



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 (SMRFs) are widely used for multi-storey buildings, yet code-based design checks the structure at one design-level force and says little about how often a given performance limit would be exceeded as shaking becomes stronger. This project carries out a performance-based seismic assessment of a G+7 steel SMRF of 20.1 m × 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, 150 mm interior and 250 mm exterior walls and a live load of 3 kN/m². Five ground-motion records from the Chamoli (1999), India–Burma (1988) and Uttarkashi (1991) earthquakes were chosen following the selection criteria of FEMA P695. The fundamental period obtained from modal analysis was used to read the spectral acceleration $S_a(T_1)$ of each record, and each record was normalised by $1/S_a(T_1)$. The normalised records were then scaled in steps of 2 and applied as time histories under the service load combinations $DL+LL \pm THAX/THAY$ and $DL \pm THAX/THAY$. The maximum storey drift was recorded for every scale factor until it exceeded 0.4 % (and further for each record), and the scale factor producing exactly 0.4 % drift was found by linear interpolation for each record. The five scale factors were arranged in ascending order, and a lognormal fragility curve was built from the mean and standard deviation of their natural logarithms. The fundamental period of the model from modal analysis was $T_1 = 2.379$ s. The scale factors that produce a maximum storey drift of 0.4 % were 6.65, 12.60, 18.58, 28.32 and 28.34 for the five records. The lognormal fit gave $\lambda = 2.8075$ and $\beta = 0.6119$, so the scale factor at 50% probability (the median) is 16.57. The structure was finally designed according to IS 800:2007 and the ductile-detailing provisions of IS 18168:2023 using the scale factor corresponding to 50 % probability. The report also reviews published IDA studies on steel, composite and concrete frames to place the work in context. Layout of the report. Chapter 1 introduces the problem, aim and scope. Chapter 2 reviews the literature on performance-based design, IDA, ground-motion selection and fragility. Chapter 3 describes the methodology. Chapter 4 describes the structural model, loads and design data. Chapter 5 presents the results and discussion. Chapter 6 gives the conclusions and the scope for future work..

Keywords- Incremental dynamic analysis; steel moment-resisting frame; fragility curve; storey drift; FEMA P695; IS 18168:2023; performance-based seismic design.



I. INTRODUCTION

1. Background

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. They have a high strength-to-weight ratio, can deform in a ductile manner, and keep the floor plan free of diagonal braces and shear walls. Lateral resistance is provided by flexure in the beams and columns and by rigid beam-column joints, 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 then 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. Performance-based earthquake engineering tries to answer these questions by describing structural response over a range of shaking intensities. Incremental Dynamic Analysis (IDA) is one of the most widely used tools for this purpose: a structural model is analysed under several ground motions, each scaled to progressively higher intensities, so that demand can be followed as intensity grows [1].

2. Motivation

Most published IDA studies of steel frames are based on non-Indian design codes, non-Indian ground motions and research software such as OpenSees (Chapter 2). 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 % probability of reaching a chosen drift limit, can then serve as a rational basis for the final design.



3. Aim and Objectives

The aim of this project 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 specific objectives are:

- 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;
- to select five ground-motion records using the criteria of FEMA P695 and normalise them with the spectral acceleration at the fundamental period;
- to scale each record in steps and record the maximum storey drift for each scale factor until a drift of 0.4 % is exceeded;
- to determine, by interpolation, the scale factor that produces exactly 0.4 % drift for each record;
- to construct a fragility curve from these scale factors and identify the scale factor at 50 % probability; and
- to design the structure according to IS 800:2007 and IS 18168:2023 using this scale factor.

4. 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 % 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 should therefore be read as indicative of the drift-based capacity of this building under the chosen records and not as a general conclusion for all steel frames.

5. Organisation of the Report

The report is arranged in six chapters. Chapter 1 gives the background, aim and scope. Chapter 2 reviews the literature. Chapter 3 explains the methodology. Chapter 4 describes the model, loads and design data. Chapter 5 presents the results and discussion. Chapter 6 gives the conclusions and the scope for future work. The appendices, the list of references and the acknowledgements follow.



II. REVIEW OF LITERATURE

1. Introduction

This chapter reviews published work that is relevant to the drift-based assessment of steel frames by IDA. The studies are grouped by theme: performance-based design and steel moment frames; the fundamentals of IDA; ground-motion selection; applications of IDA to steel and other frames; statistical treatment and fragility; collapse modelling; and the Indian code context. The chapter ends with a critical appraisal and a statement of the research gap.

2. Performance-Based Design and Steel Moment Frames

The idea of linking building performance levels to earthquake hazard levels was set out in the SEAOC Vision 2000 document [5]. A probabilistic version, in which hazard, structural response, damage and loss are chained together, was later proposed by the Pacific Earthquake Engineering Research Center [6]. In this framework the structural-response step is usually obtained from nonlinear dynamic analysis of the building under a suite of ground motions, which is exactly the role played by IDA.

