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Evaluating the Effect of Multiple Earthquakes of Different Frequency Content on Steel Buildings

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Abstract

The effects of multiple earthquakes of different frequency content on the global response of steel building are presented in this paper. Multiple earthquakes forming sequences as mainshock-aftershock pairs, three (3) steel moment resisting frames of 10-storey (H10), 15-storey (H15) and 20-storey (H20) were analyzed according to the code provisions of IS 1893 (Part 1): 2016 using sequential nonlinear response history analyses. The frequency content of mainshock and aftershock records were varied based on the PGA/PGV ratio. The result produced expected differences in structural responses conditional on the frequency content and the dynamic characteristics of each model. The outcome is measurable differences in the structural response, with the inter-storey drift and other structural response parameters changing due to the different frequency characteristics of the aftershocks. It is expected that the conclusions drawn in this paper will lead to an improvement of the IS 1893 with regards to multiple earthquake effect for an improved design and construction of steel frame structures in India, which will lead to better seismic performance.

Keywords: foreshock; aftershock; ground motion; frequency content; steel building; earthquake

Introduction

There is high uncertainty in both earthquake occurrence and the structural behaviour of a building. Due to the high uncertainty associated with such seismic action, the application of probability concepts for estimating seismic hazards at a site as well as the likely response of a structure is more than justified. Nonetheless, the design codes, researchers and earthquake engineers generally concern themselves with the building response under the main shock without recourse to the effect of multiple earthquakes (seismic sequences). A clear and typical case in the history of multiple earthquakes and its effect is the 1985 Mexico City earthquake. Several other historical earthquake records reveal the foreshock-main shock-aftershock sequence of earthquake occurrence (e.g., Alliard 2006; Kim et al. 2010; Chen et al. 2002). The Christchurch, New Zealand, earthquakes in 2011 and 2012 (Jiun-Wei et al., 2015) are a recent record of this phenomenon. Other recent multiple examples which showed the compounding effect of damage and disruption that multiple earthquakes can bring on human life and finances include Chi-Chi earthquakes in 1999, Wenchuan in 2008, Tohoku in 2011, and central Italy in 2016 (Atzori et al. 2012; Kazama and Noda 2012; Galadini et al. 2017). Multiple earthquakes are a sequence of foreshocks, main-shock and aftershocks (Agnès et al., 2018). The magnitude of the main shock determines the magnitude and frequencies of the aftershocks (e.g., Li et al. 2007; Sakai et al. 2014; Kagan and Knopoff, 1978; Reasenberg, 1999). Retrospective data shows the foreshock frequency, which is, the fraction of mainshocks that happen after a foreshock, is 10% to 40% (Jones and Molnar, 1979; Yamashina, 1981; Lindh and Lim, 1995; Abercrombie and Mori, 1996; Michael and Jones, 1998; Reasenberg, 1999). Sunasaka et al. (1993) have observed in contrast that, large mainshocks have considerably more aftershocks compared to foreshocks, which has been upheld through the research of (Kagan and Knopoff, 1976, 1978; Jones and Molnar, 1979). The authors showed these foreshocks are less frequent than aftershocks at a ratio of 2:4 for main shocks having magnitudes from 5-7. The distance of such records are between 50-500 km from the mainshock and for a time of 10-100 days before or after the mainshock (Kagan and Knopoff 1976; 1978; Jones and Molnar, 1979; Von Seggern et al., 1981; Shaw, 1993]. In the midst of these contrasting views by these researchers, a large amount of the evidence upholds the occurrence of foreshock-mainshock to be less than the frequency of mainshock-aftershock occurrences. Carlos and Stephen (1988) found from a comprehensive comparison of aftershock patterns following several moderate to large earthquakes that the occurrence of aftershocks requires a secondary redistribution of stress from the main shock and Karen et al. (2004) concludes a single physical triggering mechanism is accountable for the incidence of aftershocks, foreshocks, and multiplets. They also observed that aftershocks occur mostly outside of or near the edges of the source areas indicated by the patterns of mainshock slip. In other studies, (Zhang et al. 2013; Zhao et al. 2010; Li et al. 2014 and Yeo et al. 2009) have proven that the peak ground acceleration (PGA) of the aftershock can be as large as the main shock's even though the energy released by an aftershock is often small. This pattern was observed in the 1992 Landers earthquake where the PGA of the aftershock was 10 times greater than the PGA of the main shock (Li et al. 2014) and aftershock of the Darfield earthquake (4 days after the main shock), produced greater spectral accelerations than the main shock did, for some periods (Bruneau et al., 2010). By looking at the interval of time between earthquakes, Faisal et al. (2013) stipulated that they could happen within the same area if the time interval is short as happened Berlongfer (India) in 1988 and 1990. Therefore, an earthquake-damaged building can come under the effect of a strong aftershock or another earthquake in a short interval and may be vulnerable to such future earthquakes. However small this probability might be, experience has demonstrated its possibility and how deadly it can be. Okawa et al., (2013) showed a clear example when a 55-stories building in Japan had excessive deformation resulting in tangling of elevator wires, the subsequent confinement of passengers, and unexpected movement of fire-protection doors that caused breakage of sprinklers, etc. was again subjected to an MW 7.7 aftershock, thirty minutes after the Tohoku-Oki main shock. The material loss is estimated to reach $300 billion, a major disaster that affected economies throughout the world.

In the event of such, steel buildings which are already damaged after the first earthquake, may lose their structural capacity and become inadequate because of damage accumulation. Research findings show the damage accumulation often result in a surge in Inter-Storey Drift Ratio (IDR) (Saman and Parinaz 2018), ductility demand and maximum inelastic displacement (Efraimiadou et al. 2013). Jiun-Wei et al. (2015) witnessed significant P-Δ effects in the pushover curves of an investigation where a 35-story steel building was subjected to an earthquake sequence foreshock followed by the main shock. Typically, failure of steel buildings involves local buckling of the cross-sections due to excessive compression or shear, large residual deformation caused by exaggerated plastic strain (Zhanzhan et al., 2016), and brittle failure or fracture resulting from low-cycle fatigue.

