Proceedings of the 9th U.S. National and 10th Canadian Conference on Earthquake Engineering
Compte Rendu de la 9ième Conférence Nationale Américaine et
10ième Conférence Canadienne de Génie Parasismique
July 25-29, 2010, Toronto, Ontario, Canada • Paper No 67

PERFORMANCE EVALUATION OF TALL STEEL STRUCTURES WITH DUAL SYSTEM IN NEAR-FIELD GROUND MOTIONS COMPARING WITH FAR-FIELD GROUND MOTIONS CONSIDERING FEMA 356 PROVISIONS

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ABSTRACT

This paper illustrates performance of tall steel structures with dual system (comprised of SMRFs and RC shear walls) in near-field ground motions considering acceptance criteria introduced by FEMA356. In order to achieve this purpose and compare with far-field ground motions, some nonlinear time history analyses under near-field and far-field records are performed. The records used in this study, have been scaled in a reasonable manner. Thereafter, acceptance criteria of structures are checked. The results show that despite the appropriate performance of dual system under far-field excitations, structural performance is not acceptable under near-field ground motions. The deficiency of Iranian code(Standard No.2800) in exhibition of critical load combination for column's seismic design is another result of this study.

Introduction

Near-Fault ground motions, which have caused much of the damage in recent major earthquakes (Northridge 1994, Kobe 1995, Bam 2003), are characterized by a short-duration impulsive motion that exposes the structure to high input energy at the beginning of the record. The need exists to incorporate this special effect in the design process for structures located in the near-fault region. The near-source factors incorporated in recent codes cannot solve the problem consistently, because design procedures should pay attention to the special frequency characteristics of near-fault ground motions. Moreover, the engineering concepts of performance-based design require a quantative understanding of response to different types of ground motion at different performance levels (Alavi 2000).

Recent studies have shown that severe pulse-type ground motions may significantly increase the seismic lateral displacement and consequently the inter-story drift and inelastic rotation demands on long period structures. In extreme cases, such demands, coupled with the

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action of the gravity load, may pose a collapse hazard. This is particularly true for certain frame-type structures in which the drift demand in the story levels near the base may be amplified by severe displacement pulses and P-delta effects (Anderson 1999). Special precautions will have to be taken to insure adequate performance at both the damage control and life safety levels. Thus it was decided to carry out the study reported herein.

In this article, first 10 and 20-story steel structures with Special Moment Resisting Frames (SMRFs) and Reinforced Concrete Walls are designed on the basis of Iranian Code of Practice for Seismic Resistant Design of Buildings (Standard No.2800). Then, Nonlinear Time-History analyses under near-field and far-field records in BSE-1 level (BSE-1 is defined in FEMA356 as the lesser of the ground shaking for a 10%/50 year earthquake or two-thirds of the BSE-2 at a site) are performed and finally the acceptance criteria of structures are checked in Life Safety (LS) level.

Modeling

The structures modeled in this study, are 10 and 20-story steel structures with dual system (SMRFs and RC shear walls). The general plan of structures is shown in Fig. 1. Height of story in all levels is assumed to be 3.2 m. Columns' sections are boxes and beams consist of IPE profiles with cover plates. RC shear walls are distributed in middle span of peripheral frames. Floors are made of two-way reinforced concrete slabs. Seismic design of structures is based on provisions of Iranian Code (Standard No.2800). Soil type is supposed to be type-II and the structures have special occupancy (seismic importance factor =1.2). Uniform dead load of the floors is supposed 500 kg/m^2 and uniform live load is taken 150 kg/m^2 and 300 kg/m^2 in roof and other story levels respectively. Weight of the peripheral walls is supposed to be 300 kg/m.

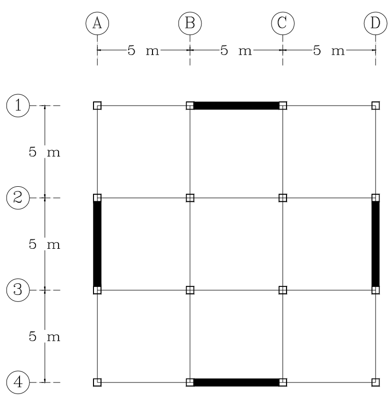


Figure 1. Typical plan of 10 and 20-story structures

Analysis

Nonlinear analyses are performed with PERFORM 3D software. It's a powerful computer program for Nonlinear Time-History and Nonlinear Static analyses. Including FEMA356 acceptance criteria and having capacity of performance point identification through Capacity Spectrum and Target Displacement methods, PERFORM 3D is extensively used in performance-based design methods.

