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Seismic performance of reinforce concrete buildings designed according to codes in Bangladesh, India and U.S.

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ABSTRACT

This paper focuses on the comparison of seismic design provisions in Bangladesh (BNBC-1993), India (IS-1893), and the U.S. (ASCE 7-10) in relation to analysis, design, and seismic performance of reinforced concrete buildings on the basis of the type of allowable analysis procedures, zoning system, site classification, fundamental vibration period of the structure, response reduction factor, importance factor, minimum design lateral force, allowable story drifts, and design response spectra.

Three geometrically similar commercial reinforced concrete buildings in high seismic regions of Bangladesh, India, and U.S. were designed and detailed per the respective codes. Three-dimensional nonlinear dynamic analyses of the designed structures were conducted. Each structure was subjected to a pair of orthogonally applied artificial ground motions compatible with the design response spectrum for each building code. The structural performance of each building was compared in terms of roof displacements, inter-story drifts, load-carrying capacity of beams and columns, and overall energy dissipation characteristics. The comparisons allowed an in-depth evaluation of the differences in the seismic performance of buildings designed according to ASCE 7-10, BNBC-1993, and IS-1893 codes. The Indian code performed better when subjected to the ground motion that is intended to represent the Indian design response spectrum.

1. Introduction

Major earthquakes have been recorded in Bangladesh, India, and the U.S. Bangladesh has experienced seven major earthquakes of magnitude over 7.0 during the last two hundred and fifty years, e.g., Bengal Earthquake of 1885 and Srimongol Earthquake of 1918. The Bhuj earthquake (M 7.7) of 2001 in India resulted in the loss of nearly 20,000 lives and severe damage to 339,000 houses. The 1989 Loma Prieta and 1994 Northridge earthquakes led to a loss of 120 lives and major damage to buildings and infrastructure. The situation is direr due to poorly constructed buildings and over population in Bangladesh and India. To minimize damage and loss of life, seismic design codes have been developed.

Design codes in the U.S. are refined and updated approximately every 3–5 years in order to keep up with advances in earthquake engineering and to incorporate research findings, and are reflected in American Society of Civil Engineers, ASCE 7-10 [2]. The Indian seismic code (IS-1893), first published in 1962, has been revised only five times in the last 50 years; the most recent revision being in 2002 after the devastating Bhuj earthquake. Bangladesh National Building Code (BNBC), developed in 1993, was officially enacted in 2006 without changing the code ([3]). According to Bari and Das [4], the value of design base shear is the least in BNBC-1993 in comparison to ASCE 7-10 and IS-1893. Some studies have pointed out a number of limitations of the code in terms of seismic hazard protection. Reinforced concrete frame buildings were heavily damaged in Bhuj earthquake, and the majority of them collapsed completely according to a reconnaissance report prepared by World Seismic Safety Initiative ([13]). Based on the observations and lessons learned from Bhuj earthquake, most of the weaknesses in the 1984 edition of IS-1893 were removed in the 2002 version of the code. Buildings designed according to the U.S. seismic provisions are generally expected to perform well.

Although the three design codes share some commonalities, it is unclear whether a building designed according to ASCE 7-10, BNBC-1993, and IS-1893 codes would perform as intended when the building is subjected to a design level ground motion that has a response spectrum comparable to the one used in design. For example, are the drift limits met? is week girder-strong column design methodology achieved? The focus of the reported research was to answer these and other questions by comparing the seismic performances of reinforced

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Table 1
Comparison of seismic provisions of ASCE 7 [2], IS 1893 [9] and BNBC 1993.

ASCE 7 [7]	IS 1893 [9]	BNBC 1993
(a) Zoning system i. Each region is assigned a location specific mapped spectral acceleration parameter (*SS*, short period and *S*1, 1 sec). ii. *SS* & *S*1 are modified for Site Class effects to get Maximum Considered Earthquake (MCE) spectral response acceleration parameters (*SMS* and *SM*1). iii. The design spectral acceleration *SDS* and *SD*1 parameters can be obtained by dividing *SMS* and *SM*1 parameters by 1.5.	i. The country is divided into 4 zones (II, III, IV and V). ii. Each zone is assigned a factor (Z), which is used to obtain the response spectrum depending on the perceived seismic hazard in that zone corresponding to MCE.	i. The country is divided into 3 zones (1, 2, 3) ii. Each zone is assigned a coefficient (Z).
(b) Site classification i. Average shear wave velocity ($\bar{v}_s$), average field standard penetration resistance ($\overline{N}$), and average undrained shear strength ($\bar{s}_u$) for the top 30.5 m are used to classify different sites.	i. Site classification depends only on the standard penetration value (N).	i. Site classification depends on shear-wave velocity and soil profile depth. Site soils are classified into four types: S_1, S_2, S_3, and S_4.
(c) Approximate fundamental period i. Approximate fundamental period for "Reinforced Concrete (RC) Moment Resisting Frame" is $T_a = 0.0466 h_n^{0.9}$, h_n in m.	i. Approximate fundamental period for "Reinforced Concrete Moment Resisting Frame", $T_a = 0.075 h_n^{0.75}$, h_n in meter.	i. Approximate fundamental period for "Reinforced Concrete Moment Resisting Frame", $T_a = 0.073\ h_n^{0.75}$, h_n in meter.
(d) Response reduction factor (R) Classification of RC moment resisting frames: i. Ordinary Moment Resisting Frame (OMRF), $R = 3$. ii. Intermediate Moment Resisting Frames (IMRF), $R = 5$. iii. Special Moment Resisting Frame (SMRF), $R = 8$.	Classification of RC moment resisting frames: i. Ordinary Moment Resisting Frame (OMRF), $R = 3$. ii. Special Moment Resisting Frame (SMRF), $R = 5$.	Classification of RC moment resisting frames: i. Ordinary Moment Resisting Frame (OMRF), $R = 5$. ii. Intermediate Moment Resisting Frames (IMRF), $R = 8$. iii. Special Moment Resisting Frame (SMRF), $R = 12$.
(e) Importance factor i. Based on the four risk categories (I, II, III, & IV), ASCE 7 has four seismic importance factors (I_e): 1.0, 1.0, 1.25, and 1.5, respectively	i. Based on the functional use and the occupancy of the buildings, IS 1893 has two importance factors (I): 1.0 and 1.5	i. Based on the five risk categories (I, II, III, IV, & V), BNBC has five seismic importance factors: 1.25, 1.25, 1.0, 1.0, 1.0, respectively.
(f) Drift criterion i. Allowable "inelastic" story drifts are limited to $0.020 H_{storey}$ for commercial buildings having Risk category I or II. ii. The allowable limits decrease as the risk category increases. Refer to Table 12.12.1, ASCE7-10.	i. Allowable "elastic" story drifts are $0.004 H_{storey}$ for all the structures irrespective of any structural or risk category. Refer to clause 7.11.1, IS 1893(Part 1): 2002.	Story Drift, Δ, shall be limited as follows: i. $\Delta \le 0.04h/R \le 0.005h$ for $T \le 0.7$ sec. ii. $\Delta \le 0.03h/R \le 0.004h$ for $T \ge 0.7$ sec. iii. $\Delta \le 0.0025h$ for unreinforced masonry structures where h = height of the building Refer to Section 1.5.6.1 BNBC.
(g) Minimum design lateral force i. Design lateral force calculated from static analysis is $V = C_s \times W$ where C_s = the seismic response coefficient $C_s = \frac{S_{DS}}{\left(\frac{R}{I_e}\right)}$ and W = the seismic weight of the building	i. Design lateral force calculated from static analysis is $V = \frac{1}{2} \times \frac{S}{g} \times \frac{I}{R} \times W$ where (S/g) = spectral response acceleration parameter for MCE response spectrum corresponding to T_a, and W = the seismic weight of the building	i. Design lateral force calculated from static analysis is $V = \frac{Z \times I \times C}{R} \times W$ where Z = Seismic zone coefficient, $C = 1.25S/T^{2/3}$, and W = the seismic weight of the building
(h) Response spectrum i. Spectral Acceleration, For $T < T_0$, $S_a = S_{DS}(0.4 + 0.6\frac{T}{T_0})$ $T_0 = 0.2.S_{D1}/S_{DS}$ ii. $T_0 > T > T_S$, $S_a = S_{DS}$, $T_S = S_{D1}/S_{DS}$ iii. $T_S > T > T_L$, $S_a = \frac{S_{D1}}{T}$ where T_L = long period transition period iv. $T > T_L$, $S_a = \frac{S_{D1} T_L}{T^2}$	i. For rocky or hard soil sites, $S_a/g = \begin{cases} 1 + 15T, & (0.0 < T < 0.10) \\ 2.5, & (0.1 < T < 0.40) \\ 1/T, & (0.4 < T < 4.0) \end{cases}$ ii. For medium soil sites, $S_a/g = \begin{cases} 1 + 15T, & (0.0 < T < 0.10) \\ 2.5, & (0.1 < T < 0.55) \\ 1.36/T, & (0.55 < T < 4.0 \end{cases}$ iii. For soft soil sites, $S_a/g = \begin{cases} 1 + 15T, & (0.0 < T < 0.10) \\ 2.5, & (0.1 < T < 0.67) \\ 1.67/T, & (0.67 < T < 4.0) \end{cases}$ iv. To get a site-specific design response spectrum, a factor $(Z/2)$ is to be multiplied.	i. According to Section 2.5.7.1 in BNBC 93, "a site-specific response spectra shall be developed based on the geologic, tectonic, seismologic, and soil characteristics associated with specific site. The spectra shall be developed for a damping ratio of 0.05 unless a different value is found." ii. "In absence of a site-specific response spectrum, the normalized response spectra given in Fig. 6.2.11 BNBC 93 shall be used with the procedure described in Section 2.5.7.2 BNBC 93".