In that regard, reckoning the risk of further damage to the built environment and the risk to human survival from aftershocks, taking the step to design for it, is essential to post-mainshock repair and retrofit decision-making, functionality, and recovery. It is very momentous to consider the unrepaired seismic damage in the evaluation of the residual capacity of a building against another earthquake. At present, however, the focus of the earthquake design codes in the world is to resist deformation in the main shock and does not take into consideration the residual seismic capacity to resist another earthquake. Some studies have nonetheless been reported in the literature about multiple earthquake effects (Elanshai et al. 1998; Amadio et al. 2003; Hatzigeorgiou 2010a; 2010b; 2010c; Yaghmaei-Sabegh and Ruiz-Garcia 2016; Hatzigeorgiou and Liolios 2010; Ruiz-Garcia and Negrete-Manriquez 2011; and Faisal et al. 2013). Notwithstanding, further studies into whether or not the ground motion characteristics of the sequence have any particular effects on a building's dynamic response is overdue.

A primary objective of this paper is to assess the effect of multiple earthquakes which have different frequency content on the dynamic response of a steel building using the code provisions of IS 1893 (Part 1): 2016. In addition, three scenarios of foreshock-main shock, main shock-main shock and main shock-aftershock are formed using the PGA/PGV ratio categorization of frequency content and taken as the input ground motions. The effect on the dynamic response through nonlinear time-history dynamic analysis of a steel building was thus conducted.

The Crux of The Problem with Steel Buildings in Earthquakes

Several design philosophies have been advanced and adopted for the design of earthquake resistant buildings. The Ultimate Limit State (ULS) and Capacity Design are just some of the widely used design philosophies in current earthquake engineering. All of these design methods aim to ensure that the seismic response of a structure is ductile and dependable. With respect to steel buildings, the tragic February 2011 earthquake in Christchurch, the 1994 Northridge earthquake and 1995 Kobe earthquake (Uang et al. 1997) triggered the debate over the efficiency these seismic design philosophies. The lives lost and the structures that either collapsed or suffered major and unrepairable damage (Laugesen, 2013) was magnanimous to warrant such a debate. Newer philosophies such as Damage Avoidance Design and Performance-Based Earthquake Engineering have emerged as an improvement over the earlier methods of design. They are more stringent, requires the structure to preserve life as well as remain operable and functional because life, education, and business are to remain uninterrupted after an earthquake (Ramhormozian et al., 2018).

The seismic performance of steel buildings is more predictable and there is very little record of steel buildings collapsing in an earthquake (Yanev et al., 1991; Tremblay et al., (1995). They have had better performance in earthquake resistance so far (AISC 1994a). Nonetheless, research shows there were evidence of significant inelastic response and several structural deficiencies in the aftermath of the Northridge earthquake. Preliminary damage assessment showed that most steel buildings failed in the following ways; buckling of braces; excessive sway; yielding (MagRae et al., 2015); failure of anchor bolts (uplift); cracking of base plates; failures of brace welded connections (Okazaki et al., 2013); failure of beam-column moment connections (Mahin, 1998), and reduction in the stiffness of the building to about 1/4 on the average (Okawa et al., 2013). In some buildings such as the ANX Buildings in Japan, which was an eight-story steel-framed reinforced concrete building with a basement floor, the Tohoku-Oki earthquake damaged the building so heavily that it had to be demolished (Okawa et al., 2013).

Foregoing literature has also revealed several intricacies about steel buildings in earthquakes. These include:

- The damage to components of steel buildings caused by seismic action could take so long to discover. The framework of a steel building mostly experience damage at the connections, anchor bolts or local buckling yet these areas are hidden by the protective finish coating. This makes their earthquake-induced damage, even grave and possible catastrophic damages turn out to be less apparent and hard to discover. MagRae et al., (2015) documents evidence of this problem after the Christchurch earthquake where significant fracture damage in some buildings was discovered seven months later. In a similar manner in the Northridge earthquake, Tremblay et al. (1995) report that brittle failure of the welded connection between brace gusset plates and inelastic elongation of anchor bolts were later discovered only after the interior finishes were removed.
- Some level of damage can only be revealed only after the removal of architectural finishes, fireproofing covering, cladding and portions of concrete slabs. Where this is the case, the full extent of damage suffered by steel buildings becomes difficult to know and such data could take months or years to collect. Northridge again was a typical example where the discovery of more failures in connections was found only after structural engineers performed a random inspection of joints in various steel structures.
- Certain type of cracks do not exit the column flange surface in the structural components and these can only be detected by means of ultrasonic inspection.

Active Faults in India

For earthquake design purposes, seismic hazard assessment is required. This can be effectively done if the active faults of a site are accurately known. That is their location, spatial extent, past earthquake activity, recurrence intervals, slip rate, etc. In India, sixty-seven (67) faults have been identified as active (Verma and Bansal, 2016) and new active faults and their geometries are still being discovered. Out of this number, fifteen (15) are in the Himalaya, up to thirty (30) in the stable Peninsular India and seventeen (17) in the adjoining foredeep (ibid). Active faults have the ability to reactivate in the future (Yeats, 2012). Most of the major faults such as Allah Bund fault, Kuchch Mainland fault, Katrol Hill fault and Bhuj fault in India are active and are the sources of large earthquakes in past (e.g. 2001 Bhuj Earthquake). The Bhuj region is characterized with some of these major faults such as Nagar Parkar Fault, the Allah

Bund Fault, Island Belt Fault, Banni Fault, Kachchh Mainland Fault, Katrol Hill Fault and South Wagad Fault (Biswas and Khattri 2002).

Cities, which have high population density, large industrial complexes and lifeline structural facilities, are located in the regions where the active faults are located. In addition, the faults extend through urban centres, which emphasize the urgent need for proper seismic design philosophies.

