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Performance-Based Seismic Assessment of a Steel Moment-Resisting Frame Designed as per IS 18168:2023 Using Incremental Dynamic Analysis

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Abstract - *Steel moment-resisting frames are widely used for multi-storey buildings, but force-based seismic design checks the structure at a single design level and does not show how likely a performance limit is to be exceeded as shaking becomes stronger. This paper presents a performance-based seismic assessment of a G+7 steel moment-resisting frame of 20.1 m by 20.1 m plan, located in Seismic Zone III (Mumbai), using Incremental Dynamic Analysis (IDA). The building was modelled in ETABS with Fe 350 steel. Five Indian ground-motion records from the Chamoli (1999), India-Burma (1988) and Uttarkashi (1991) earthquakes were selected using the FEMA P695 criteria, normalised by the spectral acceleration at the fundamental period, and scaled in steps under service-level load combinations. The fundamental period from modal analysis was 2.379 s. The maximum storey drift was recorded for each scale factor, and the scale factor giving a drift of 0.4 per cent was found by interpolation for each record. These scale factors ranged from 6.65 to 28.34, showing strong record-to-record variability. A lognormal fragility curve fitted to them gave a median scale factor of 16.57 at 50 per cent probability, which was used for the final design as per IS 800:2007 and IS 18168:2023.*

Key Words: Incremental dynamic analysis, Steel moment-resisting frame, Fragility curve, Storey drift, FEMA P695, IS 18168:2023, Performance-based seismic design

1. INTRODUCTION

Earthquakes are among the most damaging natural hazards for the built environment, and a large part of India lies in regions of moderate to high seismicity. Steel moment-resisting frames (SMRFs) are a popular structural system for multi-storey buildings because of their high strength-to-weight ratio, ductile behaviour and open floor plans. Lateral resistance is provided by flexure in the beams and columns, and the energy of an earthquake is expected to be dissipated through controlled yielding, mainly in the beams.

Conventional seismic design is force-based: the structure is proportioned for a reduced design force and checked against a drift limit at that single level of loading. Such a check does not reveal how the building behaves when shaking exceeds the design level, nor how likely it is that a particular performance limit will be crossed. Incremental Dynamic Analysis (IDA) addresses this by analyzing a structural model under several ground motions, each scaled to progressively higher intensities, so that demand can be followed as intensity grows [1].

Most published IDA studies of steel frames are based on non-Indian design codes, non-Indian ground motions and research software such as OpenSees. The Indian steel design framework has recently been extended by IS 18168:2023 [2], which works together with IS 800:2007 [3] and IS 1893 (Part 1):2016 [4]. There is therefore a need to see how a mid-rise steel moment frame designed in the Indian context responds to Indian strong-motion records using tools that practising engineers already use, such as ETABS. A fragility-based scale factor, taken at 50 per cent probability of reaching a chosen drift limit, can then serve as a rational basis for the final design.

1.1 Aim and Objectives

The aim of this work is to carry out a performance-based seismic assessment of a G+7 steel moment-resisting frame using IDA and to use the result in its design. The objectives are: (i) to model a G+7 steel SMRF of 20.1 m × 20.1 m plan in ETABS for Seismic Zone III (Mumbai) and obtain its fundamental period; (ii) to select five ground-motion records using the FEMA P695 criteria and normalise them with the spectral acceleration at the fundamental period; (iii) to scale each record in steps and record the maximum storey drift until a drift of 0.4 per cent is exceeded; (iv) to find, by interpolation, the scale factor that produces exactly 0.4 per cent drift for each record; (v) to construct a fragility curve and identify the scale factor at 50 per cent probability; and (vi) to design the structure as per IS 800:2007 and IS 18168:2023 using this scale factor.

1.2 Scope and Limitations

The study is limited to one building configuration, one lateral-load-resisting system and five ground-motion records. The drift limit of 0.4 per cent relates to the serviceability-level storey drift limit of IS 1893 [4] and is not a collapse limit. Soil–structure interaction, infill-wall stiffness beyond the wall loads, and connection

deterioration are not modelled. The results are therefore indicative of the drift-based capacity of this building under the chosen records and are not a general conclusion for all steel frames.

2. LITERATURE REVIEW

2.1 Performance-Based Design and IDA

The idea of linking building performance levels to earthquake hazard levels was set out in the SEAOC Vision 2000 document [5], and a probabilistic framework chaining hazard, structural response, damage and loss was later proposed by the Pacific Earthquake Engineering Research Center [6]. After the brittle failure of welded connections in the 1994 Northridge earthquake, the FEMA-350 guidelines [7] moved steel moment-frame design towards a reliability-based format, whose probabilistic basis was described by Cornell et al. [8]. Nonlinear static (pushover) analysis is simpler, but it can mislead when higher modes matter and cannot reflect record-to-record variability [9]; Shao and Tang [10] found that IDA results for steel frame–steel plate shear wall buildings lay between pushover curves for two different load patterns.