concrete buildings designed according to the codes from Bangladesh, India, and the U.S. For this purpose, geometrically similar reinforced concrete moment-resisting frames, used as commercial buildings, were selected and designed. The three codes were compared on the basis of the type of allowable analysis procedures, zoning system, site classification, fundamental vibration period of the structure, response reduction factor, importance factor, minimum design lateral force, allowable story drifts, and design response spectra. Nonlinear response history analysis of each structure was conducted, and a number of key metrics were used to compare the performances of the three structures.

2. Comparison of seismic provisions

ASCE 7-10 utilizes seismic design category (SDC) concept to

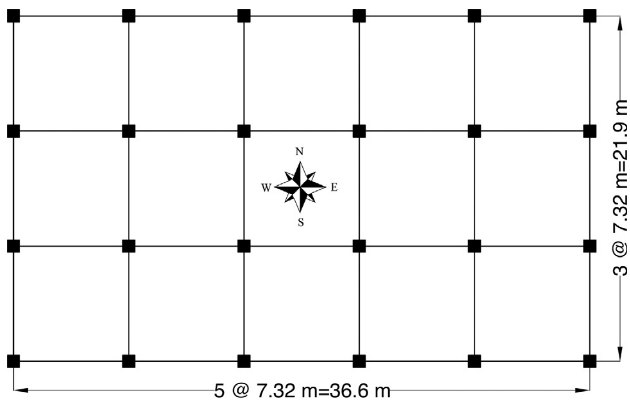


Fig. 1. Plan view of the structure.

categorize the structures according to seismic risk level. The SDC of a structure depends on the soil characteristics, geographical location, occupancy category, geometry, structural system, and vibration period of the structure. Based on the SDC, one or more of the following analysis options is permitted: (a) Equivalent Lateral Force (ELF) analysis, (b) Modal Response Spectrum analysis, and (c) Seismic Response History procedures. Reinforced concrete structures are designed and detailed in accordance with Building Code Requirements for Structural Concrete (ACI 318-14).

The structures designed in India must conform to the seismic design requirements of Indian Standard, Criteria for Earthquake Resistant Design of Structures, Part 1, General Provisions and Buildings, IS-1893 [9]. A seismic zoning system, consisting of four seismic zones, is the basis of the approach used in the Indian code. Based on height, configuration, and zone factor, either static or dynamic analysis is specified by the code. Reinforced concrete structures are designed based on Indian Standard, Plain, and Reinforced Concrete-Code of Practice, IS-456 [11]. Seismic detailing of concrete structures is performed in accordance with Indian Standard, Code of Practice for Ductile Detailing of Reinforced Concrete Structures subjected to Seismic forces, IS-13920 [10].

Seismic loadings and provisions are discussed in Chapter 2 (Part 6) of Bangladesh National Building Code, BNBC-1993 [3]. Based on the probable intensity of seismic ground motion and potential damage, Bangladesh is divided into three seismic zones (Zones 1, 2, and 3) with the northeast part of the country being the most vulnerable and southwest the least. According to BNBC-1993, seismic lateral forces on structures are determined by using either Equivalent Static Force Method or Dynamic Response Method. Buildings are classified into five structural importance categories from category (I) to (V). Reinforced concrete structures are designed and detailed in accordance with BNBC-1993.

Some of the common aspects of ASCE 7-10, BNBC-1993, and IS-1893 include: (a) Zoning system, (b) Site classification, (c) Equations for calculating the fundamental vibration period of the structure, (d) Minimum design lateral force, (e) Response reduction factor, (f) Importance factor, (g) Allowable story drifts, and (h) Design response spectrum. The most important aspects of the three codes are compared in Table 1.

