A Study on the Effect of Bracing Arrangement in the Seismic Behavior Buildings with Various Concentric Bracings by Nonlinear Static and Dynamic Analyses

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SUMMARY:
The arrangement of bracings in buildings structures affects their seismic behavior, as previous studies show, while this fact is not considered in seismic design codes. In this study a set of regular multi-story steel buildings were considered with three kinds of X, V and chevron bracing, in two placements of 'two adjacent bays' and 'two non-adjacent bays' along the building height, and their seismic behaviors were investigated. First, the buildings were designed based on the code, and then they were evaluated by both pushover and nonlinear time history analyses, and their performances were compared with the standard performance levels (PLs). Results show that in all cases, bracing arrangement in non-adjacent bays leads to lower stiffness but higher strength than in adjacent bays, and that for Immediate Occupancy PL, plastic zones appear mostly in lower stories, while for Life Safety and Collapse Prevention PLs they appear just in few lower stories.

Keywords: X, V and chevron bracings, Performance Levels, Push-Over analysis, Plastic zones

1. INTRODUCTION

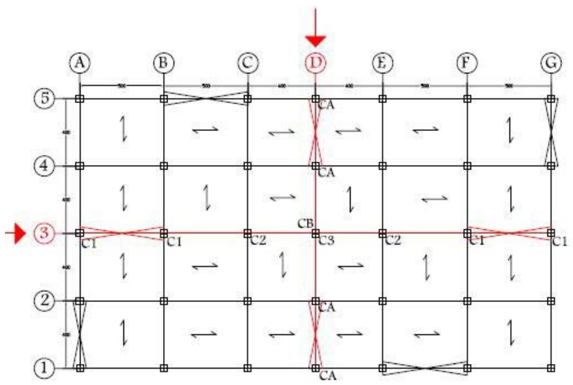
Design of earthquake resistant structures in recent years has undergone several changes and improvements which have focused mainly on altering design criteria from 'design based on resistance' to 'design based on performance'. As it is well-known, most of the current design methods of seismic codes are based on linear analysis of forces in structural elements along with controlling the inter-story drift values, and the codes try to consider the concept of structural plastic behavior by applying the so-called 'Response Modification Factor' (*R*). This is while several studies have shown that a single *R* value does not lead to an optimum design of either reinforced concrete buildings (Hosseini and Motamedi 1999, Motamedi and Hosseini 1999, Hosseini and Khoshamadi 2008) or steel buildings (Nasser Assadi and Hosseini 1999, Hosseini et al. 2004, Hosseini and Esmaeili 2006, Hosseini and Rahimi 2007, Moradi et al. 2010).

One of the most common structural systems for steel buildings is the Concentrically Braced Frame (CBFs) system, in which the most important property is the high stiffness. Therefore, usually by increasing the numbers of floors and building height, CBFs are more reasonable than moment frames, and result in more economical designs. The design codes propose using a single *R* value for design of CBFs regardless of the pattern of bracing arrangement in the building frames, however, previous studies (Hosseini et al. 2008, Hosseini and Majd 2011) have shown that the arrangement or placement of bracings in a frame does affect its seismic behavior.

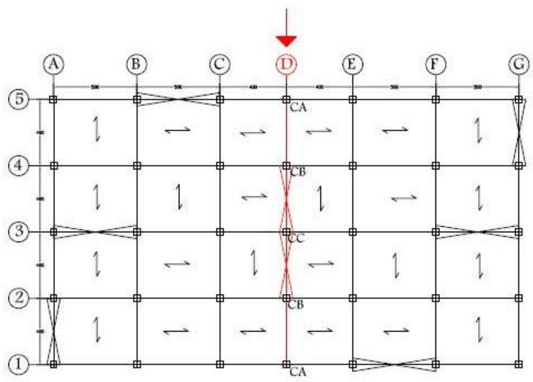
To investigate how the bracing arrangement affect the seismic behavior of steel CBF buildings, in this study a set of steel buildings with 3, 5, and 7 stories, all having 4-bay by 6-bay regular plan were considered with three kinds of concentric bracings, including X, V and inverse-V or chevron, in two placements, 1) in two adjacent bays along the building height, and 2) in two non-adjacent bays, and their seismic behaviors have been investigated. At first, the buildings were designed based on the last version of the seismic design code. Then, the buildings were evaluated by both pushover and nonlinear time history analyses, by using RAM PERFORM-3D software. The buildings' performances were compared with the standard Performance Levels (PLs) of Collapse Prevention (CP), Life Safety (LS), and Immediate Occupancy (IO). Details of these analyses are given in the following sections of the paper.