Shome et al. [11] showed that scaling a record is acceptable for estimating nonlinear response if the records are chosen suitably. Vamvatsikos and Cornell [1] then defined IDA as a procedure in which a model is subjected to records scaled to many intensity levels, giving curves of an intensity measure (IM) against a damage measure (DM). They recommended the 5 per cent-damped spectral acceleration at the first-mode period, $Sa(T_1)$, as the IM and the maximum interstorey drift ratio as the DM, and advised using several records summarised by the median and the 16th and 84th percentile curves. Practical guidance on IDA was given in a companion paper [12]. For record selection, FEMA P695 [13] defines a far-field set of 22 record pairs with criteria on magnitude, distance, site conditions and amplitude, and Baker and Cornell [14] showed that spectral shape also affects the response. Indian records are available from the PESMOS database [15].

2.2 Applications of IDA to Frames

Ghasemi Jouneghani and Haghollahi [16] used IDA on three-, five- and seven-storey moment frames with elliptic braces, modelled in OpenSees under 12 far-field records, and reported an increase of about 30 per cent in average spectral-acceleration capacity compared with conventional frames. Asgarian and Jalaeefar [17] applied IDA to 18 concentrically braced frames designed to three editions of the Iranian seismic code and found that the later editions performed better; a follow-up study of near-field motions [18] found that $Sa(T_1)$ gave the least dispersion among four intensity measures. Vahdani et al. [19] concluded from IDA of 3- to 15-storey steel moment frames that the near-fault modification factor of the Iranian code does not capture near-fault effects accurately. Ma et al. [20] obtained response modification factors of about 8.2 to 10.1 for high-strength steel eccentrically braced frames, and Li et al. [21] showed that reserve stiffness strongly governs the collapse ductility of low-ductility braced frames.

Javanpour and Zarfam [22] compared six-storey steel moment and cross-braced frames and found that the moment frame was less vulnerable and that the choice of IM and DM changed the estimated vulnerability considerably. Wang et al. [23] proposed performance levels for steel-reinforced-concrete frames from reductions in the slope of the IDA curve, Zarfam and Mofid [24] examined modal IDA as a cheaper approximation, and Sadeghpour and Ozay [25] used IDA to judge the adequacy of earlier Iranian codes for reinforced concrete frames. For fragility, the IM values at which the limit state is reached are commonly fitted with a lognormal distribution [26]; deterioration models [27], [28] are needed for collapse-level IDA but are beyond the scope of this work.

2.3 Research Gap

Limit-state drifts of 2 to 10 per cent are common in the reviewed studies [16], while the 0.4 per cent code-level drift is rarely examined. Most studies are based on Iranian, Chinese or American design practice and records, and no IDA-based study of a steel moment frame designed to IS 18168:2023 with Indian ground motions was found. The gap addressed here is a drift-based IDA assessment of a mid-rise steel moment frame, using a commonly available design package and Indian ground motions, with the result fed back into the design.

3. METHODOLOGY

The procedure follows the IDA framework of Vamvatsikos and Cornell [1], simplified for use with a commercial analysis package. The main steps are shown in Fig -1 and described in the sections below.