Table 3
Dead and live loads.

Loads	ASCE 7-10	BNBC (1993)	IS 875 (1987)
Live load	Floor: 2.4 kN/m^2 Roof: 1 kN/m^2	Floor: 3.0 kN/m^2 Roof: 1.5 kN/m^2	Floor: 2.50 kN/m^2 Roof: 1.5 kN/m^2
Concrete unit weight	23.6 kN/m^3	25.0 kN/m^3	25.0 kN/m^3
Mechanical loading	0.24 kN/m^2	0.24 kN/m^2	0.24 kN/m^2
Partition wall loads	0.72 kN/m^2	1.2 kN/m^2	1.2 kN/m^2
Cladding	4.38 kN/m	4.5 kN/m	4.5 kN/m

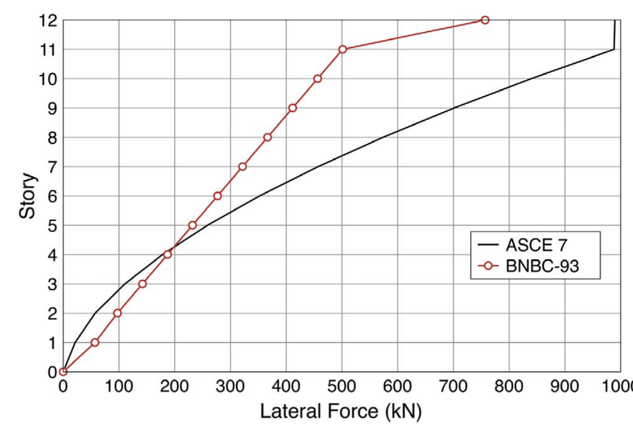


Fig. 2. Comparison of equivalent lateral forces from ASCE 7-10 and BNBC-93.

3. Analysis and design

A geometrically similar 12-story, 3-bay by 5-bay reinforced concrete special moment resisting frame was considered for all the codes (see Fig. 1). The height of the bottom story was taken as 4.27 m and the remaining stories were 3.66 m each, resulting in a 44.5 m tall building. The width and length of the structure was 21.9 m and 36.6 m, respectively with column spacing of 7.32 m in both directions. The selected structure was to represent a commercial/office building. Seismic Importance Factor (I) was taken as 1.0 according to the three codes.

The buildings were assumed to be located in high seismic regions: San Francisco (USA), Sylhet (Bangladesh), and Bhuj (India). The site soil classification and the spectral response acceleration parameters or zone factors for these buildings are shown in Table 2.

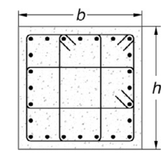
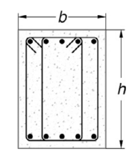
The dead load consisted of the self-weight, floors, roof, built-in partitions, cladding, and mechanical loadings. The values of gravity loads shown in Table 3 were selected according to the governing codes in each country.

Based on the calculated short period ($S_{DS} = 1.22$ g at $T_o = 0.139$ sec) and $1 - S$ ($S_{D1} = 0.85$ g at 1 sec) response acceleration parameters, the structure was classified as Risk Category II with seismic design category E in accordance with ASCE 7-10. Considering that the building is not irregular and its height is less than 48.8 m, Equivalent Lateral Force (ELF) analysis is permitted by ASCE 7-10, and this method was selected for design of the building as a special reinforced concrete frame with response modification $R = 8$ and deflection amplification $C_d = 5.5$. Using the dead load as the only effective seismic weight, the base shear was determined (5528 kN) and distributed according to

Table 2
Site locations and classifications.

Code	Location	Zone coefficient/response acceleration parameters	Site class
ASCE 7-10	San Francisco, USA	Spectral response acceleration parameters: S_S = 1.83, S_1 = 0.85	Site class D, stiff soil
BNBC (1993)	Sylhet, Bangladesh	Seismic Zone: 3, Zone factor (Z) = 0.25	Type S_3 (soft to medium stiff clay), S = 1.5
IS 1893	Bhuj, Gujarat, India	Seismic Zone: V, Zone factor (Z) = 0.36	Type II (Medium soils)

Table 4
Summary of design.

Story	Code	Dimensions $b \times h$ (mm)	Reinforcement				
			Longitudinal			Transverse	
			No. of bars	Size (mm)	ρ (%)		
(a) Columns							
1	ASCE 7	762 × 762	24	22.2	1.6	12.7 mm @ 60 mm (plastic hinge) and @100 mm (other locations)	
	BNBC	750 × 750		28	2.6	12 mm @ 125 mm (plastic hinge) and @ 300 mm (other locations)	
	IS 875	600 × 600		32	5.4	12 mm @ 100 mm (plastic hinge) and @ 300 mm (other locations)	
2–3	ASCE 7	762 × 762		22.2	1.6	12.7 mm @ 60 mm (plastic hinge) and @ 100 mm (other locations)	
	BNBC	750 × 750		28	2.6	12 mm @ 125 mm (plastic hinge) and @ 300 mm (other locations)	
	IS 875	600 × 600		25	3.3	12 mm @ 100 mm (plastic hinge) and @ 300 mm (other locations)	
4–6	ASCE 7	762 × 762		22.2	1.6	12.7 mm @ 60 mm (plastic hinge) and @ 100 mm (other locations)	
	BNBC	625 × 625		28	3.8	10 mm @ 125 mm (plastic hinge) and @ 300 mm (other locations)	
	IS 875	600 × 600		16	1.3	12 mm @ @ 90 mm (plastic hinge) and @ 150 mm (other locations)	
7–12	ASCE 7	762 × 762		19.1	1.6	12.7 mm @ 60 mm (plastic hinge) and @ 100 mm (other locations)	
	BNBC	500 × 500		22	3.6	10 mm @ 125 mm (plastic hinge) and @ 300 mm (other locations)	
	IS 875	600 × 600		16	1.3	12 mm @ @ 90 mm (plastic hinge) and @ 150 mm (other locations)	
(b) Beams							
All	ASCE 7	556 × 762	Top: 5	28.7	0.76	9.5 mm @ 100 mm (plastic hinge) and @ 200 mm (other locations)	
	BNBC	400 × 625		25	0.98	10 mm @ 125 mm (plastic hinge) and @ 200 mm (other locations)	
	IS 875	400 × 625	Bottom: 5	32	1.6	8 mm @ 125 mm (plastic hinge) and @ 250 mm (other locations)	

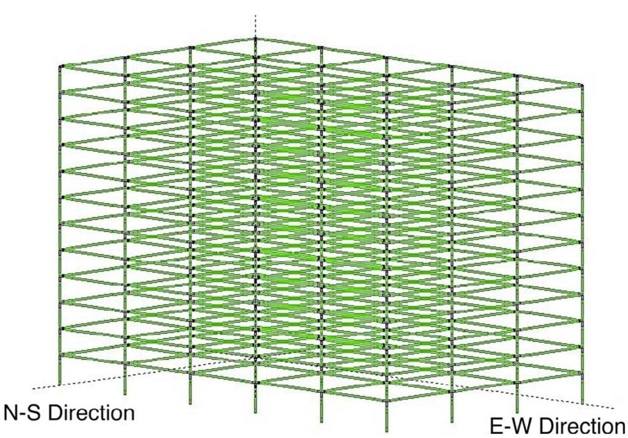


Fig. 3. 3-D structural model.

