combination and performance were evaluated. Capacity design of the building was done and the detailed buildings were subjected to pushover analysis. Fiber hinges were used to capture the nonlinearity of the buildings Ductility factor and the overstrength factor of the buildings were calculate from the building's capacity curve.

**Keywords**

Nonlinear static analysis – performance – ductility factor – over strength factor

## 1. Introduction

During an earthquake, the civil structure is subjected to the random motion at its base which induced inertial force and in turn cause stress on that structure. Hence, civil struc-tures are needed to be designed to withstand such forces besides the gravity load which they are intended to support. Cyclic ground motion, its randomness, and ground motion induced forced on the structure make the natures of the earthquake actions very different from other lateral forces, for example, wind. The frequency of occurrence of a large earthquake is very low. The probability of occurrence of a large earthquake in the service period of any civil structure is significantly small so is not economically viable to design structure to resist the earthquake-induced lateral force fully elastically. It is common to design the structure for fraction of design earthquake force for strength, and the rest of the force is resisted by inelastic deformation of material and energy dissipation device as the result the structure will survive under the worst seismic loading condition and at the same time design would be economical. Seismic consideration dominant design required a rational design approach. Structural strength should be enough for design seismic loading, however, excessive strength is not desirable for economical consideration. The seismic design includes the choice of consideration of design load, and force, analytical tech-nique, and design procedure, preference for particular structural configuration and materi-als, and aims for economically optimized design [1].

Following the Udayapur earthquake, 1988, the first attempt for development of the Na-tional Code for seismic design in Nepal begun in the Nineties as "Nepal National Build-ing Code for Seismic Design of Buildings in Nepal, NBC 105:1994"[2]. The first publi-cation was made in 1994, since then no revision and update have been made on this doc-ument. In the year between 1994 to 2019, development in research and technology, as well as knowledge, learned from the various large past earthquake and the most recent earthquake in the country, the Gorkha earthquake of 2015 April 25 and the following af-tershock, NBC

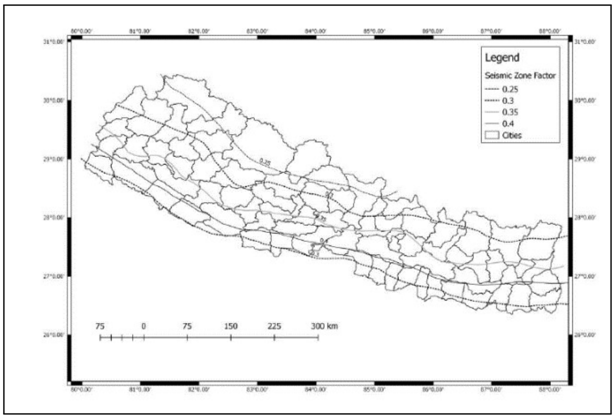



**Figure 1:** Seismic Zoning of Nepal[3]

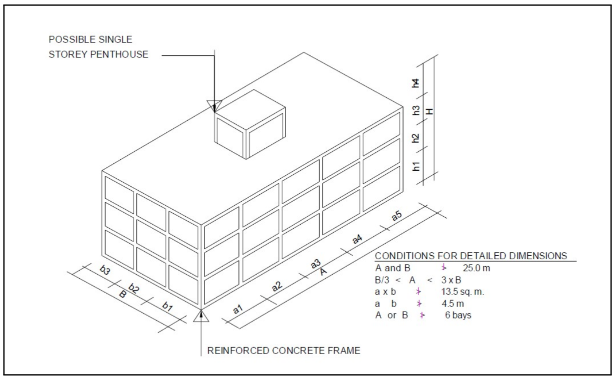



**Figure 2:** Restriction in structural layout[4]

105:1994 is in the process of the revision. This revised document of seis-mic code is “Nepal National Building Code for Seismic Design of Buildings in Nepal, NBC 105:2019” [3]. However, to this date, this document is not yet approved only the draft version is published. Seismic zone map given by NBC105:2019 is shown in Figure 1

The other document used in the country for reinforced concrete design of the building is “Nepal National Building Code Mandatory Rules of Thumb for Reinforced Concrete Buildings without Masonry Infill, NBC 205:1994”[4]. This standard is prepared to provide ready-to-use dimensions and details for various structural and non-structural elements for up to three-story RC framed ordinary residential buildings to achieve the minimum seismic safety requirements specified by NBC 105:1994[2]. However, NBC 105:1994 is in process of revision, so we can easily infer NBC 205:1994 will go revision shortly

## 2. Model Selection

Three regular building model were selected.The basic for selection of buildings models were, building category ”C” where plinth area should be less than one thousand square feet[5] and the structural layout restriction given in NBC205:1994 as shown in Figure 2.

