Criteria for performance evaluation of RC building frames using non-linear time history analysis for performance-based design

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Abstract

Displacement-based design (DBD) is emerging as the new trend for seismic design of buildings. Several displacement-based design procedures have been developed in recent times. The performances of buildings designed using these methods are usually evaluated by conducting non-linear time history analysis (NLTHA). The efficiency of performance assessment depends on proper non-linear material modelling, selection of proper earthquake records and their scaling and appropriate setting up of limit states (acceptance criteria). The present paper discusses the provisions in various seismic guidelines including ATC 63, FEMA P695 (2009) and PEER Centre report No.2010/05 and recent research findings regarding the above parameters. According to FEMA P695, only the far-field record set is required for collapse assessment as there are many unresolved issues concerning the characterization of near-fault hazard and ground motion effects. To verify this, the response spectra are plotted for 10 selected far-field and near-field ground motions and found that there is considerable increase in the response of long period structures when they are subjected to near-field pulses. Time history analysis done on a 15-storeyed frame (which is designed as per DBD) shows an increase in roof displacement of the order of two and inter-storey drift amplification of about 2.7 near the base, when near-field ground motions are used for performance assessment.

Keywords: Time history analysis, hysteretic response, spectral matching, performance objective

1. Introduction

Conventional seismic design is force-based and it aims to find out an equivalent lateral force (design base shear) which is very much less than the elastic design force by utilizing the ductility capacity of the structure. The design philosophy is to ensure that the structure possesses at least a minimum strength to withstand frequent minor earthquakes without damage, resist moderate earthquakes without significant structural damage though some non-structural damage may occur and withstand a major earthquake without collapse. Since damage is more related to displacement than forces, researchers are now developing design procedures which are based on displacement. Displacement-based design (DBD) aims to design a structure which can achieve a design displacement thus utilizing its inelastic capacity. Performance assessment of the designed frames is an important phase of DBD and this can be done using an inelastic time history analysis.

Inelastic time-history analysis (ITHA) is considered as the most accurate method for ensuring that inelastic deformations and rotations are within the design limits, when a structure is subjected to earthquake forces. A large number of subjective modelling decisions will generally be needed, and it is essential that the importance of these choices be properly understood by the analyst, who should have appropriate experience in ITHA and knowledge of material behaviour before using it for design verification (Priestley, 2007). A sound nonlinear analysis must consider inelastic material and geometric nonlinear behaviour, damping, element type selection, acceptance criteria and properly scaled ground motions (Liao et al., 2010). These parameters are discussed in detail in the following sections.

2. Inelastic behaviour of reinforced concrete

The inelastic response of members is defined by force-deformation equation describing the loading, unloading and reloading of the members (Priestley, 2007). The collective equations describing the response for a given member are termed the hysteresis rule for the member. It is important that the hysteresis rule should provide an accurate representation of the material and structural response of the member.

Identification of key deterioration and collapse modes is an important pre-cursor to the choice of non-linear analysis model. Modelling the inelastic behaviour of R C elements is a difficult task. Both strength and stiffness degradations are usually observed in beams and columns. There are three main components of deformation in an R C element which are due to flexure, shear and slippage of bars. While modelling the inelastic behaviour of R C elements, pinching of hysteresis loops may occur because of high shear force or bond slippage of steel bars (Banon, 1980).

The early research concentrated primarily on the flexural behaviour of components under monotonic loading. Research on the inelastic behaviour of reinforced concrete components under earthquake-like loading reversals was initiated in the 1960s. Under inelastic loading reversals, the ductility capacity of components is generally decreased, mainly due to the increase of shear deformation and bond deterioration. It is found that a member designed to fail in flexure tends to fail through shear under repeated loading reversals. When the number of loading reversals is increased significantly, the member tends to fail through bond at a much lower deformation than would be expected under a monotonic loading (Park, 1984).