Ground Motion Parameters

In order to study the effect of sequential ground motions with different frequency content on steel buildings, the criteria used for selecting the set of mainshock-aftershock sequences was adopted as follows: (a) the magnitudes of mainshocks and sequence aftershocks must be ≥ 6.0 and 5.0, respectively; (b) the same station recorded the accelerograms of mainshock and aftershock; (c) the peak ground acceleration (PGA) of most mainshocks and aftershocks are more than 0.1 g (an inelastic demand estimation studies criterion; Goda and Atkinson, 2009); and (d) average shear-wave velocity in the uppermost 30 m VS30 is between 100 and 1500 m/s. Using the above criteria, 8 as-recorded seismic sequences with different frequency content are selected for 4 earthquake events from the PEER NGA West2 database. The detailed information about the ground motion and the seismic sequences is shown in Table 1.

Earthquake Name	Magnitude (MW)	Station Name	MO-DY-YR (HH: MM)	PGA (cm/s^2)	PGV (cm/s)	PGA/PGV
Chi-Chi	7.62	CHY002	09/20/1999	9.798	16.614	0.59
Aftershock2	6.20		09/20/1999 (18:03)	4.233	15.982	0.26
Aftershock3	6.20		09/20/1999 (21:46)	5.559	6.343	0.88
Wenchuan	7.90	Anxiantashui	05/12/2008	18.318	29.041	0.63
Aftershock1	6.0		05/12/2008 (05:25)	0.706	0.460	1.53
Aftershock2	6.1		05/12/2008 (11:11)	0.197	0.245	0.80
Umbria Marche	6.0	Borgo-Cerreto Torre	09/26/1997 (09:40)	4.934	1.444	3.42
Aftershock8	5.2		10/12/1997 (11:08)	8.343	2.469	3.38
Aftershock2	5.5		10/14/1997 (15:23)	11.401	9.873	1.15
Mammoth lake	6.0	Convict Creek	05/25/1980 (16:34)	408.4	140.701	2.90
Aftershock1	5.7		05/25/1980 (16:49)	157.3	141.012	1.12
Aftershock2	5.9		05/25/1980 (19:44)	214.6	350.230	0.61

Table 1: Detailed information on the selected ground motion.

SeismoMatch software program was used for spectral matching of these records using the Response Spectrum function defined in IS 1893 (Part 1): 2016. The advantage of using SeismoMatch is that it is able to compute; the Fast Fourier Transform; the Arias Intensity and the different types of duration (i.e. bracketed, uniform, significant and effective durations) for the accelerograms. It also includes a specific module to perform baseline correction and filtering.

To categorize the strong motion frequency content, various definitions for earthquake frequency content have been proposed by researchers, such as PGA/PGV ratio used by Zhu et al. (1988). It groups the ground motion records into three (3) frequency classes. They are; low, medium/intermediate and high-frequency content ground motion records. The criteria follow that; Low-frequency content for PGA/PGV <0.8, Medium/Intermediate frequency content for 0.8 ≤PGA/PGV ≤1.2 and High-frequency content for 1.2 <PGA/PGV.

Site

The site for this study is Bhuj in the District of Kutch. It is a city in the State of Gujarat, Western India and is located on 23.25o N latitude and 60.67o E longitude. The investigation considered this city because of the history of earthquakes that happened in 1668, 1819, 1956 and the last of 2001, which had a magnitude of about 7.7 (www.mapsofindia.com). It is located in Seismic Zone V according to the seismic zoning map of India. This location is also appropriate because the whole of Gujarat region has an earthquake hazard of different levels from moderate to high. Thus, zones III to V. According to the Geological Survey of India (GSI), the geology of Gujarat State is characterized by hard rock terrain represented by Precambrian mctamorphites and associated intrusives; sedimentaries of Jurassic, Cretaceous and Tertiary Periods; and the traps/flows of Deccan Volcanics of Cretaceous-Eocene age. The city of Bhuj is located on of rocks of the Bhuj formation, which uncomfortably overlies the rocks of the Katrol Formation in the western mainland Kachchh, and forms a 1000 m thick sequence of friable, feldspathic and ferruginous sandstone portraying graded-bedding, ironstone, clays, and trap-pebble conglomerates with many fossiliferous horizons (GSI, 2001).

Design Philosophy

The design philosophy being developed aims to establish as close as possible, the dynamic response of steel moment resisting systems for the design basic earthquake (DBE), as specified by IS 1893, and the second is the maximum credible earthquake (MCE) which is a more severe event than the basic design earthquake. Details of each level are given in section 6.1.3 of IS 1893 (Part 1): 2016. The structure is expected to resist the earthquake load through; the ductility arising from the inelastic material behaviour, appropriate design and detailing as well as the over-strength resulting from the additional reserve strength of the structure. Under the basic design earthquake, the building is expected to respond without significant structural damage though some non-structural damage may occur and aims that structures withstand a major earthquake (MCE) without collapse.

Description of Building Model

The model (see figure 1) is a commercial building and has a total floor area of 1837.77m2. There are three (3) models in all with heights 10-story, 15-story, and 20-storeys. It is narrow with plan dimensions of 55.75m × 30.45m. Its height-to-width ratio is between 4.2:1 and 1.05:1. The floor plan is non-compact with a length-to-width ratio of 1.83:1. The lateral force-resisting system (LFRS) of the model is a framed tube system. The columns are spaced at 4.65m center-to-center distance along the long face of the building and 3m centre-to-centre distance along the short face to resist the lateral forces that are expected from the earthquake ground motion. A stiff perimeter spandrel beam ties the perimeter columns at each floor level to make the system robust. It was, however, anticipated that the spandrel beams could be blocking the wide glass expanses that building owners and occupants would desire and many intrusive internal beams would be required to connect the floor diaphragms to the tube framing. As a result, an internal frame is introduced and at suitable points, connected to the perimeter frame. However, for the connection of the inner frame with the exterior frame, the system would have been the tube-in-tube system. This arrangement permits wider column spacing on the inside of the building, which will ensure the usage of space. The exterior spandrel consist of rolled wide flanged beams and the columns are small wide flanged shapes. Floor systems are made up of 200mm thick M25 concrete slab on metal decking at each floor. The steel properties of structural elements in the numerical model were modelled to conform to clause 2.2.2 of IS 800: 2007. A steel section of E450A is used in the model with a yield strength of 540MPa. The Young's modulus of elasticity of steel was taken as E = 2.0 x 10s N/mm2 (MPa) as stipulated by clause 2.2.4.1of IS 800:2007. The columns in the model are of ISWB 600 steel sections. The exterior beams are ISWB 550 sections and ISWB 500 sections for the interior beams.

