ASSESSMENT OF SEISMIC RESPONSEPARAMETERS OF TALL BUILDINGS WITH TUBE IN TUBE STRUCTURAL SKELETON

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ABSTRACT

In this research, the basic response parameters of the nonlinear dynamic behavior of steel tall buildings with tube in tube structural skeleton were studied. The studied model consists of an exterior braced-tube system and an interior framed-tube.A number of nonlinear dynamic time history analyses were conducted for the studied model under influencing of an ensemble of free field pulse type ground motions. The studied model is a 30-story steel tube in tube structure which contains several beam, bracing and column elements. The designation process has been completed for all of the elements, members sections and the connection zones based on the Iranian national building code (steel structures - part 10). The confirmations of the principle of strong column and weak beam in all connections and the assessment of strength of panel zones have been considered in the designation process. A number of diagonal girders are used in the skeleton of the studied structure to connect efficiently the exterior braced tube to the interior rigid framed tube systems. This improvement would cause an efficient and better distribution of lateral loads between the two resistant bents of the entire structure. The other helpful advantage is to attain to the minor stress ratios in member sections to achieve the more economical structure.

INTRODUCTION

This paper investigates the consequences of well-known characteristics of far-fault and near-fault ground motions on the seismic response of tall buildings with tube in tube structural skeleton. Additionally, pulse-like ground motions are utilized in a separate study to gain further insight into the effects of high-amplitude pulses on structural demands. The studied structural model with a new configuration of resistant bents has been designed based on the Iranian seismic code 2800. This new configuration of flexural and shear bents can be considered as an efficient engineering design criterion in the designation process of flexible tall structures, especially those ones which are constructed in near fault zones (Movahed et al 2014).

The buildings were designed for equivalent static loads but it seems that their overall performances under dynamic loading caused by strong ground shaking still to be unknown (Yousuf and Bagchi 2010). On the other hand, the recorded strong motions in near fault areas contain large amplitude and long period pulses in their acceleration and velocity time histories (Shung and Lili 2007). The mentioned wave-like features can be generally viewed in the first part of velocity time history of various strong earthquake records which are influenced by forward directivity effects (Lee et al 2000). Near-field ground motions with directivity effects usually tend to have high PGV/PGA ratio, may contain distinct pulses in acceleration, velocity, and displacement time histories. These powerful ground motions can generate much higher base shears, inter-storey drifts and roof displacements in high-rise buildings as compared to quake tremors which do not contain any pulses (Malhotra 1999).

Based on the research results, it was notified that the near-field ground motions with pulses can induce dramatically high response configurations in fixed-base buildings (Bertero et al 1978). Anderson and Bertero pointed out that the wide acceleration pulses are especially damaging if the width of the pulse is large compared with the natural period of the structure (Anderson and Bertero 1987). Furthermore, Hall et al in their study related to the buildings subjected to artificially generated pulse-like ground motions, indicated that the demands imposed by the displacement pulses in the near-field ground motions can far exceed the capacity of flexible high-rise buildings designed based on current standards (Hall et al 1995). Iwan stated that the pulses in the near-field ground motions travel through the height of the buildings as waves, and that the conventional techniques using the modal superposition method and the response spectrum analysis may not capture the real effects of these pulses (Iwan 1997).

The studied model of this paper has been designed using the lateral load distribution which specified in the Iranian seismic code 2800 3th edition. The ensemble of the chosen records contains a number of strong ground motions which have been recorded in far and near fault areas.

STRUCTURAL MODELS AND DESIGN CONSIDRATION

The structural model in this research consists of one steel tall building with tube in tube structural skeleton in 30-story form. A typical floor plan and resistant bents of the model and the section properties of members give in Figures 1, 2 and Table 2, respectively. The applied dead load was considered 0.5 $\mathrm{ton/m^2}$ for all floors. Yet the live load was also determined 0.2 $\mathrm{ton/m^2}$ for the floors and 0.15 $\mathrm{ton/m^2}$ on the roof. The weight of columns and beams of all structural skeletons was defined about 0.05 $\mathrm{ton/m^2}$ which was added to the dead load. The floors consist of reinforced concrete slab supported by steel beams and girders which are located in the both of gravity and moment-frames. A36 steel with nominal yield strength of 248.29 MPa is used for all beams, braces and columns.