Most of the early work in the inelastic analysis of concrete structures was based on bilinear systems. However, it was soon realized that R C elements do not offer the large energy dissipation capacity which is inherent in a bilinear system. A more general stiffness degrading model for reinforced concrete was first introduced by Clough (1966). This model has the advantage over the bilinear model that the loading stiffness is modified as peak rotation increases. Takeda (1970) developed a non-linear model which can closely reproduce the behaviour of R C elements in flexure. The model has a bilinear envelope curve, and it is designated to dissipate energy at low cycles once the cracking point is exceeded. Emori (1978) and Takayanagi (1979) later introduced modifications into the Takeda model to take into account the slippage and shear pinching effects.

The moment-rotation relationship for the modified Takeda model is shown in figure 1. The primary curve for the model is a bilinear curve which changes slope at the point of yielding. Two other modifications were introduced by Litton (1975). The first is for stiffness degradation in the unloading part. Instead of unloading with initial slope (k_0), the parameter α is used to modify the unloading stiffness k_u. The second modification is for reloading stiffness k_l. Instead of loading towards the point of maximum (B), another point (A), which is set by parameter β is aimed at. Parameter α decreases the unloading stiffness and parameter β increases the reloading stiffness.

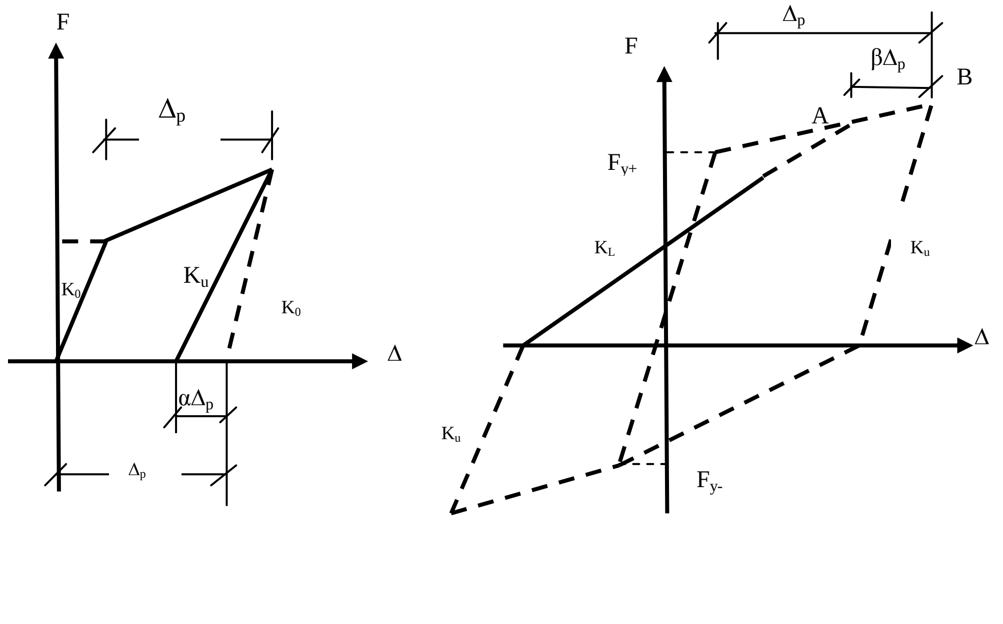

Figure 1 Parameters α and β for modified Takeda Model

2.1 Material constitutive laws

The strain hardening characteristics of steel and the Bauschinger effect can be best represented by the Ramberg-Osgood model (1967). The Ramberg-Osgood equation can be written as

$$\varepsilon = \frac{\sigma}{E} + \alpha \frac{\sigma_0}{E}\left(\frac{\sigma}{\sigma_0}\right)^{\eta} \tag{1}$$

The commonly used value of η is 5 or more. The hardening behaviour of the material depends on the material constants α and η. The value $\alpha\sigma_0/E$ is taken as the yield offset. Elastic strain at yield is σ_0/E. The value of α is taken corresponding to a strain of 0.2%.