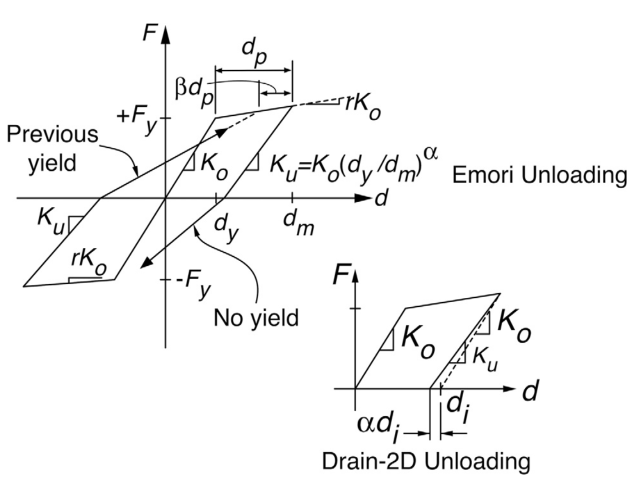


Fig. 4. Modified takeda hysteresis model (Adapted from Ruaumoko Manual (Carr, 2016)).

ASCE 7-10. The lateral force distribution over the building height is shown in Fig. 2.

For the structure located in Bangladesh, the Equivalent Static Force Method was used to determine the seismic lateral forces as the structure is regular and under 75 m in height in accordance with BNBC-1993. The total base shear in a given direction was determined from the equation described in Table 1. The response modification coefficient (R) for special moment resisting concrete frames was taken as 12. The base shear was equal to 3808 kN. The distribution of the lateral forces is shown in Fig. 2.

According to IS-1893, regular buildings of height greater than 40 m in Zone IV or V have to be analyzed by dynamic analysis, i.e., response spectrum method or response history method, but guidelines to the former are only provided in the code. A special moment resisting frame with $R = 5$ was selected. The structure was analyzed for the design response spectrum in ETABS NL ([6]). The design base shear calculated by this method was 3594 kN.

3.1. Modeling and analysis of structure for design

The structures were modeled three dimensionally in the commercial structural analysis and design software ETABS NL (Version 9.6). The columns were assumed to be fixed at the foundation. Rigid diaphragm action of the slab was simulated. Dead load, live load, and seismic loads were applied as static load on the structure according to the U.S. and Bangladesh codes. Based on the Indian code, the seismic load was taken into account through response spectrum analysis method.

3.2. Design of structures

Using the calculated design forces, the columns and beam members were designed and detailed as per the applicable provisions of the ACI 318-14 [1], IS-456, and BNBC-1993 for the most severe load combinations, as described elsewhere ([12,17]). The materials used were: (a) concrete compressive strength, f'_c, = 48 MPa and ASTM Gr. 60 reinforcing steel (yield strength, f_y, = 414 MPa) conforming to ACI 318-14; (b) M50 concrete (f_{ck} = 50 MPa) confirming to IS-456 [11] and Fe 415 Grade reinforcement (f_y = 415 MPa) confirming to IS-456 [11]; and (c) concrete with f'_c = 32 MPa and Grade 400 reinforcing bars

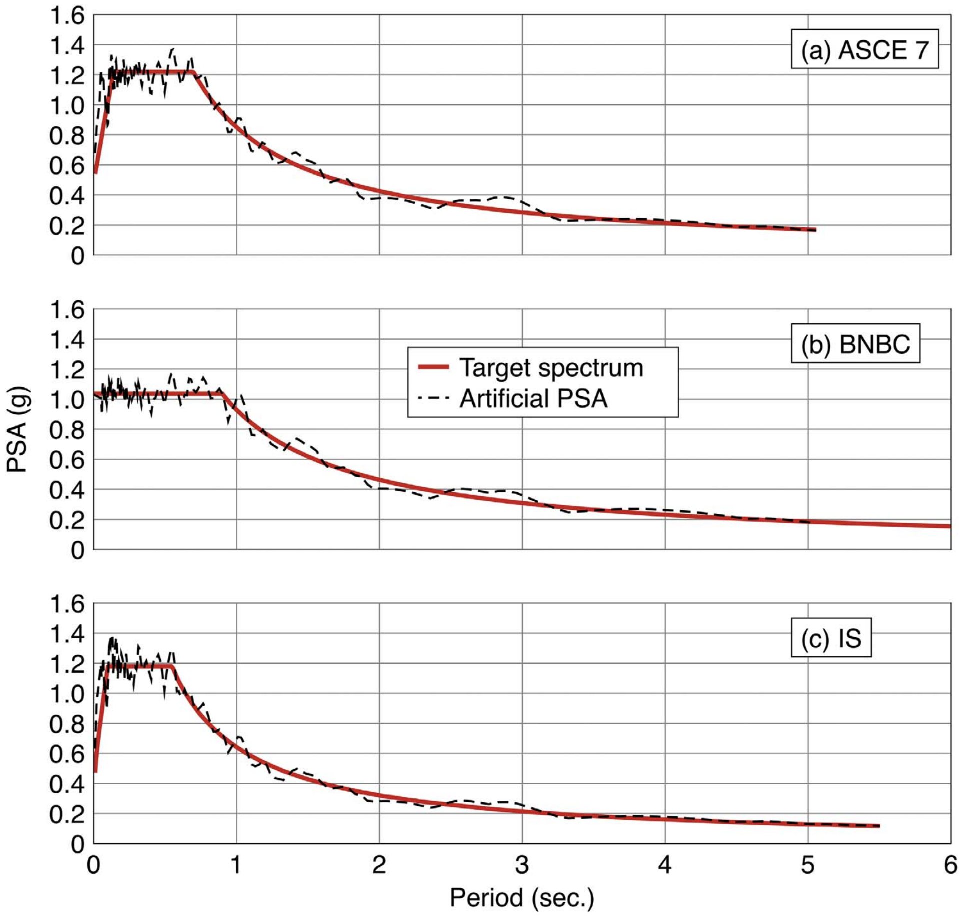


Fig. 5. Design and artificial spectra.