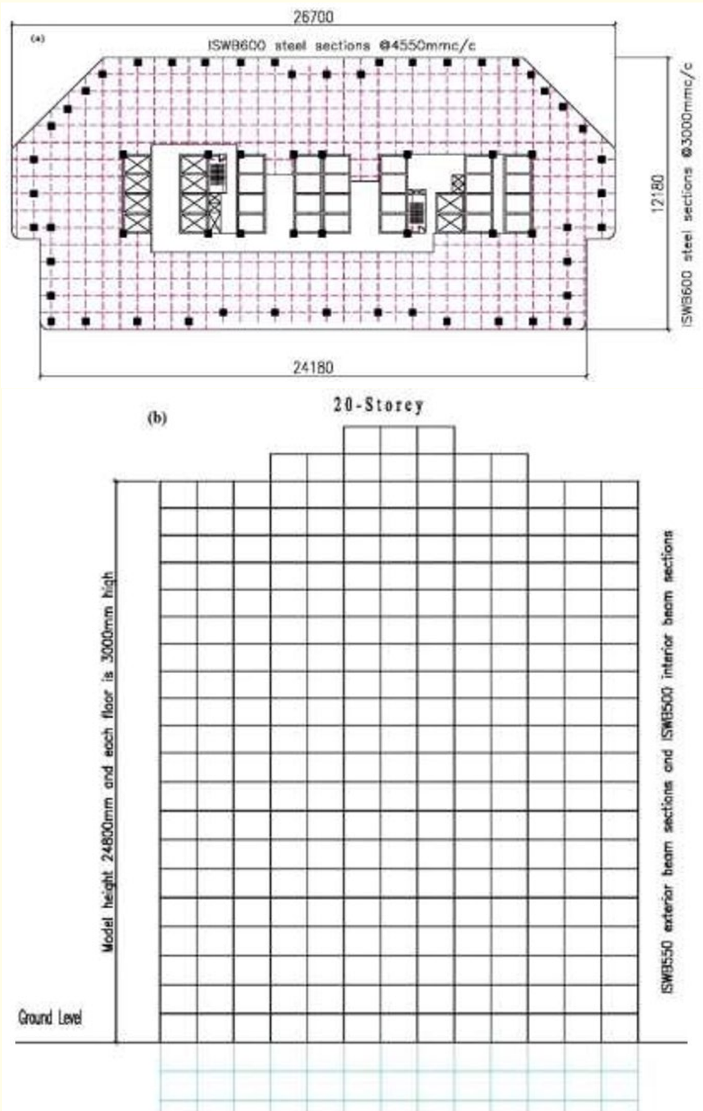


Figure 1: Typical (a) floor plan and (b) elevation of the building's analytical model.

Analysis

In this paper, the Static nonlinear Pushover Analysis method was used for estimating seismic demands and capacities of structural components of the building models. The Dynamic Nonlinear Time History Analysis (THA) was used to complement the evaluation of the building's structural response at a 5% damping ratio. Load factors and load combinations used are in the form 1.2 Dead Load (DL) + 0.5 Live Load (LL) ± 2.5 Earthquake Load (EL) and 0.9 Dead Load (DL) & 2.5 Earthquake Load (EL) and consistent with the provisions of IS 800: 2007 and clause 6.3.4.1 of the IS 1893 (Part 1): 2016. All unit weights of building materials and stored materials were obtained from IS 875 (Part 1): 2013 for computing the dead load and live load from IS 875 (Part 2):2013. Full gravity dead loads were considered along with 50% of live loads since the live load is greater than 3 KN/m2. Live load on a typical floor was assumed 3.5 KN/m2 everywhere, and 17.5 KN/m wall load on both interior and exterior beams. Seismic masses were allotted as joint masses, which were converted from the dead load of the section, concrete slab weight, and 50% of live loads.

Results and Discussion

The static analysis method is deficient in capacity for investigating the response of structures under dynamic load such as earthquakes. Nonetheless, it remains a quick and effective means for estimating the effects of changing the strength and stiffness of a building. Since it works well with short buildings and gives an accurate estimate of seismic demand(D'Ambrisi et al. 2009), the static pushover analysis was performed on the three (3) model, applying a pattern of lateral forces proportional to the first mode displacement shape. The relationship between the base shear and the roof displacement from the analysis are presented in figure 2.

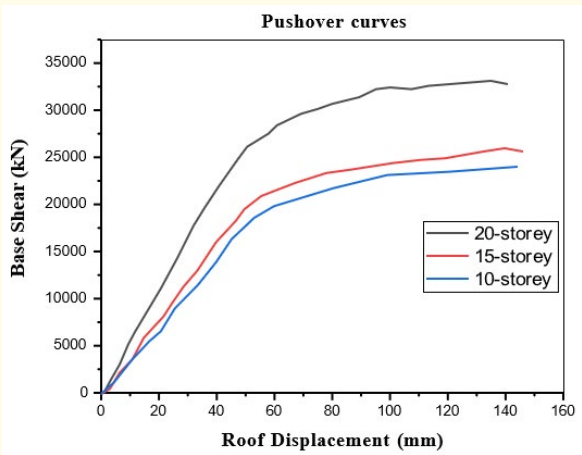


Figure 2: Pushover curves for the building models.

It is observed that the H20 (20-storey) model has greater global initial lateral stiffness and the strength H15 (15-storey) and H10 (10-storey) models. These characteristics have been basically observed to be the same for both X and Y-directions.