2. INTRODUCTION THE CONSIDERED BUILDINGS

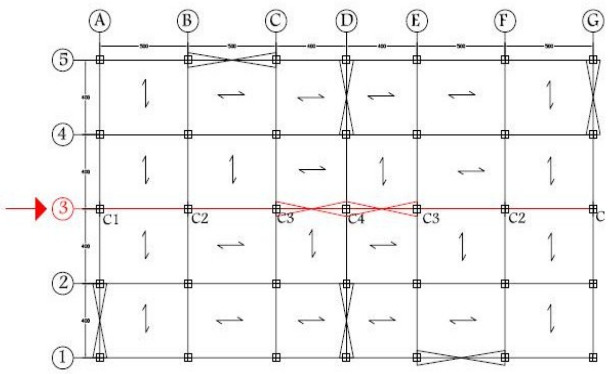
A set of steel buildings with 4- by 6-bay regular plans, having 3, 5, or 7 stories, have been considered with three kinds of concentric bracings, including X, V and invert V or chevron. Two different patterns for location of braced bays have been used, including one with adjacent braced bays and the other with non-adjacent braced bays. Regarding that the majority of existing buildings in Iran have been designed for earthquake loadings by using the Iranian Standard No. 2800 (Iranian Code for Seismic Resistant Design of Buildings), which is very similar to UBC-97, the considered buildings were designed based on that standard. For calculation of the lateral loads factor in all cases the soil type B was used, since most of the existing constructions in Tehran are on this type of soil. Furthermore, the AISC-ASD89 code was used for design of steel sections of the considered buildings, which is in concurrent use with UBC-97. Plans of the considered buildings and the selected frames for analyses are shown in Figure 1.



a) bracings in non-adjacent bays



b) bracings in adjacent bays



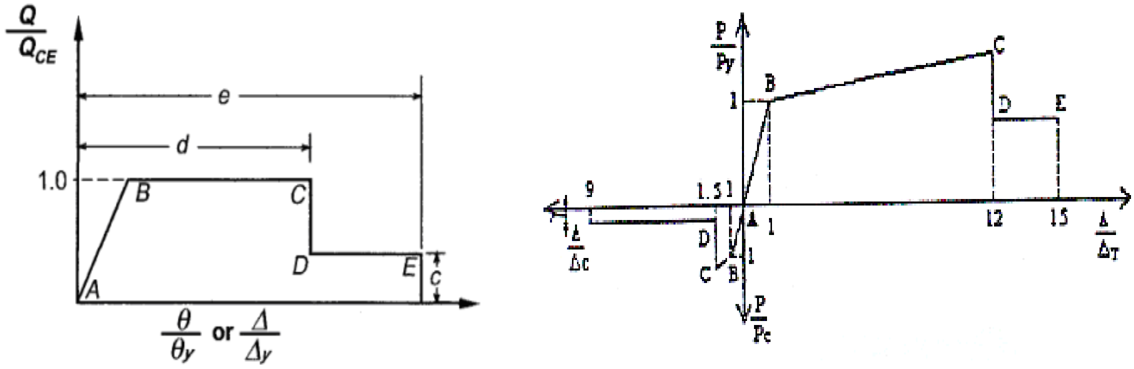
c) bracings in adjacent bays of 4-bay frames of 6-bay frames

Figure 1. Plans of the considered buildings, on which the selected frames for analyses are shown by arrows

A simple rule has been used for naming the analyzed frames based on the number of stories, number of bays and the location of braced bays, as follows. A name consisted of a letter F (Frame) followed by a letter T, for bracings in two adjacent bays (Together), or a letter O for bracings in nonadjacent bays (Open bay in between), followed by the number of stories and finally number of bays completely introduces each frame. For example, FT76 means a 7-story frame with 6 bays having adjacent bracings, and FO54 means a 5-story frame with 4 bays and non-adjacent bracings.

3. MODELING OF BUILDING'S STRUCTURE

To perform the Nonlinear Time History Analyses (NLTHA) and evaluate the vulnerability of considered buildings the building frames have been modeled by Ram-Perform 3D software (Powell 2000). The nonlinear or inelastic behavior of various structural members, including beams, columns and bracing elements were introduced to the software based on the FEMA 356 guidelines (ASCE 2000), as shown in Figure 2.



(a) (b)

Figure 2. Inelastic model used for beams and columns, (a), and for bracing elements, (b)

Parameters used in Figure 2, which define the inelastic behavior of bracing elements, are calculated by following formulas, based on the limit state stress values of steel in compression and tension, F_a and F_y, respectively.

$$P_C = 1.7 F_a A \quad (1)$$

$$\Delta_C = \frac{P_C L}{AE} \quad (2)$$

$$P_y = F_Y A \quad (3)$$

$$\Delta_T = \frac{P_y L}{AE} \quad (4)$$

In these formulas A is the cross-sectional area and L is the length of element. In Figure 3 a sample of inelastic behavior graph related to a bracing element made of two UNP120 is shown.