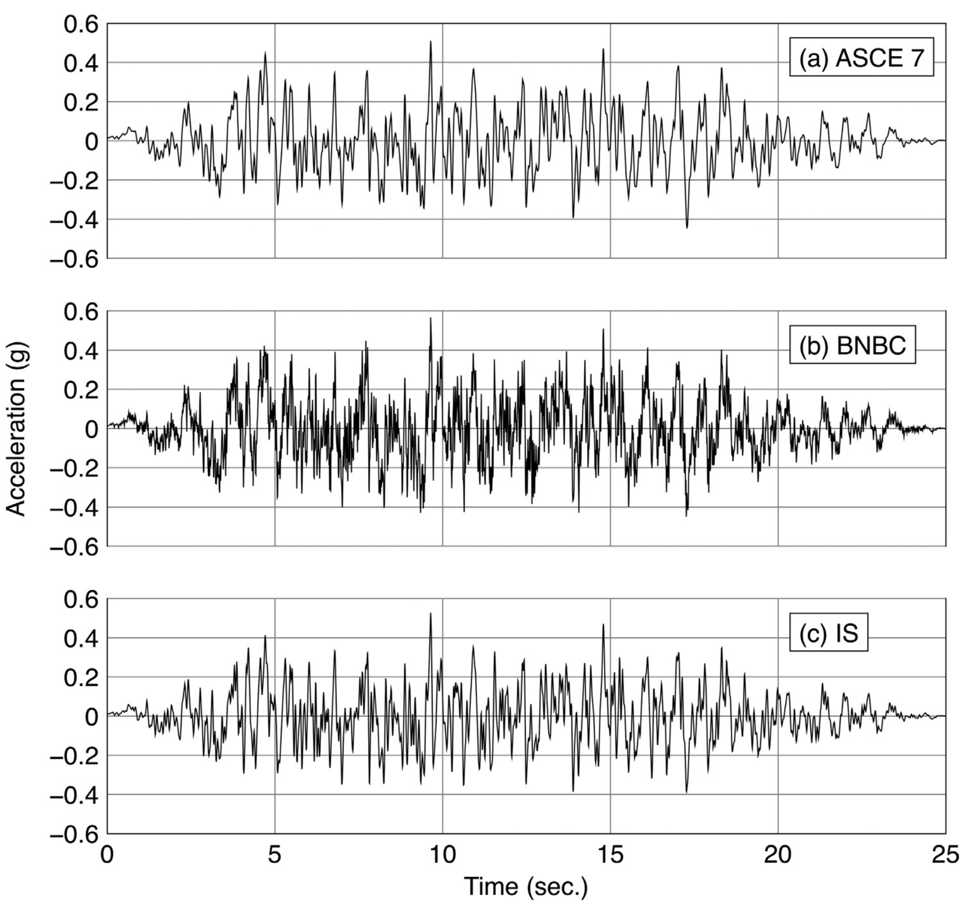


Fig. 6. Artificial accelerograms.

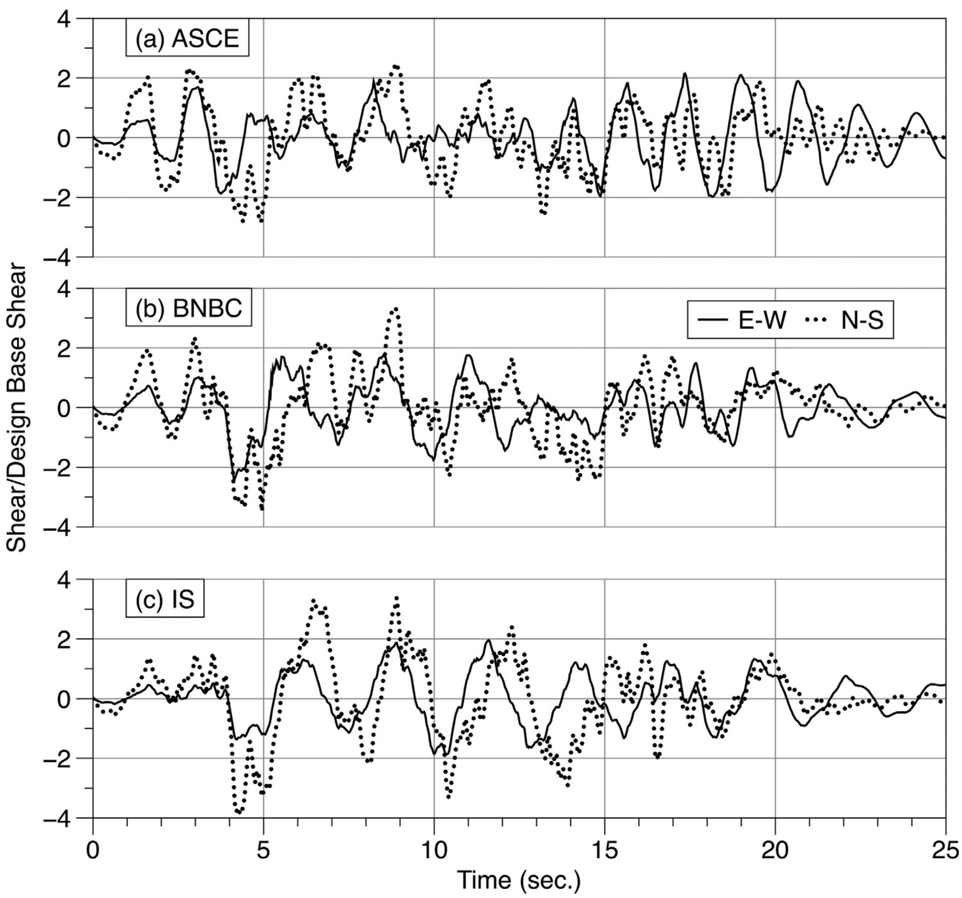


Fig. 7. Normalized base shear history.

Table 5
Peak normalized base shears and time.

Direction	Building	V_{max}/V_{design}	Time (sec.)
E-W	ASCE	2.2	17.35
	BNBC	2.4	4.15
	IS	2.0	11.60
N-S	ASCE	2.8	4.93
	BNBC	3.5	4.95
	IS	3.9	4.33

($f_y = 400$ MPa) conforming to BNBC-1993. These values are consistent with common practice in the U.S., Bangladesh, and India. The selected beam and column dimensions and reinforcement are summarized in Table 4.

4. Nonlinear dynamic response history analysis

Ruaumoko nonlinear dynamic analysis program ([5]) is a suite of applications specifically designed for dynamic inelastic analysis of structures subjected to earthquake loading. Ruaumoko 3D, which is a core program of Ruaumoko suite, was used to conduct three-dimensional nonlinear response history analysis of each structure in an effort to compare the global responses and performance of various members.