The brittle fracture of welded beam-column connections in the 1994 Northridge earthquake led to the SAC research programme and to the FEMA-350 guidelines [7]. These guidelines moved steel moment-frame design towards a reliability-based format with explicit confidence levels. The probabilistic basis behind them was described by Cornell et al. [8], who expressed the check as a comparison of a factored demand with a factored capacity. Lee and Foutch [9] used nonlinear time-history analysis to evaluate steel frame buildings designed to the new criteria, while Foutch and Yun [10] examined how modelling choices influence the computed response of steel moment frames. Together, these studies made nonlinear dynamic analysis, with interstorey drift as the main demand parameter, the reference method for assessing steel frames. Nonlinear static (pushover) analysis is simpler, but Krawinkler and Seneviratna [11] pointed out that it can mislead when higher modes or progressive stiffness changes matter and that it cannot reflect record-to-record variability. Shao and Tang [12] found the same for steel frame-steel plate shear wall buildings of 10, 15 and 20 storeys: IDA base-shear-roof-displacement points lay between the pushover curves obtained for uniform and inverted-triangular load patterns, and the



difference grew with height. For an eight-storey frame, as studied here, a dynamic procedure is therefore preferable.

3. Incremental Dynamic Analysis

Before IDA was formalised, Shome et al. [13] had shown 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 structural model is subjected to one or more records, each scaled to many intensity levels, giving curves of an intensity measure (IM) against a damage measure (DM). They recommended the 5 %-damped spectral acceleration at the first-mode period, $S_a(T_1)$, as the IM and the maximum interstorey drift ratio as the DM for first-mode-dominated structures, and they showed that $S_a(T_1)$ leads to less scatter than peak ground acceleration (PGA).

The same paper describes the typical shapes of IDA curves, which begin with an elastic segment and then show hardening, softening or even non-monotonic behaviour, and may end in a flat line at dynamic instability. Two ways of defining a limit state were discussed: a DM-based rule, in which a drift value is chosen, and an IM-based rule, in which the point where the curve flattens is used. Because one record cannot represent the variability of earthquakes, the authors advised using several records and summarising them by the median and the 16th and 84th percentile curves [1]. A companion paper [14] gave practical advice on selecting the IM, tracing the curves efficiently and summarising the results. The central advantage of IDA over single-level analysis is that the capacity for a given limit state can be read off every record, which gives a distribution of capacities and not just one number.

4. Ground-Motion Selection

The FEMA P695 methodology [15] defines a far-field set of 22 record pairs and gives criteria on magnitude, source-to-site distance, site conditions and minimum amplitude so that the set is not biased towards a particular site or structure. Baker and Cornell [16] showed that spectral shape, expressed through epsilon, affects the response in addition to $S_a(T_1)$, which matters when long-period structures are analysed. Indian records are available from the PESMOS strong-motion database [17]. The present project uses the FEMA P695 criteria as a guide for choosing five records from Indian events, which is



fewer than the record numbers used in most of the studies reviewed below (03 to 30 records). The effect of this smaller sample on the dispersion is discussed in Chapter 5.

5. Application of IDA to Steel and Other Frames

1. Moment frames and elliptic-braced moment frames

Ghasemi Jouneghani and Haghollahi [18] used IDA to study moment-resisting frames fitted with an elliptic brace (ELBRF) and compared them with conventional intermediate moment frames. Three-, five- and seven-storey buildings with a storey height of 3 m, a bay width of 6 m and steel of 235 MPa yield strength were modelled in OpenSees and analysed under 12 far-field records. $S_a(T_1)$ was the IM and the maximum interstorey drift ratio was the DM. Immediate Occupancy was associated with a drift of about 2 % and Collapse Prevention with 10 %. The elliptic braces yielded first and absorbed much of the input energy, and the average spectral-acceleration capacity increased by about 30 % compared with the conventional frames. At a 50 % probability of exceeding Immediate Occupancy, the capacities for the ELBRF were 1.88 g, 1.00 g and 0.62 g for the three heights, against 1.50 g, 0.71 g and 0.41 g for the moment frames. The ductility, overstrength and response modification factors decreased as the buildings became taller.

The same research group evaluated the elliptic-braced frame by pushover analysis for 3-, 5-, 7- and 10-storey buildings and proposed response modification factors [19], and later compared pushover-based demand estimates, including modal pushover, with nonlinear time-history results [20]. These studies show a consistent workflow of model, pushover, record selection, IDA and extraction of performance factors. Their capacity is expressed at drift levels of 2 % or more, which are far above the 0.4 % code-level drift used in the present work; the comparison is therefore one of method and not of numerical values.