The pushover curves are converted into the capacity curve as shown in figure 3. These curves indicate the stiffness of the models is almost the same.

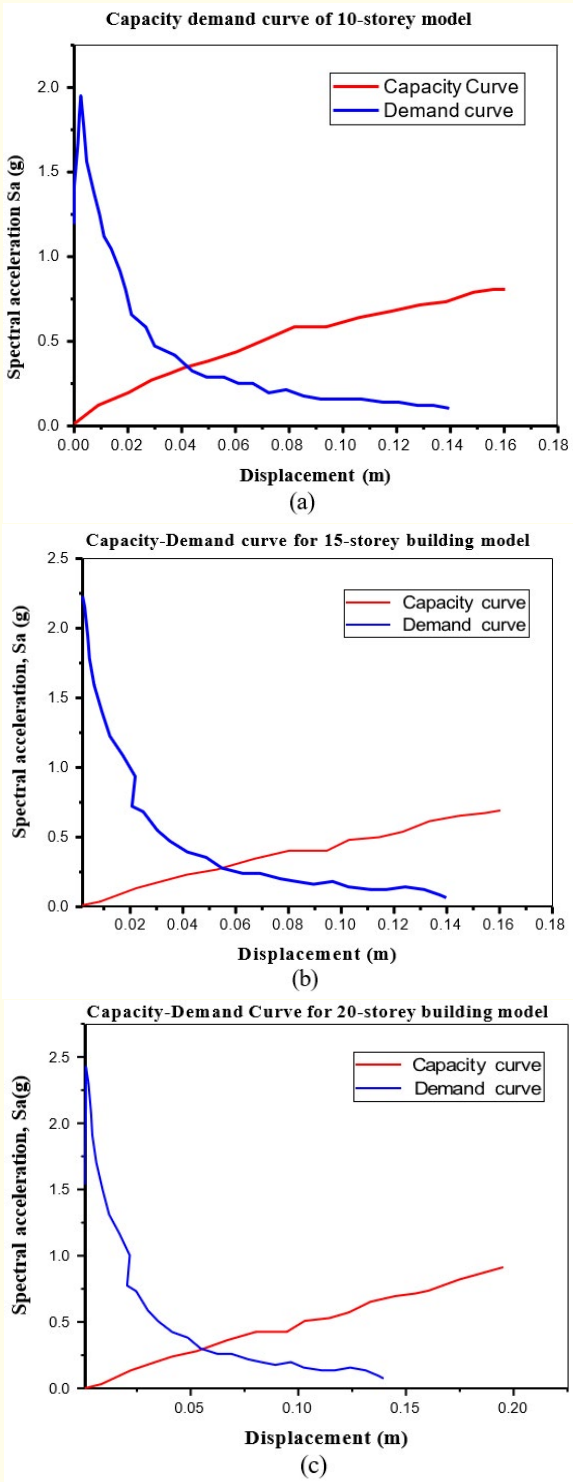


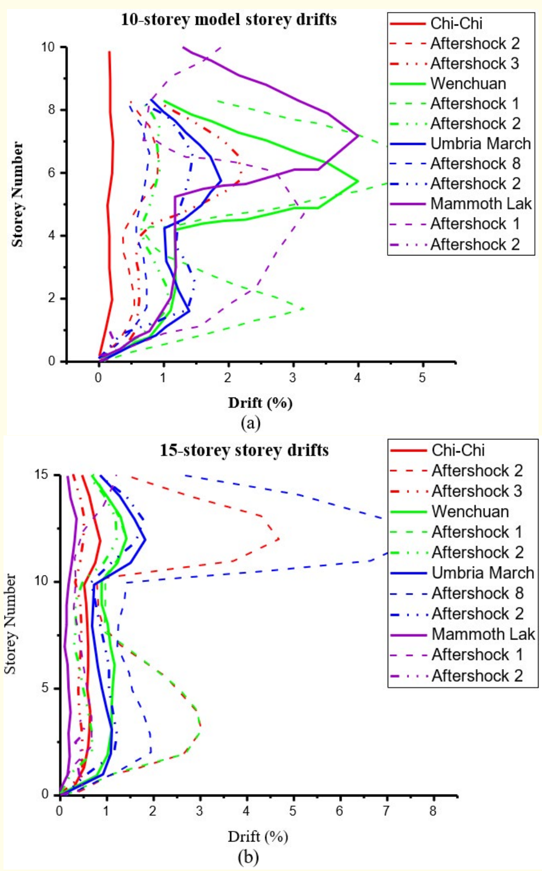
Figure 3: Capacity-Demand Curves (a) H10 (b) H15 and (c) H20.

Responses from Dynamic Nonlinear Time History Analysis

The frequency content of ground motion will always reflect in the dynamic response of a structure even when the ground motions are normalized with respect to the peak ground acceleration. Therefore, enough ground motions are required to appropriately capture the ideal response of the structure. Twelve (12) ground motions comprising of four (4) mainshocks and eight (8) aftershocks were used in the dynamic nonlinear time history analysis. The responses are presented in the section below.

Responses along the building height

Responses along the building height are presented in term of storey drifts (figure 4) and inter-storey drift (figure 5). The drift results show a maximum drift amount of 5% in the H10 model. this drift is caused by aftershock 1 of Wenchuan earthquake which has a high-frequency content with PGA/PGV ratio of 1.53. The drift caused by the Wenchuan mainshock is also high, a 4% drift. In the H15 model, however, the Umbria Marche aftershock 8 caused the maximum drift of 7.5% which also has a high-frequency content with a PGA/PGV ratio of 3.38. it can be observed that this drift is higher than that of the H10 model and this is likely because the PGA/PGV ratio of this aftershock is higher than that of the Wenchuan aftershock 1. In addition, the Chi-Chi aftershock 2 and Wenchuan mainshock both of which have low-frequency contents, caused 4.8% and 3.1% drifts respectively in the H15 model. As is clear from the graphs, the maximum drift of all models is clustered around ¾ of the model heights. In addition, the aggregate of structural drifts of taller structures (H20) is smaller than that of shorter structures (H10 and H15).