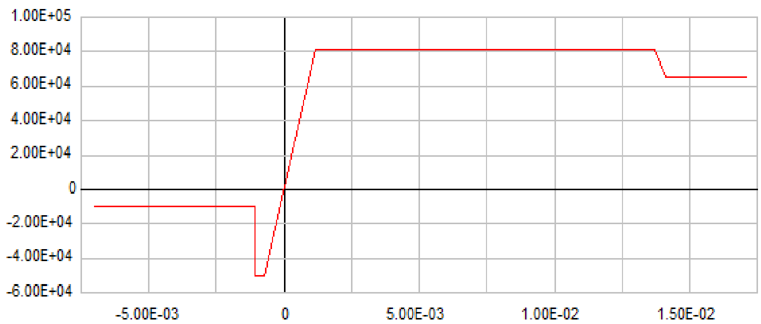


Figure 3. A sample of inelastic behavior graph related to a bracing element made of two UNP120 introduced to Ram-Perform 3D software

4. NONLINEAR STATIC AND NONLINEAR TIME HISTORY ANALYSES

In nonlinear static method, the increasing forces in the two control states by force and drift are applied. In this method, the structure response is controlled only with the main mode and the mode shape remains constant during the analysis. In this study nonlinear static method with drift control is used for function assessment of all structures. For NLTHA of various building models, eight set of accelerograms (one more than the recommended number by most of seismic codes, including Iranian Standard no. 2800), recorded on soil type B, all having the PGA level around 0.35g, which is maximum PGA value in the Iranian code, were used. The specifications of the used earthquakes are given in Table 1.

Table 1. The specifications of applied accelerograms

Event	Scaled PGA (g)	PGD (cm)	PGV (cm/s)	Significant Duration (sec)	R (km)	Soil Type (USGS Geomatrix Soil Class)
Landers 1992.06.28	0.406	5.97	11.0	36.32	25.2	B
Kocaeli, Turkey 1999.08.18	0.523	35.57	39.5	10.27	17.0	B
Imperial Valley 1979.10.15	0.286	7.95	18.6	36.23	26.5	B, A
Chi-Chi 1999.09.20	0.377	10.37	39.3	25.91	15.2	B
Loma Prieta 1989.10.18	0.446	3.79	15.2	12.62	19.9	B, B
El Centro 1950.5.18	0.409	29.0	38.0	25.52	-	B
Tabas, Iran 1978.09.16	0.424	127.0	117.0	15.86	-	B
Duzce, Turkey 1999.11.12	0.490	8.19	13.7	15.10	15.6	B, B

5. STATIC AND DYNAMIC ANALYSIS RESULTS FOR THE MODELS STUDIED

5.1. Nonlinear static analysis results

5.1.1. Base Shear-Roof Displacement Diagrams

In this section the results of non-linear static analysis of all samples 3-, 5- and 7-storey with invert V-(Chevron), V-, and X-shaped bracings are presented in form of the base shear – roof displacement diagrams. For a better comparison the corresponding curves, for the three bracing systems are illustrated in one figure. Figures 4 and 5 show the results related to FT36 and FO36, respectively.

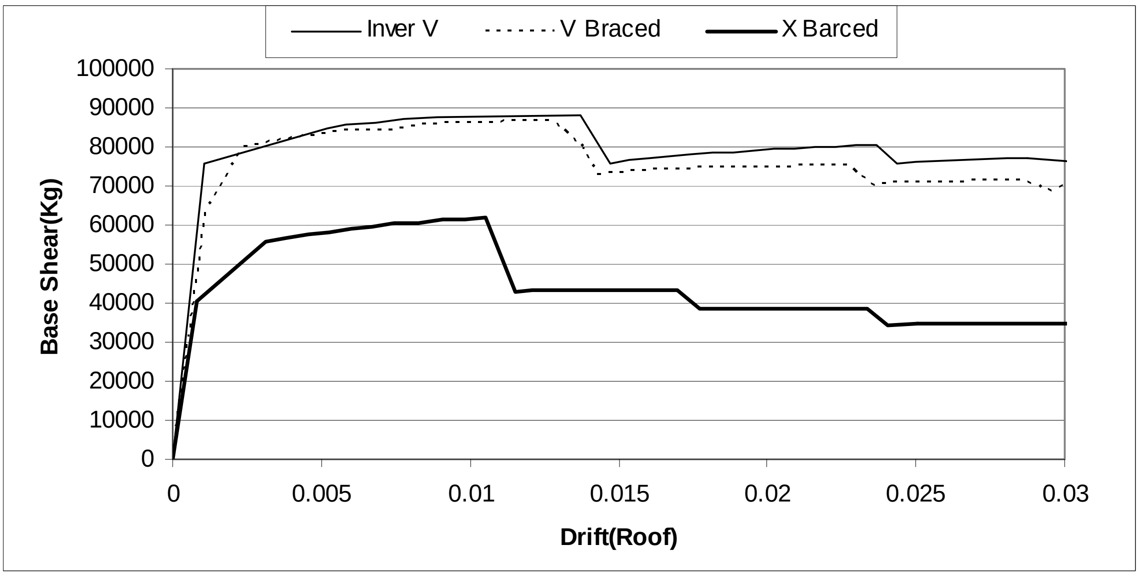


Figure 4. Base shear – roof displacement curves of FT36 for three types of bracings