2. Braced frames and code-based comparisons

Asgarian and Jalaeefer [21] applied IDA to 18 concentrically braced frames of 5, 8 and 12 storeys, with X and chevron bracing, designed to the first, second and third editions of the Iranian seismic code.



Fibre-based beam-column elements with initial imperfections were used in OpenSees, and 20 records were scaled over 14 intensity levels from 0.1 g to 2.4 g. Frames designed to the first edition were noticeably weaker and more flexible, while the second and third editions gave similar and better performance. The study also reported that the response modification factor was lowered from 7 to 6 between editions. A follow-up study of near-field motions [22] compared four intensity measures and found that the first-mode spectral acceleration gave the least dispersion, supporting the choice of $S_a(T_1)$ made in this project. Vahdani et al. [23] used IDA on steel moment frames of 3 to 15 storeys to examine the near-fault modification factor of the Iranian code and concluded that it does not capture near-fault effects on inelastic response accurately.

Ma et al. [24] used IDA on 18 high-strength steel eccentrically braced frames of 8, 12 and 16 storeys with three link lengths and two steel grades (Q460 and Q690). They obtained response modification factors of about 8.2 to 10.1, displacement amplification factors of about 8.3 to 9.8 and overstrength factors of about 4.1 to 6.4. The IDA-based performance curves agreed best with pushover curves under a uniform load pattern. Li et al. [25] studied collapse of low-ductility concentrically braced frames idealised as single-degree-of-freedom systems with reserve strength and stiffness after brace fracture. Reserve stiffness had a much larger influence on collapse ductility than reserve strength, and a global drift limit was found to be potentially unconservative in high seismic regions.

3. Other systems and approximate methods

Javanpour and Zarfam [26] compared six-storey steel moment frames and cross-braced frames with IDA in OpenSees, using PGA and S_a as intensity measures and displacement and drift as damage measures. The moment frame showed lower vulnerability, and the choice of IM and DM changed the estimated vulnerability considerably. Wang et al. [27] proposed performance levels for steel-reinforced-concrete frames from reductions in the slope of the IDA curve and reported 16 %, 50 % and 84 % fractile curves for an eight-storey frame. Zarfam and Mofid [28] examined modal IDA, in which the structure is represented by equivalent single-degree-of-freedom systems, for an eight-storey reinforced concrete frame under 30 records; the approximation saved computing time but could underestimate the response at higher intensities and in the upper storeys, and at least three modes were needed for drift.



Sadeghpour and Ozay [29] analysed 48 reinforced concrete systems designed to two editions of the Iranian code with 2112 analyses following FEMA P695, which shows how IDA can be used to judge the adequacy of a design code.

6. Summary of the Reviewed Studies

Table 2.1 summarises the main features of the studies that used IDA on frame structures.

Table 2.1: Summary of selected IDA studies on frame structures

Ref.	System / height	Records and software	IM and DM	Main finding
[18]	ELBRF vs IMRF; 3, 5, 7 storeys	12 far-field; OpenSees	Sa(T1); max drift	≈30 % higher Sa capacity with elliptic brace
[21]	X and chevron CBFs; 5, 8, 12 storeys	20 records; OpenSees	Sa(T1); max drift	Later code editions perform better; R lowered from 7 to 6
[22]	CBFs under near-field motions	Near-field set	Four IMs compared	Sa(T1) shows least dispersion
[23]	Steel moment frames; 3 to 15 storeys	Far- and near-fault sets	Sa; drift	Code near-fault factor not accurate
[24]	High-strength steel EBFs; 8, 12, 16 storeys	Real records	IDA vs pushover	$R \approx 8.2-10.1$; uniform load pattern matches IDA

Ref.	System / height	Records and software	IM and DM	Main finding
[25]	Low-ductility CBFs (SDOF)	Moderate and high region suites	Collapse ductility	Reserve stiffness governs collapse
[26]	Steel MRF vs cross-braced; 6 storeys	OpenSees	PGA, Sa; displacement, drift	MRF less vulnerable; IM/DM choice matters
[27]	SRC frame; 8 storeys	Multi-record	Sa(T1); max drift	Limit states from IDA slope reduction



[12]	Steel frame–SPSW; 10, 15, 20 storeys	SAP2000	Base shear–displacement	IDA lies between two pushover patterns
[28]	RC frame; 8 storeys	30 records	Displacement, drift	Modal IDA cheaper but can underestimate

7. Fragility Curves and Statistical Treatment

A fragility curve gives the probability that a limit state is reached or exceeded for a given intensity. When IDA is used, the IM value at which the limit is reached is taken from every record and the resulting values are fitted with a distribution, most often a lognormal one defined by a median and a logarithmic standard deviation [8], [30]. Baker [30] compared fitting techniques and gave closed-form expressions for fitting a lognormal fragility function to IDA-based data. Javanpour and Zarfam [26] compared lognormal and logistic models and found that the lognormal model gave vulnerability values about 5 % to 20 % higher, which shows that the choice of distribution matters. In this project the scale factors at 0.4 % drift are sorted and a curve is fitted from their mean and standard deviation; the scale factor at 50 % probability is the median capacity.