4.1. Geometry

The analytical model (Fig. 3) consisted of a total of 312 nodes and 744 members (456 beams and 288 columns). The column bases were fixed (i.e., all 6 DOFs were restrained) based on the assumption that the foundation system is adequate and does not fail when subjected to seismic loading. To simulate rigid floor diaphragm action, all the horizontal degrees of freedom in a given floor were slaved to a single node; hence, all the nodes in a given floor would have identical horizontal displacements.

4.2. Member hysteretic models

Inelastic sectional response of a member (in particular reinforced concrete members) depends not only on its cross-sectional geometry but also on its material characteristics, which are affected by the constitutive relationships for unconfined concrete, confined concrete, and steel reinforcement. The following material models were selected: (a) Hognestad [8] stress-strain model for unconfined concrete; (b) Mander, Priestly, and Park model [14] for confined concrete; and (c) bilinear stress-strain curve with strain hardening for reinforcing steel. Sectional analysis software XTRACT ([18]) was used to generate moment-curvature relationships (M-ϕ diagrams) and axial load-moment interaction diagrams (P-M diagrams) to determine the various parameters needed for the selected hysteretic models.

The modified Takeda model ([16]) was used to model the hysteretic characteristics of the beams and columns (see Fig. 4). The bilinear factor (r) for this model was determined by moment-curvature relationships that were idealized as bilinear curves. The values of α, β, and displacement ductility factors were selected based on the experience of the authors ([12]) and others ([7]).

The capacity of the columns subjected to bi-directional moments was found by generating three-dimensional axial load-bending moment interaction surfaces. The columns in the reported study were square columns; hence, the moment capacities about both axes of bending were the same. The capacity surface was an ensemble of slices of the P-M capacity diagrams about the full range of bending axes (0–180°) that were calculated from XTRACT. A MATLAB [15] code was written to construct the capacity surface and check whether the demands (axial

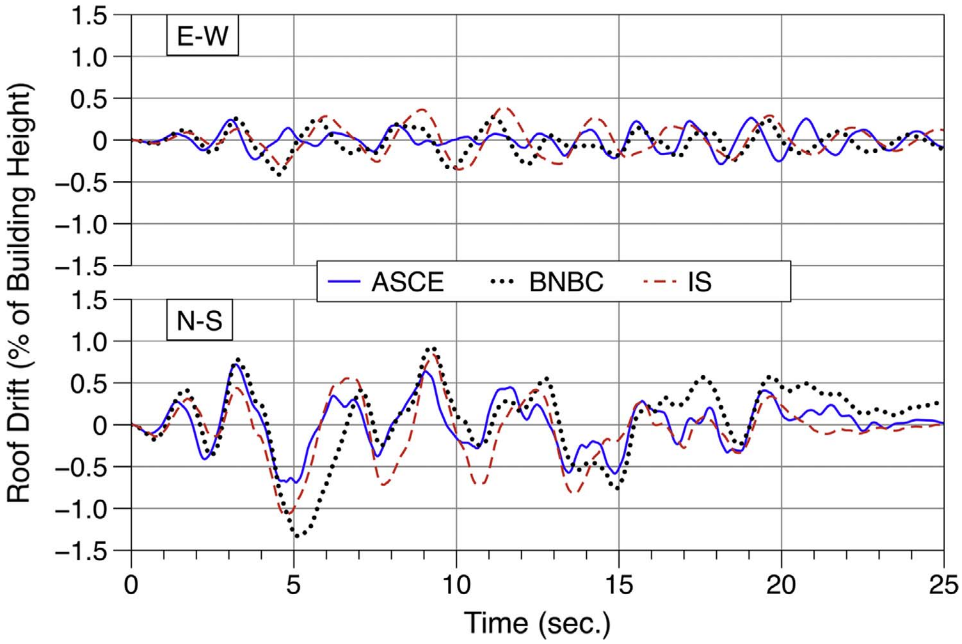


Fig. 8. Roof drift history.

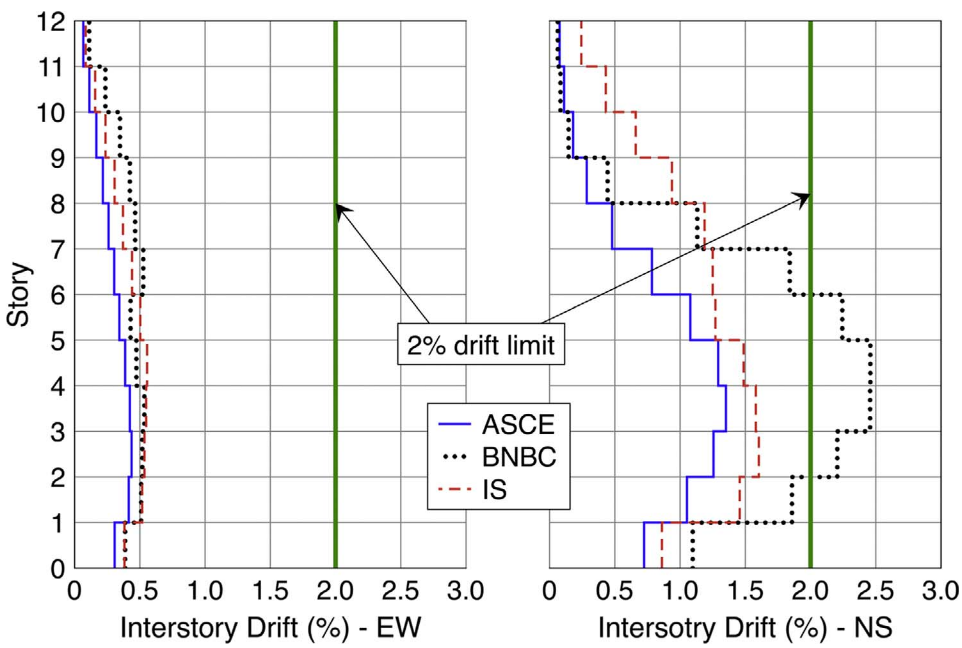


Fig. 9. Inter-story drifts.

load and moments about both axes of bending) obtained from the nonlinear analyses are within the surface (i.e., having adequate capacity) or fall outside the surface (corresponding to failure).

4.3. Selection of artificial ground motions

Considering the paucity of recorded ground motions in Bangladesh, response-spectrum compatible artificial ground motions were generated and used for dynamic analyses by SIMQKE, which is a part of Ruaumoko software suite. Generation of artificial ground motions was based on matching the design response spectra. As evident from Fig. 5, the 5% damped response spectra of the artificially generated ground motions reasonably match the target design spectra.