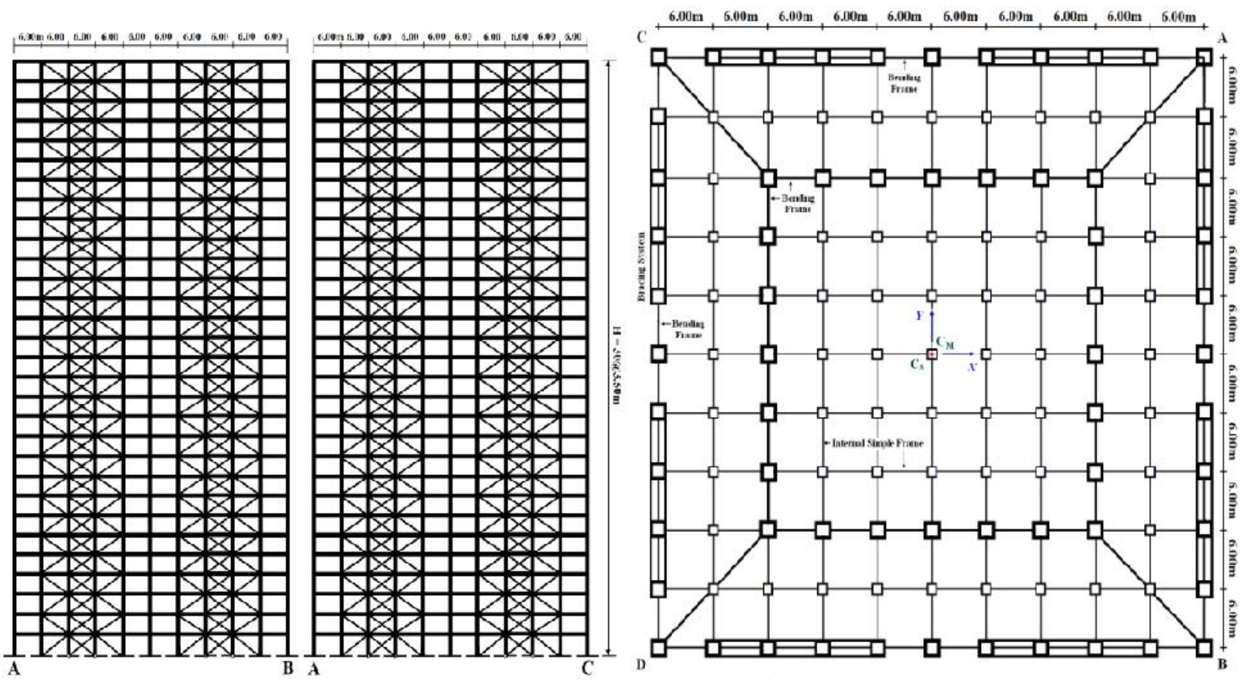


Figure 1. The plan and elevation of the studied structure, C_M: mass center, C_s: shear center

The design lateral load coefficient related to the seismic lateral load has been calculated according to the Iranian seismic code 2800 3th edition and the structure modal vibration periods related to the three Cartesian directions are shown in Table 1. The basic notifications which contain the evaluation of the seismic drift limits of stories, the confirmation of the principle of strong column and weak beam in all connections and the assessment of strength of panel zone, have been considered in the designation process. The diagonal girder elements are used in plan of structure to connect exterior frame panels to interior ones for better distribution of lateral load and attain to the minor stress ratios in the member sections of the structure.

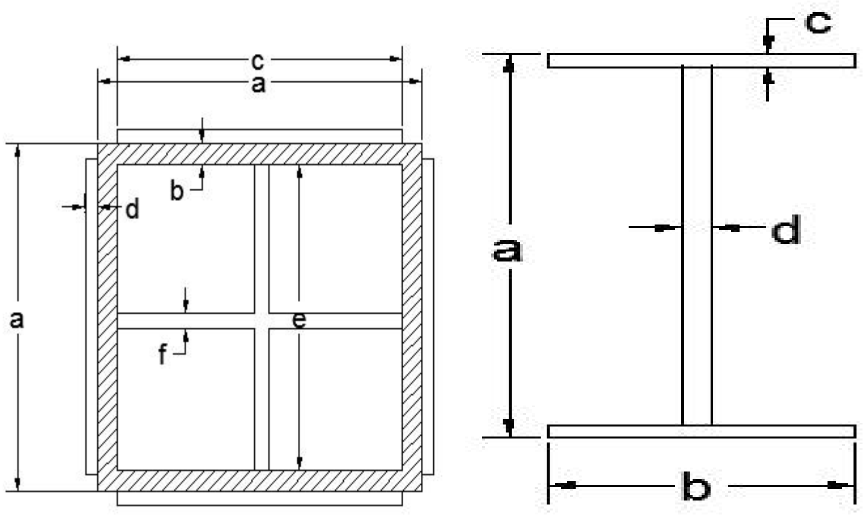
However, the seismic demands in the fundamental and higher modes must be evaluated by taking into consideration the fact that the system moves from the elastic state to inelastic state (Kalkan 2003). Additionally, the seismic P-Δ effects can be a great threat for overall stability of buildings during the powerful ground motions as well as the strong aftershocks that usually follow the ground main shock (Krishnan et al 1998). It is noticeable that the buildings which are designed according to Iranian seismic code 2800, have the assumed performance level of "Life Safety". The section properties of members of the studied structure are presented in both Figure 2 and Table 2.

