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# Comparative Study of Seismic Design and Performance of OMRF Building Using Indian, British, and European Codes

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**Abstract:** In India, damage cause by some major earthquakes, such as India/Nepal 2015, Sikkim 2011, Kashmir 2005, Bhuj 2001, Latur 1993, and Uttarkashi 1991, have raised alarms to professionals. The probability of seismic risk is higher in more densely populated Indian cities, such as Bhuj, Kashmir, Sikkim, Uttarkashi, as they come under the highest seismicity zone in India. Therefore, our primary interest is to investigate the seismic performance evaluation of the buildings in these seismic prone areas. Significant research has been conducted on the seismic performance of existing buildings. However, investigations on the seismic performance of a building with different country codes for the same earthquake event has not been explored, which is crucial in providing a deeper knowledge of the seismic performance of buildings. This paper presents a comparative study of an Ordinary Moment Resistant Frame (OMRF) building designed using three major codes, Indian (IS: 456-2000, IS: 1893-2002), British (BS: 8110-1997) and European (EC-2, EC-8). Six typical building models considered with earthquake (WiEQ), and without earthquake (WoEQ), and their assessments were interpreted using non-linear static analysis for determining their seismic performance. Seismic performance is compared in terms of base shear coefficient (BSC) and drift ratio that shows WiEQ models, at the drift ratio of 1.5%, the BSC was as follows; 0.78, 0.88, and 0.96 for the models designed for British, Euro, and Indian codes, respectively. The results show that the building models, that have been designed for the Indian codal provisions for both cases, performed well as compared to the other country codes. Base shear and drift ratio are the vital parameters that vary considerably among the building models. This aspect of the Indian code makes it a safer design methodology with higher reserve strength and a reasonably good displacement capacity before reaching the Collapse Prevention (CP) performance level.

**Keywords:** OMRF building; pushover analysis; base shear coefficient; drift ratio

## 1. Introduction

### 1.1. General Overview

Buildings protect humans against extreme natural events, e.g., climate and weather. However, once a natural disaster, such as an earthquake occurs, these buildings can cause catastrophic damage in terms of human and economic loss, if not appropriately designed. India has experienced some of the great seismic events such as India/Nepal 2015, Sikkim 2011, Kashmir 2005, Bhuj 2001, Latur 1993, and Uttarkashi 1991. As noted in Jain et al. [1] and Kumar [2], the first significant work on earthquake-resistant construction emerged after the 1931 Baluchistan (now in Pakistan) earthquakes. Over the last 25 years, more than 25,000 people were killed due to building collapses from earthquakes in India [3]. In India, most of the buildings, which were designed prior to 2001 (the year of the Bhuj earthquake) were designed only for dead load and live load combinations, without any earthquake loading (WoEQ), making them vulnerable to collapse in the event of an earthquake [2,3]. As a result, on 26 January 2001, the Bhuj earthquake claimed the lives of 13,800 persons due to building collapses; about 130 multi-storey buildings collapsed in Ahmedabad alone, at a distance of 200 km from the epicenter [1]. This was the largest number of multi-storey building collapses in history of India [1,4]. Are practising engineers and designers following the building code? Are building and regulating authorities adhering to the standards? The codal provisions in India, IS -456 [5] and IS 1893-part 1 [6] were examined, and many questions arose concerning the safety aspect of building codes.

### *1.2. A Brief on Buildings in India*

In India, different types of buildings are constructed using various types of material in both urban and rural areas. In building construction, either from rural or urban areas, locally available materials, coupled with a lack of engineering input and skilled labour makes the building more vulnerable to lateral forces [7]. Based on the vulnerability of different types of buildings, a detailed catalogue was prepared by the National Disaster Management Authority (*NMDA*), an agency established by the Ministry of Home Affairs [8]. Many researchers performed seismic vulnerability studies on such building types and modelled a damage scenario for several areas [9–11]. In India, 62% of buildings have an reinforced cement concrete (RCC) framed structure, rather than a masonry load-bearing structure [12–14]. At the time of the Bhuj earthquake, most of the buildings had reinforced frames with unreinforced masonry infill, which do not provide additional stiffness or strength to the buildings. Humar et al. [15] and Ghosh [16] stated that most of the multi-storey residential buildings in the metropolitan cities of India have stilt-level parking for automobiles. Hence, there was a complete absence of an infill wall, which result in a critical change in a building's stiffness [15,16].