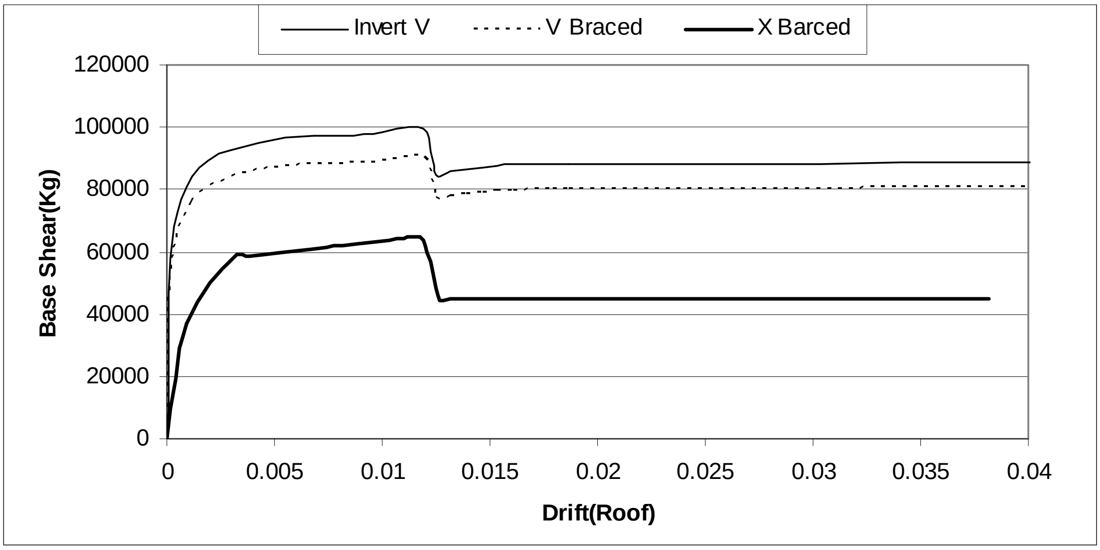


Figure 5. Base shear – roof displacement curves of FO36 for three types of bracings

It can be seen in Figures 4 and 5 that in both cases of FT and FO systems the invert V- and V-shape bracings have higher stiffness and strength than X bracing, and that the invert V is a little stronger than V bracing. However, comparing each curve in Figure 4 with its corresponding curve in Figure 5 shows that in case of FT systems the initial stiffness in higher, but the maximum strength is lower. To better see the effect of bracing arrangement in case of each bracing type separately, Figures 6 to 8 show the curves related to FT and FO systems together, for chevron, V, and X bracing, respectively, in case of 5-story 6-bay buildings.

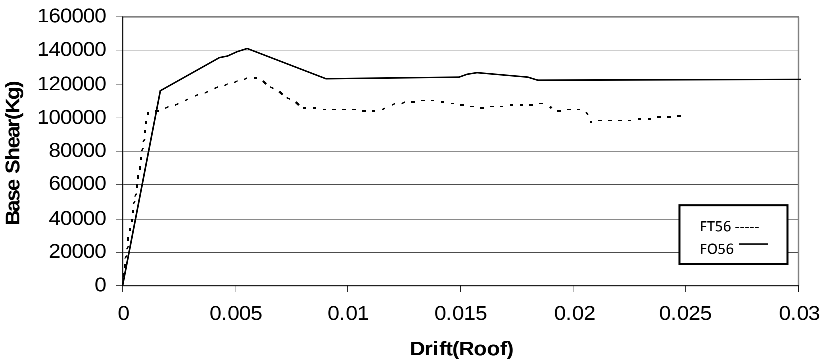


Figure 6. Base shear – roof displacement curves of 5-story 6-bay buildings in two arrangements of chevron bracings in adjacent and distant bays