8. Collapse and Deterioration Modelling

A full assessment requires a model that can reproduce strength and stiffness loss. Ibarra and Krawinkler [31] proposed a hysteretic model with cyclic and in-cycle deterioration for studying global collapse, and Lignos and Krawinkler [32] calibrated the deterioration parameters for steel beams from a large experimental database. These tools are needed for collapse-level IDA but are beyond the scope of this project, which stops at a small drift limit related to serviceability.

9. Indian Code Context

Seismic design of buildings in India follows IS 1893 (Part 1):2016 [4], which defines the seismic zones, the design response spectrum and limits on storey drift. Steel members are designed using the limit-



state method of IS 800:2007 [3]. IS 18168:2023 [2] adds provisions for the earthquake-resistant design and ductile detailing of steel structures. The author could find no published IDA-based fragility study of a steel moment frame designed to this standard.

10. Critical Appraisal and Research Gap

The reviewed work leads to the following observations:

- IDA is the accepted method for estimating capacity and its variability, and $S_a(T_1)$ with maximum interstorey drift is the most commonly used IM–DM pair [1], [22].
- Most studies use OpenSees with fibre or hinge models and run the analysis to a large drift or collapse. Limit-state drifts of 2 % to 10 % are common [18], while the 0.4 % code-level drift is rarely examined.
- Studies are mostly based on Iranian, Chinese or American design practice and records. No study found uses an Indian-code steel moment frame with Indian ground motions..
- The choice of statistical distribution affects the fragility estimate [26], so it must be stated clearly.
- The gap addressed here is the lack of a drift-based IDA assessment, using a commonly available design package and Indian ground motions, of a mid-rise steel moment frame designed under IS 800:2007 and IS 18168:2023, with the result fed back into the design.

III. Methodology

1. Overview

The procedure adopted in this project follows the general IDA framework of Vamvatsikos and Cornell [1], simplified for use with a commercial analysis package. The main steps are shown in Figure 3.1 and described in the sections below.

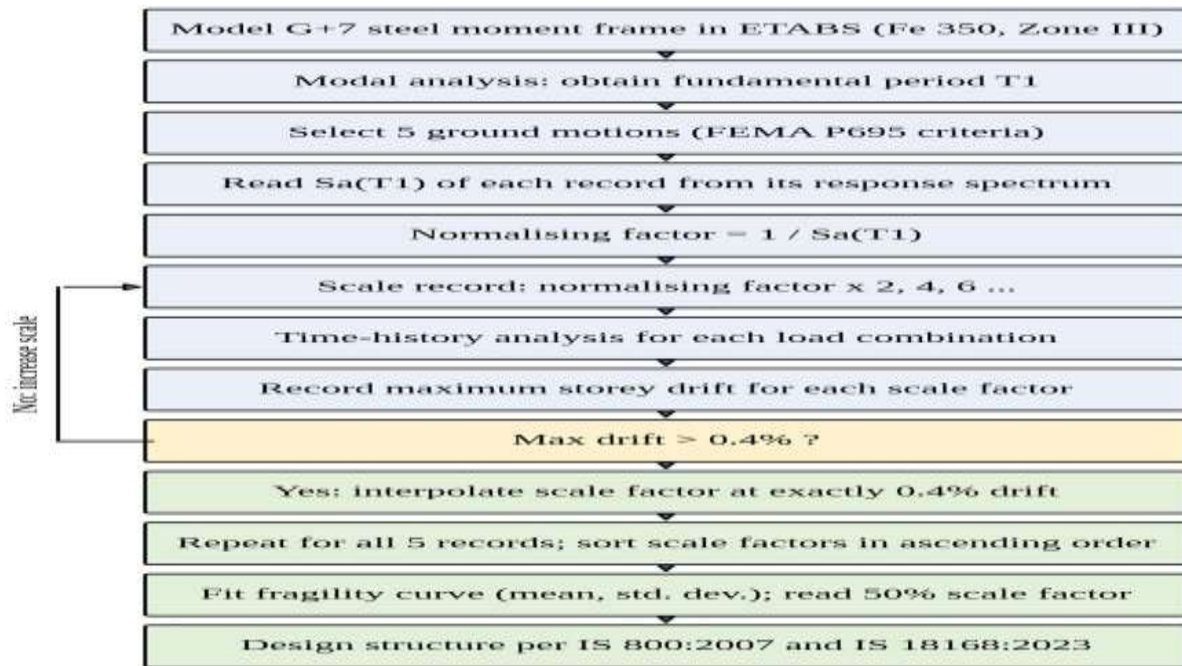


Figure 3.1: Methodology adopted for the IDA-based assessment

2. Structural Model

A G+7 steel moment-resisting frame building of 20.1 m × 20.1 m plan was modelled in ETABS. Fe 350 steel was used for all members. The building is located in Seismic Zone III (Mumbai). Dead load, live load, wind load and earthquake load 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/m² was applied on the floors. The details are given in Chapter 4.



