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An assessment of IS codal provisions for the design of low rise steel moment frames through incremental dynamic analysis

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Abstract The current Indian Standard (IS) code for seismic design of structures (IS 1893:2002) specifies the use of time history analysis for structures with height greater than 40m. However, for structures less than 40m it recommends the concept of equivalent static analysis. This study attempts to investigate the adequacy of the current design code when it comes to the actual evaluation of structures shorter than 40 m subjected to seismic loading using dynamic analysis as opposed to the code specified static analysis. Incremental dynamic analysis, which subjects a structure to a progressively increasing series of intensity measures, has been adopted here for the purpose. Three 2D moment resisting steel structures under the 1991 Uttarkashi and the 2001 Bhuj earthquakes (both of which predate the current IS1893) have been studied—a single storeyed portal frame, a 2 storey 3 bay frame and a 3 storey 2 bay frame. While it can be argued that two records are never enough for any generalization, and that only a full probabilistic analysis can determine if the limiting collapse prevention probability has been exceeded for these structures, the IS code in both cases does significantly under predict the seismic demands on the structures. At the same time, and perhaps why the codal provisions usually work, the structural capacities are in most cases underestimated as well. These suggest that a thorough study is in order and that there is scope for rationalization in the IS codal provisions.

Keywords Base shear · Steel · Deflection · Collapse · Earthquake · Incremental dynamic analysis · Indian Standard code · Pushover analysis · Capacity demand ratio

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Abbreviations and symbols used

A_k	Design horizontal seismic coefficient for mode k
A_h	Design horizontal seismic coefficient (calculated according to IS code)
Φ_{ik}	Mode shape of the ith floor due to kth mode
ω_n	Natural (fundamental) vibration frequency of structure
EDP	Engineering Damage Parameter (also called Damage Measure)
h_i	Height of the floor (storey height)
I	Importance factor associated with the structure
IDR	Inter-storey Drift Ratio (Defined as storey displacement/storey height)
IM	Intensity measure
MDOF	Multi degree of freedom
P_k	Modal participation factor
PGA	Peak Ground Acceleration
PGV	Peak ground velocity
Q_{ik}	Shear force on ith floor in kth mode
R	Response reduction factor
S_a	Spectral acceleration (pseudo spectral acceleration in true sense)
S_d	Spectral displacement
S_e	Elastic slope of S_a-IDR graph
SDOF	Single degree of freedom system
UDL	Uniformly distributed load
V_b	Base Shear
W_i	Seismic weight of ith floor
W_s	Seismic weight of the structure
Z	Zone in which a structure is located

1 Introduction

The engineering challenges from major earthquakes in the Indian context cannot be overemphasized. More than about 60% of the land area is considered prone to shaking of intensity VII and above (MMI scale). In fact, the entire Himalayan belt is considered prone to great earthquakes of magnitude exceeding 8.0, and in a short span of about 50 years, four such earthquakes (in Assam (1897), in Kangra (1905), in Bihar-Nepal (1934) and in Assam Tibet (1950)) have occurred (Jain and Nigam 2000). Although earthquake engineering applications started quite early in our country, extensive damage during several moderate earthquakes in recent years (Shekhar et al. 2004) indicate that earthquake risk in the country has been increasing alarmingly and that there is significant scope of improvement in our design codes.

In India, IS1893 (BIS 2002) is the main code outlining seismic design provisions. For low-rise structures (<40 m high) the code focuses the design approach on base shear and its distribution to various floors of a building in an equivalent static sense (either taking the first mode of vibration of a cantilever column, or through a modal analysis taking into account the first few modes). An effort has been made in this paper to study the actual response of three low rise 2-D steel moment-resisting frame structures (designed to IS1893 and IS800 (BIS 2007) provisions) to two recent earthquake records (1991 Uttarkashi as captured from station Bhatwari and 2001 Bhuj as captured from station Ahmedabad), to find their yield and ultimate capacities through full dynamic analyses, and to compare the results with the codal provisions.

2 Structural design and evaluation methodologies

The current Indian Standard code for designing/evaluating a structure for seismic loading is IS 1893:2002. IS 1893 recommends the use of dynamic analysis for certain structures (regular structures with height greater than 40 m and irregular structures with height greater than 12 m in zone V). In this study however, dynamic analysis is not used for computing the design base shear. This is because the structures used in this study, in addition to being shorter than 40 m, have almost their entire modal masses concentrated in the first mode of vibration, as detailed below. IS 1893 (Clause 7.8.4.2) suggests incorporating only those modes for which the sum of modal masses is at least 90% of the total seismic mass to compute the design base shear for each floor in each mode. The shear force on the ith floor in kth mode is

$$Q_{ik} = A_k \phi_{ik} P_k W_i \tag{1}$$

where, A_k = design horizontal seismic coefficient for mode k, P_k = modal participation factor, W_i = the weight of the ith floor, Φ_{ik} refers to the mode shape of the ith floor due to kth mode.

The first example (a single storeyed portal frame) being one storied, the modal mass corresponding to the first mode equals 100% of the total seismic mass, hence the dynamic method for computing the individual modes and then the base shear (as mentioned in IS code) becomes more or less similar to the static procedure. In the second example (a two storey three bay frame), the contribution of first mode towards modal mass is 92.4%. Hence, the computation of design base shear using dynamic method is not needed. In the third example (a three storey 2 bay frame), the contribution of first mode towards modal mass is 89%. Being very close to the required 90% mark, dynamic method is not adopted in this case either.

2.1 IS code based equivalent static evaluation of structure

The equivalent static load approach defines a series of forces acting on a building to represent the effect of earthquake ground motion, typically defined by a seismic design response spectrum. It assumes that the building responds in its fundamental mode. For this to be true, the building must be low-rise and must not twist significantly when the ground moves. Since higher modes are not accounted for, the accuracy of this method may be limited. To account for effects due to "yielding" of the structure, modification factors (like reduction factors) are used. The structures designed by this method could be rather conservative (as can be seen in the example in later sections).

IS1893:2002 uses a horizontal acceleration spectrum factor, A_h, which on multiplication with the seismic weight of the structure (W_s) gives the base shear, V_b (Clause 7.5.3, IS 1893:2002).

$$V_b = A_h \times W_s \tag{2}$$

W_s is the sum of the seismic weights of all the floors of the building. The seismic weight of a floor is the sum of the factored dead plus factored live loads. A_h can be found for a structure by knowing the following factors (Clause 6.4.2, IS 1893):

1. Zone factor (Z): It is a factor to obtain design spectrum taking into account the location of the building. The country is divided into 4 different zones according to the likelihood of severe earthquakes to occur (maximum considered earthquake). Z values for different zones are given below in Table 1.

Table 1 Seismic zone factors

Seismic zone	II	III	IV	V
Z	0.10	0.16	0.24	0.36

Seismic zone V indicates a zone with high seismic activity where the chances of a major earthquake occurring is significant. This factor Z is for maximum considered earthquake.