Nonlinear Time-History Analysis

Nonlinear Time-History analysis is one of the most precise methods for determination of structure response to earthquake excitations. Nonlinear Time-History analysis in PERFORM is accomplished on the base of constant average acceleration method (also known as Newmark b=1/4 method) (RAM 2000).

Earthquake Records

At least, three pairs of horizontal ground motion records shall be used. If three time-history analyses are performed, the maximum response of the parameter of the interest shall be used for design. If seven or more pairs of horizontal ground motion records are used, the average response of the parameter of interest may be used for design (FEMA356 2000, Standard No.2800 2005). Near-field and far-field record data used for Time-History analyses in this article are shown in Table 1.

Time Step

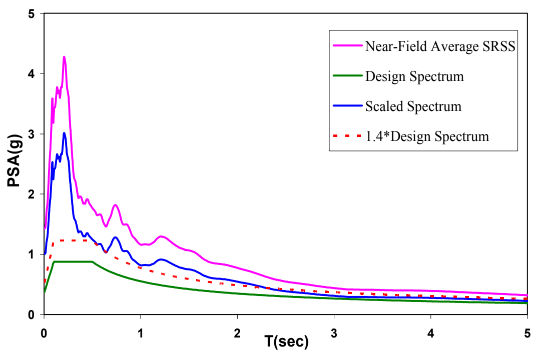
The structure's response shall be captured accurately. Time step determination has direct impression on structural response. For linear multi-degree-of-freedom structure, one method for choosing the time step is to identify the highest mode that contributes significantly to the response and use a time step equal to 1/12 of the period for the mode. Since most structures increase in period as they yield, a time step based on linear behavior should usually be short enough for calculating inelastic response. Also, ground motion shall be captured accurately. Therefore, the time steps determined to analyze structures shall not be greater than the time steps in which ground motions are recorded even for a very long period structure (RAM 2000). In this article, time steps for time-history analyses selected equal to 0.005 sec.

Table 1. Properties of near-field and far-field earthquake records

	Earthquake Name	Evidence Year	Station	Direction	PGA cm/sec^2	PGV cm/sec	PGD cm	Ms
Near Field	Tabas	1978	9101	H1	820	97.8	39.9	7.4
				H2	836	121.4	94.6	
	Bam	2003	Governorship	H1	623	59.7	20.8	6.8
				H2	778	123.7	34.5	
	Landers	1992	24 Lucerne	H1	770	31.9	16.4	7.4
				H2	707	97.6	70.3	
Far Field	San Fernando	1971	262 Palmdale Fire Station	H1	119	12.3	2.7	6.6
				H2	148	8.1	1.9	
	Loma Prieta	1989	1652 Anderson Dam	H1	239	20.3	7.7	7.1
				H2	235	18.4	6.7	
	Imperial Valley	1979	6604 Cerro Prieto	H1	166	11.6	4.3	6.9
				H2	154	18.6	7.9	

Records Scaling

Time-History analyses shall be performed with data sets (two horizontal components) of appropriate ground motion time histories that shall be scaled reasonably. Based on FEMA356 recommendations, for each data set, the square root of the sum of the squares (SRSS) of the 5%-damped site-specific spectrum of the scaled horizontal components shall be constructed. The data sets shall be scaled such that the average value of the SRSS spectra does not fall below 1.4 times the 5%-damped spectrum for the design earthquake for periods between 0.2T and 1.5T seconds (T is the fundamental period of the building) (FEMA356 2000). Iranian Code (Standard No.2800)has similar method for earthquake data scaling.



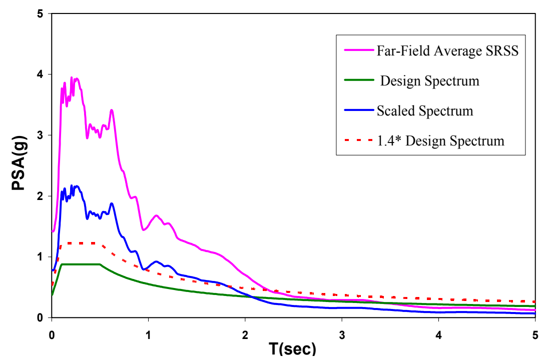
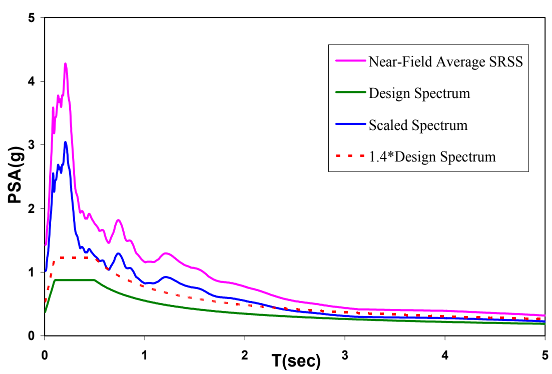