## 3. METHODOLOGY

This study consist of the sequences of the task which were Linear analysis of selected models, evaluation of performance, design of buildings using capacity design procedure, Pushover analysis of selected models,and development of capacity curve.

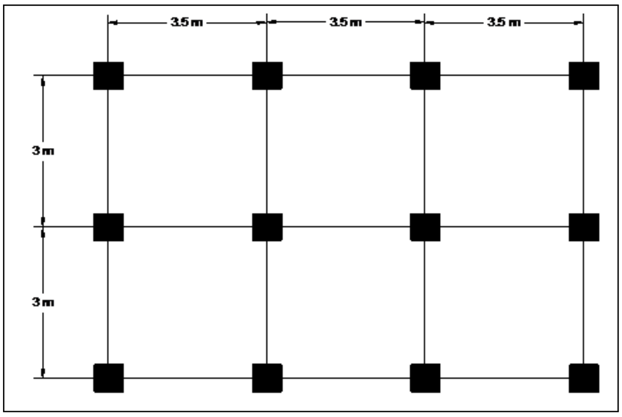
### 3.1 Linear analysis

Analysis and design of the building was done with the aid of finite element software SAP2000 V 21.2[6]. Beam and column were modeled using Frame element and Slab was modeled as shell elements[7]. IS 456:2000[8] was used for reinforced concrete design . General assumption considered in numerical model for analysis are listed below.

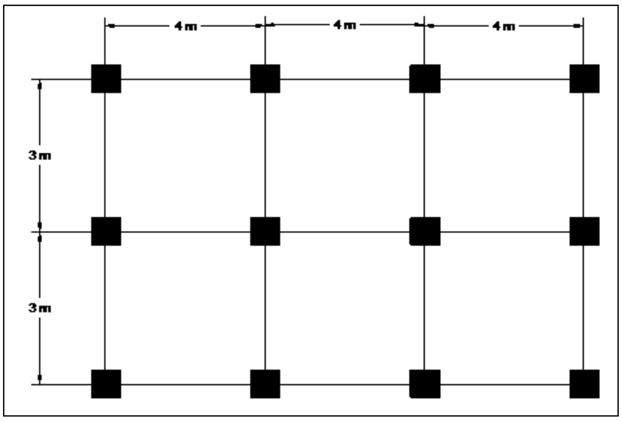
1. Foundation is assumed rigid, so soil-structure interaction is not considered.
2. Foundation is not modeled and restrain has been applied at plinth level.
3. Beam rest centrally on column to avoid local eccentricity.
4. Size of the column and beam are kept same throughout the building.
5. Diaphragm is assumed rigid.
6. Frame infill wall interaction and infill wall stiffness is not considered.

Building material, cross-section size, the grade of materials, and all loads in building other than earthquake loading are given below.

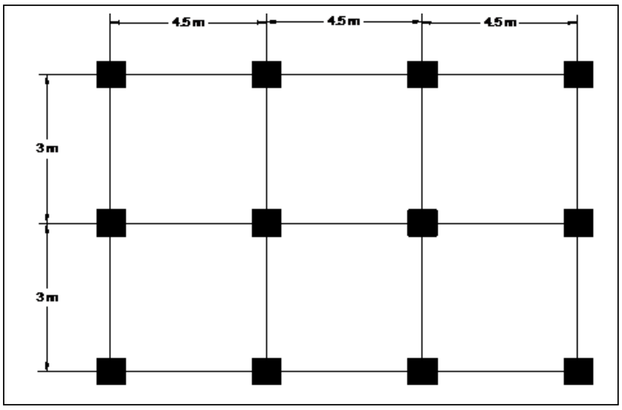
- Size of column: $(350 \times 350)$ mm
- Size of beam including slab depth: $(355 \times 250)$ mm
- Slab depth: 100 mm