3. Selection of Ground Motions

Five ground-motion records were selected using the criteria of FEMA P695 [15] and taken from a strong-motion database [17]. 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 details of the records are listed in Chapter 5.

4. Intensity Measure and Normalisation

The intensity measure is the 5 %-damped spectral acceleration at the fundamental period, $S_a(T_1)$ [1]. The fundamental period T_1 was obtained from the modal analysis of the ETABS model ($T_1 = 2.379$ s, Section 4.3). For each record, the response spectrum was generated and the spectral acceleration at T_1 was read from it. The record was then normalised by multiplying it by the factor $1/S_a(T_1)$, so that all records have the same spectral acceleration at T_1 before scaling.

5. Scaling and Analysis

Each normalised record was multiplied by increasing scale factors in steps of 2 (that is, 2, 4, 6, ...) times the normalising factor. For each scale factor a time-history analysis was run for the service load combinations $DL+LL \pm THAX$, $DL+LL \pm THAY$, $DL \pm THAX$ and $DL \pm THAY$, where THAX and THAY are the time-history cases in the two plan direction. The maximum storey drift was recorded for each scale factor. The scaling was continued until the maximum storey drift exceeded 0.4 %, and was then carried further for each record, to maximum drifts of between 0.68 % and 2.19 % (Table 5.2). The same procedure was repeated for each of the five records.

6. Damage Measure and Limit State

The damage measure is the maximum storey drift ratio, which is the maximum relative displacement between two consecutive floors divided by the storey height, expressed in per cent [1]. The limit state considered is a drift of 0.4 %, 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 [18].

7. Interpolation of the Scale Factor

For each record, the two consecutive scale factors that bracket the drift of 0.4 % were identified. If SF_1 and SF_2 are the scale factors giving drifts $\delta_1 < 0.4\%$ and $\delta_2 > 0.4\%$, the scale factor at exactly 0.4 % was found by linear interpolation:

$$SF_{0.4} = SF_1 + (0.4 - \delta_1)(SF_2 - SF_1)/(\delta_2 - \delta_1) \quad (3.1)$$

8. Fragility Curve

The five interpolated scale factors were arranged in ascending order. Their mean μ and standard deviation σ define a cumulative probability curve of the scale factor at which the drift limit is reached:

$$P(SF \leq s) = \Phi[(s - \mu)/\sigma] \quad (3.2)$$

where Φ is the standard normal cumulative distribution function. If a lognormal distribution is used, as is common in the literature [30], the mean and standard deviation are calculated from the natural logarithms of the scale factors and the equation becomes:

$$P(SF \leq s) = \Phi[(\ln s - \ln \theta)/\beta] \quad (3.3)$$

where θ is the median and β the logarithmic standard deviation. The scale factor at 50 % probability equals μ in Equation 3.2 and θ in Equation 3.3. In this study the lognormal form (Equation 3.3) was adopted: $\ln \theta$ was taken as the mean of the natural logarithms of the scale factors and β as their sample standard deviation $(n - 1)$. This scale factor was used for the design stage.

9. Design of the Structure

The structure was designed with the 50 %-probability scale factor, in accordance with IS 800:2007 [3] for strength and stability and with the ductile-detailing provisions of IS 18168:2023 [2]. The checks include section classification, bending strength with lateral-torsional buckling, shear strength and the interaction of shear and moment. A spreadsheet prepared for the beam checks is described in Appendix I.



IV. STRUCTURAL MODELLING AND DESIGN DATA

1. Description of the Building

The building is a residential G+7 steel structure with a square plan of 20.1 m × 20.1 m. The lateral load is resisted by steel moment-resisting frames in both directions. Table 4.1 lists the data used for the model. The floor plan is shown in Figure 4.1. Items in square brackets are to be completed from the ETABS model.