2. Importance factor (I): This factor assumes a value of 1 and 1.5, the latter being for structures of higher importance.
3. Response Reduction factor (R): Response reduction factor depends on the allowable system ductility and represents the ratio of maximum seismic force on a structure during a specified ground motion (if it were to remain elastic) to the design seismic force. Thus, actual seismic forces are reduced by a factor R to obtain design forces. In the present study, all the structures have been analysed with an R value of 5.
The intent of R is to simplify the structural design process such that only linear elastic static analysis is needed for building design. Such a design philosophy implies that structural inelastic behaviour is expected. The three factors contributing towards R are system overstrength, structural redundancy, and system ductility.
However, there is no clear demarcation of the contribution of each factor in R. As a result, a system with low ductility but high overstrength (+ redundancy) and a system with high ductility and a low overstrength (+ redundancy) would have the same value of R. For a portal framed structure, reduction of response due to redundancy becomes insignificant. Assuming a symmetric structure, it is clear that the loads would be symmetric as well. So, hinges at different locations would form simultaneously and hence structural redundancy becomes almost zero. IS code does not address this. Also, the present R factor is not dependent on the natural period of the structure and does not address the redistribution of forces once non-linearity sets in. Apparently, the reduction of the design forces due to R is compensated by factor of safety imposed on the structure while designing its components. The most important source of uncertainty in seismic performance evaluation of structures is in ground motion itself (for example the structure in Sect. 3.3). Although, in general, greater importance should be given to ductility based reduction than to system overstrength, this is beyond the scope of present study.
4. Structural Response factor (S_a/g): It is a factor that determines the acceleration response spectrum of the structure subjected to earthquake ground vibration and depends on the natural period of vibration and damping of the structure (clause 6.4.5 IS 1893:2002).

Once these factors are known A_h can be evaluated as:

$$A_h = \frac{ZIS_a}{2Rg} \tag{3}$$

Equation (2) can then be used to find the total base shear, V_b. As seen in Eq. (3), the zone factor Z is further divided by 2 for reducing the maximum considered earthquake to design basis earthquake (Jain and Murthy 2005). This reduction is based on the assumption that the structure would not be subjected to the maximum considered earthquake during its lifetime. The storey forces (horizontal force, Q_i) are computed as per clause 7.7 of the code:

$$Q_i = V_b \frac{W_i h_i^2}{\sum_{j=1}^{n} W_j h_j^2} \tag{4}$$

Here, W_i = seismic weight of ith floor, h_i = height of the floor measured from base.

2.2 Pushover analysis based evaluation of structure

Nonlinear pushover analysis, or collapse mode analysis, is a simple and efficient technique to predict the seismic response of a structure often in lieu of a full dynamic analysis. A pushover analysis provides us with the capacity to monitor initial yielding and gradual post elastic plastic behaviour of both individual elements and overall structural system (Hasan et al. 2002). In pushover analysis the forces are applied incrementally in every step until the design (or ultimate) force is reached. Using the results from this analysis, the actual nonlinear dynamic response of the structure can be estimated.

The non linear pushover analysis in this work is performed using IDARC-2D with force control (Valles et al. 1996). It is assumed that no in-cycle strength degradation, and thus no negative stiffness, occurs during the force control. Non linearity both in terms of geometry and material are incorporated. The controlling parameter of the analysis is the base shear (V_b): the analysis stops when the base shear exceeds its design/final value or if the structure collapses before the design/final base shear V_b is reached. Structural collapse is defined by failure of Cholesky decomposition of the stiffness matrix, which signifies that the matrix is no longer positive definite.

IDARC uses the following system of equations to analyze the structure:

$$[K_t]\{\Delta u\} = \{\Delta F\} - \{\Delta P_v\} - \left\{\Delta P_{fr}\right\} - \{\Delta P_{hi}\} - \{\Delta P_{iw}\} + C_{corr}\{\Delta F_{err}\} \tag{5}$$

where, $[K_t]$ = tangent stiffness matrix, $\{\Delta u\}$ is the vector with the increment of lateral displacements; $\{\Delta F\}$ is the vector with the increment in lateral forces; $\{\Delta P_v\}$, $\{\Delta P_{fr}\}$, $\{\Delta P_{hi}\}$, and $\{\Delta P_{iw}\}$ are the vector with the increment of forces in viscous dampers, friction dampers, hysteretic dampers, and infill panels respectively; C_{corr} is a correction coefficient (usually taken as one); and $\{\Delta F_{err}\}$ is the vector with the unbalanced forces in the structure.

IDARC uses a single step unbalanced force correction method to deal with the unbalanced force in static pushover analysis. The same approach is also taken in non linear dynamic analysis, as discussed in the next subsection. The force increment is taken to be very small so that the magnitude of unbalanced force (which is computed when moments, shear and stiffness are being updated in the hysteresis model) does not increase beyond acceptable limits (Au and Yan 2008).

2.3 Incremental dynamic analysis (IDA) based evaluation of structures

Incremental dynamic analysis (Vamvatsikos and Cornell 2002) conducts a series of non linear dynamic runs of the structure which can then be used to correlate the performance of the structure with the seismic demand. The method is highly efficient in terms of predicting the response of the system and has become a valuable tool in seismic engineering. Simpler options like linear static procedure, linear dynamic procedure, non linear static procedure are available as given in SAC FEMA (Cornell et al. 2002). However, the degree of accuracy is much lesser than in IDA.

Incremental dynamic analysis procedure:

Incremental dynamic analysis involves a series of non linear dynamic analysis of a structure. The basic steps are:-

1. ***Select an intensity measure, IM***: Intensity measure is a quantity used to describe the intensity of ground motion of which spectral acceleration, S_a is the most suitable variable.

S_a corresponds to the first natural mode of vibration of the required structure and is obtained for a single horizontal component of earthquake (any one of the earthquake components is sufficient). It is the maximum acceleration that the ground motion will cause in a linear oscillator with a specified natural period and damping. Strictly speaking, it is actually the "pseudo spectral acceleration", defined as the spectral displacement times the natural frequency squared:

$$S_a = S_d \omega_n^2 \tag{6}$$

that is commonly used as a measure of spectral acceleration. However, the difference between the two is small enough (for a structure with light damping and short time period) to be neglected (Chopra 2002).

2. ***Select an engineering demand parameter, EDP***: Also known as structural state measure, EDP is a quantity used to describe the relevant response of structure due to the prescribed seismic loading. Many EDPs are in vogue and its selection depends on the type of structural assessment. For assessing structural damage, peak Inter-story drift ratio, IDR is the natural choice as it is closely related to the damage caused to the structure (Chopra and Goel 2001). Local failure can be identified easily using IDR and hence this is selected as the EDP in this paper. The dispersion for IDR is higher as IM increases.
3. ***Increment IM***: IM needs to be incremented successively to capture the entire scenario of structural state- from elastic behaviour to its eventual collapse. There are various models proposed for increasing IM in repeated loops of which the method of scaling up the record by a constant step and increasing the step size every run is used here. In this method, once non convergence is reached, analysis is performed at an IM in between the last known convergent scaling up factor and the non convergent scaling up factor.
4. ***Run the time domain analysis***: The non linear dynamic analysis is performed using IDARC-2d which uses a combination of Newmark-Beta integration and pseudo force method. Non linearity both in terms of geometric and material are incorporated. Material non linearity is accounted by using bilinear hysteretic model (along with stiffness degradation, strength deterioration and slip lock). A bilinear hysteretic model suggests that as soon as hinge formation occurs at a particular node, it is unable to bear further loads i.e. its bending stiffness becomes zero. Mass proportional damping is adopted in this paper.
5. ***Obtain the EDP***: Repeat step 2–4 until structure collapses
6. ***Plot IM versus EDP***: from above.