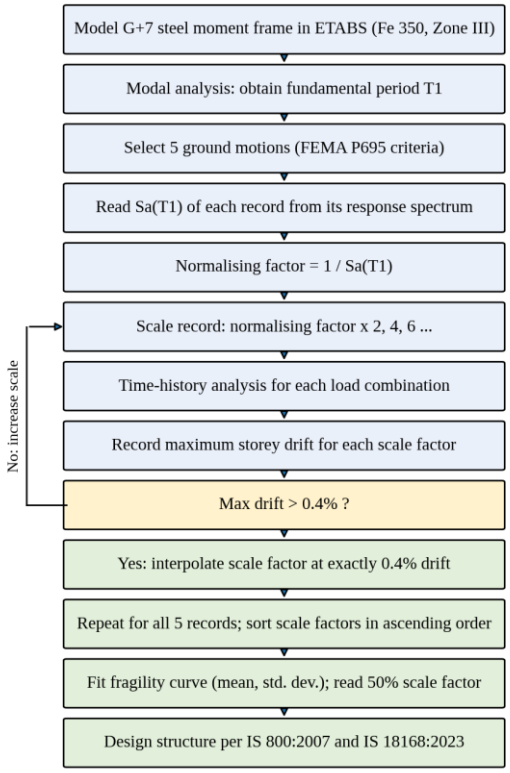


Fig -1: Methodology adopted for the IDA-based assessment

3.1 Structural Model

A G+7 steel moment-resisting frame building of 20.1 m × 20.1 m plan (3 × 3 bays of 6.7 m, Fig -2) was modelled in ETABS. Fe 350 steel was used for all members, with ISMB 250 beams and columns made of two ISMB 250 sections placed toe to toe. The floor is a 100 mm deck slab. The building is located in Seismic Zone III (Mumbai) on soil type II, with an importance factor of 1, a response reduction factor of 5 and 5 per cent damping. Dead load (DL), live load (LL), wind load (WL) and earthquake load (EQ) were defined as load patterns. Interior walls of 150 mm and exterior walls of 250 mm thickness were applied as line loads, and a live load of 3 kN/m^2 was applied on the floors. The wind load used a basic wind speed of 44 m/s and terrain category 3. The input data are summarised in Table -1.

Table -1: Building, material and load data

Parameter	Value
Structural system	Steel moment-resisting frame
Number of storeys	G+7
Plan dimensions	20.1 m × 20.1 m (3 × 3 bays of 6.7 m)
Structural steel	Fe 350 (f_y = 350 MPa)
Elastic modulus of steel	2 × 10^5 MPa
Beams and columns	ISMB 250; 2-ISMB 250 toe to toe
Floor slab	100 mm deck slab
Wall thickness	150 mm interior, 250 mm exterior
Live load	3 kN/m^2
Location, seismic zone	Mumbai, Zone III (IS 1893 (Part 1):2016)
Soil type, importance factor, R	Type II, 1, 5
Wind load	Basic wind speed 44 m/s, terrain category 3, importance factor 1
Damping ratio	5 %
Uncertainty factors	E350: R_y = 1.2, R_u = 1.1

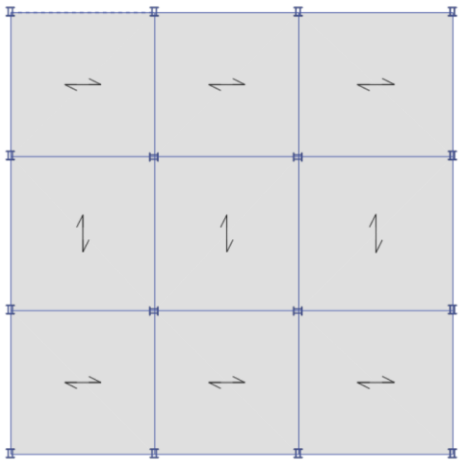


Fig -2: Typical floor plan of the ETABS model (3 × 3 bays)

3.2 Ground Motions and Normalisation

Five ground-motion records were selected using the criteria of FEMA P695 [13] and taken from a strong-motion database [15]. They are from the Chamoli (Gopeshwar) earthquake of 28 March 1999, the India–Burma earthquake of 6 August 1988 (stations Bhaigao and Dipun) and the Uttarkashi earthquake of 19 October 1991 (stations Bhatwari and Uttarkashi). The intensity measure is the 5 per cent-damped spectral acceleration at the fundamental period, Sa(T_1) [1]. The period T_1 was obtained from the modal analysis of the ETABS model. For each record, the response spectrum was generated, Sa(T_1) was read from it, and the record was multiplied by 1/Sa(T_1) so that all records have the same spectral acceleration at T_1 before scaling.

3.3 Scaling and Analysis

Each normalised record was multiplied by increasing scale factors, in steps of 2, times the normalising factor. For each scale factor a time-history analysis was run for the service load combinations DL+LL ± THAX, DL+LL ± THAY, DL ± THAX and DL ± THAY, where THAX and THAY are the time-history cases in the two plan directions. The maximum storey drift was recorded for each scale factor. Scaling was continued until the drift exceeded 0.4 per cent and was then carried further for each record.