Figure 2. Near-field and far-field record scaling, 10 story structure



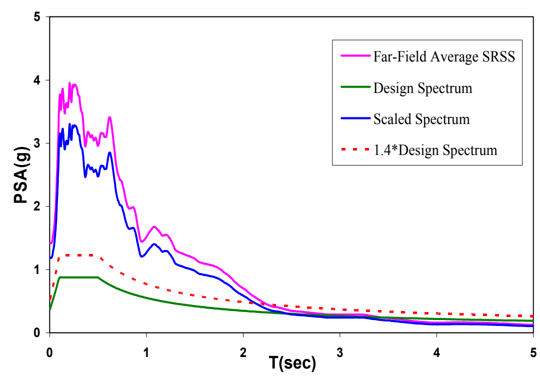


Figure 3. Near-field and far-field record scaling, 20 story structure

Multidirectional Seismic Effects

Structures shall be analyzed for seismic motion in any horizontal direction but when the structure has one or more columns which form a part of two or more intersecting frame elements, in nonlinear analyses, the requirement that multidirectional excitation effects be considered is satisfied by designing elements or components for the forces and deformations associated with 100% of the seismic displacements in one horizontal direction plus the forces associated with 30% of the seismic displacements in the perpendicular horizontal direction (FEMA356 2000).

Nonlinear Properties of Elements

Beams and Columns

The whole properties of beam and column elements such as plastic hinge properties are determined according to FEMA356 recommendations.

Panel Zones

Panel Zones (PZs) have been shown to provide stable strength and stiffness characteristics well into the inelastic range, even after several cycles of inelastic deformation. PZs also have a greater slope of strain-hardening compared to the plastic hinge characteristics of a wide flange shape and are lesser prone to local buckling. Therefore, due to their stable hysteretic behavior, PZs are considered to be a very good source of energy dissipation. This point is considerable particularly in near-field ground motions where the energy imposed on the structure is too excessive (Schneider 1998).

Simple analytical models that do not include PZ deformations result in non-conservative estimations of drift and the lateral strength of the frame. Neglecting PZ distortions underestimates drift by as much as 10% and overestimated the base shear strength by 30% (Schneider 1998).

The model for a panel zone component is shown in Fig. 4.

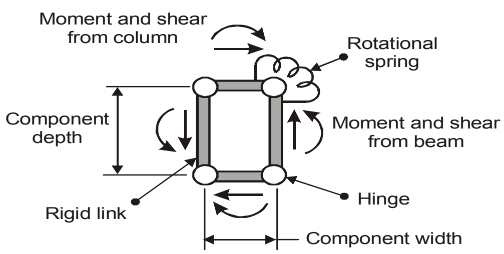


Figure 4. Model for panel zone component (RAM 2000)

This is the Krawinkler model. It consists of four rigid links, hinged at the corners. The moments and shears from the columns and beams act on the rigid links. A rotational spring provides the connection strength and stiffness. This spring has a linear or nonlinear moment-rotation relationship.

For a steel connection you can specify the column and beam sizes, in this case, Krawinkler formulas for calculating PZ properties are as follows (Krawinkler 1987):

Initial stiffness of rotational spring $= 0.95 d_b d_c t_p G$ (1)

Strength at yield point $= 0.55 F_y t_p 0.95 d_b d_c$ (2)

Hardening stiffness $= 1.04 b_{fc} t_{fc}^2 G$

(3)

Deformation at ultimate point = 4 (Deformation at Y point) (4)

Where; d_b = beam depth, d_c = column depth, t_p = panel zone thickness, b_{fc} = column flange width, t_{fc} = column flange thickness, G = shear modulus and F_y = yield stress.