Table 1: The seismic lateral load coefficient (C), the static base shear (V) and the modal vibration periods (T)

Model	Height	C	V(ton)	T_1(sec) First Lateral Mode	T_2(sec) Second Lateral Mode	T_3(sec) Initial Torsional Mode
Tube in Tube System	30-story	0.068	5226	2.48	1.85	1.49

Table 2: Structural member properties of the studied tube in tube model

Model	Stories Group	Exterior Columns	Interior Columns	Beams
Tube in Tube System	1-5	C100x3.0x95x1.5x95x2.0	C60x3.0x55x2.0	B65x55x2.5x2.0
	6-10	C100x3.0x95x1.5x95x2.0	C55x2.5x50x2.0	B65x55x2.5x2.0
	11-15	C85x3.0x80x2.0	C50x2.5x45x2.0	B65x55x.2.5x2.0
	16-20	C80x3.0x75x2.0	C45x2.5x40x2.0	B65x55x2.5x2.0
	21-25	C75x3.0x70x2.0	C40x2.5x35x2.0	B65x55x2.0x1.5
	26-30	C70x3.0x65x2.0	C35x2.5x30x2.0	B65x55x2.0x1.5



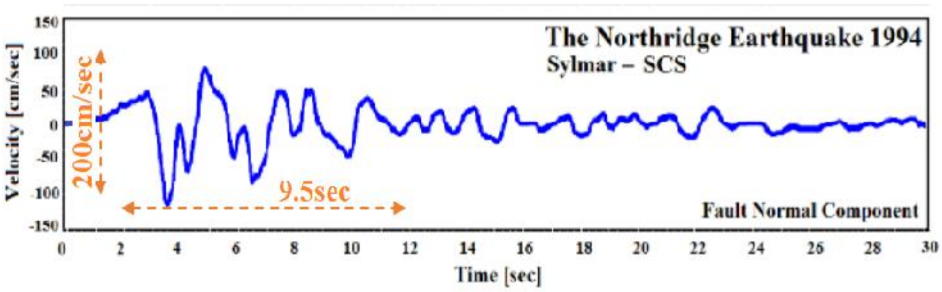
C a x b x c x d x e
Column Section

B a x b x c x d
Beam Section

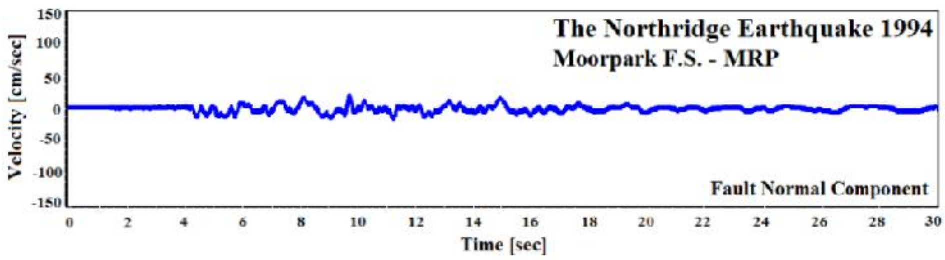
Figure 2. The Structural Model: (a) Typical beam section, (b) Typical column section