The artificially generated ground motions were 25 sec long, see Fig. 6. The peak accelerations for ASCE, BNBC, and IS response spectrum compatible records were 0.51 g, 0.57 g, and 0.53 g, respectively. The buildings were subjected to ground motions in both directions. Considering that the building is "weaker" in the N-S direction than the E-W direction (3 vs. 5 bays), 100% of the ground motion was applied in the N-S direction plus 30% of the ground motion in the E-W direction to simulate the most critical loading scenario. The same artificial ground motion was used in both directions, but the acceleration amplitudes were multiplied by 0.30 for the E-W direction.

5. Results and discussions

The performance of each of the structures was evaluated in terms of (a) normalized base shear, (b) roof and inter-story drifts, (c) yielding of beams and failure of columns, and (d) overall energy dissipation. Each of these metrics is discussed separately. In the following discussion, the structure in Bhuj, India; San Francisco, U.S.; and Sylhet, Bangladesh are referred to as IS, ASCE, and BNBC building, respectively.

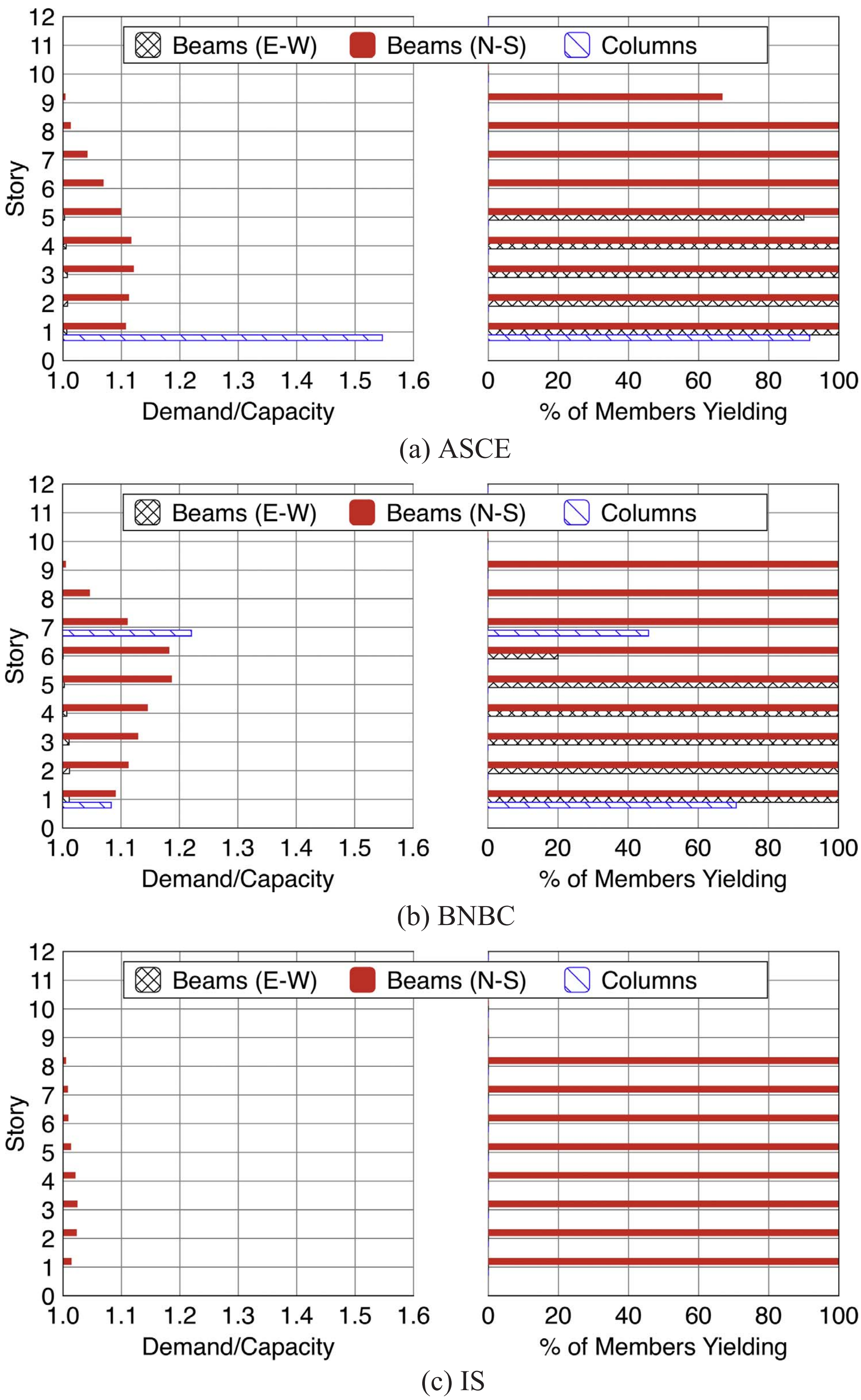


Fig. 10. Assessment of yielding in beams and columns.

5.1. Base shear

The calculated base shear history for each building was normalized with respect to its respective design value. The normalized base shear history is shown in Fig. 7. The peak normalized shear and the corresponding time are summarized in Table 5. The three acceleration records (Fig. 6) had generally similar frequency characteristics; however, the base shear histories are noticeably different. In the N-S direction, the peak values occurred within the first 5 sec for all the buildings, but the time corresponding to the peak values are above 10 sec for the ASCE and IS buildings in the E-W direction. As explained previously, 100% of the ground motion was applied in the "weak direction" (N-S) in conjunction with 30% of the ground motion in the other direction. Therefore, the base shears in the N-S direction are larger than their counterparts in the E-W direction: 1.27, 1.46, and 1.95 times larger for the ASCE, BNBC, and IS building, respectively. All the buildings were

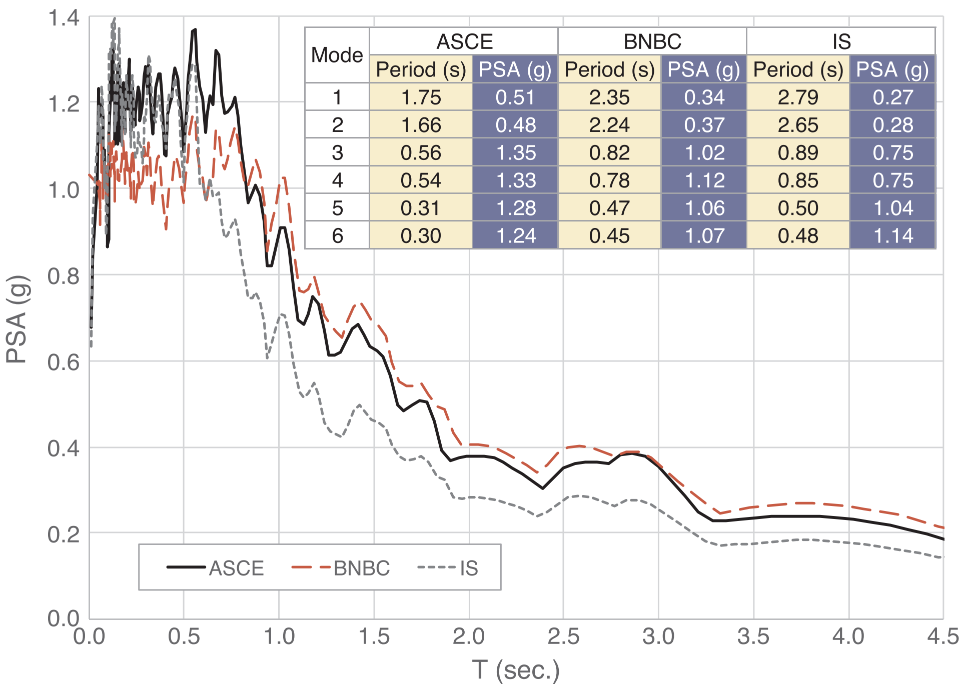


Fig. 11. Comparison of pseudo acceleration response spectra.