Table 4.1: 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
Number and size of bays (X and Y)	3 bays in each direction, 6.7 m each (20.1 m ÷ 3, equal spans; Figure 4.1)
Storey height	21m
Structural steel	Fe 350 (fy = 350 MPa)
Modulus of elasticity of steel	2×10^5 MPa
Beam and column sections	ISMB 250 & 2-ISMB 250 TOE TO TOE.
Floor slab	100mm thick deck slab Figure 4.1
Interior wall thickness	150 mm (On external Beam)
Exterior wall thickness	250 mm (On internal Beam)
Live load	3 kN/m ²
Location and seismic zone	Mumbai, Zone III (IS 1893 (Part 1):2016)
Soil type, importance factor, response reduction factor	II, 1, 5
Wind load parameters	basic wind speed- 44, Terrain- 3, Importance factor-1
Damping ratio	5%

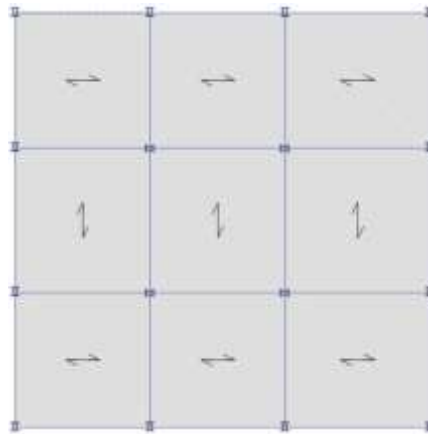


Figure 4.1: Typical floor plan of the ETABS model (3 × 3 bays); arrows show the direction of one-way floor load distribution

2. Load Cases and Combinations

Dead load (DL), live load (LL), wind load (WL) and earthquake load (EQ) were defined as static load patterns. For the IDA, the service-level combinations in Table 4.2 were used together with the time-history cases THAX and THAY.

Table 4.2: Load combinations used in the study

Purpose	Combination
Time-history (IDA) analysis	DL + LL ± THAX; DL + LL ± THAY; DL ± THAX; DL ± THAY
Design as per IS 800 / IS 1893	As per IS 800, Table 4

3. Modal Analysis

The modal analysis of the model gave the fundamental period used for normalising the records. Table 4.3 gives the first three modes. The first mode ($T_1 = 2.379$ s) is translational in



the X direction, with a mass participation ratio of 0.8182, and the second mode (2.352 s) is translational in the Y direction, with a ratio of 0.8156. The third mode has a period of 2.096 s.

Table 4.3: Modal periods and mass participation ratios

Mode	Period (s)	UX	UY	UZ
1	2.379	0.8182	1.792×10^{-5}	0
2	2.352	2.483×10^{-5}	0.8156	0
3	2.096	0.0007	0.0006	0

V. RESULTS AND DISCUSSION

1. Ground-Motion Records

The five records used and their spectral acceleration at the fundamental period are given in Table 5.1.

Table 5.1: Ground-motion records used in the study

No.	Earthquake and date	Station	Sa(T1) (g)	1/Sa(T1)
GM1	Chamoli (Gopeshwar), 28-03-1999	Chamoli	0.05596	17.86
GM2	Uttarkashi, 19-10-1991	Bhatwari	0.0496	20.16
GM3	Uttarkashi, 19-10-1991	Uttarkashi	0.0184975	54.06
GM4	India–Burma, 06-08-1988	Bhaigao	0.015191	65.82
GM5	India–Burma, 06-08-1988	Dipun	0.03168	31.565

2. Maximum Storey Drift versus Scale Factor

Table 5.2 gives the maximum storey drift obtained for each scale factor and record. A dash means that the record was not analysed at that scale factor. Values above 0.4 % are shown in bold.

Table 5.2: Maximum storey drift (%) for each scale factor



Scale factor (× normalising factor)	GM1	GM2	GM3	GM4	GM5
1	0.0223	0.0345	0.0631	0.0072	0.0083
2	0.0431	0.0643	0.1147	0.0135	0.0134
4	0.0861	0.1360	0.2437	0.0237	0.0236

Scale factor (× normalising factor)	GM1	GM2	GM3	GM4	GM5
6	0.1292	0.1841	0.3611	0.0405	0.0346
8	0.1641	0.2455	0.4815	0.0540	0.0409
10	0.2039	0.3389	0.6048	0.0675	0.0577
12	0.2584	0.3683	0.7222	0.0810	0.0692
14	0.3015	0.4736	0.8455	0.0945	0.0808
16	0.3282	0.4940	0.9230	0.1080	0.0923
18	0.3876	0.6089	1.0862	0.1215	0.1008
20	0.4307	0.6772	1.2066	0.1350	0.1154
22	0.4737	–	–	0.1485	0.1269
24	0.5168	–	–	0.1620	0.1384
26	0.5599	–	–	0.1755	0.1479
28	0.5743	–	–	0.1659	0.1615
30	0.7650	–	–	1.5457	1.6418
32	0.8161	–	–	1.6488	1.7512
34	0.8671	–	–	1.7518	1.8017
36	0.9181	–	–	1.8549	1.9701
38	0.9691	–	–	1.9579	2.0831
40	1.0201	–	–	2.0610	2.1890
42	1.0711	–	–	–	–
44	1.1221	–	–	–	–