Table 3: The selected earthquake records

Ground Motion	Component	Duration (sec)	PGA (g)	PGV (cm/s)	PGD (cm)	Magnitude M_W	PGV/PGA (sec)	PGD/PGV (sec)
Tabas 1978 Tabas City - 3.0km	LN	30.00	0.836	97.7	39.9	7.4	0.12	0.40
	TR		0.851	121.3	94.5		0.14	0.78
	UP		0.688	45.5	17.0		0.06	0.37
Bam 2003 Bam City - 1.0km	LN	30.00	0.635	59.6	20.7	6.6	0.09	0.34
	TR		0.793	123.7	37.4		0.16	0.30
	UP		0.999	37.66	10.11		0.03	0.26
Manjil 1990 Abbar – 12.25km	LN	30.00	0.51	21.42	4.40	7.3	0.04	0.20
	TR		0.49	26.26	6.34		0.05	0.24
	UP		0.54	22.89	13.12		0.04	0.57
Northridge 1994 Sylmar (SCS) - 6.40km	LN	30.00	0.897	102.23	45.28	6.7	0.11	0.44
	TR		0.612	117.47	54.16		0.19	0.46
	UP		0.586	34.59	25.63		0.06	0.74
Northridge 1994 Rinaldi (RRS) - 7.10km	LN	30.00	0.472	72.72	19.82	6.7	0.15	0.27
	TR		0.838	166.87	29.79		0.19	0.17
	UP		0.852	51.01	11.71		0.06	0.22
Northridge 1994 Tarzana (TAR) – 15.00km	LN	30.00	0.990	77.60	30.45	6.7	0.07	0.38
	TR		1.779	113.6	33.22		0.06	0.29
	UP		1.048	73.69	20.52		0.07	0.27
Northridge 1994 Arleta (ARL) – 9.20km	LN	30.00	0.308	23.2	10.57	6.7	0.08	0.45
	TR		0.344	40.6	15.04		0.12	0.37
	UP		0.552	18.4	8.83		0.03	0.47
Northridge 1994 Moorpark (MRP) – 28km	LN	30.00	0.19	20.20	4.79	6.7	0.11	0.24
	TR		0.29	20.70	4.24		0.07	0.20
	UP		0.16	7.90	0.9		0.05	0.11



(a)



(b)

Figure 3: The velocity time history of the fault normal component: (a) The Sylmar record-SCS; (b) The Moorpark record-MRP

THE ENSEMBLE OF CHOSEN EARTHQUAKE RECORDS

The main criterion which was considered to select of strong ground motions for performing non-linear time history analyses is the existence of high amplitude and long period coherent pulse or a multiple pulse system in the velocity time history of each earthquake record. Based on Figure 3, the coherent velocity pulses are quite distinctive for the Rinaldi and Sylmar records 1994, such pulses do not exist in a typical far-fault ground motion like the Moorpark record 1994. Researches results show that energized high-amplitude velocity pulses are able to place severe inelastic demands on middle to high rise structures (Hall et al 1995).

The selected earthquake records are classified in two groups. The group1 consists of the near-field records with long period coherent velocity pulses. This group contains the Bam, Tabas and Manjil Iranian records as well as the three extremely powerful ground motions entitled SCS, RRS and TAR records due to the 1994 Northridge earthquake. Meanwhile the second group is collected by recorded components of the near-field earthquake motions which contain non-coherent velocity spikes. These mentioned records are both of the Arleta (ARL) and Moorpark (MRP) ground motion events due to the 1994 Northridge earthquake. In this research the characteristics of the three translational components of each earthquake record were applied in X & Y directions of the plan and Z axis of the structural model. The nonlinear time history analyses were carried out based on these components in three directions of the studied structure.

Since in general the seismic response variations of a structure depends on the entire frequency band of the input ground motions (Krishnan et al 1998), the database of Table 3 which compiled for nonlinear time history analyses, constitutes a representative number of ground motions from a variety of tectonic events. A total of 8 three-component corrected records were selected with the notification points respect to frequency content, strong motion duration and spikes amplitudes. Based on the result of this research, the existence of high values for PGV parameter and long period pulses in the velocity time history are generally considered as the potentiality of the ground motion which to cause huge structural damage. Yet, for the same peak ground acceleration (PGA) and duration of strong shakings, the ground motions with directivity pulses can generate much higher demands in high-rise buildings (Malhotra 1999).

NON-LINEAR DYNAMIC ANALYSIS AND RESPONSE PARAMETERS

In this research the dynamic behavior factors of the designed structure under influencing of the selected strong ground motions were evaluated. The response parameters have been obtained based on the conducting a number of non-linear dynamic time history analyses. The illustrated outputs of the analyzed model contain the maximum absolute acceleration, the maximum relative velocity, the both of maximum displacement and drift of all stories, respectively. In modeling process of the studied structure, the ability of performing non-linear behavior for all beams, columns and braces were assigned based on introducing the moment M3, the interacting P-M2-M3 and the single P hinges based on FEMA 356 (Figure 4).

For the ductile model, the beams and columns ends, the braces medians and ends are modeled as plastic hinges with strain hardening zone relative to the elastic stiffness of the corresponding element. In the analytical model of the studied structure, the P–Δ effects are included too. The beam-to-column joints are modeled in three dimensions using rigid panel zone elements while the gravity columns are modeled using plastic hinge elements. These analytical abilities have been assigned to the studied structural model (Figure 1) to simulate the damage mode, accurately and efficiently. To denote the aforementioned analytical meanings, the total collapse mechanisms related to the nonlinear skeletal mode of the studied structure subjected to the TAB and SCS records are illustrated in the Figure 5.