**(a)** Model 1




**(b)** Model 2




**(c)** Model 3

**Figure 3:** Plan layout of building model

- Floor height: 3.35 m
- No of story : 3
- Grade of concrete: M20
- Grade of Steel: Fe500
- Imposed loads on the roof: $1.5KN/m^2$
- Imposed load on the floors: $2KN/m^2$
- Brick wall on peripheral wall: 230 mm
- Brick wall on inner wall: 130 mm
- Density of concrete:$25KN/m^3$
- Density of brick wall including plaster: $20KN/m^3$
- Parapet wall height of half brick wall thickness : 1 m

### 3.1.1 Seismic Load and Design load combination

Seismic load and design load combination were adopted accordance with NBC 105:2019[3]. Equivalent static method was used. Seismic zone four which gives higher zone factor coefficient of 0.4 as shown in Figure 1 was considered. Ductility factor of 4, and over strength factor of 1.5 for ultimate limit state, and over strength factor 1.25 for serviceability limit state was considered.The effective stiffness of cracked section as given in Table 1 was considered in analysis.

**Table 1:** Effective stiffness of different component [4]

| S.No | Component | Flexural Stiffness | Shear Stiffness |
|---|---|---|---|
| 1 | Beam | $0.35E_cI_g$ | $0.40E_cA_w$ |
| 2 | Columns | $0.70E_cI_g$ | $0.40E_cA_w$ |
| 3 | Wall-cracked | $0.50E_cI_g$ | $0.40E_cA_w$ |
| 4 | Wall-uncracked | $0.80E_cI_g$ | $0.40E_cA_w$ |

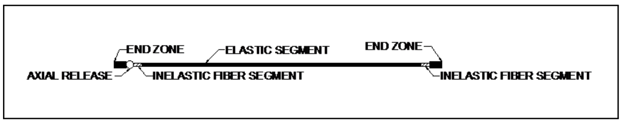
### 3.1.2 Story drift definition

Two type of drift definition were used in study, which is defined below.

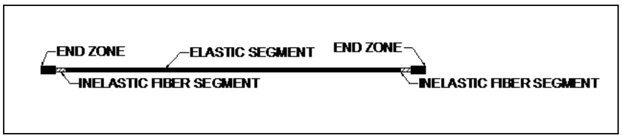
1. Inter story drift: This is the drift between two adjacent stories defined by the ratio of inter-story lateral displacement divided by story height.This is a local phenomenon that takes place be-tween two adjacent stories so it only reflects displacement response at the local floor level.
2. Global drift: This is the average drift of the structure defined by the ratio of roof displacement divided by total height of building. It reflects the global response of the structure.

### 3.2 Nonlinear static analysis

Pushover analysis, was done using a performance base design software, PERFORM 3D version 7[9]. Beam and column component were modeled as frame type element[10]. Typical frame element consist of rigid end zone at the both end, hinge element and inelastic fiber cross section after the rigid end zone at both end and elastic cross section at remaining portion. Beam and column element are shown in Figure 4. Inelastic response were modeled using inelastic fiber section. Modeling of beam element using inelastic fiber section on the structural system with rigid floor diaphragm axial compression force is developed in the beam section[10]. So, to prevent this effect axial release was made on beam compound component.




**(a)** Beam assembly




**(b)** Column assembly

**Figure 4:** Beam and Column element

Asumption made in numerical modeling in nonlinear analysis are listed below.

1. Expected failure mode is flexural failure. So other failure mode such as anchorage failure of longitudinal bars, lap-splice failure, shear failures, premature buckling of longitudinal bars were not considered.
2. Beam column joint is expected to remains essentially elastic.
3. Slab contribution was not considered in defining inelastic hinge properties of beam.
4. To account for shear deformation shear stiffness of 0.4 times Shear area was used[11].
5. Mander concrete model were used to model confined and unconfined concrete used in fiber inelastic segment[12]. However, in analytical model trilinear approximation as show in Figure 5 was used.