The algorithm is described in Fig. 1.

As in pushover analyses, IDARC uses a single step unbalanced force correction method to deal with the unbalanced force in non-linear dynamic analysis. The unbalanced force (which is computed when moments, shear and stiffness are being updated in the hysteresis model) is applied at the next iteration. As mentioned in previous section this is not very accurate as it modifies the actual input loads. As a result when the magnitude of unbalanced force increases, so does the error in the model. The unbalanced force is correlated with the time step for which the integration is performed, so if the time step is small enough, the method gives quite accurate answers. For a general building, the time step should be smaller than 0.005 s (Valles et al. 1996).

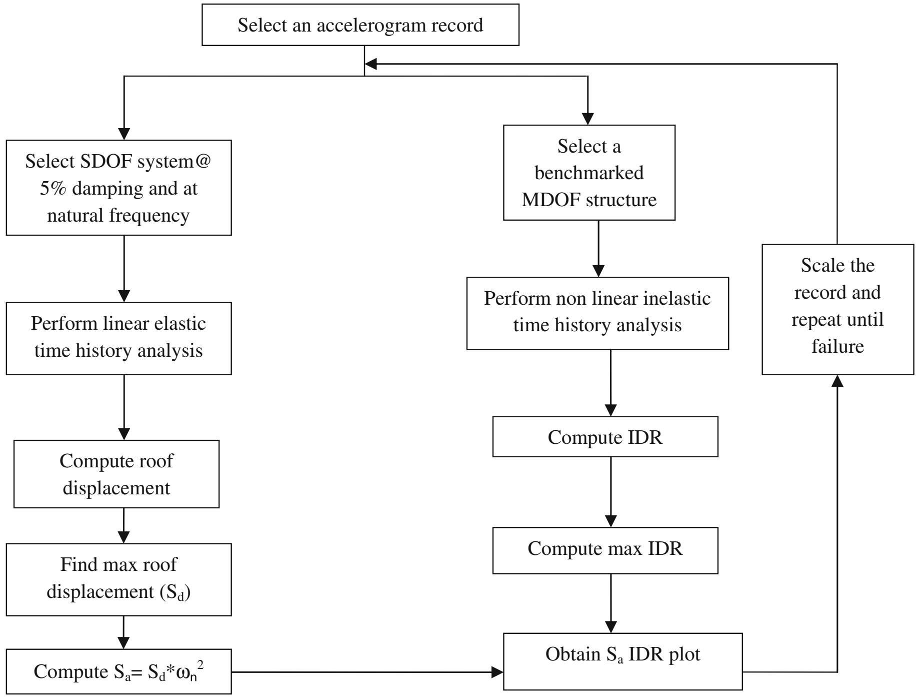


Fig. 1 Algorithm of incremental dynamic analysis

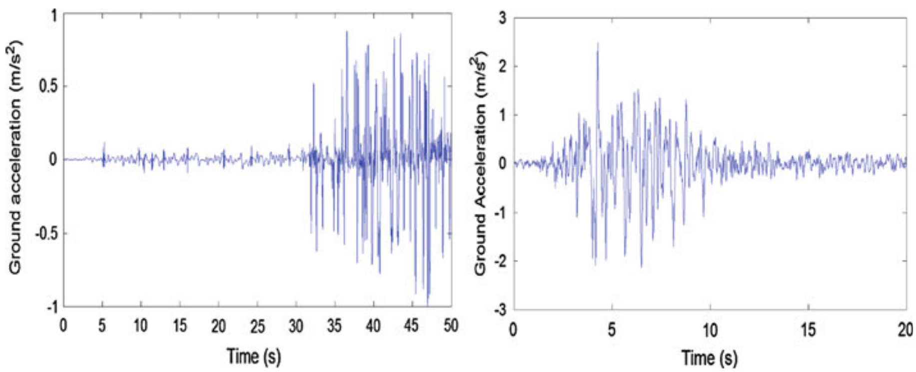


Fig. 2 Bhuj (*left*) and Uttarkashi (*right*) time history

2.4 Computing the drift capacity

If dynamic behaviour is taken into account, the drift capacity of a structure depends on the earthquake record. As stated above, the 1991 Uttarkashi (as captured from station: Bhatwari) and 2001 Bhuj (as captured from station: Ahmedabad) earthquake ground motion records (Fig. 2) are adopted in this paper. Both of these events predate the current edition of IS 1893.

Non-linear dynamic analysis for each earthquake record is run from a very small value of S_a to the value of S_a where the structure collapses. The drift capacity of the structure, used to identify the point of collapse, is computed as follows (Yun et al. 2002):

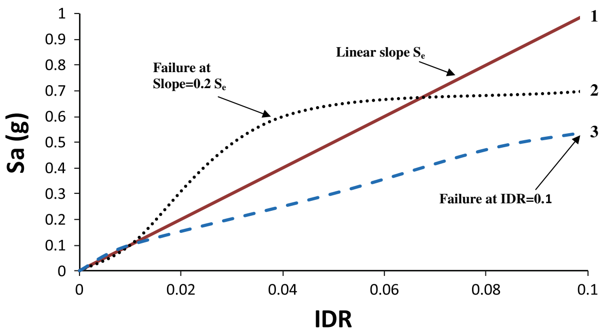


Fig. 3 Computation of drift capacity of structure

1. Elastic time history analysis (Chopra 2002) is performed on an accelerogram record which produces a point on an S_a—IDR graph. The slope of the straight line joining this point and the origin gives the elastic slope of accelerogram (S_e).
2. Similarly, S_e corresponding to other accelerograms are found and the median elastic slope is obtained.
3. Non-linear time history analysis of the building subjected to the relevant accelerogram is performed. The points are plotted on the S_a-IDR graph.
4. Drift capacity of the structure is determined from the slope of the above curve (subject to certain limiting conditions) as described below (Fig. 3), and is measured in terms of IDR.

In Fig. 3, Line 1 represents the line whose slope is equal to the median elastic slope (S_e). Lines 2 and 3 represent the two rather extreme cases that may arise during an IDA run. It is often seen that on scaling the record beyond a certain level, many members start failing in quick succession. Line 2 represents this case. Due to progressive degradation of the stiffness matrix, the slope of S_a versus IDR curve decreases. The structure is assumed to have failed as soon as the slope drops below a limiting value (in the range $0.2S_e \sim 0.3S_e$)—in this paper we have taken the limit conservatively as $0.3S_e$. Nevertheless, the drift capacity computed this way must be limited to 0.10. Line 3 shows another case in which structural members do not fail in quick succession. As a result, the S_a versus IDR curve remains fairly linear and its slope does not deviate much from S_e. For this case as well, structural failure is said to occur when IDR reaches a value of 0.10.

3 Seismic investigation of three lowrise steel frames

In this section, we analyse the seismic capacities of three steel moment frames designed to IS standards (members are designed as laterally unsupported). All are assumed to be located in zone V. The allowable deflections, moments and shears (per IS codes) for the three buildings are ascertained first; the corresponding demands (per IS codes and using equivalent base shear in static pushover analyses) are computed. The demands under two real earthquake records (Fig. 2)—Uttarkashi and Bhuj—are obtained through a full dynamic analysis. Finally, the values of deflection, moment and shear in the frames corresponding to first yield and collapse are computed through incremental dynamic analysis. The capacities and demands are compared at the end and a critical appraisal of the codal provisions is made in Sect. 4.