### *1.3. Past Studies and Codes*

Many studies have been conducted on the seismic performance of old masonry [17–22], RC buildings [23–28], and a combined RC-masonry building [29]. Bayraktar et al. [30] conducted an extensive study on 90 RC buildings to investigate their performance during the 2011 Van earthquakes in Turkey. Itti et al. [31] presented a comparative study on the seismic provisions of Indian and international building codes (IBC) [32] for RC buildings concerning the lateral deflection of the buildings for special moment-resisting frame of IBC, i.e., implementing the concept of lesser force and more deflection. However, they did not discuss issues related to reinforcement and its performance in relation to seismic events. Climent and Zahran [33] studied the seismic performance of RCC building frames with a wide beam in a seismic-prone Mediterranean area before the introduction of modern codes. However, they did not compare the results with buildings that have been designed with following present building codes; also, they considered the effect of the wide beam. Chaulagain et al. [34] conducted a study in Nepal on four different design practices and analyzed them with a non-linear static and dynamic approach. Li et al. [35] studied the timber-steel hybrid system and its performance in a seismic environment. Ali et al. [36] estimated the seismic performance of masonry buildings in the form of base shear coefficient (BSC) as it can be conveniently compared without giving specific importance to input excitation and peak ground acceleration (PGA). Kaur and Singh [37] presented a review article on earthquake analysis of buildings using various codal provisions from India, American, Europe, and New Zealand. In this comparative study, it was found that the European code stipulates a more conservative design than other countries.

The current research aims to study the comparison of the codal provisions among the different major country codes, i.e., British, European, and Indian codes for the earthquake analysis.

Ramanjaneyulu et al. [38] reported the various codal provisions on the design and detailing of European and Indian code. Moreover, Ng et al. [39] summarized the comparison between the Euro code 2 and BS 8110 codes and found that both codes have some different codal provisions, but the designer may come at the same design following either code.

### 1.4. Problem Statement

The seismic codes vary in specific regions or countries, and each code has different control parameters, which are used to enhance the seismic performance of a building. Sufficient research has been done on the seismic performance of existing buildings. However, there is still inadequate information on the performance of a building. If a building is designed for a specific seismic region using different codal provisions, the seismic performance may significantly vary. To the best of the author's knowledge, so far no one has compared the performance evaluation of Ordinary Moment Resistant Frame (OMRF) buildings designed with, and without, seismic loading among different seismic codes. Therefore, it is crucial to conduct comparative research to investigate essential parameters that can be used for next generation of seismic design codes.

### 1.5. Significance and Methodology

In the present study, an attempt has been made to conduct the competitive study of the OMRF buildings for without earthquake (WoEQ) and with earthquake (WiEQ) models with different country codes by employing pushover analysis in terms of BSC versus Drift ratio. Six G+3 storied OMRF buildings are designed for both the WoEQ and WiEQ condition in accordance with three different building codes viz., Indian, British, and European for the Indian seismic environment of Zone V. A comparison of steel is made for all the models, such as those designed without earthquake loading, Indian ($M_{IC}$); British ($M_{BC}$); Euro code ($M_{EC}$) and designed with earthquake loading, Indian ($M_{ICE}$); British ($M_{BCE}$); Euro code ($M_{ECE}$). Further, static pushover analysis is performed, and the performance of the models is compared in terms of base shear coefficient (BSC) and drift ratio (DR) for both the WoEQ and WiEQ condition.

## 2. Model and Design

A 3D model of an ordinary moment-resisting RCC (G+3) storey framed building is made with, and without, earthquake loading. The models were designed for India's highest Seismic Zone-V, with a medium soil profile and a base dimension of 8.44 m × 9.66 m (27′8″ × 31′8″). The total height of the building is 14 m (45′11″) with a 3 m (9′10″) floor height. Figure 1 and Table 1 show the plan, and preliminary data, respectively for the designed model. The stress-strain relationship of the concrete used is per IS 456 [5]. Table 2 shows the recommended values for partial safety $(\gamma_m)$ material as per BS 8110 [40], EC 2 [41] and IS 456 [5]. In the current study, six model of three multi-storey (i.e., G+3) rigid joint space frame buildings are considered. The cross-sectional dimensions of the structural members are as follows: Beams are two sizes viz., 350 × 450 mm (1′1.8″ × 1′5.8″) and 350 × 500 mm (1′1.8″ × 1′7.7″), columns are 300 × 300 mm and slab depth is 120 mm (4.7″). The details of the cross-sectional elevation of the building are shown in Figure 2a,b. The clear cover of the beam and column are 25 mm, and 40 mm respectively.