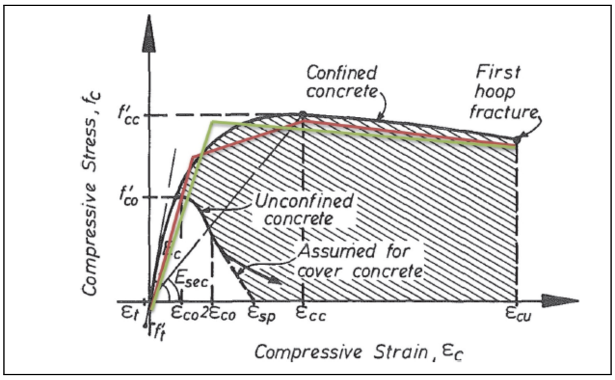



**Figure 5:** Stress-strain model for confined and unconfined concrete[12], with simplified bilinear and trilinear approximations[11].

6. In fiber inelastic segment, concrete inside the rectangular hoop (stirrup) was modeled as Mander confined concrete and concrete outside the rectangular hoop, cover concrete, was modeled as Mander unconfined concrete.
7. To capture the nonlinear response of beam and column element fiber elements as shown in Figure 6 was used.

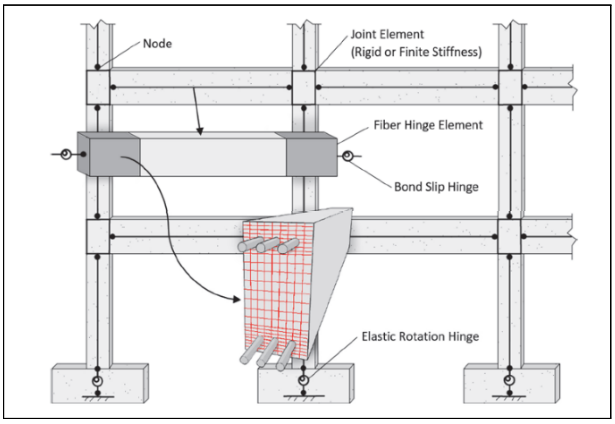



**Figure 6:** Overview of a typical reinforced-concrete moment-resisting frame system, showing the fiber-type model idealization[11].

8. For beam – column joint modeling the approach followed here was adjusting the stiffness of beam and column offset within the joint panel regions as shown in Figure 7[11].
9. Hinge length of 0.5 times Section depth was used[1]. This length is used a reasonable length for a fiber segment in reinforced concrete element[10].
10. Material property were taken from sap2000 material property database which shown in Figure 8.

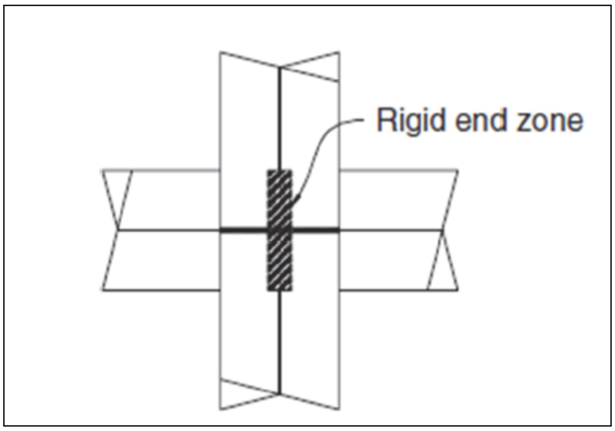
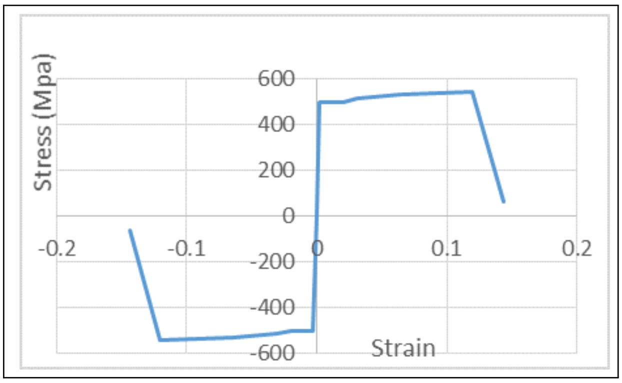
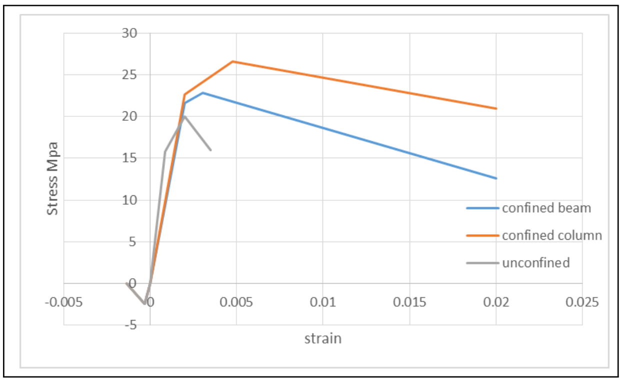