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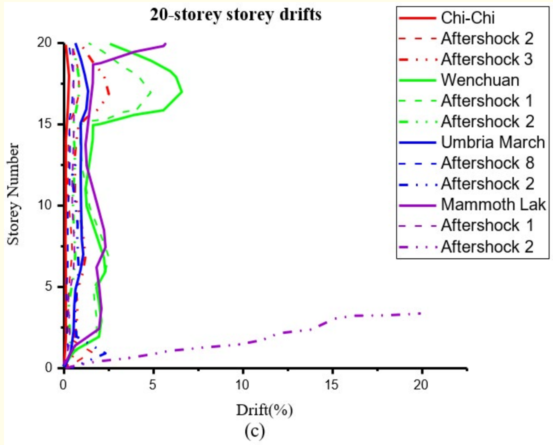
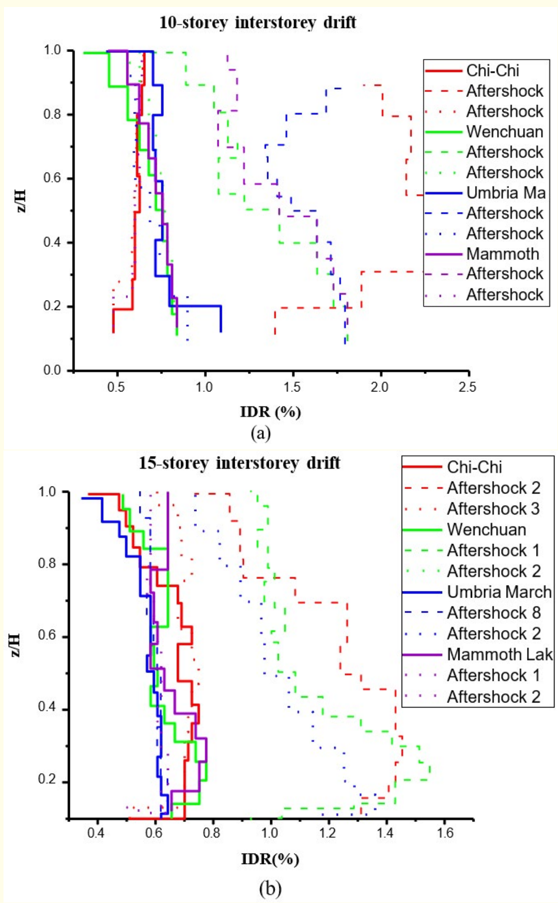


Figure 4: Storey drifts (a) H10 (10-storey model), (b) H15 (15-storey model) and (c) H20 (20-storey model).



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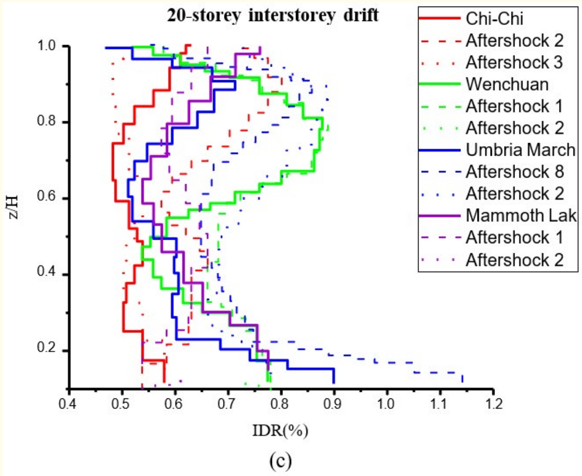

Figure 5: Inter-storey drifts H10 (10-storey model), (b) H15 (15-storey model) and (c) H20 (20-storey model).

The inter-storey drift ratio (IDR), the difference between the roof and respective floor displacements shows that a maximum of 2.5% (see figure 5a) under the influence of Chi-Chi aftershock 2 was observed in the H10 model. The graphs show the possibility of residual drift presence which resulted from the mainshocks.

It would be observed that the inter-storey drifts clustered between 0.5% to 1.15% and is around ¾ of the model height.

Conclusion

The effect of multiple earthquakes of different frequency content on the dynamic response of steel buildings have been investigated using the provisions of IS 1893 (Part 1): 2016 and IS 800: 2007, the Indian Earthquake Resistant Design and Steel Design codes respectively. The authors findings and conclusion are in agreement with the conclusions drawn by Li and Ellingwood (2007) who subjected two steel frames of 9 and 20 storey high, to artificially generated mainshock-aftershock earthquake sequences and Ruiz (2012) who showed that the connection between the dominant period of aftershock that is a measure of its frequency content and the period of vibration of the frame, determines the response of steel moment-resisting frames under real mainshock-aftershock sequences. As observed in figures 4 and 5 above, the frequency content of the aftershocks have an influence on the storey and inter-storey drifts and will ultimately influence the damage pattern that will develop in the building.

References

1. Alliard PM. "Mainshocks and aftershocks sequences database" (2006). http://www.polymtl.ca/structures/en/telecharg/index.php
2. Kim KH., et al. "Seismogenic structures of the 1999 Mw 7.6 Chi-Chi, Taiwan, earthquake and its aftershocks". Tectonophysics 489.1 (2010): 119-127.
3. Chen KC., et al. "Conjugate thrust faulting associated with the 1999 Chi-Chi, Taiwan, earthquake sequence". Geophys. Res. Lett 29.8 (2002): 118-1-4.
4. Jiun-Wei Lai., et al. "Seismic Evaluation and Retrofit of Existing Tall Buildings in California: Case Study of a 35-Story Steel Moment-Resisting Frame Building in San Francisco". PEER Report No.2015/14, Pacific Earthquake Engineering Research Center Headquarters at the University of California, Berkeley (2015).