Unlike steel, concrete shows different behaviour under tension and compression. A section will crack after the first few cycles and there would be no tensile contribution after that point. Concrete also shows a different behaviour when confined. Behaviour of confined and unconfined concrete upto peak stress (f_c') is almost the same, but their unloading slopes are different (Banon, 1980). Concrete when properly confined can carry compressive forces well beyond its unconfined ultimate strain. However, the overall behaviour of a section is dominated by steel and any reasonable approximation in concrete stress-strain curve will have little effect on moment-curvature relationship.

2.2 Moment-curvature behaviour of a section

The moment-curvature relation, developed based on stress-strain curves of Modified Mander model (which can be used for both confined and unconfined concrete), is linear up to yield point. Section stiffness (*EI*) is the slope of the M-φ curve. Other points on the curve may be defined by setting different values of concrete strain (ε_c). But, moment-rotation parameters are the actual input for modelling the hinge properties and this can be calculated from the moment-curvature relation.

Axial load on a member also greatly modifies the shape of M-φ curve. Moderate axial load on a member increases its yield moment and initial section stiffness, but it limits the capacity of a member to sustain high strains. The presence of axial compression reduces ductility and accelerates strength decay. When a building is subjected to dynamic loads, axial loads in the column change at each time step. Variation of axial load around the average axial load (gravity load) may be quite significant for perimeter columns. The calculated M-φ relationships represent an average behaviour for members with axial load.

In the earthquake resistant design of R C frames, codes usually specify that positive moment capacity of a girder has to be not less than 50 percent of its negative moment capacity. Hence, most members in a real building have different areas of steel at top and bottom. Furthermore, both yield moment and stiffness of a non-symmetric section differ in the loading directions. This is also the case with T-sections. A simple method of analysing such sections is to use an average stiffness and to have different yield moments in negative and positive directions for the Takeda model.

2.3 Shear behaviour

In most analytical studies of R C structures, shear deformation is assumed to be elastic. Recent studies have shown that shear deformations have an inelastic behaviour which is quite different from flexural behaviour. While the shear strength can be determined by means of various empirical expressions, shear distortions in the post cracking and post yielding regions are difficult to establish (Saatcioglu, 1991). Compression field theory is one of the approaches that have been shown to produce good predictions of shear force-shear deformation primary curve for members under combined shear, flexure and axial loading.

Current design practice requires structural members to be proportioned to yield in flexure prior to shear failure. In order to prevent shear failure, design codes prescribe specifications (e.g., ductile detailing requirement of IS 13920:1993) for adequate shear reinforcement, corresponding to the ultimate moment capacity level.

2.4 Bond slippage

In an interior beam- column joint, beam reinforcement passing through the joint may be more susceptible to bond failure. Penetration of yielding from both sides of the joint can destroy the bond totally within the joint (Saatcioglu, 1991). Slippage of beam reinforcement in this subassembly can produce substantial deformations at member ends. This type of bond slip deformation is characterized by excessive pinching of hysteresis loops. In such cases, pinching of hysteresis loops should be included in the analytical model for reinforcement slip. However if total loss of bond in the joint is prevented, pinching of hysteresis loops is reduced significantly.

3. Site hazard and ground motion

Maximum considered earthquake (MCE) are generally described with the probabilistic criteria specified corresponding to the risk of a 2% probability of exceedance within a 50-year period, which is equivalent to a return period of 2,475 years. Design-basis earthquake (DBE) is defined as the earthquake ground motion that is two-thirds of the corresponding MCE ground motion. It corresponds to risk of a 10% probability of a 50-year period having a return period of 475 years. Serviceability earthquake has a probability of exceedance of 50% in 50 years (ATC 40).

Ground motion records provide the most direct approach for analyzing the performance of a structure. The PEER Next Generation Attenuation (NGA) database is an update and extension to the PEER Strong Motion Database and provides a larger set of records, more extensive metadata, with some corrections made to information in the original database. But, NGA site includes only acceleration time-history files.