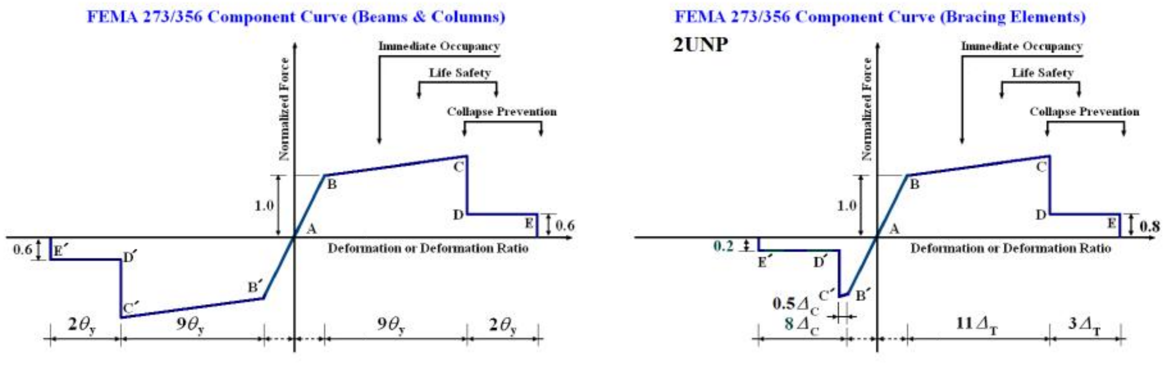
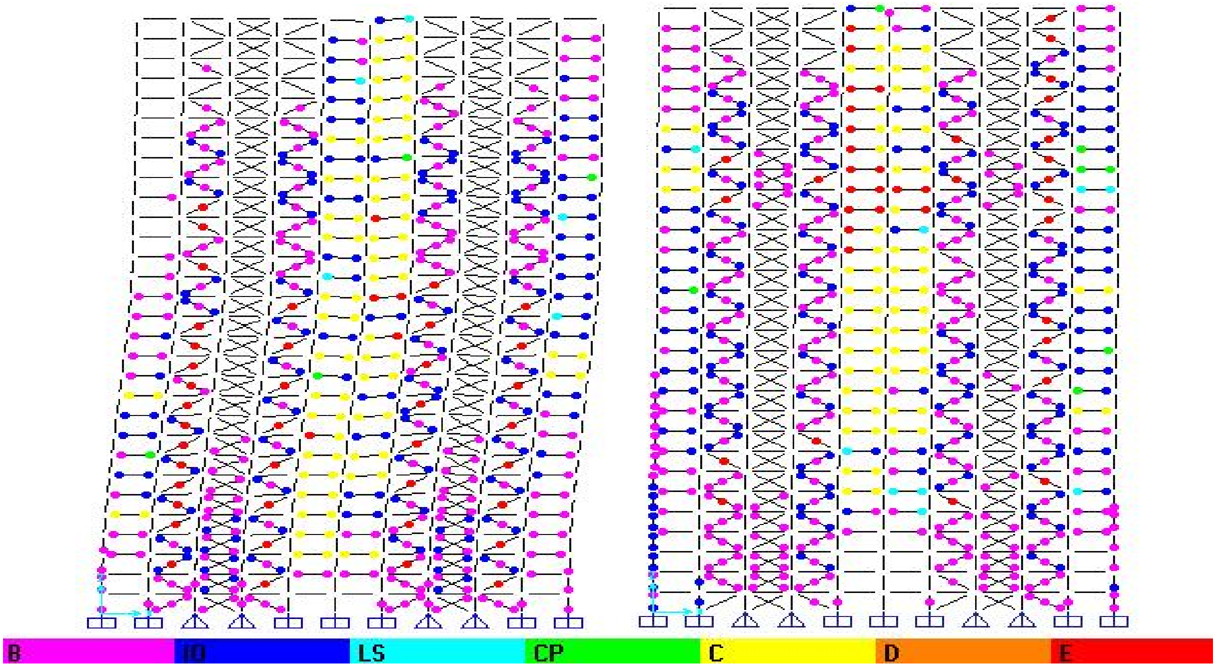


Figure 4: Force-deformation relationship of a plastic hinge (Fema 356)

Figure 4 shows a typical force-deformation relationship to define the behaviour of plastic hinges by FEMA-356 and also the required acceptance criteria of immediate occupancy (IO), life safety (LS), collapse prevention (CP) and fully collapsed (C) performance levels. In Figure 4, the point A corresponds to unloaded condition of hinge deformation. The point B represents the initial yielding of structural elements. The general hinge deformation in Figure 4 shows a probable strength degradation at the point D where the element might show sudden failure after this point. The symbolic progressive failure of the element can be defined by reaching the point D to E.