subjected to at least twice the design base shear in either direction, which is noteworthy when comparing different design codes.

5.2. Drifts

Fig. 8. illustrates the roof drift normalized with respect to the total building height. The largest drift of 1.3% is in the N-S direction of the BNBC building. In either the N-S or E-W direction, the ASCE building experienced the least roof drift. The roof drifts of all the buildings are acceptable (i.e., < 2%). The inter-story drifts are plotted in Fig. 9. With the exception of floors 3-6 in the N-S direction of the BNBC building, all the inter-story drifts are smaller than 2%. The maximum inter-story drift of 2.46% for floors 4 and 5 in the BNBC building does not meet the target limit of 2%.

5.3. Failure/yielding

In an effort to assess the performance of the beams and columns, the maximum demand-to-capacity ratios (abbreviated as D/C) are plotted in each floor in Fig. 10. In the same figure, the percentage of the members in a given floor with D/C exceeding 1 is also plotted. The largest D/C (1.55) occurred in the first-floor columns of the ASCE building, where the moments in 22 out of the 24 columns exceeded the boundaries of the P-M interaction diagram. Failure of 92% of the columns in the first floor of the ASCE building suggests story-level hinging would likely occur in the first floor. In the case of the BNBC building, the columns in floors 1 and 7 failed with D/C of 1.08 and 1.22, respectively. Out of a total of 24 columns in each floor, 11 and 17 columns in floors 1 and 7 failed, respectively, corresponding to 71% and 46% of the total number of columns. Column failure was concentrated in the first floor of the ASCE building, but it was spread over two floors in the BNBC building. The likelihood of story-level hinging in the BNBC building is less than the ASCE building. None of the columns in the IS building failed.

The difference between the performance of the columns in the three buildings is attributed to the characteristics of the ground motions. Although the peak accelerations of the three artificially generated ground motions are comparable (ASCE: 0.51 g, BNBC: 0.57 g, and IS: 0.53 g), their acceleration response spectra are different. In Fig. 11, the pseudo acceleration (PSA) response spectra, and the vibration periods of the first six modes and the corresponding values of PSA are compared. The ground motion used to analyze the ASCE building has the largest PSA for all the six modes – on average 25% and 18% larger than IS and BNBC, respectively. Had the IS building been subjected to the ASCE compatible ground motion, all the columns in the first floor of the IS building would have also failed.

Considering the larger accelerations applied in the N-S direction, the beams in this direction yielded (D/C > 1) more significantly than the beams in the E-W direction. For all the buildings, 100% of the N-S beams in the first 9 floors yielded (see Fig. 10). The largest D/C (1.19) was found to be in floor 5 of the BNBC building, and the least D/C of 1.02 occurred in the 4th floor of the IS building. In the case of the ASCE building, the largest level of yielding of the N-S beams occurred in floor 3 with D/C being 1.12. None of the E-W beams in the IS building experienced yielding. The level of yielding for the beams in the E-W direction was minimal for the other two buildings, the largest D/C was equal to 1.01.

5.4. Energy dissipation

To capture the overall energy dissipation characteristics of the three buildings, the area under the roof drift vs. the base shear was computed. This area represents an aggregate measure

of the total energy dissipated through yielding of the beams and columns. From Table 6, it is evident that the level of energy dissipation in the E-W direction is significantly smaller than that in the N-S direction because of the smaller level of inelasticity in the strong direction

Table 6
Dissipated Energy.

Direction	Building	Energy (kJ)
E-W	ASCE	1537
	BNBC	1432
	IS	723
N-S	ASCE	25,368
	BNBC	24,948
	IS	10,988

(i.e., E-W). In both directions, the amount of dissipated energy for the IS building is approximately one-half of the energy that the ASCE and BNBC buildings dissipated, which is consistent with no yielding of the columns and smaller level of yielding in the beams. Energy dissipation of the ASCE and BNBC buildings is fairly close.

6. Conclusion

When subjected to response-spectrum-compatible artificial ground motions, the buildings designed according to the practice in Bangladesh, India, and the U.S. experienced base shears several times larger than their design base shears. Therefore, it was possible to compare the building codes under scenarios for which large levels of inelastic deformations are expected.

The building designed according to ASCE 7-10 exhibited the largest stiffness in terms of roof and inter-story drifts. The building in Bangladesh designed per BNBC-1993 was generally the most flexible structure, with a number of the floors not meeting the 2% inter-story drift limit. As designed, the beams in the first 9 floors of the three buildings yielded. The beams in the BNBC building had the most amount of yielding, as evident from having the largest moment demand/capacity ratio. The frames were designed based on strong column-weak beam design methodology. Nevertheless, 92% of the columns in the first floor of the ASCE building exceeded their capacity (i.e., axial load and bending moment were extended beyond the P-M diagram boundaries), suggesting a story-level hinging, while the remaining columns did not yield. For the BNBC building, there was less likelihood of story-level hinging as column yielding was spread between two floors. All of the columns in the IS building performed as intended, i.e., they did not yield while the beams developed and exceeded their flexural capacities. The difference in the performance of the columns is attributed to the characteristics of the three artificially generated ground motions, for instance, all the columns in the IS building would have also failed if it had been subjected to the ground motion used to evaluate the ASCE building. The level of overall energy dissipation in the ASCE and BNBC buildings was comparable, which is consistent with yielding in the beams and columns of these two buildings. The IS building had the smallest energy dissipation because a large percentage of its members did not yield and the demand/capacity ratios were smaller than the other two buildings.

Aggregating the above results from different metrics, it can be concluded that the structure designed according to the Indian code performed better when subjected to the ground motion that is intended to represent the Indian design response spectrum. Although the drift limits were met, slightly larger members would have made the stiffness of the IS building comparable to the ASCE building. Additional studies are needed to evaluate these codes for a suite of ground motions and other structural systems.

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