3.1 Selection of ground motion records

Since earthquake magnitude (M) and distance (R) of the rupture zone from the site of interest are the most common parameters related to a seismic event, it is evident that the simplest selection procedure involves identifying these characteristic (M, R) pairs. The geotechnical profile is known to influence seismic motions by modifying both their amplitude and the computed response spectra. In order to introduce the soil profile into the selection process, site classification and strong-motions recording sites must be known with a high degree of confidence. Generally speaking, shear-wave velocity at the uppermost 30m can be used as a suitable metric for site classification, although there are cases where deeper soil structure can also exert a strong influence. Apart from the soil profile, strong-motion duration constitutes a complementary criterion for the selection of real records. The maximum acceleration over maximum velocity (a/v) ratio has been proposed as a complementary measure for the selection process (Evangelos et al., 2010).

The selection of real accelerograms is often performed on the basis of compatibility between the response spectra and a corresponding 'target' spectrum as defined by code provisions or computed directly through a probabilistic seismic hazard analysis (PSHA). Spectral matching is the most commonly proposed earthquake record selection method by seismic codes. For an assessment of structural performance, the ground motion intensity measure (*IM*) customarily adopted is spectral acceleration near the fundamental period of the structure with a damping ratio of 5%. Firstly, this choice is driven by convenience since seismic hazard curves in terms of S_a (T_1) are either readily available or easily computed. It is clear that S_a is related to both structural and seismic motion characteristics, while the PGA of a record, which constituted a commonly used IM in the past, accounts only for strong motion features.

National seismic codes prescribe general guidelines but do not provide specifics for selecting the type of earthquake records required for nonlinear dynamic analysis purposes. The period range for spectral matching varies among code provisions. Moreover, the minimum number of records required for structural analysis is three in all cases and when a set of at least seven ground-motions is used, the structural engineer is allowed to compute the mean structural response. Otherwise, only a maximum response value is computed if three to six recordings are used. As per ASCE7-05, the ground motions shall be scaled such that the average value of the 5 percent damped response spectra for the suite of motions is not less than the design response spectrum for the site for periods ranging from 0.2T to 1.5T where T is the natural period of the structure in the fundamental mode for the direction of response being analyzed.

3.2 Far field and near field ground motion records

The ATC-63 guidelines include two suites of ground motions and procedures for scaling these relative to the seismic hazard intensities of the Seismic Design Categories. One set of records, termed the “Far-Field” record set, includes twenty-two ground motion pairs recorded at sites located greater than 10 km from fault rupture. The second “Near- Field” set includes twenty-eight pairs of motions recorded at sites located within 10 km of the fault. Records in each set were selected to provide an unbiased suite of motions that represent strong ground motion shaking with earthquake magnitudes of 6.5 to 7.9. Within each set, the records are normalized by their peak ground velocities to reduce the scatter while preserving variations that are consistent with variations observed in ground motion attenuation functions.

As per FEMA P695, only the Far-Field record set is required for collapse assessment. This is done for reasons of practicality, and in recognition of the fact that there are many unresolved issues concerning the characterization of near-fault hazard and ground motion effects. The Near-Field record set is provided as supplemental information to examine issues that could arise due to near-fault directivity effects, if needed. If the building is located within 10 km of an active fault, then the Near-Field record set should be selected for collapse evaluation, otherwise the Far-Field record set should be used.

As per PEER Report 2000/02, near-fault motions are greatly different from ground motions away from the fault. Our traditional understanding of site conditions is based on observations of site effects away from the fault. Ground shaking near a fault rupture is characterised by a pulse with very high energy input. Prior to the Turkey earthquakes, there were a total of 8 recordings world-wide recorded at distances less than 20 km from crustal earthquakes with magnitude greater than 7.0. The Chi-Chi (Taiwan) earthquake added an additional 68 recordings. Near-fault effects are not adequately described by uniform scaling of a fixed response spectral shape; the spectrum becomes richer in long periods as the level of the spectrum increases.