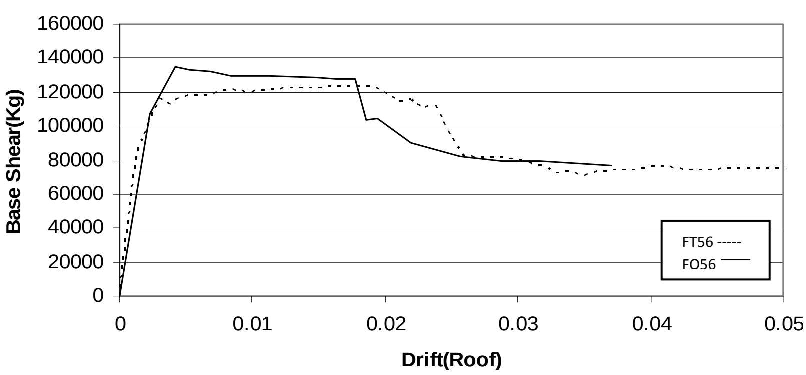


Figure 7. Base shear – roof displacement curves of 5-story 6-bay buildings in two arrangements of V bracings in adjacent and distant bays

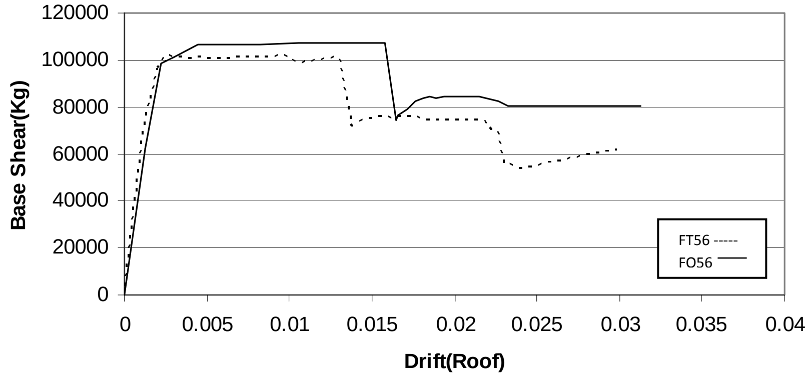


Figure 8. Base shear – roof displacement curves of 5-story 6-bay buildings in two arrangements of X bracing in adjacent and distant bays

It can be seen in Figures 6 to 8 that for all three bracing types, in case of FT systems the initial stiffness in higher, but the maximum strength is lower. Comparison of Figures 6 to 8 shows again that chevron and V bracings have higher stiffness and strength than X bracing, and that the chevron bracing is a little stronger than V bracing.

5.1.2. Plastic zones

To see how the bracing arrangement affects the formation of plastic zones in structural elements of the building, the 7-storey 6-bay buildings with V bracing are considered. Figures 9 to 11 show the plastic zones created in bracing elements in three PLs of IO, LS and CP for comparison.