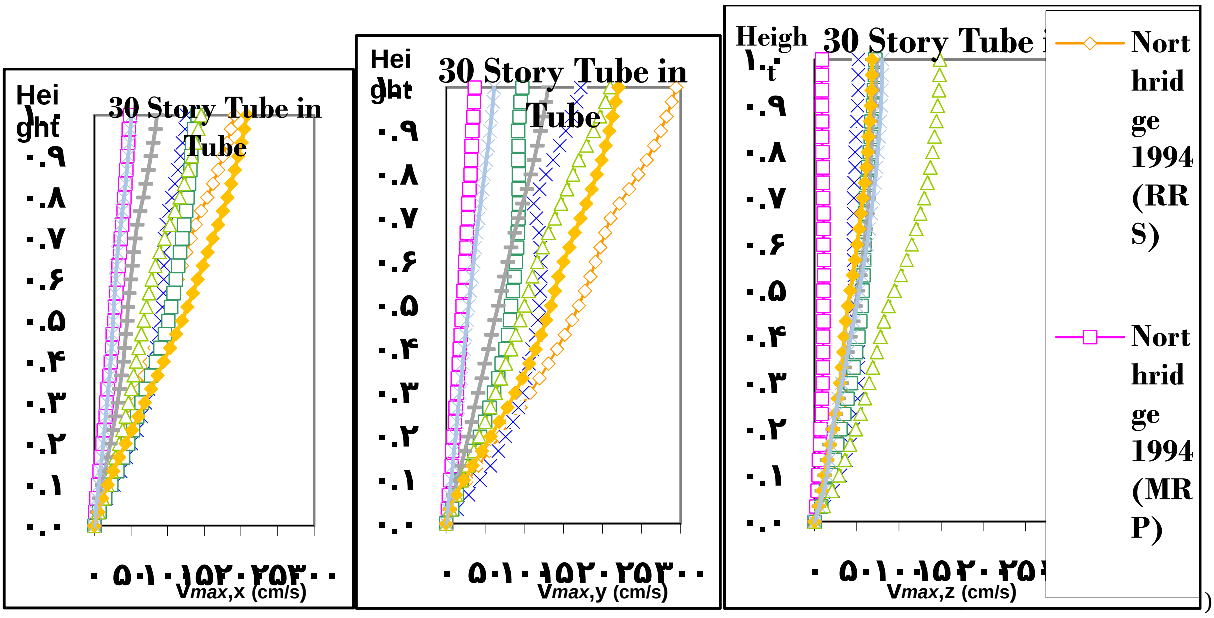


(a) (b)

Figure 5: The formed FEMA plastic hinges mechnisem: (a) The Tabas earthquake (TAB) (b) The Northridge earthquake (SCS)

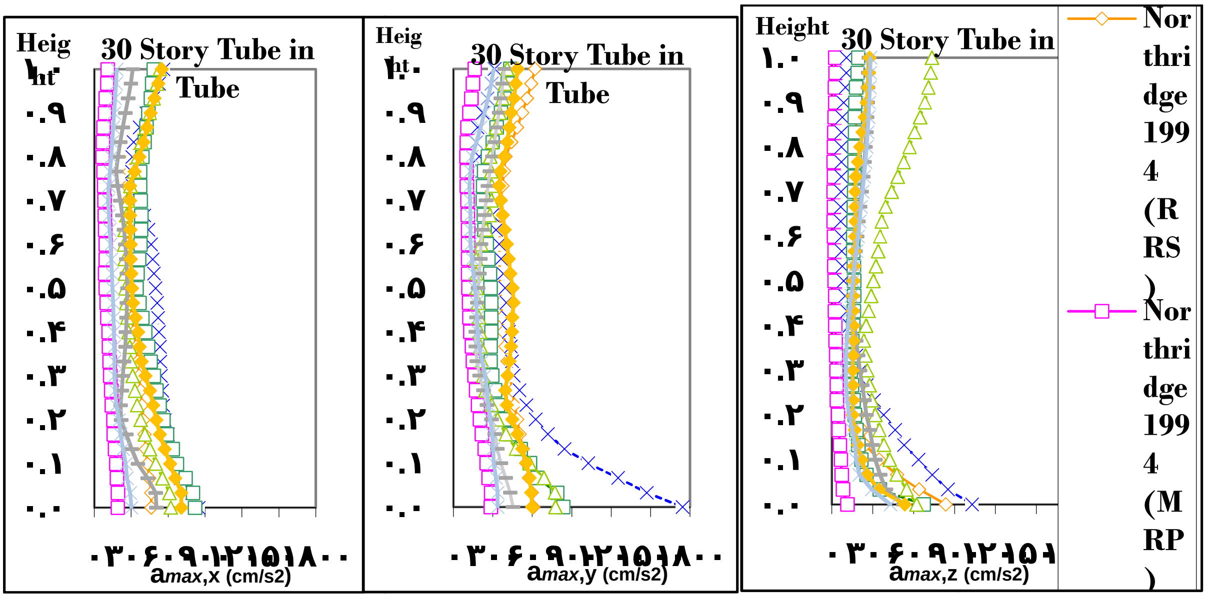
The analytical simulations carried out in this study show that tall buildings with tube in tube structural skeleton can be subjected to large displacement demands at the arrival of the velocity pulse, in which the earthquake resistant system needs to dissipate considerable input energy in a single or relatively few plastic cycles. This large seismic demand would impact the resistant structures with limited ductility capacity. In contrast, far-fault motions build input energy more gradually and though the displacement demands are on average lower than the demands in near-fault records. Hence, the structural system is subjected to significantly more plastic cycles. This finding is significant in the development of testing protocols and damage models incorporating low-cycle fatigue. Yet, it is founded that the elastic part of the rotation is almost negligible which suggests that for ductile elements with significant inelastic behavior, the peak component deformation is generally equivalent to the plastic deformation (El-Bahy et al 1999). Non-linear analyses of the structure were performed using the program SAP2000 version 14.2.2. It is noticeable that the Newmark and Hilber time integration schemes were utilized in all of the non-linear dynamic analyses (Bathe 1996). The seismic response parameters for the 30-story studied model are illustrated in Figure 6 to Figure 8.