Shear Walls

ATC-40 recommends a hinge length equal to the smaller of one half the cross section depth and one half the wall height (ATC40 1996).
To accept the shear wall performance, the maximum plastic hinge rotation shall be lesser than the limits introduced in FEMA356 acceptance criteria for shear walls. In this article, a more accurate method is used to evaluate shear wall performance. In this method, the maximum tensile and compressive strain in steel and concrete, in shear wall plastic hinge zone are controlled. In accordance to FEMA356 provisions, without confining transverse reinforcement, the maximum usable strain at the extreme concrete compression fiber shall not exceed 0.002. Maximum compressive strains in longitudinal reinforcement shall not exceed 0.02 and maximum tensile strains in longitudinal reinforcement shall not exceed 0.05 (FEMA356 2000).

Evaluation of Performance and Acceptance Criteria of the Structures

In order to control structures performance in near-field earthquakes to compare with those in far-field earthquakes and to control design provisions of Iranian Code (Standard No.2800), acceptance criteria of structures are examined in BSE-1 for Life Safety (LS) performance level.

10-Story Structure Performance

Elements Performance

Beams performance is acceptable in LS level (Fig. 5). If higher performance level (like immediate occupancy (IO)) is considered, structure shall be retrofitted (especially in near-fault regions). The whole PZs performance is acceptable in Life Safety level (Fig. 5). Due to large stiffness of the structure (because of shear walls existence), Shear strain (plastic rotation) in PZs is low, consequently, the entire PZs performance is acceptable.

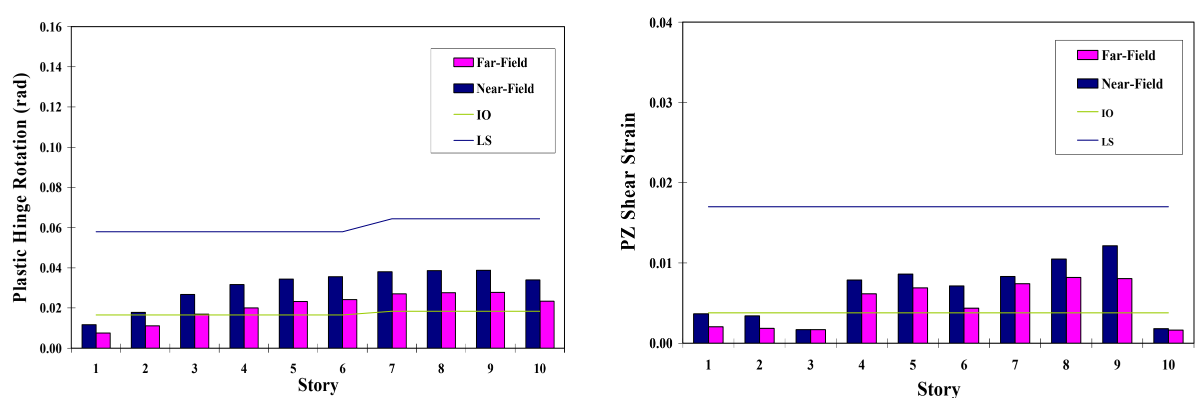


Figure 5. Maximum plastic rotation of beams and maximum shear strain of PZs

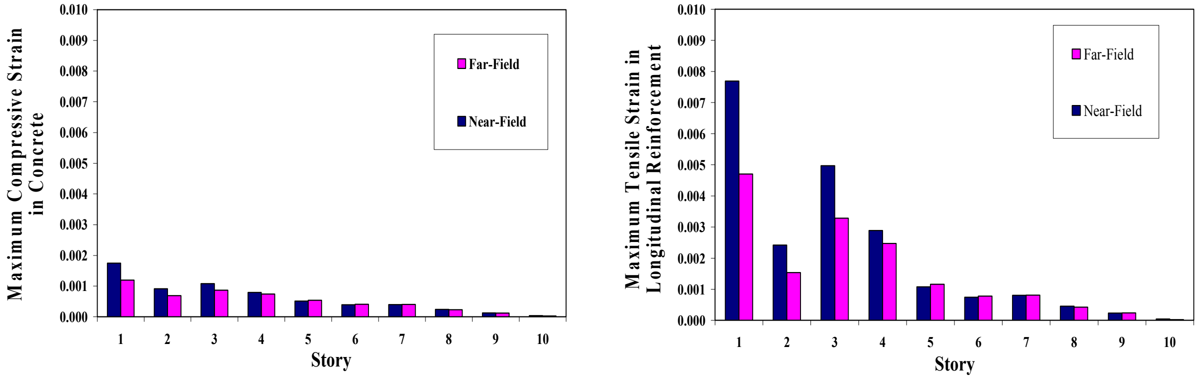


Figure 6. Maximum tensile strain in steel bars and maximum compressive strain in concrete