46	1.0978	-	-	-	-
48	1.2241	-	-	-	-
50	1.2751	-	-	-	-
52	1.3261	-	-	-	-
54	1.3771	-	-	-	-
56	1.4281	-	-	-	-
58	1.4791	-	-	-	-
60	1.5301	-	-	-	-

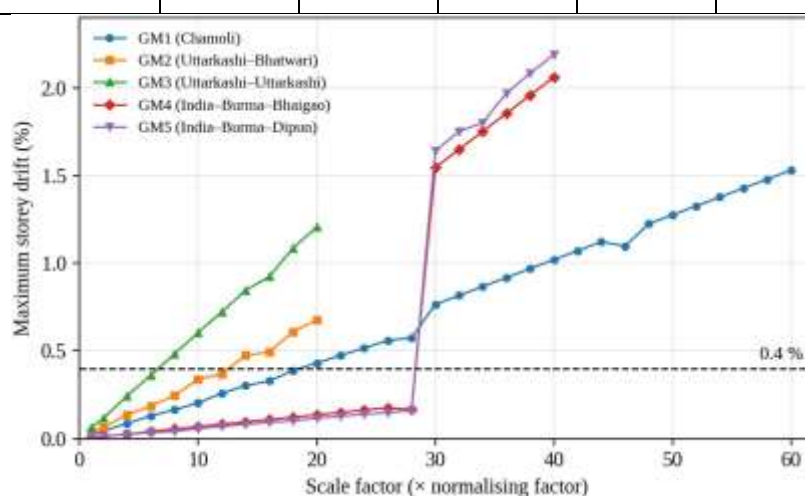


Figure 5.1: Maximum storey drift versus scale factor for the five ground motions

3. Scale Factor at 0.4 % Drift

The scale factors bracketing the drift of 0.4 % and the interpolated values from Equation 3.1 are given in Table 5.3.



Table 5.3: Interpolated scale factor at 0.4 % storey drift

Record	SF ₁	δ_1 (%)	SF ₂	δ_2 (%)	SF at 0.4 %
GM1	18	0.3876	20	0.4307	18.575
GM2	12	0.3683	14	0.4736	12.602
GM3	6	0.3611	8	0.4815	6.646
GM4	28	0.1659	30	1.5457	28.339
GM5	28	0.1615	30	1.6418	28.322

4. Fragility Curve

The scale factors were sorted in ascending order (Table 5.4) and the fragility curve of Figure 5.2 was obtained by fitting a lognormal distribution (Equation 3.3). The natural logarithms of the scale factors have a mean $\lambda = 2.8075$ and a sample standard deviation $\beta = 0.6119$, so the median scale factor is $\theta = \exp(\lambda) = 16.57$. For comparison, the empirical probability of the i -th ranked value is shown as $i/(n+1)$, with $n = 5$.

Table 5.4: Sorted scale factors and empirical probability

Rank i	Record	SF at 0.4 %	$\ln(\text{SF})$	$P = i/(n+1)$
1	GM3 – Uttarkashi (Uttarkashi)	6.646	1.8940	0.167
2	GM2 – Uttarkashi (Bhatwari)	12.602	2.5339	0.333
3	GM1 – Chamoli	18.575	2.9218	0.500
4	GM5 – India–Burma (Dipun)	28.322	3.3436	0.667
5	GM4 – India–Burma (Bhaigao)	28.339	3.3443	0.833
	Mean of $\ln(\text{SF})$, λ		2.8075	
	Standard deviation of $\ln(\text{SF})$, β		0.6119	
	Median $\theta = \exp(\lambda)$	16.57		