**Figure 7:** stiffness adjustment of beam and column offsets (Elwood et al., 2007)

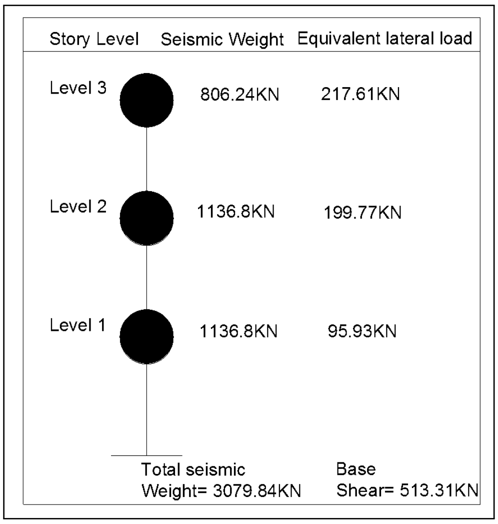



**(a)** Stress – strain (Fe 500)

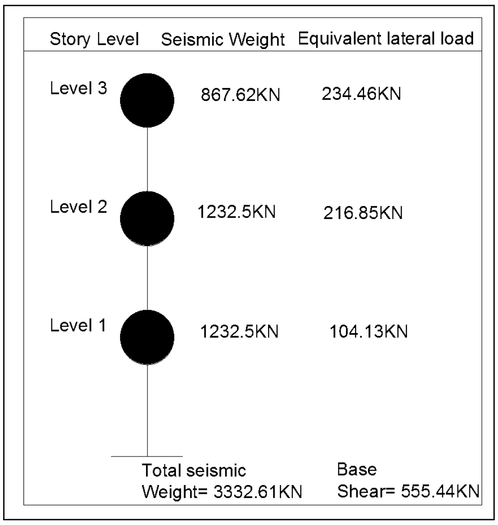



**(b)** Mander concrete model:Trilinear approximation(M20)

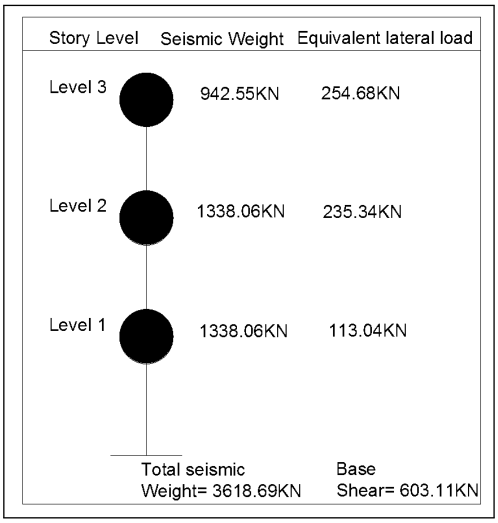
**Figure 8:** Nonlinear stress- strain diagram of rebar and concrete