5. Atzori S., et al. “The 2010-2011 Canterbury, New Zealand, seismic sequence: Multiple source analysis from InSAR data and modeling”. J. Geophys. Res 117.B8 (2012): B08305.
6. Kazama M and T Noda. “Damage statistics (Summary of the 2011 off the Pacific Coast of Tohoku Earthquake damage)”. Soils Found 52.5 (2012): 780-792.
7. Galadini F., et al. “Chapter 2: Seismic source and surface rupture”. In Engineering reconnaissance following the 2016 central Italy earthquakes: Version 2. edited by P. Zimmaro and J. P. Stewart, Rep. No. GEER-0500. 33-50. Hurricane, Sandy: Geotechnical Extreme Events Reconnaissance Association (2017).
8. Agnès Helmstetter, Didier Sornette and Jean-Robert Grasso. “Mainshocks are Aftershocks of Conditional Foreshocks: How do Foreshock Statistical Properties Emerge from Aftershock Laws”. File InvOmoriRevised formatted 29 (2018).
9. Li Q and Ellingwood BR. “Performance evaluation and damage assessment of steel frame buildings under main shock-aftershock earthquakes sequences”. Earthq. Eng. Struct. Dyn 36.3 (2007): 405-427.
10. Sakai K and Murono Y. “Fundamental study on evaluation of main shock-aftershock ground motions for seismic design”. J. Japan Soc. Civ. Eng. Ser A1, JSCE 70.4 (2014): I644-I653.
11. Kagan YY and L Knopoff. “Statistical search for nonrandom features of the seismicity of strong earthquakes”. Phys. Earth Planet. Int 12 (1976): 291-318.
12. Kagan YY and L Knopoff. Statistical study of the occurrence of shallow earthquakes”. Geophys. J. R. Astr. Soc 55 (1978): 67-86.
13. Reasenberg PA. “Foreshocks occurrence before large earthquakes”. J. Geophys. Res 104 (1999): 4755-4768.
14. Jones LM and P Molnar. “Some characteristics of foreshocks and their possible relationship to earthquake prediction and premonitory slip on fault”. J. Geophys. Res 84 (1979): 3596-3608.
15. Yamashina K. “Some empirical rules on foreshocks and earthquake prediction”. in Earthquake Prediction, edited by D. W. Simpson and P. G. Richards, AGU Maurice Ewing series: 4, Washington, D.C (1981): 517-526.
16. Lindh AG and MR Lim. “A clarification, correction and updating of Parkfield California, earthquakes prediction scenarios and response plans”. U.S. Geol. Surv. Open File Rep (1995): 95-695.
17. Abercrombie RE and J Mori. “Occurrence patterns of foreshocks to large earthquakes in the western United States”. Nature 381 (1996): 303-307.
18. Michael AJ and LM Jones. “Seismicity alert probabilities at Parkfield, California”. revisited, Bull. Seis. Soc. Am 88 (1998): 117-130.
19. Sunasaka Y and Kiremidjian AS. “A method for structural safety evaluation under main shock- aftershock earthquake sequences”. Report No. 105, The John A. Blume Earthquake Engineering Center, United States (1993).
20. Von Seggern D, SS Alexander and C-B Baag. “Seismicity parameters preceding moderate to major earthquakes”. J. Geophys. Res 86 (1981): 9325-9351.
21. Shaw BE. “Generalized Omori law for aftershocks and foreshocks from a simple dynamics”. Geophys. Res. Letts 10 (1993): 907-910.
22. Carlos Mendoza and Stephen H Hartzel. “Aftershock Patterns and Main Shock Faulting”. Bulletin of the Seismological Society of America 78.4 (1998): 1438-1449.
23. Zhang S, Wang G and Sa W. “Damage evaluation of concrete gravity dams under mainshock-aftershock seismic sequences”. Soil Dyn. Earthq. Eng 50.1 (2013): 16-27.
24. Zhao B and Taucer F. “Performance of infrastructure during the May 12, 2008, Wenchuan earthquake in China”. J. Earthq. Eng 14.4 (2010): 578-600.
25. Li Y, Song R and Lindt JWVD. “Collapse fragility of steel structures subjected to earthquake mainshock-aftershock sequences”. J. Struct. Eng 140.12 (2014): 04014095-1-10.
26. Yeo GL and Cornell CA. “A probabilistic framework for quantification of aftershock ground motion hazard in California: Methodology and parametric study”. Earthq. Eng. Struct. Dyn 38.1 (2009): 45-60.
27. Bruneau Michel., et al. “Preliminary Report on Steel Building Damage from The Darfield Earthquake of September 4, 2010”. Bulletin of The New Zealand Society for Earthquake Engineering 43.4 (2010): 351-359.