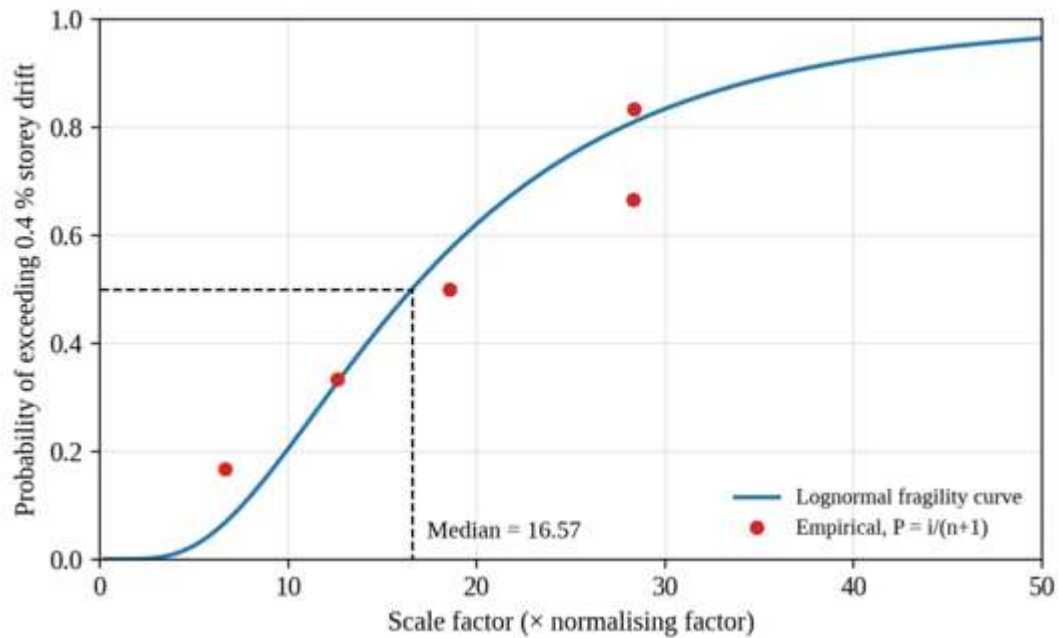


Figure 5.2: Lognormal fragility curve for a maximum storey drift of 0.4 %

5. Design Using the 50 % Scale Factor

The scale factor at 50 % probability was 16.57 (that is, 16.57 times the normalising factor $1/S_a(T_1)$). The structure was designed with this value as described in Section 3.9. The member sizes before and after the revision are given in Table 5.5.



Table 5.5: Member sizes before and after redesign

Member	Final section
Beams	ISMB 250
Columns	2- ISMB 250 (Toe to Toe)

6. Discussion

The scale factors at 0.4 % drift range from 6.65 (GM3, Uttarkashi station) to 28.34 (GM4, India–Burma record at Bhaigao), a ratio of about 4.3. At a scale factor of 10 the maximum storey drifts are 0.204 %, 0.339 %, 0.605 %, 0.068 % and 0.058 % for GM1 to GM5. The Uttarkashi-station record is therefore the most damaging for this building at the drift level considered, and the two India–Burma records 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 %) and then rises about tenfold between scale factors of 28 and 30 (to 1.55 % and 1.64 %). GM1 also shows a smaller jump over the same interval (0.57 % to 0.77 %). A change that appears at the same scale factor in three records points to a common cause, for example the onset of yielding or a change in the analysis settings, and should be checked in the ETABS model. Because the 0.4 % crossing of GM4 and GM5 lies inside this interval, their interpolated scale factors (28.34 and 28.32) are approximate, and additional runs at intermediate scale factors (for example 28.5, 29 and 29.5) would refine them.

The logarithmic standard deviation $\beta = 0.61$ is large, which reflects strong record-to-record variability, and with only five records the estimate itself is uncertain (Section 2.4). According to the fitted curve, the probability that the 0.4 % drift limit is reached is about 20 % at a scale factor of 10, 62 % at 20 and 50 % at the median of 16.57.



VI. CONCLUSIONS AND SCOPE FOR FUTURE WORK

1. Summary of the Work

A G+7 steel moment-resisting frame of 20.1 m × 20.1 m plan, located in Seismic Zone III (Mumbai), was modelled in ETABS. Five ground motions from Indian earthquakes were selected using FEMA P695 criteria, normalised with the spectral acceleration at the fundamental period and scaled in steps. For each record the scale factor that gives a maximum storey drift of 0.4 % was found by interpolation, and a fragility curve was constructed from the five values. The scale factor at 50 % probability was then used in the design of the structure as per IS 800:2007 and IS 18168:2023.

2. Conclusions

The following conclusions are drawn from the study:

- 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).
- The scale factors that produce a maximum storey drift of 0.4 % range from 6.65 to 28.34 for the five records, which shows large record-to-record variability.
- A lognormal fit to the five scale factors gives $\lambda = 2.8075$ and $\beta = 0.6119$, and the scale factor at 50 % probability is 16.57.
- 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.

3. Scope for Future Work

- Use a larger set of records (the 22 pairs of FEMA P695, or at least 10 to 20 Indian records) to obtain a more reliable dispersion.



- Perform fully nonlinear IDA with plastic hinges or fibre elements and extend the analysis to higher limit states such as Life Safety and Collapse Prevention.
- Fit a lognormal fragility function by maximum-likelihood methods [30] and compare it with the mean–standard-deviation approach used here.
- Include the effect of soil conditions, near-field motions and spectral shape through epsilon or average spectral acceleration [16].
- Extend the study to frames of different heights and to braced or dual systems, and compare the results with the studies reviewed in Chapter 2.

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