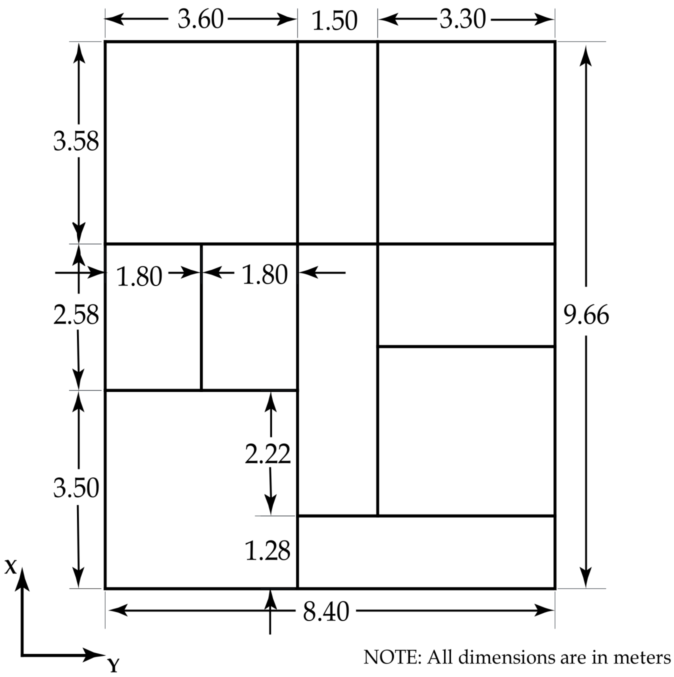


**Figure 1.** Typical building plan.

**Table 1.** Preliminary data for G+3 space frame.

| | | |
|---|---|---|
| 1) | Type of Structure | Multi-storey rigid jointed space frame |
| 2) | Seismic Zone of India | V |
| 3) | Number of stories | (G+3) |
| 4) | Imposed load including Floor Finishes | 2.5 kN/m$^2$ (52.25 lbf/ft$^2$) |
| 5) | Outer Brick wall load (due to 230 mm) | 13 kN/m (891 lb/ft) |
| 6) | Inner Brick wall/Partition load (due to 110 mm) | 6 kN/m (411 lb/ft) |
| 7) | Depth of slab | 120 mm (4.7″) |
| 8) | Materials | M25 concrete and Fe415 steel |
| 9) | Modulus of elasticity of concrete | $2.10 \times 10^5$ N/mm$^2$ ($4.40 \times 10^9$ lb/ft$^2$) |

**Table 2.** Material Partial Factors of Safety ($\gamma_m$) for all three codes.

| Code | Concrete in Flexure or Axial Load | Concrete in Shear | Concrete in Bond | Reinforcement Steel |
|---|---|---|---|---|
| BS 8110-1997 [40] | 1.50 | 1.25 | 1.40 | 1.15 |
| EN 1992-2004 [42] | 1.50 | 1.50 | 1.50 | 1.15 |
| IS 456-2000 [5] | 1.50 | 1.50 | 1.50 | 1.15 |