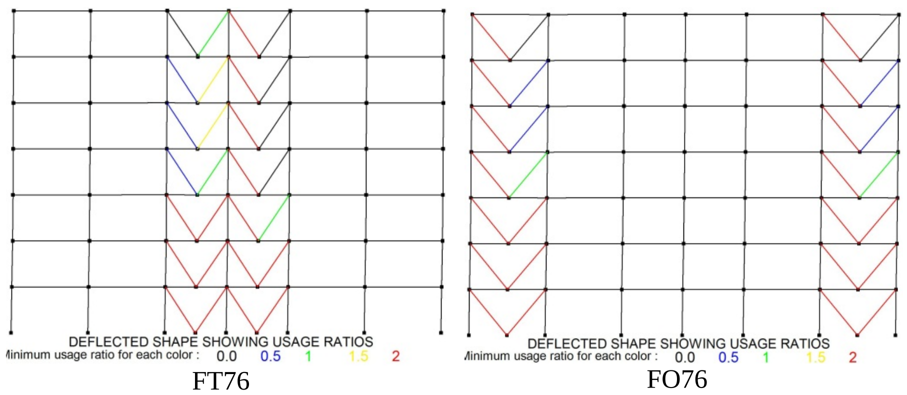


Figure 9. Plastic zones in FT76 and FO76 buildings with V bracings in IO PLs

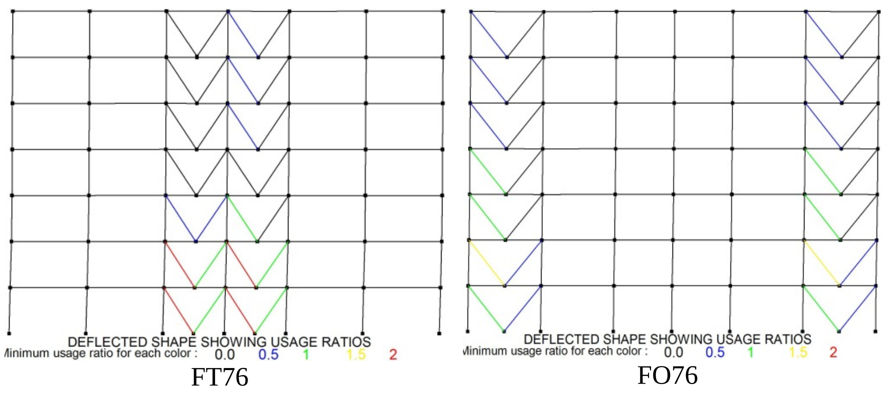


Figure 10. Plastic zones in FT76 and FO76 buildings with V bracings in LS PLs

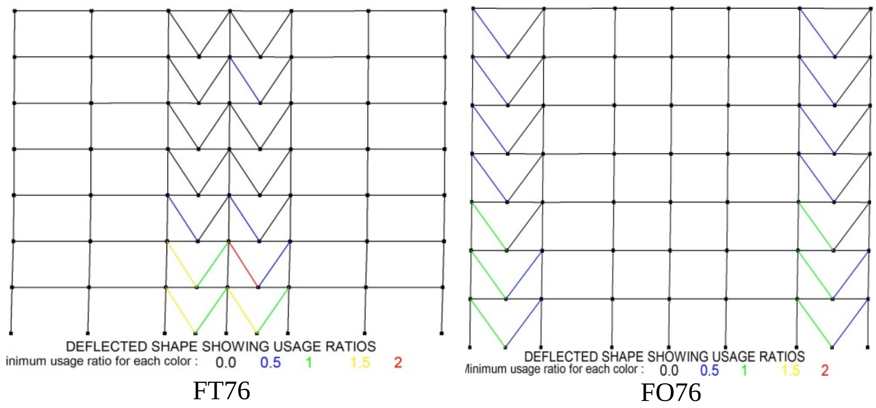


Figure 11. Plastic zones in FT76 and FO76 buildings with V bracings in CP PLs

It can be seen in Figures 9 to 11 that for IO PL the plastic zones occur in all stories or most of lower stories in all cases, while for LS and CP PLs the plastic zones occur only in the lowest story or just few lower stories. This means that the employed seismic design code has resulted in more conservative design of upper stories compared to the lower stories of the studied buildings.

5.2. Nonlinear Dynamic analysis results

5.2.1. The results of nonlinear time history analyses

Figures 12 to 14 show samples of the roof displacement time history in case of three of the 7 used earthquakes, including Kocaeli, Imperial Valley, and El Centro.

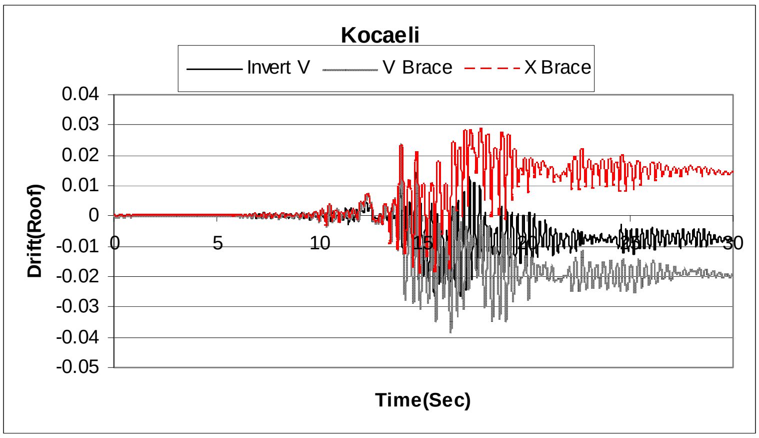


Figure 12. Sample roof displacement nonlinear time histories subjected to Kocaeli Centro earthquake

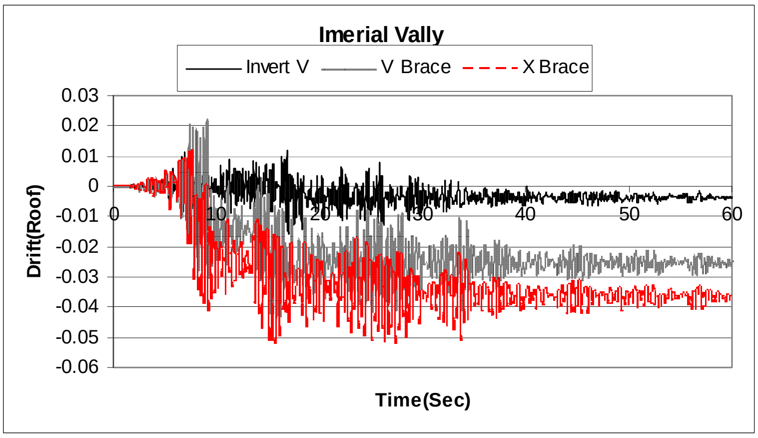


Figure 13. Sample roof displacement nonlinear time histories subjected to Imperial Valley earthquake