3.4 Damage Measure and Interpolation

The damage measure is the maximum storey drift ratio, expressed in per cent [1]. The limit state is a drift of 0.4 per cent, which corresponds to the storey drift limit of IS 1893 (Part 1):2016 [4]. This is a serviceability-type limit and is considerably smaller than the collapse-prevention drifts used in many IDA studies [16]. For each record, the two consecutive scale factors SF_1 and SF_2 that bracket the drift of 0.4 per cent (drifts $\delta_1 < 0.4$ per cent $< \delta_2$) were identified, and the scale factor at exactly 0.4 per cent was found by linear interpolation:

$$SF_{0.4} = SF_1 + (0.4 - \delta_1)(SF_2 - SF_1) / (\delta_2 - \delta_1) \qquad (1)$$

3.5 Fragility Curve and Design

The five interpolated scale factors were arranged in ascending order and a lognormal fragility curve [26] was fitted:

$$P(SF \le s) = \Phi[(\ln s - \ln \theta) / \beta] \qquad (2)$$

where Φ is the standard normal cumulative distribution function, θ is the median and β the logarithmic standard deviation. $\ln \theta$ was taken as the mean of the natural logarithms of the scale factors and β as their sample standard deviation $(n - 1)$. The scale factor at 50 per cent probability equals θ. It was used in the design of the structure according to IS 800:2007 [3] for strength and stability and the ductile-detailing provisions of IS 18168:2023 [2].

3.6 Design Considerations

The following considerations were adopted in the design as per IS 800:2007 [3] and IS 18168:2023 [2].

(a) The expected yield and tensile stresses of steel were taken as $R_y f_y$ and $R_u f_u$, where R_y and R_u are the material strength uncertainty factors of Table 1 of IS 18168:2023 [2]; for E350 steel $R_y = 1.2$ and $R_u = 1.1$.

(b) The load combinations were taken from Table 4 of IS 800:2007 [3], with the earthquake load (EL) as per IS 1893 (Part 1):2016 [4]. In addition, the combinations of Clause 5.5 of IS 18168:2023 [2] were considered:

$$1.2\ DL + \lambda\ LL \pm 1.0\ EL_m \qquad (3)$$

$$0.9\ DL \pm 1.0\ EL_m \qquad (4)$$

where $EL_m = \Omega \times EL$ is the estimated maximum equivalent earthquake load, Ω is the overstrength factor (3.0 for SMRFs) and λ is the partial safety factor for live load (0.25 for live load up to 3.0 kN/m^2).

(c) The beam and column sections were checked against the limiting width-to-thickness ratios of Table 2 of IS 18168:2023 [2], with $\varepsilon = (250/f_y)^{1/2}$ and $C_a = P_u/(P_y/\gamma_{m0})$. For I-sections the outstanding flange limit is $9.0\varepsilon/\sqrt{R_y}$ for beams and columns, and the web limit is $44.5\varepsilon/\sqrt{R_y}$ for beams. For columns the web limit is $(72.7\varepsilon/\sqrt{R_y})(1 - 1.04C_a)$ for $C_a \le 0.118$ and $(24.9\varepsilon/\sqrt{R_y})(2.68 - C_a)$ but not less than $44.4\varepsilon/\sqrt{R_y}$ for $C_a > 0.118$. For $f_y = 350$ MPa and $R_y = 1.2$ these give 6.94 (flange) and 34.33 (beam web), and $56.09(1 - 1.04C_a)$ or $19.21(2.68 - C_a) \ge 34.26$ for the column web.

4. RESULTS AND DISCUSSION

4.1 Modal Analysis

The modal analysis of the model gave a fundamental period $T_1 = 2.379$ s. The first mode is translational in the X direction with a mass participation ratio of 0.8182, the second mode (2.352 s) is translational in the Y direction with a ratio of 0.8156, and the third mode has a period of 2.096 s. The spectral acceleration of each record at T_1 and the resulting normalising factors are given in Table -2.

4.2 Storey Drift versus Scale Factor

Fig -3 shows the maximum storey drift obtained for each scale factor and record. At a scale factor of 10 the maximum storey drifts are 0.204, 0.339, 0.605, 0.068 and 0.058 per cent for GM1 to GM5. The Uttarkashi-station record (GM3) is therefore the most damaging for this building at the drift level considered, and the two India–Burma records (GM4, GM5) are the least damaging. For GM4 and GM5 the drift grows slowly and almost linearly up to a scale factor of 28 (0.17 and 0.16 per cent) and then rises about tenfold between scale factors of 28 and 30 (to 1.55 and 1.64 per cent).