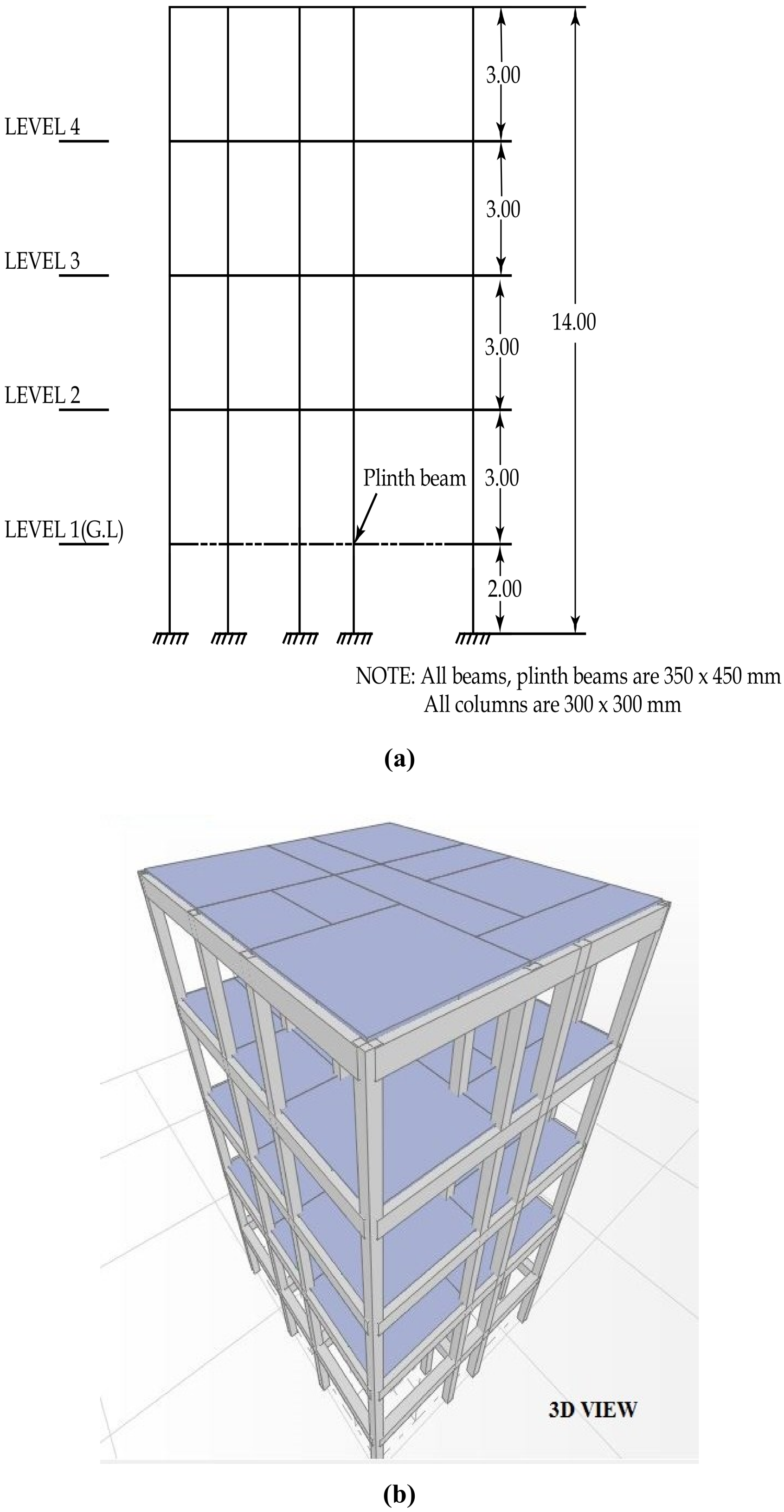


**Figure 2.** (**a**) Cross-sectional elevation of building with beams and column size, (**b**) SAP2000 three dimensional (3D) model.

### *2.1. Loading*

The loads on the building that were considered are as follows: Imposed load, including floor finishes, is 2.5 kN/m$^2$, the outer brick wall load (due to 230 mm) is 13 kN/m, and the inner brick wall load (due to 110 mm) is 6 kN/m Table 1. This has only given us an insight into the design adopted by various codes. The following load combinations are considered as per the clause 6.3.1.2 mentioned in IS 1893-Part I [6]:

**a.** 1.5 (Dead Load + Imposed Load)
**b.** 1.2 (Dead Load + Imposed Load ± Earthquake Load)
**c.** 1.5 (Dead Load ± Earthquake Load)
**d.** 0.9 Dead Load ± 1.5 Earthquake Load

Table 3 illustrates the values of partial factors of safety for the loadings, and a necessary load combination stipulated by the three codes. From Table 3, it can be noted that there is a slight difference in the dead load partial safety factor whereas in the case of live load, EC2 [43,44] and IS 456 [5] have the same partial safety factor of 1.5, but BS 8110 [40] has 1.6. Table 4 shows the design parameter for earthquake loading IS 1893 [6] considered in the study.

**Table 3.** Basic load combinations and partial safety factors $(\gamma_f)$ at the ultimate limit state (adapted from [36]).

| Code | Dead Load (DL) | Live Load (LL) |
|---|---|---|
| BS 8110-1997 [40] | 1.40 | 1.60 |
| EN 1992-2004 [42] | 1.35 | 1.50 |
| IS 456-2000 [5] | 1.50 | 1.50 |

**Table 4.** Parameter used for earthquake loading.

| Particulars | Values |
|---|---|
| 1. Damping Ratio (concrete) | 5% |
| 2. Seismic coefficients $(A_h) = \left(\frac{Z}{2}\right)\left(\frac{I}{R}\right)\left(\frac{S_a}{g}\right) Z = 0.36, I = 1, R = 3$ (OMRF), $\left(\frac{S_a}{g}\right) = 2.5$<br>$T = 0.09\text{h}/\sqrt{\text{d}}$ $T_x = 0.09(14/\sqrt{8.44}) = 0.4337$ sec<br>$T_z = 0.09(14/\sqrt{9.66}) = 0.4054$ sec | 0.15 |

### *2.2. Reinforcement*

The minimum or maximum amount of percentage of steel reinforcement should not violate the limits stipulated by the codes. Table 5 gives the minimum and maximum percentage of steel reinforcement specified by BS 8110 [40], EC2 [41] and IS 456 [5].

**Table 5.** Minimum and maximum percentage of steel reinforcement.

| Members | BS 8110-1997 [40] | | EC 2 [41] | | IS 456-2000 [5] | |
|---|---|---|---|---|---|---|
| | Min. % | Max. % | Min. % | Max. % | Min. % | Max. % |
| Beam | 0.13 | 4.00 | 0.30 | 4.00 | - | 4.00 |
| Column | 0.40 | 6.00 | 0.30 | 8.00 | 0.80 | 4.00 |

### *2.3. Methodology*

Static non-linear pushover analysis was performed on the six models to determine their respective performance, and the base shear coefficient (BSC) of all the models was compared. To identify the performance of the buildings, in 1982, the Applied Technology Council (ATC) took the first comprehensive initiative to evaluate the performance of existing buildings. Further, the Federal Emergency Management Agency (FEMA) released a handbook on the seismic evaluation of existing buildings in FEMA-178 [45] after the ATC report. In 1998, the first standard guideline for analysis, the American Society of Civil Engineers (ASCE) converted FEMA 178 [45] into FEMA 310 [46] as an approved national consensus standard document. The guideline suggests that buildings must be evaluated in three phases, i.e., a screening phase, evaluation phase, and a detailed evaluation phase. The procedure used in the evaluation of a building is displacement-based instead of force-based as proposed in ATC-14 [47]. Now FEMA 178 introduces a displacement-based procedure, which is consistent with the procedures outlined in FEMA 273 [48,49].