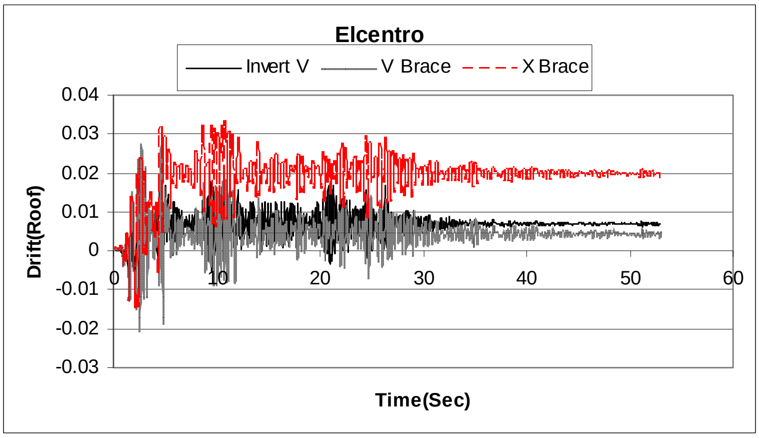


Figure 14. Sample roof displacement nonlinear time histories subjected to El Centro earthquake

It can be seen in Figures 12 to 14 that the trend of going to inelastic range, and the amount of displacement response of buildings with various types of bracings is highly dependent on the earthquake characteristics. However, it can be realized that in most cases the systems with X bracing give higher response values, and the systems with chevron bracing show lower responses. To get a better understanding of the effect of bracing pattern on the inelastic seismic behavior of the considered buildings the maximum lateral drift ratio values of all stories of the all 7-story buildings of FO76 and FT76 types with various bracing types, subjected to the 8 selected earthquakes, are given, respectively, in Tables 2 and 3 for comparison.

Table 2. Maximum lateral drift ratio values of FO76 buildings with various types of bracings, subjected to the 8 selected earthquakes

Bracing Type / Story No.	**Earthquake Name**							
X	**Chi-Chi**	**Duzce**	**El Centro**	**Imperial Valley**	**Kocaeli**	**Landers**	**Loma Prieta**	**Tabas**
1	0.0015	0.0018	0.0017	0.0018	0.0018	0.0015	0.0014	0.0015
2	0.0021	0.0033	0.0040	0.0025	0.0025	0.0022	0.0021	0.0022
3	0.0034	0.0045	0.0055	0.0053	0.0041	0.0038	0.0027	0.0034
4	0.0025	0.0044	0.0032	0.0050	0.0033	0.0036	0.0033	0.0025
5	**0.0062**	***(0.0121)***	**0.0110**	**0.0088**	**0.0114**	**0.0119**	**0.0109**	**0.0062**
6	0.0050	0.0063	0.0072	0.0041	0.0081	0.0081	0.0090	0.0050
7	0.0033	0.0039	0.0036	0.0034	0.0038	0.0042	0.0041	0.0033
Invert V	**Chi-Chi**	**Duzce**	**El Centro**	**Imperial Valley**	**Kocaeli**	**Landers**	**Loma Prieta**	**Tabas**
1	0.0012	0.0016	0.0013	0.0016	0.0016	0.0018	0.0012	0.0012
2	0.0017	0.0024	0.0019	0.0024	0.0037	0.0034	0.0021	0.0017
3	0.0024	0.0028	0.0024	0.0026	0.0032	0.0029	0.0028	0.0025
4	0.0033	0.0032	0.0028	0.0028	0.0034	0.0046	0.0036	0.0034
5	0.0050	0.0038	0.0033	0.0032	0.0041	0.0044	0.0045	0.0050
6	**0.0060**	**0.0040**	**0.0036**	**0.0035**	**0.0047**	**0.0054**	**0.0047**	***0.0061***
7	0.0052	0.0037	0.0033	0.0033	0.0040	0.0046	0.0046	0.0053
V	**Chi-Chi**	**Duzce**	**El Centro**	**Imperial Valley**	**Kocaeli**	**Landers**	**Loma Prieta**	**Tabas**
1	0.0016	0.0021	0.0020	0.0017	0.0020	0.001754	0.00178	0.001632
2	0.0026	0.0039	0.0050	0.0031	0.0045	0.003052	0.00314	0.002691
3	0.0029	0.0046	0.0063	0.0039	0.0057	0.005347	0.00333	0.002962
4	0.0033	0.0054	0.0090	0.0057	0.0065	0.006537	0.00452	0.003335
5	0.0038	**0.0055**	***0.0103***	**0.0065**	**0.0070**	**0.007529**	**0.00600**	0.003846
6	**0.0042**	**0.0055**	0.0091	**0.0065**	0.0069	0.007444	0.00596	**0.004293**
7	0.0037	0.0050	0.0082	0.0056	0.0067	0.006318	0.00468	0.003744