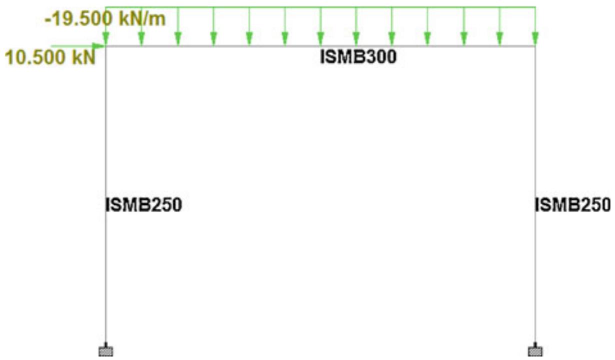


Fig. 4 Simple portal frame (height = 4 m, width = 6 m)

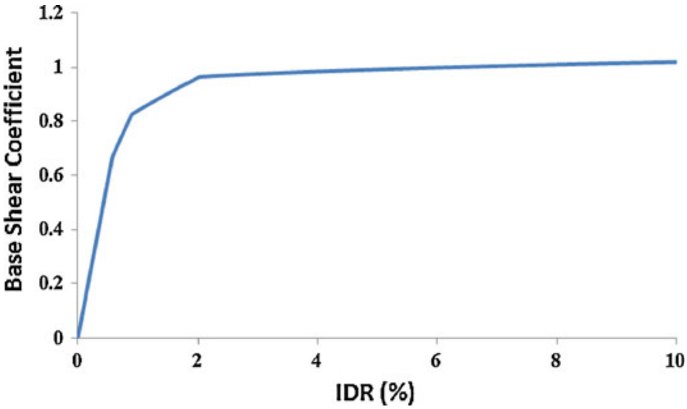
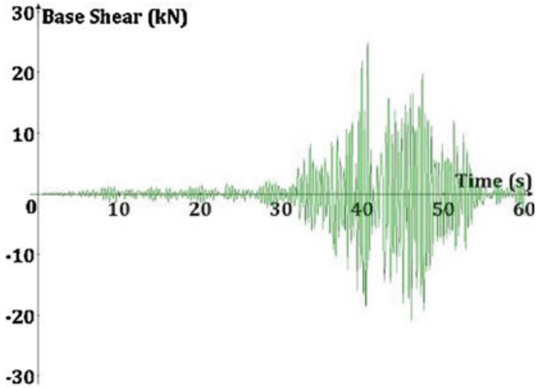


Fig. 5 Pushover graph for portal frame

3.1 Single storeyed portal frame

A simple portal steel moment resisting frame (a one storey one bay frame with column height 4 m and beam length 6 m) located in Zone V, designed according to the current IS codes (BIS 2002, 2007), is taken as our first example (Fig. 4). The natural period of the structure is 0.374 s. Pushover analysis of the structure, time history analysis as well as Incremental Dynamic Analysis are performed as described below. The building is assumed to be an industrial building with no crane load. The structural steel has a yield stress of 250 MPa.

As per IS1893:2002, the design base shear coefficient (A_h) is 0.09. The factored seismic weight of the structure is 117 kN, giving the value of design base shear (V_b) as 10.5 kN. As the structure is a single storeyed portal frame, this entire load is applied to the upper left node of the frame and the structure is designed as per this load. The column and beam sections thus obtained are ISMB 200 and ISMB 300 respectively. The horizontal deflection is 3.2 mm based on elastic analysis whereas the allowable value is 16 mm (= height/250). Other codes specify different criteria for deflection. The Immediate Occupancy level in SAC-FEMA (2000) corresponds to IDR = 1% which amounts to storey height/100. Eurocode EC8 (Marino et al. 2005) presents a drift limit of 1% for structures with no non-structural



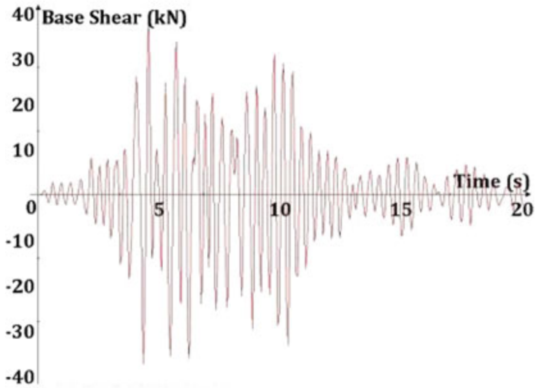


Fig. 6 Base shear versus Time graph for Bhuj (*top*) and Uttarkashi (*bottom*) records for single storey portal frame

elements (the structures presented in this paper has no non-structural elements). Japanese code (Marino et al. 2005) specifies a limit of 0.83%.

The combined axial and bending demand capacity ratios (Sec. 9.1.3.1 of IS800:2007),

$$\left(\frac{F_b}{N_d}\right) + \left(\frac{M_y}{M_{ndy}}\right) + \left(\frac{M_z}{M_{ndz}}\right) < 1 \tag{7}$$

are found to be 0.88 for the beam (at its centre) and 0.86 for the right columns (at its top). Thus, the governing design criterion is combined axial and bending capacity of the beam.

3.1.1 Pushover analysis

IDARC, with its force control option, is used for pushover analysis of the portal frame. The structural response is fairly linear up to $A_h = 0.64$ beyond which the response becomes non linear and finally the structure fails (Fig. 5), failure being defined as the point when the stiffness matrix no longer remains +ve definite. The structural drift at the last known safe state ($A_h = 1.07$) is 30%.

The structural drift at the design value of $A_h = 0.09$ (which occurs at the design shear of $V_b = 10.5$ kN) is 3.2 mm. The deflection limit as per IS code of 0.4% is reached when $A_h = 0.46$ or $V_b = 53.5$ kN. First yielding of the structure (at A_h of 0.64 and IDR of 0.56% i.e., a peak displacement of 22.4 mm) occurs at the bottom of the right column.

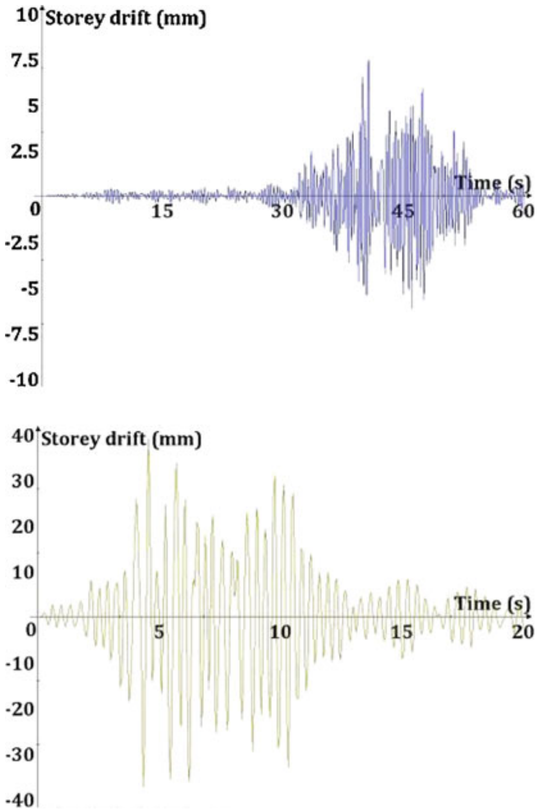


Fig. 7 Inter-storey drift ratio versus time graph for Bhuj (*top*) and Uttarkashi (*bottom*) records for single storey portal frame

3.1.2 *Time history analysis*

In order to estimate the seismic demands more accurately, we now perform time history analysis of the structure subjected to Uttarkashi and Bhuj earthquake records. More accurate estimates of the corresponding capacities, both at yield and at collapse, are estimated later from an incremental dynamic analysis of the structure.