In seismic design or the assessment of buildings, modern codes, including the British Code (BS 8110-1997), Euro Code-8 (EC 8), and Indian Code (IS 1893-2002), consider four main methods of structural analysis: Linear static (or simplified modal), linear dynamic (typically multimodal with response spectrum), non-linear static (pushover analysis), and non-linear dynamic.

It has the advantage of providing information on many response characteristics that cannot be obtained from an elastic static or elastic dynamic analysis. Non-linear static analyses is recognized as a practical tool to evaluate the seismic behavior of structures. Tumsek et al. [50] suggested the use of static analysis, a simplified non-linear method for the seismic assessment of buildings for its retrofitting and strengthening purpose in Slovenia. This method was further refined in the subsequent year [51], based on the storey-mechanism approach.

Pushover analysis is an approximate analysis method in which the structure is subjected to systematic increasing lateral forces with a vertical distribution until a target displacement is reached. Pushover analysis is a process of systematic elastic analysis, superimposed to plot a force-displacement curve of the overall structure. A two-dimensional (2D) or 3D model that includes load-deformation diagrams of all the elements plotted and gravity loads are applied. A known load distribution pattern is applied as discussed in the aforementioned load combinations. The load increment process continues until the elements yield. The elements are modified as per stiffness, and then loading is applied again so that the additional members will yield. The analysis stops when control node displacement reaches the target displacement, or the structure becomes unstable due to the formation of the mechanisms. Here, the mid-top portion of the building is taken as the control node and the target displacement is predicted to 0.3 m. The top floor displacement is plotted with respect to the base shear to get the pushover curve of the structure.

The various levels of a building's performance can be obtained from discrete damage states identified from a continuous spectrum of possible damage states *viz., O-Operational, IO- Immediate Occupancy, LS- Life Safety and CP- Collapse Prevention*. The structural performance levels based on the roof drifts are the same as those in FEMA 356 [52]. For the reader's convenience, this has been reproduced and is shown in Table 6.

**Table 6.** Performance levels (adapted from [53]).

| Performance Levels | Structural Performance | Non-Structural Performance |
|---|---|---|
| Operational (*O*) | Very light damage No permanent drift substantially original strength and stiffness. | Negligible damage, Power & other utilities are available. |
| Immediate Occupancy (*IO*) | No permanent drift, Substantially original & stiffness, Minor cracking, Elevators can be restarted, Fire protection operable. | Equipment & content secure but may not operate due to mechanical/utility failure. |
| Life Safety (*LS*) | Moderate damage, Some permanent drift, Residual strength & stiffness in all stories, Gravity elements function, Building may be beyond economical repair. | Falling hazard mitigated but extensive systems damage. |
| Collapse Prevention (*CP*) | Severe damage, Large permanent drift, little residual strength & stiffness, gravity elements function, some exits blocked, Building near collapse. | Extensive damage. |

### 2.4. Properties of Hinges

This study has been undertaken using a P-M2-M3 hinge, which means that the axial force is in one direction and the biaxial moment is in two mutually perpendicular directions to that of the axial force direction. FEMA-356 [52] or ACI 318-02 [54] ($\Phi$=1) is used to define the three-dimensional yield surface of the P-M2-M3 hinges for concrete. The important point to remember is that design forces (Pu, M2, and M3) must lie within the interaction surface. The M2-M3 interaction diagram is drawn for a constant axial load (design force). In the present work, the maximum monitored displacement magnitude of 0.3 m (control displacement) has been adopted at the center of the top storey.

## 3. Results and Discussion

The amount of steel reinforcement obtained from the design guidelines for all the six models is detailed in Table 7. It is observed that the model designed for WoEQ detailing with Indian code ($M_{IC}$) provides 40.6% and 35.1% more steel than the British ($M_{BC}$), and Euro Code ($M_{EC}$), respectively. Similarly, WiEQ detailed model provides 65.5% and 43.5% more steel than the British ($M_{BCE}$) and Euro code ($M_{ECE}$). The overall reinforcement for the WoEQ model designed in accordance with the British code contains less steel, and the Indian code has the highest amount of steel of all the codes.

**Table 7.** Comparison of design reinforcements.

| Type of Model | BS 8110-1997 [40] | | EC 2, EC 8 [41,44] | | IS 456 [5], IS 1893 [6], IS 13920 [55] | |
|---|---|---|---|---|---|---|
| | $M_{BC}$ | $M_{BCE}$ | $M_{EC}$ | $M_{ECE}$ | $M_{IC}$ | $M_{ICE}$ |
| Steel (kg) | 28915 | 33269 | 30073 | 38378 | 40658 | 55053 |
| % Change | 15.10% | | 27.60% | | 35.40% | |

All these structures are safe for their load combinations, and the cross-sectional dimensions of the structural elements (i.e., beam and column) are considered the same in all the models to make them comparable. Further, the comprehensive explanation of obtained results from the non-linear static analysis is described below.