**(a)** Model 1




**(b)** Model 2



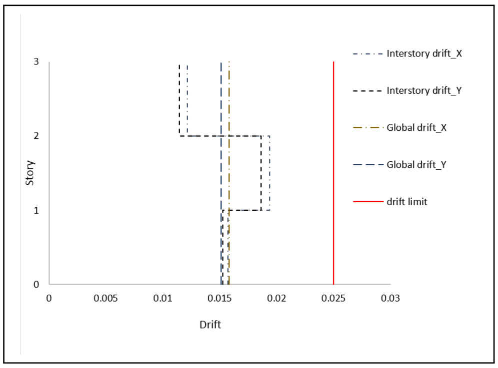

**(c)** Model 3

**Figure 9:** Seismic weight and equivalent lateral load summary

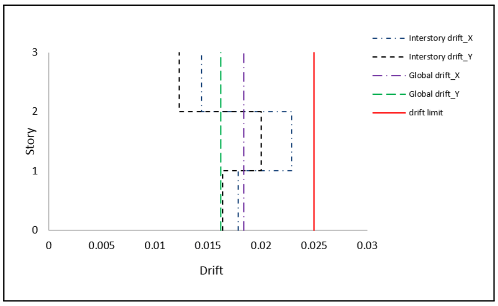
## 4. Result and Discussion

### 4.1 Linear analysis

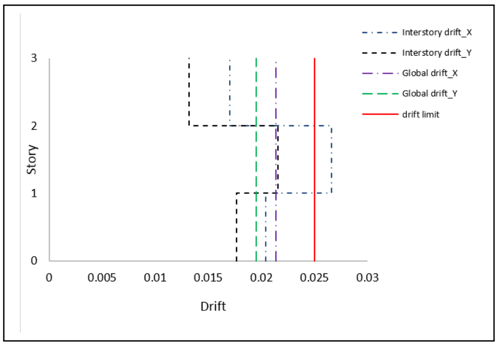
Seismic weight and equivalent seismic lateral load, Vibration periods, and drift value are presented below.




**(a)** Model 1

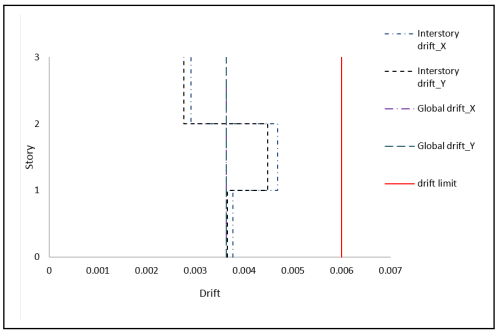



**(b)** Model 2

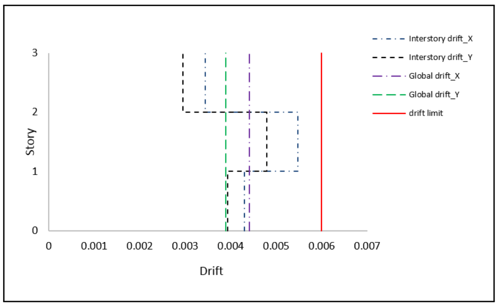



**(c)** Model 3

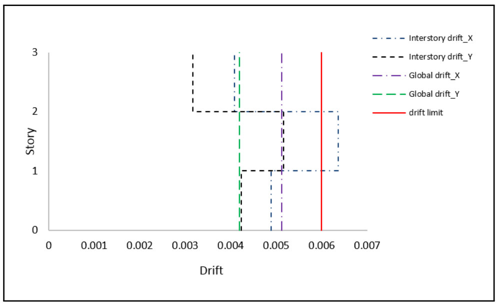
**Figure 10:** Ultimate limit state performance check




**(a)** Model 1




**(b)** Model 2




**(c)** Model 3

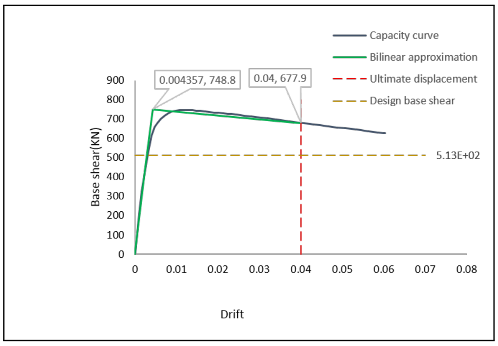
**Figure 11:** Serviceability limit state performance check

### 4.2 Discussion

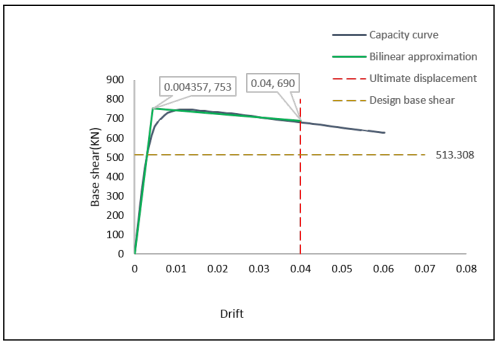
Model 1 and model 2 satisfied performance limit sate. However, model 3 did not satisfy both the performance limit state. The inter-story drift limit exceeded the limiting value for the ultimate limit state and serviceability limit sate lateral loading. In Model 1 and 2, the maximum span of the beam adopted was 3.5 m and 4 m respectively. However, in model 3 beam spanning of 4.5 m was used. From the above observation, a conclusion can be made than for beam spanning up to 4 m the provided size of column and beam satisfied the performance requirement stipulate in NBC 105:2019. However, for beam spanning up to 4.5m the provided size of the column fails to satisfy the performance requirement. The provided column size was not sufficient and it needs to be revised