A bilinear hysteresis stress strain model is selected for performing the time history analysis. Coefficient of damping is chosen to be 5%. The analysis is run for 60 s for Bhuj and 20 s for Uttarkashi (Figs. 6, 7).

For Bhuj (Uttarkashi) ground record, the peak base shear (Fig. 6) in the portal frame (V_b) is found to be 46.4 kN (39.0 kN) at time 40.3 s (4.56 s), and the peak displacement (Fig. 7) is found to be 0.195% (0.29%) also at time 40.3 s (4.56 s). The figures in parenthesis in the previous sentence refer to the Uttarkashi ground record.

Detailed results, for both Bhuj and Uttarkashi records, are summarized in Table 2. These will be discussed in detail along with the results of the other two structures, in Sect. 4.

3.1.3 *Incremental dynamic analysis*

As discussed in Sect. 2.4, failure of a dynamically excited structure can also be given in terms of S_a and the structural capacity in terms of drift ratio is presented here. We keep linearly amplifying the record and thus incrementing the IM (Fig. 8). The first yielding for Bhuj

Table 2 Summary of results for portal frame (B = Bhuj, UK = Uttarkashi)

Criteria	Capacity (per IS code)	Pushover (using factored loads)			Time history analysis		IDA			
		Design demand (per IS code)	Yield	Collapse	B actual	UK actual	B yield	UK yield	B collapse	UK collapse
Column no.	2	3	4	5	6	7	8	9	10	11
IDR (%) (at roof)	0.4	0.08	0.56	inf	0.195	0.29	0.79	0.725	2.7	3.80
Total Base Shear (kN)	23.4	10.8	75 (C2 bottom)	126	26.4	39	100 (both columns)	97.3 (both columns)	112	112.8
Beam Moment* (kNm)	54.7	45	99	140	21.1	33.4	87	82	106.6	109.3
Column Moment* (kNm)	59.2	47	100	116	26.4	41.2	102	102	104.5	105.1
Spectral acc (g)	– –	2.5	–	–	0.23	0.33	0.88	0.8	2.4	2.2
PGA (g)					0.1	0.24	0.4	0.6	1.05	1.58

* At critical cross section

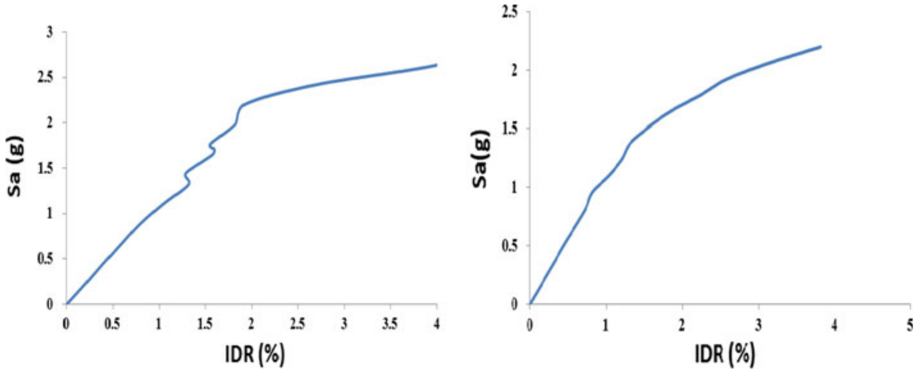


Fig. 8 IDA curves for portal frame. *Left*: Bhuj loading and, *right*: Uttarkashi loading

record anywhere in the structure occurs for an S_a of 0.88 g (at centre of the beam). The peak base shear for this amplified record is identified and is found to be $A_h = 0.85$. The peak IDR for the same record is 0.79%. Collapse occurs not via mechanism formation but when the structure violates the $0.3^* S_e$ law as mentioned in previous section, at a PGA of 1.05 g when $S_a = 2.4$ g. Just prior to collapse, the peak values of base shear reached is $V_b = 112$ kN ($A_h = 0.95$) and the peak IDR is 2.7%.

Moving to the Uttarkashi record, the first yielding record anywhere in the structure occurs for $S_a = 0.8$ g, corresponding to which $A_h = 0.835$. The peak IDR for the same record is 0.725%. Collapse occurs (because it violates the rule described in Sect. 2.4) when the slope of the graph becomes less than 0.3 which occurs at a PGA of 1.58 g when $S_a = 2.2$ g. Just prior to the collapse, the peak values reached are: $V_b = 112.8$ kN($A_h = 0.96$), IDR = 3.8%.

3.1.4 Analysis of the portal frame results

The results from static pushover analysis, actual time history analysis and incremental dynamic analysis, for both Bhuj and Uttarkashi records, are summarized in Table 2. Five structural response categories are selected—deflection, base shear, moments in critical sections and spectral acceleration; the PGA is also listed where relevant. It should be noted that all design demands (column 3) include the load factor of 1.3, and except deflection, all capacities (column 2) incorporate the capacity factor of 1.2. The base shear capacity of 23.4 kN has been found to be that value which causes the combined axial and bending capacity-demand ratio of the beam to be 1.0.

Let us first look at the time history analyses of the portal frame under the actual Bhuj and Uttarkashi records. Clearly, there is significant difference in each category of response under the two records—a difference that would perhaps be missed in a code-specified static analysis. Further, while the ratios of the structural responses for these two records are around 1.5, the Uttarkashi PGA is 2.4 times that of the Bhuj PGA, underscoring the unsuitability of PGA as an intensity measure.

When each of these two records is scaled up, we first reach yield, and if the amplification is continued, we encounter collapse. In other words, we get the yield and collapse structural capacities in each category of response under the given excitation. As may be expected for this SDOF structure, although the responses under the actual records vary by a factor of about 1.5, the yield capacities under the two records are very close (within 5—10% of each other), and the collapse capacities are even closer (with the exception of the roof deflection).