Table 3. Maximum lateral drift ratio values of FT76 buildings with various types of bracings, subjected to the 8 selected earthquakes

Bracing Type / **Story No.**	**Earthquake Name**							
X	**Chi-Chi**	**Duzce**	**El Centro**	**Imperial Valley**	**Kocaeli**	**Landers**	**Loma Prieta**	**Tabas**
1	0.0016	0.0013	0.0019	0.0017	0.0015	0.0017	0.0018	0.0013
2	0.0023	0.0016	0.0029	0.0020	0.0021	0.0030	0.0026	0.0019
3	0.0090	0.0018	0.0043	0.0055	0.0048	0.0047	0.0026	0.0037
4	0.0100	0.0023	0.0032	0.0038	0.0036	0.0036	0.0027	0.0051
5	***(0.0147)***	**0.0080**	**0.0114**	**0.0092**	**0.0080**	**0.0103**	**0.0094**	**0.0127**
6	0.0078	0.0063	0.0050	0.0053	0.0044	0.0067	0.0069	0.0101
7	0.0031	0.0028	0.0020	0.0028	0.0023	0.0030	0.0031	0.0034
Invert V	**Chi-Chi**	**Duzce**	**El Centro**	**Imperial Valley**	**Kocaeli**	**Landers**	**Loma Prieta**	**Tabas**
1	0.0015	0.0016	0.0015	0.00154	0.0019	0.00148	0.0019	0.0017
2	0.0020	0.0027	0.0020	0.00206	0.0022	0.00185	0.0022	0.0022
3	0.0021	0.0029	0.0020	0.00206	0.0022	0.00186	0.0022	0.0023
4	0.0022	0.0037	0.0029	0.00201	0.0020	0.00200	0.0022	0.0023
5	0.0030	0.0040	0.0037	0.00203	0.0031	0.00308	0.0033	0.0033
6	**0.0037**	***0.0053***	**0.0044**	**0.00301**	**0.0037**	**0.00380**	**0.0041**	**0.0040**
7	0.0027	0.0047	0.0036	0.00180	0.0031	0.00330	0.0038	0.0033
V	**Chi-Chi**	**Duzce**	**El Centro**	**Imperial Valley**	**Kocaeli**	**Landers**	**Loma Prieta**	**Tabas**
1	0.0018	0.0016	0.0019	0.00157	0.0020	0.00168	0.0019	0.0018
2	0.0023	0.0022	0.0023	0.00186	0.0030	0.00194	0.0029	0.0020
3	0.0031	0.0031	0.0020	0.00200	0.0030	0.00210	0.0032	0.0020
4	0.0035	0.0032	0.0020	0.00205	0.0021	0.00199	0.0023	0.0020
5	0.0038	0.0039	0.0034	0.00223	0.0022	0.00205	0.0032	0.0032
6	**0.0048**	***0.0060***	**0.0047**	**0.00358**	**0.0040**	**0.00346**	**0.0044**	**0.0043**
7	0.0036	0.0047	0.0033	0.00266	0.0028	0.00247	0.0032	0.0032

It can be seen in Tables 2 and 3 that the maximum drift in the 7-story buildings (shown by **bold** figures in the tables) has usually occurred in the 5th or the 6th story of the building for all of used earthquakes, and the relative maximum among the maximum drift values (shown by ***italic bold*** figures) does not occur necessarily for one earthquake for buildings with various bracing type. However, it is noticeable that the absolute maximum drift value (shown by ***(italic bold)*** figures in a pair of parentheses) has always happened in the buildings with X bracing. It can also be seen in Tables 2 and 3 that, in average, chevron bracing leads to less drift values in the building.

6. CONCLUSIONS

Based on the results of pushover and NLTHA analyses performed on the buildings considered in this study it can be concluded that:

- In all three kinds of bracings the arrangement in non-adjacent bays leads to lower stiffness but higher strength than arrangement in adjacent bays.
- In all cases the chevron bracing leads to higher stiffness compared to the other two types, while the other two types show almost the same stiffness.
- The amount of ultimate resistance for chevron bracing is around 50% higher than the X bracing. This means that using the same value for response modification factor of all types of concentric bracing does not seem appropriate, and the design codes needs some revision in this regard.
- The employed seismic design code has resulted in more conservative design of upper stories compared to the lower stories of the buildings. This means that using a single value for response modification factor is not adequate.

- For IO performance level plastic zones occur in all stories or most of lower stories in all cases, while for LS and CP performance levels plastic zones occur only in the lowest story or just a few lower stories. This means that the employed seismic design code has resulted in more conservative design of upper stories compared to the lower stories of these buildings, and seismic design codes need revision in this regard as well.

Finally, it should be noted that to make a general conclusion and to propose modifications for the present seismic design codes more research should be conducted with regard to other types of building structures.

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