The maximum relative velocity and the maximum absolute acceleration of all stories are illustrated in Figures 6 and 7, respectively. From the preliminary information generated through the evaluation of the studied model discussed above, it is clear that the building respond differently to far-fault and near-fault ground motions. Based on the result of this research, the both aforementioned response parameters influenced intensively by the near-field earthquake records and also are appropriately higher than those values related to the far-field recorded motions. These results can be referred to the nature of strong ground motions which contains forward directivity effects. These types of earthquake records enable to display wave like features in their time histories, especially in the form of high amplitude coherent velocity pulses. Additionally, the distribution of the floors acceleration in the height of the structure contain larger values of this parameter in comparison with the results due to the far-field and those near-field earthquake records which would not display velocity pulses or even velocity spikes (Movahed et al 2014).



(a) (b) (c)

Figure 6. The maximum seismic relative velocity of stories; (a) X direction of plan; (b) Y direction of plan; (c) Z direction of the studied structure



(a) (b) (c)

Figure 7. The maximum seismic absolute acceleration of stories; (a) X direction of plan; (b) Y direction of plan; (c) Z direction of the studied structure

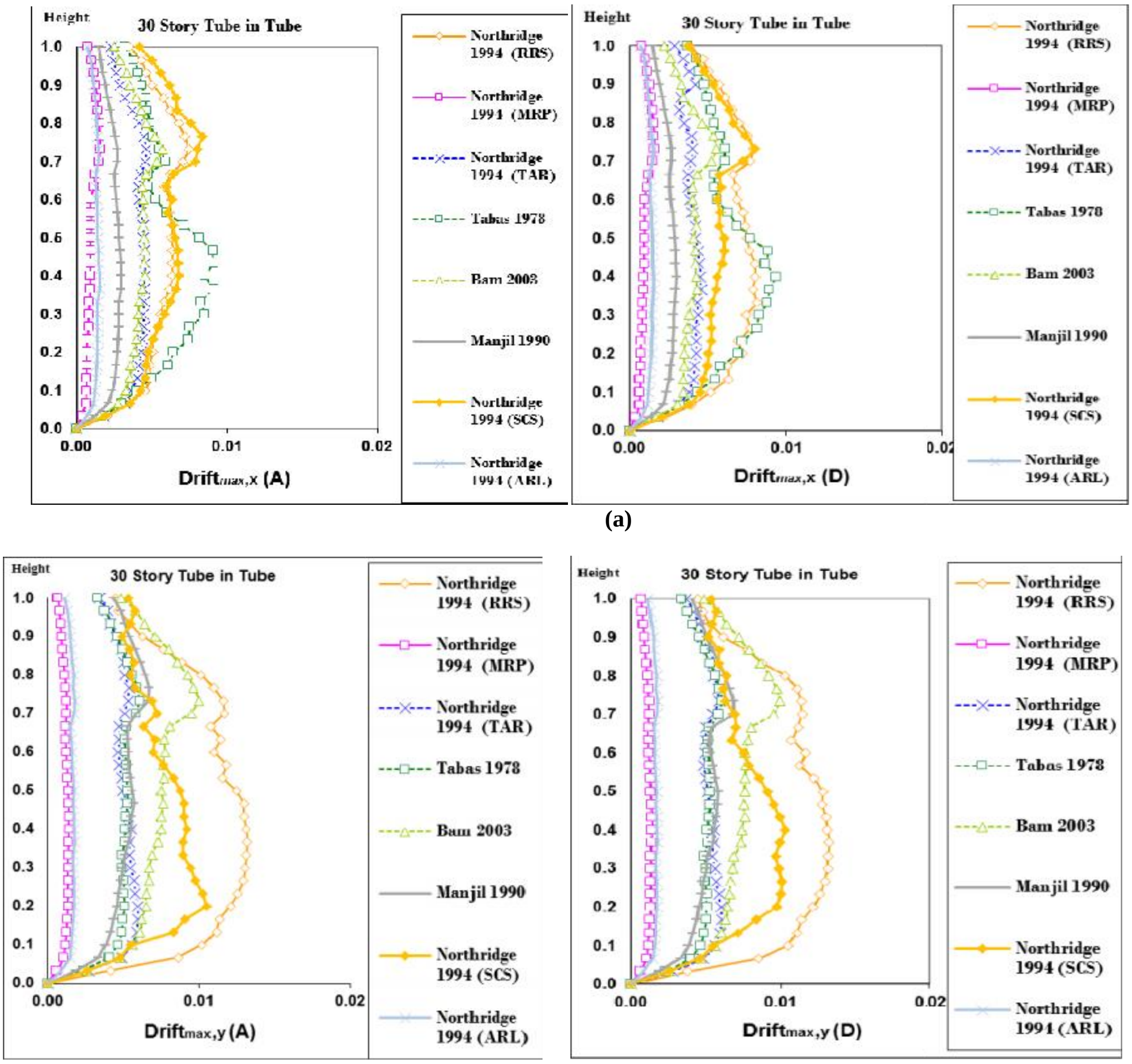


Figure 8.The maximum seismic drift of stories; (a) X direction of structure; (b) Y direction of structure;

The maximum drifts for each story of the analyzed model are shown in 8 which are related to the X and Y directions, respectively. The analytical assessments carried out in this study show that tall buildings can be subjected to large displacement demands at the arrival of the velocity pulse that require the structure to dissipate large amounts of kinetic energy in relatively few plastic cycles. This demand would impact tall structures with low ability of ductile behavior. Furthermore, even strong far-field records as well as the near-filed ground motions which contain neutral directivity effects, may release the kinetic energy of the ground quake tremors more gradually. Therefore, the seismic displacement demands would tend to an average limit which is distinctly lower than those of the forward directivity influenced records. Hence cumulative effects are not usually pronounced in far-fault ground motions.