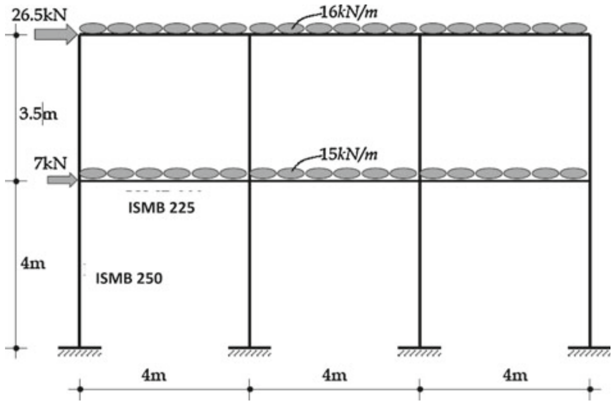


Fig. 9 Description of structure and member-load properties

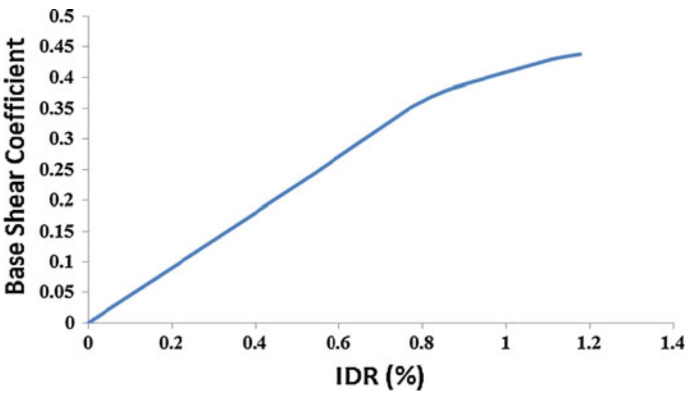


Fig. 10 Pushover graph for the two storey frame

We now compare the efficacy of the static analysis for this problem. The design demand column (column 3) is compared with the time history analysis columns; the pushover yield column is compared with the IDA yield columns; the pushover collapse column is compared with the IDA collapse columns. Finally the capacity (as per IS code) column is compared with the static and the IDA capacity columns (in yield as well as in collapse).

The design demand (per IS code) severely underestimates the actual (i.e., peak dynamic) deflection and base shear demands on the structure; the other demands are comparable. It should be remembered that the design demand column is the result of factored loads whereas the time history analyses make use of unfactored loads.

The pushover yield capacities are comparable to the IDA yield capacities; although pushover analysis seems to be conservative in estimating deflection and base shear capacities. The collapse capacities tell a different story: the pushover analysis overestimates all collapse capacities compared to the IDA counterparts.

The code-specified capacity (column 2) grossly underestimates the capacity in every category of response, by a factor of at least 1.5 at yield, and at least 2.0 at collapse.

We now analyze two more structures in the same manner as above and pull the inferences together in Sect. 4.

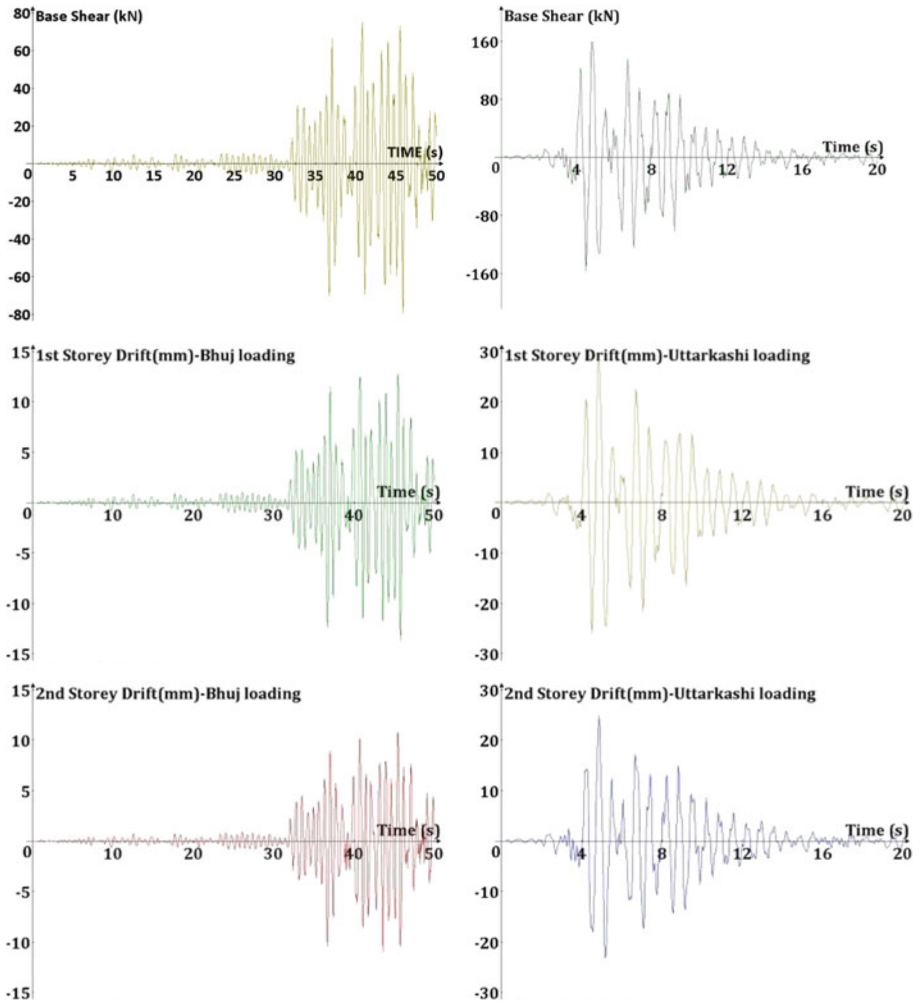


Fig. 11 Base Shear, 1st storey drift and 2nd storey drift for 2 storey structure subjected to Bhuj loading (*left*) and Uttarkashi loading (*right*)

3.2 Two storeyed three bay frame

The second structure analysed in this paper is a 2 storey 3bay steel moment resisting frame (Fig. 9) designed according to the current IS codes (BIS 2002, 2007). Damping coefficient is taken as 5% as before. The first mode period of the structure is 0.70 s. The member sections are found to be ISMB 225 and ISMB 250 for beams and columns respectively. The yield strength is taken to be 250 MPa.

The governing criterion is strength. Combined axial and flexural check is performed and beam (1) is found to be critical with the ratio of 0.98 followed by column (2) with the ratio of 0.92.

When subjected to pushover loading (Fig. 10), It is seen that the structure collapses at a base shear coefficient of 0.45. A point worth noting is that the slope of the curve (Fig. 10)

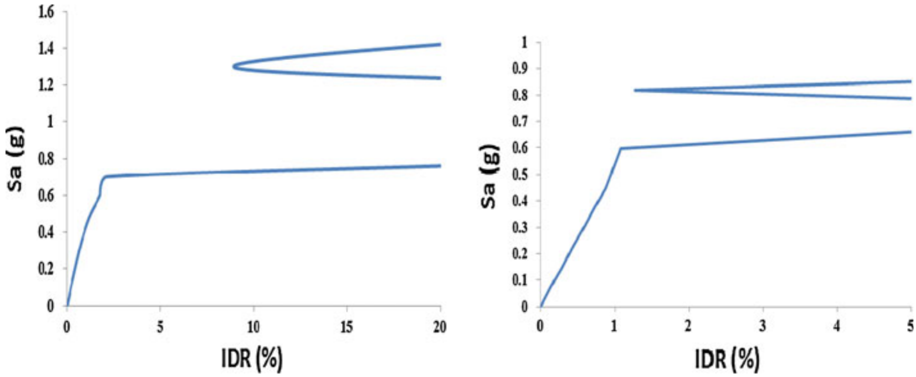


Fig. 12 IDA Curve due to Bhuj (*left*) and Uttarkashi (*right*) loading respectively for two storey frame

just prior to collapse is not too small as compared to the initial slope suggesting that prior to the structural collapse, hinge formation has not occurred in all the members.