28. Faisal A, Majid TA and Hatzigeorgiou GD. "Investigation of story ductility demands of inelastic concrete frames subjected to repeated earthquakes". Soil Dyn. Earthq. Eng 44.1 (2013): 42-53.
29. Izuru Okawa., et al. "Recorded Responses of Building Structures during the 2011 Tohoku-Oki Earthquake with Some Implications for Design Practice". Earthquake Spectra 29.S1 (2013): S245-S264.
30. Stephen A Mahin. "Lessons from damage to steel buildings during the Northridge earthquake". Engineering Structures 20.4-6 (1998): 261-270.
31. Saman Yaghmaei-Sabegh and Parinaz Panjehbashi-Aghdam. "Damage assessment of adjacent fixed- and isolated-base buildings under multiple ground motions". Journal of Earthquake Engineering (2018).
32. Efraimiadou S, Hatzigeorgiou GD and Beskos DE. "Structural pounding between adjacent buildings Accepted Manuscript subjected to strong ground motions". Part I: The effect of different structures arrangement. Earthquake Engineering & Structural Dynamics 42.10 (2013): 1509-1528.
33. Zhanzhan Tang, Xu Xie and Tong Wang. "Residual seismic performance of steel bridges under earthquake sequence". Earthquakes and Structures 11.4 (2016): 649-664.
34. Elnashai AS, Bommer JJ and Martinez-Pereira A. "Engineering implications of strong-motion records from recent earthquakes". In: Proceedings of 11th European conference on earthquake engineering (1998).
35. Amadio C, Fragiacomo M and Rajgelj S. "The effects of repeated earthquake ground motions on the non-linear response of SDOF systems". Earthq. Eng. Struct. Dyn 32.2 (2003): 291-308.
36. Hatzigeorgiou GD. "Ductility demand spectra for multiple near-and far-fault earthquakes". Soil Dynamics and Earthquake Engineering 30.4 (2010a): 170-183.
37. Hatzigeorgiou GD. "Behavior factors for nonlinear structures subjected to multiple near-fault earthquakes". Computers & Structures 88.5 (2010b): 309-321.
38. Hatzigeorgiou GD. "Ductility demands control under repeated earthquakes using appropriate force reduction factors". Journal of Earthquake and Tsunami 4.03 (2010c): 231-250.
39. Yaghmaei-Sabegh S and Ruiz-García J. "Nonlinear response analysis of SDOF systems subjected to doublet earthquake ground motions: A case study on 2012 Varzaghan-Ahar events". Engineering Structures 110 (2016): 281-292.
40. Hatzigeorgiou GD and Liolios AA. "Nonlinear behaviour of RC frames under repeated strong ground motions". Soil Dynamics and Earthquake Engineering 30.10 (2010): 1010-1025.
41. Ruiz-García J and JC Negrete-Manriquez. "Evaluation of drift demands in existing steel frames under as-recorded far-field and near-fault main shock-aftershock seismic sequences". Eng. Struct 33.2 (2011): 621-634.
42. Indian standard 1893 (2016), Criteria for Earthquake Resistant Design of Structures Part 1 General Provisions and Buildings (Sixth Revision), Bureau of Indian Standards, Manak Bhavan. 9 Bahadur Shah Zafar Marg, New Delhi 110002 (2016).
43. Uang C-M., et al. "Seismic response of an instrumented 13-story steel frame building damaged in the 1994 Northridge earthquake". Earthquake Spectra 13.1 (1997): 131-149.
44. Laugesen R. (201). "Rock steady".
45. Ramhormozian S., et al. Dynamic Performance Analysis and System Identification (SI) of a Low Damage Multi-Storey Structural Steel Building (2018). under two Moderately Severe Earthquake Events using Structural Health Monitoring (SHM) Data". NZSEE Conference (2018).
46. Yanev. Dabovski Ch., et al. "A geodynamic model of the Alpine magmatism in Bulgaria". Geologica Balc 21.4 (1991): 3-15.
47. Tremblay R., et al. "Performance of steel structures during the 1994 Northridge earthquake". Canadian Journal of Civil Engineering 22.2 (1995): 338-360.
48. American Institute of Steel Construction, Inc., Northridge Steel Update I (1994a).
49. MacRae GA., et al. "Lessons from steel structures in Christchurch earthquakes". Proceedings, 8th International Conference on Behavior of Steel Structures in Seismic Areas (STESSA), Shanghai, China (2015).
50. Taichiro Okazaki., et al. "Damage to Steel Buildings Observed after the 2011 Tohoku-Oki Earthquake". Earthquake Spectra 29.S1 (2013): S219-S243.

51. Mithila Verma and Brijesh K Bansal. "Active fault research in India: achievements and future perspective". Geomatics, Natural Hazards and Risk 7.1 (2016) 65-84.
52. Yeats R. "Active faults of the world". Cambridge University Press (2012): 634.
53. Biswas SK and Khattri KN. "A geological study of earthquakes in Kachchh, Gujarat". India. J Geol Soc India 60 (2002): 131-142.
54. PEER. "Pacific Earthquake Engineering Research Centre: PEER Motion Database". University of California.
55. Zhu TJ, Tso WK and Heidebrecht AC. "Effect of peak ground a/v ratio on structural damage". Journal of Structural Engineering (ASCE) 114 (1988): 1019-1037.
56. Seismo Soft. SeismoStruct - A computer program for static and dynamic nonlinear analysis of framed structures (2003). https://seismosoft.com.
57. Geological Survey of India (2001). https://mines.gov.in/writereaddata/UploadFile/chap8.pdf
58. Indian standard 800 (2007). General Construction in Steel — Code of Practice (Third Revision), Bureau of Indian Standards, Manak Bhavan. 9 Bahadur Shah Zafar Marg, New Delhi 110002 (2007).
59. Indian standard 875 (1987 Reaffirmed 2008), Code of Practice for Design Loads (Other than Earthquake) for Buildings and Structures Part 1 Dead Loads - Unit Weights of Building Materials and Stored Materials (Second Revision), Bureau of Indian Standards, Manak Bhavan. 9 Bahadur Shah Zafar Marg, New Delhi 110002 (2010).
60. Indian standard 875 (1987 Reaffirmed 2008), Code of Practice for Design Loads (Other than Earthquake) for Buildings and Structures Part 2 -Imposed Loads (Second Revision), Bureau of Indian Standards, Manak Bhavan. 9 Bahadur Shah Zafar Marg, New Delhi 110002 (2010).
61. Indian standard 875 (2015+Amendment 2016), Design Loads (Other than Earthquake) for Buildings and Structures - Code of Practice Part 3 Wind Loads (Third Revision), Bureau of Indian Standards, Manak Bhavan. 9 Bahadur Shah Zafar Marg, New Delhi 110002 (2015).
62. Indian standard 456 (2000), Plain and Reinforced Concrete-Code of Practice, (Fourth Revision), Bureau of Indian Standards, Manak Bhavan. 9 Bahadur Shah Zafar Marg, New Delhi 110002 (2007).
63. Quanwang Li and Bruce R Ellingwood. "Performance evaluation and damage assessment of steel frame buildings under main-shock-aftershock earthquake sequences". Earthquake Engineering and Structural Dynamics 36 (2007): 405-427.
64. Ruiz-García J. "Mainshock-aftershock ground motion features and their influence in building's seismic response". J Earth Eng 16.5 (2012): 719-37.

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