Although the response spectrum provides the basis for specification of design ground motions, there is a growing recognition that the response spectrum is not capable of adequately describing the seismic demands presented by brief impulsive near-fault ground motions. This indicates the need to augment the response spectrum with a time domain representation of near-fault ground motions, preferably in the form of simplified pulses whose parameters such as period and peak velocity can be related to earthquake magnitude, fault distance, rupture directivity condition and site conditions (Somerville, 2000).

A comparison of response spectra obtained using the programme “Seismospect” for 10 near-filed and far-field ground motions taken from PEER database are shown in figure 2. It is clear from the figure that near-field pulses have higher spectral accelerations at longer periods.

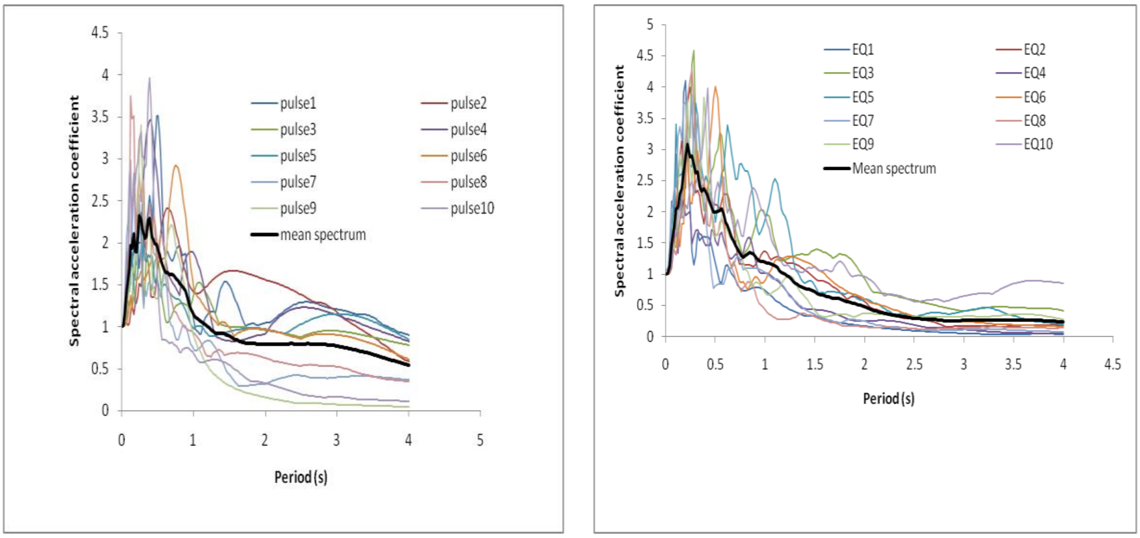

Figure 2 Normalized response spectra for (a) near-field ground motions, strike normal components (b) far-field ground motions

3.3 Scaling methods

Two principal procedures are used for ground motion modification: direct scaling and spectral matching. The direct scaling procedure consists of determining a constant scale factor by which the amplitude of an accelerogram is increased or decreased. Because elastic response spectra correspond to linear response of single-degree-of freedom systems, the same scale factor applies to spectral accelerations at all periods. In contrast, spectral matching adjusts the frequency content of accelerograms until the response spectrum is within user-specified limits of a target response spectrum over a defined period band (PEER center report No.2010/05).

The scaling process involves two steps namely, normalization and scaling. As per FEMA P695, individual records in each set are first “normalized” by their respective peak ground velocities. This step is intended to remove unwarranted variability between records due to inherent differences in event magnitude, distance to source, source type and site conditions, without eliminating overall record-to-record variability. Second, normalized ground motions are collectively scaled (or “anchored”) to a specific ground motion intensity such that the median spectral acceleration of the record set matches the spectral acceleration at the fundamental period, *T*, of the index archetype that is being analyzed.