The two-storey structure is now subjected to Bhuj and Uttarkashi records, and the response time histories are shown in Fig. 11.

When subjected to incremental dynamic analysis, the structure exhibits an interesting phenomenon called resurrection for both records (Fig. 12). There is an intermediate zone of collapse between $S_a = 0.7\,\mathrm{g}$ and $1.3\,\mathrm{g}$ for Bhuj loading and $S_a = 0.7\,\mathrm{g}$ and $0.8\,\mathrm{g}$ for Uttarkashi loading. The structure becomes unstable in these ranges. However, it resurrects back from failure and reaches collapse at $S_a = 1.4\,\mathrm{g}$ and $0.9\,\mathrm{g}$ respectively.

Table 3 summarizes the results of the pushover, time history and IDA analysis of the structure subjected to Bhuj and Uttarkashi ground motion records. The results are discussed in detail in Sect. 4.

3.3 Three storeyed 2 bay frame

The third structure analysed in this paper is a 3 storey 2 bay steel moment resisting frame (Fig. 13) designed according to the current IS codes (BIS 2002, 2007). Beam length and column height are 5 and 4 m respectively. Damping coefficient is taken as 5% as before. The fundamental time period of structure is 0.945 s. The member sections are found to be ISMB 350 and ISMB 250 for beams and columns respectively. The yield strength is taken to be 300 MPa.

The UDL shown here is the factored load obtained by multiplying actual load by a factor of 1.3. Horizontal load represents that arising from equivalent static analysis (earthquake load) as obtained from IS 1893:2002. As for the other two structures, The governing criterion is strength. Combined axial and flexural check show that beam (1) is critical with the ratio of 0.98 and column (2) is critical with the ratio of 0.99.

Figure 14 shows the pushover graph for the frame when subjected to incremental static horizontal loads. One point to be noted here is the decrease in the base shear capacity of the structure prior to collapse (not shown). This maximum value is considered as the ultimate base shear.

Figure 15 show the IDA curves when the structure is subjected to Bhuj and Uttarkashi records respectively. In case of Bhuj record, the structural failure occurs when IDR becomes 10% (see Sect. 2.4). In case of Uttarkashi record, the structure exhibits a phenomenon called

Table 3 summary of results for 2 storey 3 bay frame

Criteria	Capacity (per IS code)	Pushover (using factored loads)			Time history analysis		IDA			
		Design demand (per IS code)	Yield	Collapse	B actual	UK actual	B Yield	UK yield	B collapse	UK collapse
Column no.	2	3	4	5	6	7	8	9	10	11
IDR 1st floor %	0.40	0.20	0.47	Inf	0.34	0.72	0.69	0.88	2.50	2.50
IDR 2nd floor %	0.40	0.20	0.48	Inf	0.32	0.72	0.64	0.83	2.86	2.86
Total Base Shear (kN)	43.5	43.5	108 (beam 1, right)	221	80	163	165 (beam 1, left)	185 (col. 2, bottom)	213	210
Beam Moment* (kNm)	39	37	70	77	27	73	54.7	76.9	76.2	79
Column Moment* (kNm)	59.2	43	62.7	104	47	97	93.8	103.1	103	104
Spectral acc (g)		2.5	–	–	0.17	0.37	0.33	0.45	0.7 / 1.4	0.75
PGA (g)					0.11	0.24	0.19	0.30	0.42	0.5

* At critical cross section

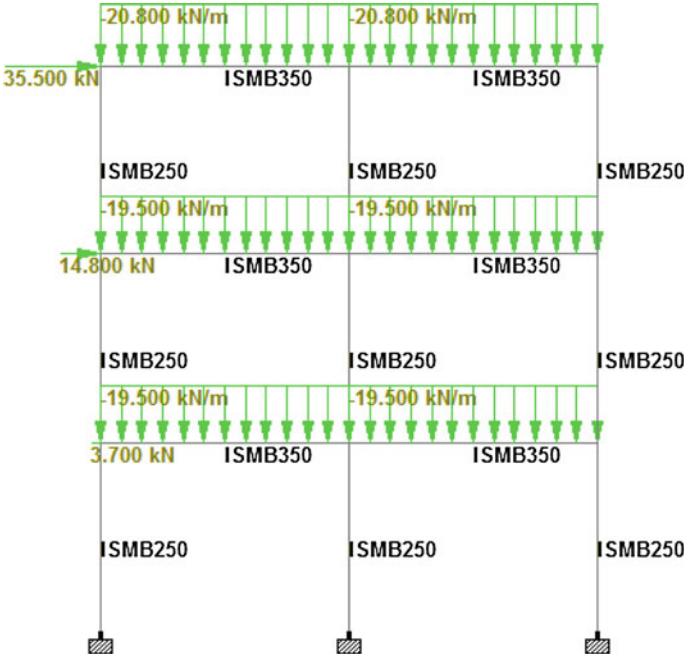


Fig. 13 Three storey two bay frame (column height = 4 m, beam length = 5 m; beam and column numbers increase left to right, then bottom to top)

weaving where in the structural response decreases at a certain step even when S_a increases. In this case, the structural slope falls almost to 0.3^*S_e (see Sect. 2.4) and at that point it is assumed that the structure fails. The pushover, time-history and IDA results are summarized in Table 4 below.

The R factor is a serious source of underdesign in this structure. Both Bhuj and Uttarkashi earthquakes are moderate when compared to earthquakes like Kobe, 1995 where PGA was 1.28 g. It can be seen that the structure yields under unscaled Uttarkashi earthquake ground motion record. A stronger earthquake can prove disastrous for this kind of structure.

4 Discussion of results—true demand and capacity versus codal provisions

IS1893 prescribes design base shear for structures below 40 m. Three steel moment resisting 2-D framed structures were analyzed, all in Seismic Zone V, subjected to Bhuj and Uttarkashi records. Here, we compare the codal provisions with the actual performance of the three structures. Four response categories are chosen: peak IDR, total base shear, critical beam moment and critical column moment. In each of these categories, and for each structure, the comparisons are made in terms of four ratios given below. The numbers in parentheses refer to column entries of Tables 1, 2 and 3.

A. **Codal CDR.** (Code specified factored capacity / code specified factored demand) = (2)/(3).
B. **Capacity ratio.** (Actual capacity / code specified factored capacity)= min((10),(11))/(2) for shear and moments, and min((8),(9))/(2) for IDR.