The pushover curve for WoEQ is shown in Figure 3. At a displacement of 0.07 m, the Indian code reaches a collapse prevention point (*CP*) around a base shear of 767 kN. Whereas, the British and Euro code shows a corresponding base shear of 400 kN, and 200 kN, respectively. It is observed that all the models fail suddenly after this point. It can be seen from the plot that at a very small level displacement, all three models follow the same path of the curve, i.e., 197 kN. The base shear of $M_{IC}$ became almost constant with slight increases for the displacement of 0.065 m and then collapsed. The immediate occupancy (*IO*) level and life safety (*LS*) level are from 0.02, to 0.05 m, respectively for an almost constant base shear of ~197 kN.

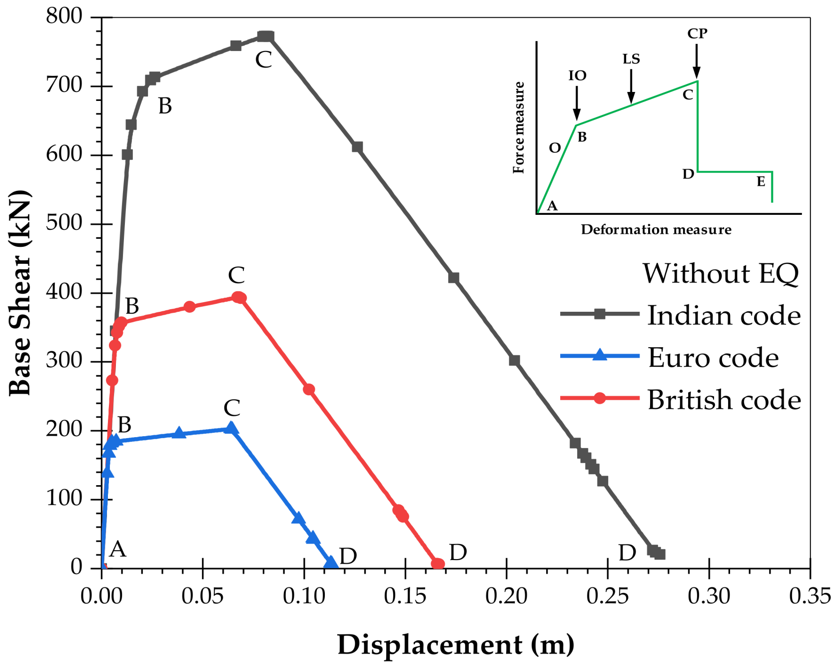


**Figure 3.** Pushover curve of models without earthquake loading.

Further, the path followed by $M_{BC}$ and $M_{IC}$ continued and $M_{BC}$ starts deviating at the base shear level of 350 kN with 0.025 m displacement. There is very less increment in base shear (~50kN) in $M_{BC}$ over a displacement level of 0.025 m to 0.06 m. Base shear is almost double that of $M_{EC}$ at *CP* for $M_{BC}$. The $M_{IC}$ shows a significant increase in base shear on the same path of the curve, going up to 675 KN of base shear for 0.035 m of displacement and then it increases slowly to the level of 767 KN for a strain level of 0.07 m and then collapses. It shows that the resistance offered by $M_{IC}$ is much higher than the other two codes viz., $M_{BC}$ and $M_{EC}$. The sudden failure of all these models is due to the fact that there is no provision for ductility reinforcement in their design. It can also be seen from Table 7 that more steel is provided in $M_{IC}$ than $M_{BC}$ and $M_{EC}$, hence base shear capacity of $M_{IC}$ is on the higher side. In all the cases, the *IO* and *LS* level ranges from ~0.025m to ~0.06m of the displacement level.

Figure 4 shows the pushover curve for the model design with earthquake loading viz., $M_{BCE,}$ $M_{ECE,}$ and $M_{ICE}$, where the curve shows a significant increase in the base shear for the earthquake designed models. The curve shows that all three models followed the same path up to the base shear level of 1250 KN for the displacement of 0.03m, which is six times the base shear achieved in the WoEQ models. At this level, $M_{BCE}$ starts deviating from the curve path and with a very slight change in the base shear level, it passes through *IO* (0.05 m) *LS* (0.075 m) and then *CP* (0.250 m) and still shows a significant displacement of 0.3 m. Further, $M_{ECE}$ and $M_{ICE}$ continue on the same curve up to the base shear level of 1800 KN and here $M_{ECE,}$ differs from the path with a very low increase in base shear of 300 KN the displacement level of 0.25 m. Once the base shear crosses the *LS* point, there is a very slight increase in base shear whereas displacement is continuous up to *CP*. In $M_{ICE,}$ the base shear level at *CP* is 2300 KN with a strain level of 0.23 m. All the codal provisions ensure safety in terms of providing additional reinforcement for ductility. It can be seen from both the analyses that there is ~200 % increase in base shear whereas the level increases for *CP* up to 228 % in the case where a building is designed with an earthquake provision. The plot shows that the India code performed better than the other codes. Although, the design and detailing is expensive. Further, if we comprise the base shear level by 10%, then the European code saves 43 % of steel. Table 8 shows the load and displacement capacity of a building designed using various codes of practices viz., Indian, British, and Euro.