4. Performance evaluation

Performance evaluation is based on the results of nonlinear dynamic analyses. It requires judgment in interpreting analytical results, assessing uncertainty, and rounding of values for design. Performance objectives (PO) are selected and expressed in terms of expected levels of damage resulting from expected levels of earthquake ground motions. A performance level represents a distinct band in the spectrum of damage to the structural and non-structural components and contents, and also considers the consequences of the damage to the occupants and functions of the facility (Bertero and Bertero, 2002). An example for the quantification of the POs to control structural, non-structural and contents damage is shown in Table 1. The performance levels are keyed to limiting values of measurable structural response parameters, such as drift and ductility (monotonic and cumulative), structural damage indexes (*DM*), storey drift indexes (*IDI*), and rate of deformations such as floor velocity, acceleration and even the jerk (in the case of frequent minor earthquake ground motions). The structural damage index is expressed as

$$DM = \frac{\delta - \delta_y}{\delta_{umon} - \delta_y} + b\frac{E_{H\mu}}{F_y \delta_{umon}} \tag{2}$$

where δ is the maximum displacement during earthquake ground motion, δ_y the yield displacement, δ_{umon} is the ultimate displacement at impending collapse under monotonic loading, $E_{H\mu}$ is the hysteretic energy dissipated by plastic deformation, F_y is the yield strength of the system and b is a parameter controlling strength degradation.

Table 1 Quantification of Performance Objectives (Bertero and Bertero, 2002)

Performance level	*EQ return period*	*Structural damage (local DM index)*	*Non-structural damage (IDI)*	*Contents damage (Floor acceleration)*
Fully operational	*43*	*0.2*	*0.003*	*0.6g*
Operational	*75*	*0.4*	*0.006*	*0.9g*
Life safety	*475*	*0.6*	*0.015*	*1.2g*
Near collapse	*970*	*0.8*	*0.020*	*1.5g*

When the performance levels are selected, the associated limiting values become the acceptability criteria to be verified in later stages of the design. Once the limit value of the parameter has been selected for a particular earthquake hazard level to completely define the design criteria, it is still necessary to define the acceptable conditional probability of going beyond that limit state (failure probability).

4.1 Performance verification of an example frame

A 15-storeyed frame designed by direct displacement-based design method is selected for the present study. The frame is assumed to have three bays, with a bay width of 6m. The storey height is typically 3.3m, with the ground storey having a height of 4.5m above fixed column bases. The design PGA is taken as 0.6g. The design criteria for DBD is to limit the maximum inter-storey drift in the range of 2%-3% and hence tall frames located in low to moderate seismic intensity regions will behave either elastically or with limited inelasticity. This is in contradiction to FBD wherein frames are designed for ductility levels of 3-5 making it more flexible. In order to study how such frames behave under design earthquake, the frame is intentionally designed for a displacement ductility of 4. It is subjected to 10 far-field and 10 near-field ground motions scaled to a PGA of 0.6g. Table 2 shows a comparison of the average values of peak responses and the corresponding design values. There is an increase in roof displacement of the order of two and inter-storey drift amplification of about 2.7 at the bottom level, when near-field ground motions are used for performance assessment. But the average drift amplification is only 1.4 times the drift under far-field ground motion.

Table 2 Comparison of average responses under near-field and far-field earthquakes

Parameter	*Due to near-field ground motions*	*Due to far-field ground motions*	*Design values*
Maximum roof displacement	*1.06m*	*0.52m*	*1.01m*
Maximum inter-storey drift	*3.4%*	*2.4%*	*2.0%*
Base shear	*2077kN*	*1679kN*	*735kN*
Ductility demand	*1.325*	*elastic*	*4*

The variation of inter-storey drift along the height of the frame under near-field and far-filed ground motions are shown in figure 3. Due to near-field pulses, inter-storey drift is more towards the lower third portion of the frame whereas for far-filed ground motions, inter-storey drift is more in the lower as well as the upper third portions of the frame. The higher mode effects of EQ3 and EQ8 can be clearly seen in the response.