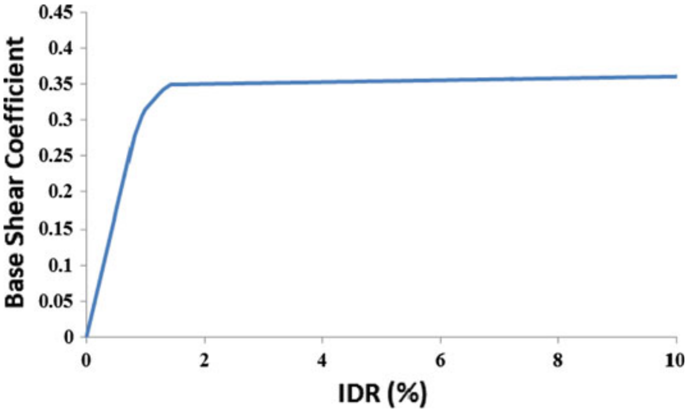


Fig. 14 Pushover graphs for the 3 storey 2 bay frame

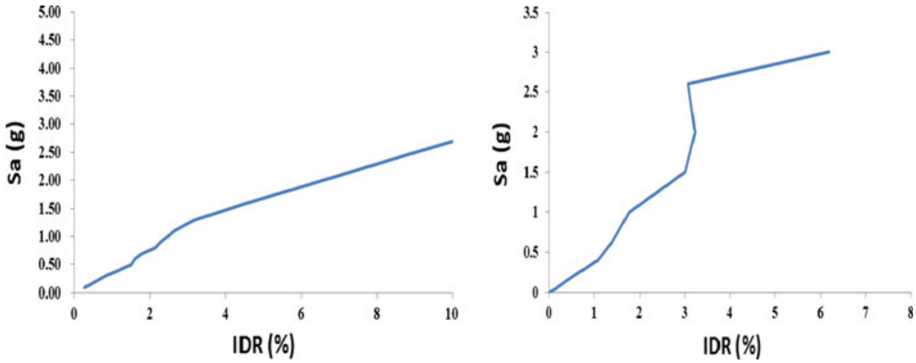


Fig. 15 IDA curves for 3 storey frame—Bhuj (*left*) and Uttarkashi (*right*) records

C. **Demand ratio.**(Actual demand/ code specified factored demand)= max((6),(7))/(3)
D. **Actual CDR.** (Actual capacity/ actual demand)= min((10)/(6),(11)/(7)) for shear and moment, and min((8)/(6),(9)/(7)) for IDR.

In the above ratios, the code specified factored capacity refers to column 2 while the code specified factored demand refers to column 3 of Tables 1, 2 and 3. The actual capacity refers to the ultimate capacity in column 10 and 11 for base shear and moments, and to the yield capacity in columns 8 and 9 in case of IDR. The actual demand refers to columns 6 and 7 of the three Tables. For a ratio involving the actual capacity, the lower result between the two records—Bhuj and Uttarkashi, is adopted; while for a ratio involving actual demand, the higher result between the two earthquake records is selected.

Figure 16 describes the four ratios in each of the four response categories for the three structures. The first set of bars in each figure shows the codal CDRs. Since the code specified capacities and demands include the respective design factors in them, it is not surprising that the critical codal CDR for each structure is very close to one.

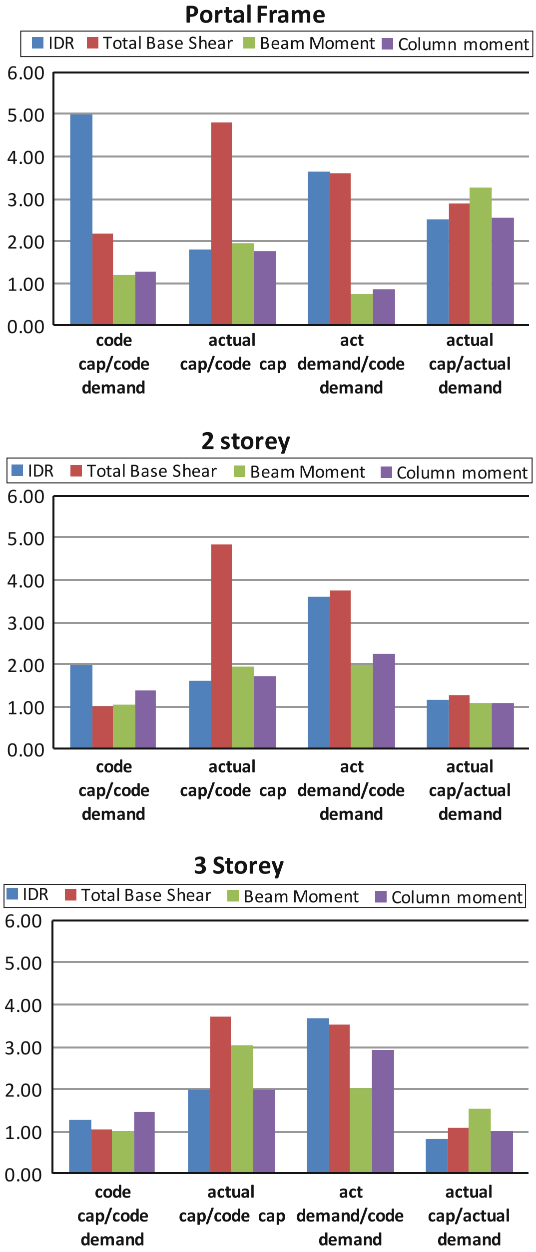
The second set of bars in each figure shows the capacity ratios. Clearly the ratios are at around 2.0 or higher for each structure and might (erroneously) suggest an amount of conservatism in the codal provision. Such false sense of security is lost when one looks at the third set of bars—the demand ratios. The demand ratios are greater that one in most cases, and

Table 4 Summary of results for 3 storeyed frame

Criteria	Capacity (per IS code)	Pushover (using factored loads)			Time history analysis		IDA			
		Design demand (per IS code)	Yield	Collapse	B actual	UK actual	B yield	UK yield	B collapse	UK collapse
Column no.	2	3	4	5	6	7	8	9	10	11
IDR 1st floor (%)	0.4	0.2575	0.7575	inf	0.48	0.95	0.87	0.79	10	6.175
IDR 2nd floor (%)	0.4	0.31	0.905	1129	0.6	1.01	1.16	0.86	10	4.5
IDR 3rd floor (%)	0.4	0.215	0.625	40	0.42	0.61	0.77	0.54	10	2.65
Total base shear (kN)	56	54	158	234	103	191	178	173	210	208
Beam moment* (kNm)	89	87	165	293	96	176	179	147	271	273
Column moment* (kNm)	64.8	44	122	133	80	129	127	128	130	131
Spectral acc (g)		2.5	–	–	0.22	0.35	0.4	0.3	2.7	3
PGA (g)					0.10	0.24	0.19	0.20	2.0	1.8

* At critical cross section

Fig. 16 Codal provisions versus actual demand and capacities of the three structures



sometimes almost as high as four. Clearly, the IS codes severely underestimate the seismic demands; this has been qualitatively acknowledged in IS1893:2002 clause 6.1.3.

The final set of bars in the three figures show the actual CDRs. It should be noted that the capacities and demands in these CDRs are unfactored. Thus the fact that some of these CDRs are less than one is cause for concern. Nevertheless, the actual safety of each structure under the respective earthquake records can only be found after appropriate reliability analyses of the structures (e.g., Cornell et al. 2002).

5 Conclusions

This study looked at the adequacy of the current Indian seismic design code for low-rise steel structures by subjecting three representative 2-D frames to incremental dynamic analysis under Uttarkashi and Bhuj earthquake records. It is seen that the equivalent static method recommended by the IS code for low-rise structures significantly underpredicts both seismic demands and structural capacities in four different response categories. Further, the actual capacities of the structure may be inadequate in the face of real seismic demands. Although two records are never enough to make such generalization, it should be borne in mind that the latest revision of the code was released not long after these two earthquakes. Further, it is also true that only a full probabilistic analysis can determine whether these structures actually exceed the limiting collapse prevention probability, e.g., 2% in 50 years (SAC-FEMA 2000). Nevertheless, this study does underscore that a comprehensive investigation is in order and that there is scope for rationalization in the IS codal provisions.

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