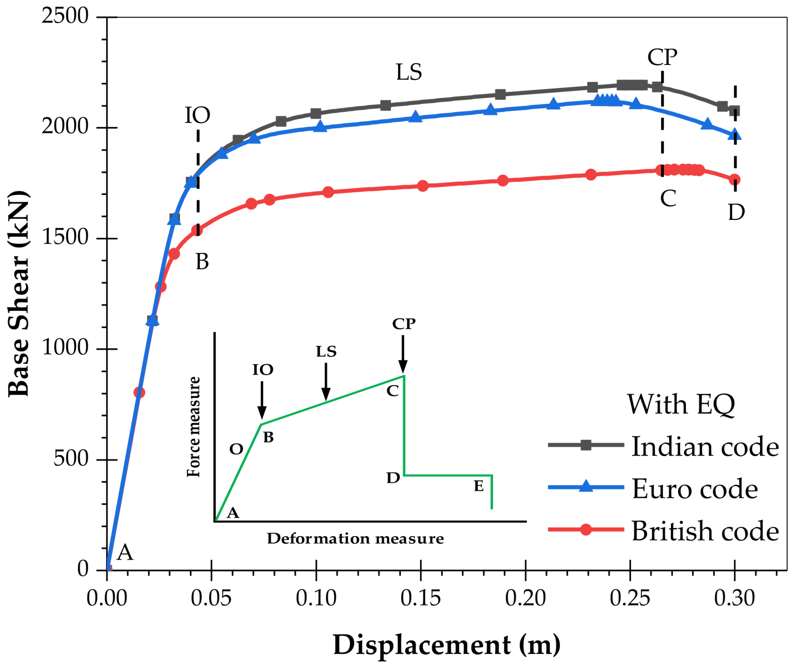


**Figure 4.** Pushover curves for models with earthquake loading.

**Table 8.** (a) Table of ultimate load along with displacement.

| Sl. No. | Code | Loading Condition | Ultimate Load (kN) | Displacement (m) |
|---|---|---|---|---|
| 01 | Indian | WoEQ | 767.54 | 0.08 |
| | | WiEQ | 2118.65 | 0.25 |
| 02 | British | WoEQ | 393.73 | 0.07 |
| | | WiEQ | 1810.70 | 0.27 |
| 03 | Euro | WoEQ | 202.75 | 0.06 |
| | | WiEQ | 1716.68 | 0.21 |

From Table 8 and the pushover curves of the building for WoEQ Figure 3 and WiEQ Figure 4 loading conditions, it can be said that a building designed in accordance with the Indian code has a reasonably better load, as well as a displacement capacity, before reaching the *CP* level. In the case of WoEQ loading, there is an increase in the load capacity of 95% and 279% over the British, and Euro codes, respectively. Whereas, in the case of WiEQ loading, the corresponding increases are about 17%, and 23.42%, respectively. It is observed that for a large displacement capacity without strength and stiffness degradation, the Indian code gives a 19% and 26% increase in displacement capacity, compared to the British, and Euro code, respectively for WoEQ loading. Whereas, for WiEQ loading, the increase is about 11.68% with reference to the Euro code and is almost comparable w.r.t the British code. These characteristics of the Indian code make it a safer design methodology with higher reserve strength and a reasonably good displacement capacity before reaching the collapse prevention (*CP*) performance level. The model is compared in terms of base shear and drift ratio. The base shear coefficient (BSC) is the ratio of base shear to the total weight of the model. The drift ratio (%) is the ratio of the top lateral displacement to the height of the model. The allowable limit of the drift ratio in the British code, European code, and Indian Code is 1.5 %, 1.5%, and 1.2 % respectively.

Figure 5 shows the behavior of the building models designed for WoEQ loading. It can be seen that the frame behaved in a linearly elastic manner up to a BSC value of 0.18, 0.10, and 0.33 for the British, Euro, and Indian codes of design. At the BSC value of 0.20, 0.12, and 0.38, there was a sudden drop in the pushover curves of the British, Euro, and Indian models, which indicates the failure of the building, since the model is designed for WoEQ loading, and there is no incorporation of ductility reinforcement in the structural members, which leads to its sudden collapse. Even though all the models failed due to a lack of ductility reinforcement, the model, designed in accordance with the Indian code of practice, resisted the maximum base shear compared to the other codes. It shows that the performance of the $M_{IC}$ structure of WoEQ is better due to the higher safety factor in the loads, material provided in the building code.