The design base shear force is 735kN corresponding to a displacement ductility of 4. The average base shear demand due to near-field and far-field ground motions is 2077kN and 1679kN respectively. The base shear due to far-field earthquake is almost two times the design base shear and this will be taken care of by the over-strength of the frame which was initially assumed during design. But higher value of design ductility resulted in large inter-storey drift and member end chord rotations. Hence, the desirable value of design ductility is in the range of 2-2.5. The amplification in base shear, maximum roof displacement and inter-storey drift shows the need for incorporating near-fault ground motion effects in the design response spectrum.

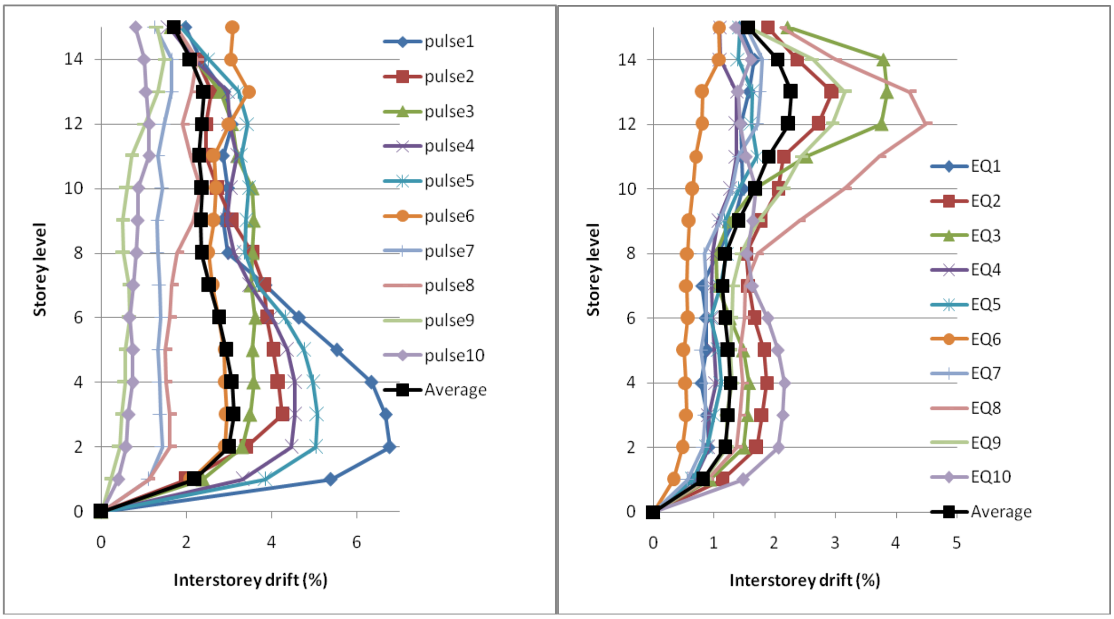

Figure 3 Maximum inter-storey drift ratio of 15-storeyed frame under a) near-field ground motions b) far-field ground motions

5. Conclusions

A comprehensive review on performance evaluation of structures using non-linear time history analysis is presented in this paper. Non-linear material properties and the selection of ground motion and scaling procedures are discussed in detail. It is clear from the above discussion that flexure dominant response shows stable hysteretic loops whereas shear as well as bar slippage causes pinching effects. Setting up of limit states for performance evaluation is usually done as per designer's choice, based on performance objectives specified by owners and risk managers. The provisions in various seismic guidelines including ATC 63, FEMA P695 (2009) and PEER Centre report No.2010/05 on performance verification are discussed. Properly executed, these guidelines help in designing buildings that are capable of achieving the seismic performance objectives and hence result in uniform risk structures. Response spectra generated for a suite of near-field and far-field ground motions shows an increase in response due to near-field earthquakes in the long period range. This conclusion is justified through the non-linear time history analysis of a 15-storeyed frame using far-field and near-field earthquakes and found that there is considerable increase in base shear demand, maximum roof displacement and inter-storey drift due to near-field earthquake.

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