In accordance with former recommendations the maximum compressive strain in concrete fibers is limited to 0.002 and the maximum tensile strain in steel bars (armatures) is limited to 0.05. Figures 9 and 10 show that shear walls performance is acceptable.
For steel columns under combined axial compression and bending stress, where the axial column load is less than 50% of the lower-bound axial column strength, PCL, the column shall be considered deformation-controlled and the maximum permissible plastic rotation demands on them, in radians, shall be as indicated in table 5-6 and 5-7 of FEMA356, dependent on the axial load present and the compactness of the section. Analyses results show that deformation-controlled columns performance is acceptable in Life Safety level. Flexural loading of columns, with axial compressive forces exceeding 50% of the P_{CL}, shall be considered force-controlled and shall conform to Eqs. (5) ~ (8):

$$\frac{P}{P_{CL}} + \frac{C_{mx} M_x}{\left[1 - \frac{P}{P_{ex}}\right] M_{CLx}} + \frac{C_{my} M_y}{\left[1 - \frac{P}{P_{ey}}\right] M_{CLy}} \leq 1 \tag{5}$$

$$\frac{P}{AF_{ye}} + 0.85 \left[\frac{M_x}{M_{PCLx}} + \frac{M_y}{M_{PCLy}} \right] \leq 1 \tag{6}$$

$$M_x \leq M_{PCLx}, M_y \leq M_{PCLy} \tag{7}$$

Where; M = bending moment, M_{CL} = lower-bound flexural strength of column, P_e = Euler critical load (without safety factor), C_m = a coefficient assuming no lateral translation of the frame that conservatively is taken 1 for all cases, M_{PCL} = lower-bound plastic strength of cross section, F_{ye}= expected yield stress and P = axial load of column. Also

$$Q_{CL} = P_{CL} = 1.7 F_a A \tag{8}$$

Where; F_a = permissible axial stress of column considering lower-bound strength of the element and , A = cross section of column.
Results show that peripheral columns performance in 4th and 5th stories, under Time-History Analyses in far-field is not acceptable. Therefore Iranian Code provisions has not satisfied

design requirements. In near-field regions, in addition to 4th and 5th stories, column performance in 1th and 2th stories is not acceptable.

Inter-story Drifts

In accordance with FEMA356 recommendations, drift values indicated in this code are not intended to be used as acceptance criteria for evaluating the acceptability of a rehabilitation design; rather, they are indicative of the range of drift that typical structures containing the indicated structural elements may undergo when responding within the various structural performance levels. In spite of this, in many research articles, the maximum permissible inter-story drift of structures is limited to 1.5%. Thus the inter story drifts of 10-story structure are not acceptable under near-field excitations (Fig. 7).

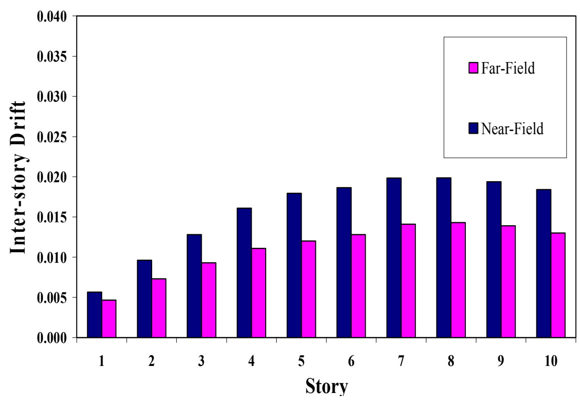


Figure 7. Maximum inter-story drifts in 10-story structure

20-Story Structure Performance

Elements Performance

Beams performance is acceptable in Life Safety level (Fig. 8). The whole PZs performance is acceptable in Life Safety level (Fig. 8). Shear walls performance is acceptable too (Fig. 9).

Force demand is to some extent more than capacity of peripheral columns in 20-story structure. This phenomenon is extended to include more columns under excitations of near-field ground motions.