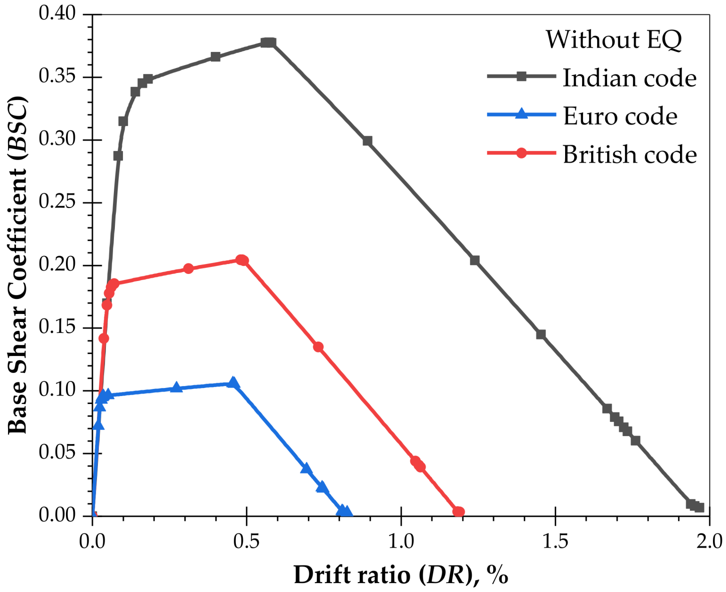


**Figure 5.** Base Shear Coefficient versus Drift Ratio for without earthquake (WoEQ) model.

Figure 6 shows the building models designed for WiEQ loading. It can be seen that the frame behaved in a linearly elastic manner up to a BSC value of 0.62, 0.78, and 0.82 for the British, Euro, and Indian codes of design. For a drift ratio value of 0.5 to 1.65, the structures maintained constant BSC values and resisted for a long time, demonstrating non-linearity behavior. The elasto-plastic behavior of the structure was observed with the increase in drift ratio and an increment of BSC from 0.5 to 1.65, which is due to the presence of ductility reinforcement in the building, and the $M_{ICE}$ structure performed well compared to $M_{ECE}$ and $M_{BCE}$. The results show that the Indian standard is very conservative in its approach compared to the other two codes.

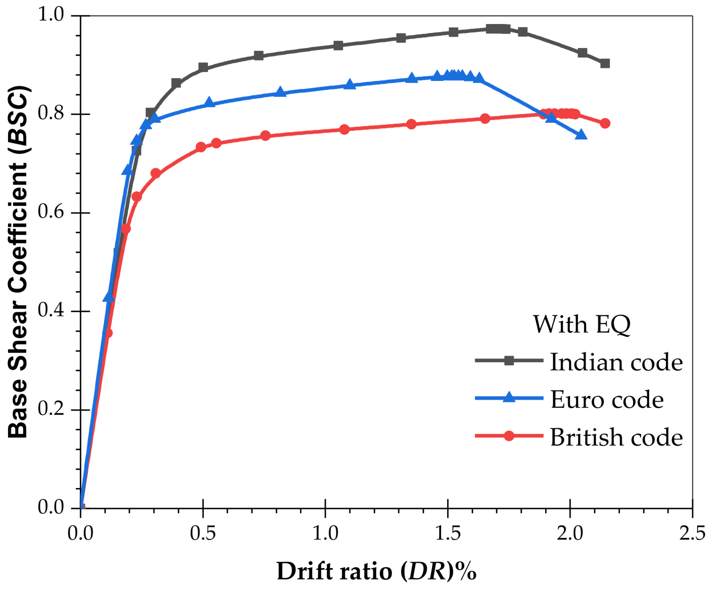


**Figure 6.** Base Shear Coefficient versus Drift Ratio for with earthquake (WiEQ) model.

## 4. Conclusions

The present study compared the performance of six buildings designed using three codal provisions namely, Indian, British, and European. Six four-storied typical ordinary moment resistant frame buildings were designed with, and without, earthquake loading conditions. The following conclusions can be drawn:

- For the WoEQ loading condition, the amount of steel required to comply with the Indian code is 40.6% and 35.1% more than the British, and Euro code, respectively. Further, for buildings designed with WiEQ, the amount for steel required to comply with the Indian code is 66.5% and 43.5% more than the British code, and European code, respectively. This may be due to various safety factors applied on materials, ductility provisions, and the minimum criteria of reinforcement in the various codes.
- The pushover analysis results show that buildings designed in accordance with the Indian code perform significantly better in a seismic environment compared with the British and European codes. In the case of WoEQ loading, there is an increase in the load capacity of 95% and 279%, and for WiEQ loading, the corresponding increases are about 17% and 23.42% over the British code, and Euro code, respectively. Further, for WoEQ loading case, the Indian code gives a 19% and 26% increase in displacement capacity compared to the British code, and Euro code, respectively. Whereas, for WiEQ loading, the increase is about 11.68% w.r.t the Euro code and is almost comparable w.r.t the British code.
- It is also observed that for a large displacement capacity without strength and stiffness degradation, the Indian code provides a 19% and 26% increase in displacement capacity compared to the British code, and Euro code, respectively for WoEQ loading.
- The study also concludes that, for the same level of hazard at different places on earth, one should have a uniform design and detailing provisions.

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