The peak inter-story drift is usually the most reliable measure to evaluate the structural performance. The report FEMA 356 proposes peak inter-story drift limits of 0.007, 0.025, and 0.05 for the immediate occupancy (IO), life safety (LS) and collapse prevention (CP) performance levels, respectively. In case of near-fault records, they impose higher demands than far-fault records though the maximum drift which is generally concentrated at the lower to middle story levels. The largest demand is caused by the Rinaldi (RRS) record which produced about 1.3 percent inter-story drift at the seven to fourteen stories.

CONCLUSIONS

The strategy was to place steel tall building with tube in tube structural skeleton under influencing of strong earthquakes and compute its seismic response parameters. The chosen earthquake records consist of some strong ground motions which have been recorded in both near and far fault areas respect to a causative fault. The results are presented in outputs of the analyzed model, contain the maximum absolute acceleration and the relative velocity as well as the maximum drift of each story. According to observations hold for near-field records, the demands at the intermediate levels are considerably higher of the entire data set and yet, the Rinaldi (RRS) record generated the highest demand i.e. 1.3 percent inter-story drift at the 7 to 14 stories. Three intensively powerful near-field ground motions i.e. the RRS, SCS and Bam records created significant demands at lower to middle story levels, particularly. The Rinaldi (RRS) record causes a general shift in demands from the upper to lower stories. However, the variation in story demand for the far-fault records is less significant.

The analytical results were illustrated and studied in accordance with the special characteristics of those selected earthquake records which contain coherent velocity pulses. As a general result, the presented response parameters indicate that because of special structural configuration that is used in skeleton of the studied tube in tube model, two main response parameters i.e. the seismic base shear and stress ratios of sections would considerably be reduced. On the other hand, the total seismic behavior of the studied model should extremely be influenced by strong earthquake records, specifically those ones that enable to display long period wave like pulses in their velocity time histories. Consequently, it is important to control of the lateral displacement and drift of stories in the design process of steel tall buildings with tube in tube structural skeleton in near zones of active fault.

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