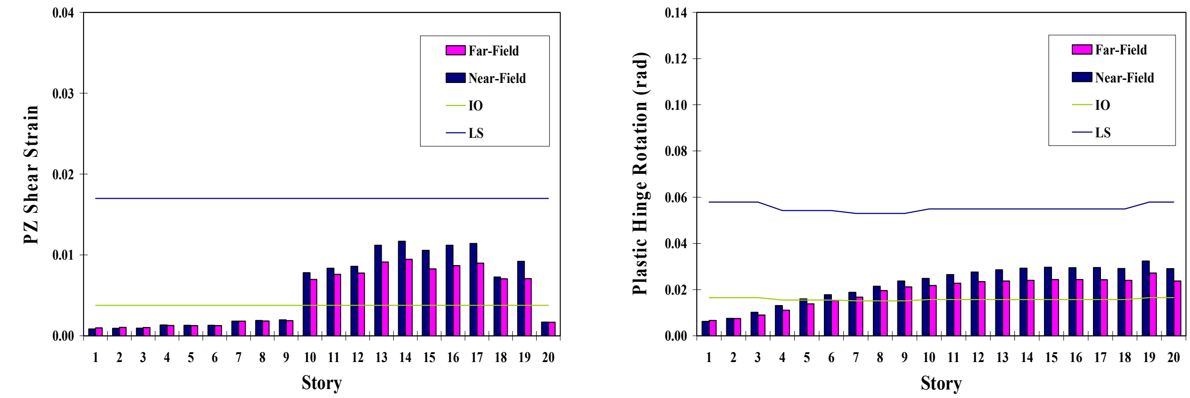


Figure 8. Maximum plastic rotation of beams and maximum shear strain of PZs

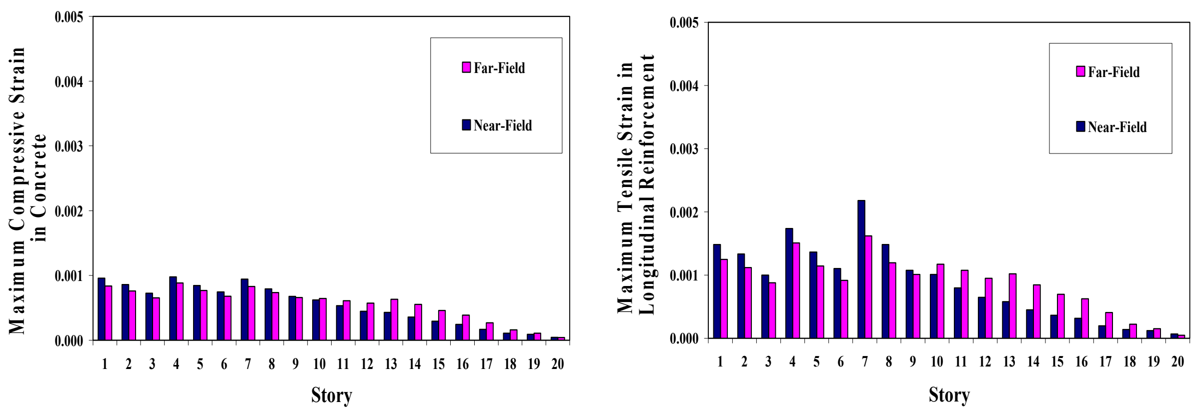


Figure 9. Maximum tensile strain in steel bars and maximum compressive strain in concrete

Inter-story Drifts

Inter-story drifts of 20-story structure are more than allowable inter-story drifts specially in upper stories under near-field excitation (Fig. 10).

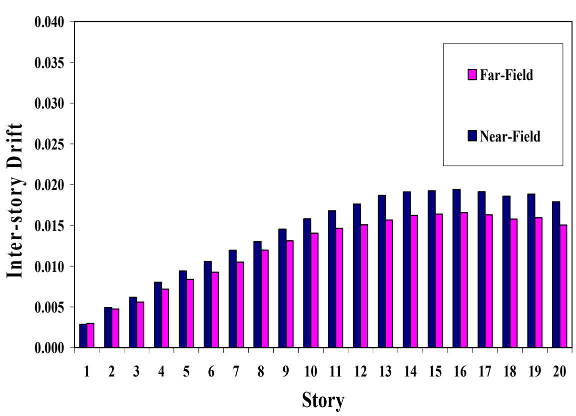


Figure 10. Maximum inter-story drifts in 20-story structure

Conclusions

In accordance to results extracted from analyzing these structural models, deficiency of Iranian Code (Standard No.2800) in exhibition of critical load combination for column's seismic design seems to be considerable. This is concluded from the fact that a lot of Forced-controlled columns capacity didn't satisfy force demand requirements in any of analyses. Even so, performing analyses on more and various structures can confirm this assertion.

The stiffening of the structure by addition of shear wall will reduce the inter-story drift, however, it will also shorten the period of the structure and consequently move it towards a region of higher spectral acceleration response. It is more critical in near-field excitation where spectral acceleration zone is more extensive. Hence, it results in higher lateral forces which make it necessary to increase the strength to such high values that makes the design uneconomical.

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