### 4.3 Capacity curve

The capacity curve obtained from the pushover analysis was used for evaluation of over strength and ductility factor.The definition of ultimate displacement was taken as the maximum displacement with a ten percent reduction of the ultimate capacity or the displacement where the reinforcement bar fracture or buckles, or ultimate drift of 0.04 whichever occurs first. The yield strength of the buildings was taken as the mean of the two value of capacity after post yield.

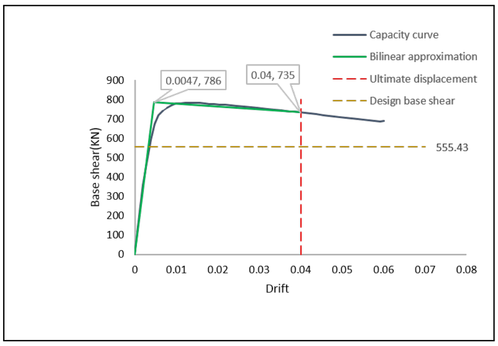



**(a)** X-direction

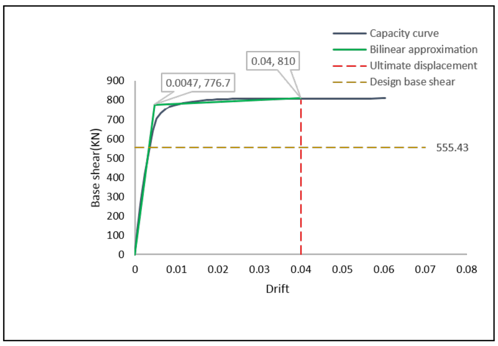



**(b)** Y-direction

**Figure 12:** Capacity curve model 1

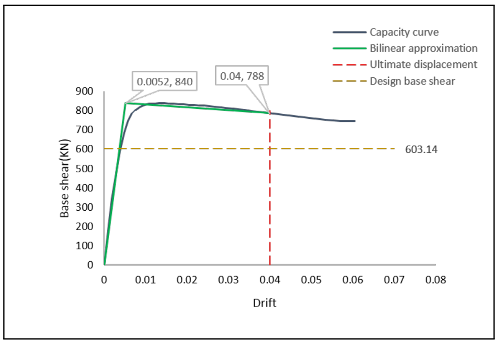



**(a)** X-direction

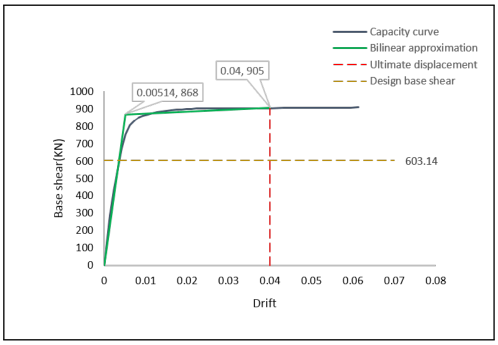



**(b)** Y-direction

**Figure 13:** Capacity curve model 2




**(a)** X-direction




**(b)** y-direction

**Figure 14:** Capacity curve model 3

## 5. Conclusion

After the interpretation and through observation of the results following conclusions were made.

1. NBC 105:2019 draft yields a higher value of base shear coefficient compared with NBC105:1994. In this study, for selected building models NBC 105:2019 generated the base shear coefficient of 0.167 for 475 years return period earthquake for site class B. If the same building were analyze using NBC105:1994 it would have generated base shear coefficient of 0.09 for 475 year return period earthquake.
2. For beam spanning up to 4.5m the provided size of the column fails to satisfy the performance requirement. The provided column size was not sufficient to prevent inetrstory drift limits.
3. The average ductility factor of 7.8 was obtained which is more than the minimum ductility requirement of NBC 105:2019 which is 4, while the average overstrength factor of 1.33 was obtained which is less than that NBC 105:2019 considered which is 1.5

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