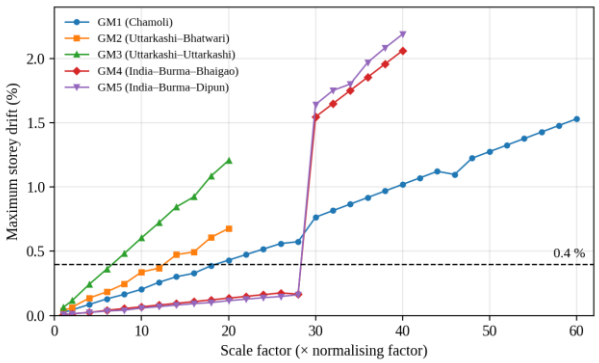


Fig -3: Maximum storey drift versus scale factor for the five ground motions

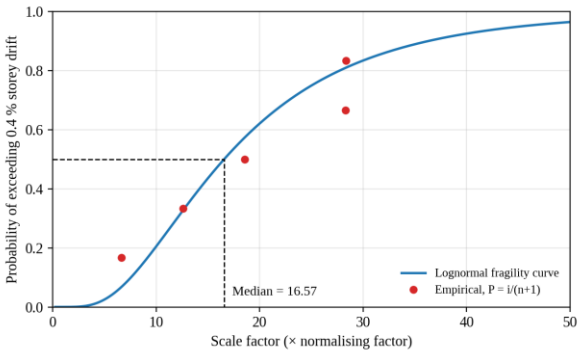


Fig -4: Lognormal fragility curve for a maximum storey drift of 0.4 per cent

4.3 Scale Factor at 0.4 per cent Drift

The scale factors at exactly 0.4 per cent drift, obtained from Equation (1), are given in Table -2. They range from 6.65 (GM3) to 28.34 (GM4), a ratio of about 4.3. Because the 0.4 per cent crossing of GM4 and GM5 lies inside the sharp jump between scale factors of 28 and 30, their interpolated values (28.34 and 28.32) are approximate, and additional runs at intermediate scale factors would refine them.

Table -2: Ground-motion records and scale factor at 0.4 per cent drift

No.	Earthquake (station), date	Sa(T_1) (g)	SF at 0.4 %
GM1	Chamoli (Gopeshwar), 28-03-1999	0.05596	18.58
GM2	Uttarkashi (Bhatwari), 19-10-1991	0.04960	12.60
GM3	Uttarkashi (Uttarkashi), 19-10-1991	0.01850	6.65
GM4	India–Burma (Bhaigao), 06-08-1988	0.01519	28.34
GM5	India–Burma (Dipun), 06-08-1988	0.03168	28.32

4.4 Fragility Curve and Design

The scale factors were sorted in ascending order (6.65, 12.60, 18.58, 28.32, 28.34) and a lognormal distribution was fitted (Fig -4). The natural logarithms of the scale factors have a mean of 2.8075 and a sample standard deviation β of 0.6119, so the median scale factor is θ = exp(2.8075) = 16.57. According to the fitted curve, the probability that the 0.4 per cent drift limit is reached is about 20 per cent at a scale factor of 10 and 62 per cent at 20. The large β reflects strong record-to-record variability, and with only five records the estimate itself is uncertain.

The structure was designed with the 50 per cent scale factor of 16.57 (that is, 16.57 times the normalising factor $1/Sa(T_1)$) as described in Section 3.5. The final sections are ISMB 250 for beams and 2-ISMB 250 (toe to toe) for columns.

5. CONCLUSIONS

A G+7 steel moment-resisting frame of 20.1 m × 20.1 m plan in Seismic Zone III (Mumbai) was assessed by IDA using ETABS and five Indian ground-motion records selected with the FEMA P695 criteria. The following conclusions are drawn:

1) The fundamental period of the structure is 2.379 s; the first mode is translational in the X direction (mass participation ratio 0.8182) and the second (2.352 s) is translational in the Y direction (0.8156).
2) The scale factors that produce a maximum storey drift of 0.4 per cent range from 6.65 to 28.34 for the five records, which shows large record-to-record variability. The Uttarkashi-station record is the most damaging and the India–Burma records are the least damaging.
3) A lognormal fit to the five scale factors gives a logarithmic mean of 2.8075 and β = 0.6119; the scale factor at 50 per cent probability is 16.57, and this value was used for the design as per IS 800:2007 and IS 18168:2023 (beams ISMB 250, columns 2-ISMB 250 toe to toe).
4) The procedure links ground-motion intensity to a drift limit using ETABS in a simple way. The estimates for the India–Burma records depend on a sharp jump in response and the dispersion from five records is uncertain, so more records and finer scale steps are advisable.

Future work may use a larger set of records (the 22 pairs of FEMA P695 or 10 to 20 Indian records), fully nonlinear IDA with plastic hinges or fibre elements extended to Life Safety and Collapse Prevention limit states, maximum-

likelihood fitting of the fragility function [26], the effect of soil conditions, near-field motions and spectral shape [14], and frames of different heights or braced and dual systems.

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