X-Bracing Configuration and Seismic Response

Saeed Rahjoo, Babak H. Mamaqani

***Abstract*—**Concentric bracing systems have been in practice for many years because of their effectiveness in reducing seismic response. Depending on concept, seismic design codes provide various response modification factors (R), which itself consists of different terms, for different types of lateral load bearing systems but configuration of these systems are often ignored in the proposed values. This study aims at considering the effect of different x-bracing diagonal configuration on values of ductility dependent term in R computation. 51 models were created and nonlinear push over analysis has been performed. The main variables of this study were the suitable location of X–bracing diagonal configurations, which establishes better nonlinear behavior in concentric braced steel frames. Results show that some x-bracing diagonal configurations improve the seismic performance of CBF significantly and explicit consideration of lateral load bearing systems seems necessary.

***Keywords*—**Bracing configuration, concentrically braced frame (CBF), Push over analyses, Response reduction factor.

I. INTRODUCTION

THE response Reduction Factor (R) which is widely used in most of the seismic design codes all over the world, is trying to compensate the effects of ductility of the system to withstand seismic load. The ultimate capacity of each structural system depends on its structural configuration and specifications, including type of bracing and size of bracing elements in case of braced frames. Consequently, the codes give various values of R depending on the lateral load bearing system of the building. For example, some codes [1], [2] suggest a value of 5 for the case of Ordinary Concentrically Braced Frame (OCBF), and a value of 6 for the case of Special Concentrically Braced Frame (SCBF). This value in Iranian code of practice for seismic resistant design of buildings is 6 (there are no ordinary or special classification). However, the R-values in codes do not depend on the number of braced bays and their relative location, or even the overall pattern of bracing while the number of braced bays in a frame is important considering their effects on redundancy. Several analytical and experimental studies have been performed on braced frames since early 70s, of which some experimental works will be briefly reviewed here.

Shaishmelashvili and Edisherashvili [3] have done an experimental study on dynamic characteristics of large-scale models of multi-story steel frame buildings with different vertical bracings. They have tested some large-scale models of a 9-story building with 12 different bracing schemes in free and forced (resonance) vibration states. Suzuki et al. [4] performed an experimental study on the elasto plastic behavior of tensile braced frames to obtain the restoring force characteristics of low-rise steel structures. Wakabayashi and his colleagues [5] did some experimental studies on the elasto plastic behavior of braced frames under repeated horizontal loading. In a part of those studies, experiments of one story-one bay braced frames were conducted to investigate the hysteretic behavior of this kind of steel frames whose braces were made of built-up H-shapes and whose columns and beams were made of rolled H-shapes. Lee and Bruneau [6] studied the energy dissipation of compression members in concentrically braced frames by reviewing the available experimental data. Design and detailing requirements of seismic provisions for CBFs were specified based on the premise that bracing members with low KL/r and b/t will have superior seismic performance. However, they claimed that relatively few tests have investigated the cyclic behavior of CBFs, and hence, it is legitimate to question whether the compression member of a CBF plays a significant role as what has been typically assumed implicitly by the design provisions.

Shademan et al. [7]-[10] have studied the effect of Bracing configuration on limit state nonlinear behavior of steel braced frame and response reduction factor values with different configuration including diagonal, X and chevron bracing configurations; the Results indicate that proper configuration which is usually neglected in current codes and design process, can significantly affect at least ductility dependent term of the response reduction factor and maximum corresponding values maybe more than 1.5 times of suggested values in building codes [1], [2] but in other cases, especially medium-rise frames with fewer bays, it is nearly half of the suggested value. They [11], [12] also have conducted six experiments in one third scale to investigate the effect of different bracing configurations on seismic behavior of concentrically braced frame (CBF) inelastic behavior. They concluded that CBF with adjacent bracing have greater response reduction factor compared to other configurations. Ahamady Jazany et al. [13] have analytically studied effect of Use of Sliding-Tension on energy dissipation of CBF. They showed that this system possesses a stable hysteretic behavior with regular form without decrease in stiffness and resistance. Also, Ahamy Jazany et al. [14] have conducted an experimental and analytical study to investigate the effect of masonry infill on the seismic performance of CBFs. They showed that the presence of masonry infill could increase the lateral stiffness and load carrying capacity of the special CBF by 33% and 41%, respectively. One of the simplest methods for nonlinear analysis of complex structures is nonlinear static analysis, also known as pushover analysis. Despite its limitations, pushover analysis could provide valuable information about capacity of structures, deformation demand,

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discontinuity in strength distribution, and potential of energy absorption. To evaluate the seismic behavior and determination of ductility factor, over strength factor and distribution of plastic hinges in structures with different x-bracing configurations, push over analysis is used in this research.

II. Analysis: Model Description and Naming Convention

The analytical models selected based on typical practice of frames in Iran. The bay width of models was considered to be 5 meters and the heights of stories to be 3 meters. Models with different number of stories and bays were used to investigate the effect of X-bracing system placement on R-factor, over strength factor and plastic hinge distribution. In order to study these parameters, frames with 3, 5 and 7 bays and 6, 12, 18 stories were modeled. The form of NS –TYPE and X is applied for naming the models, where N is the number of bays, S is the abbreviation of span, and X declares the type of bracing configuration respectively which is x-configuration here. Fig. 1 shows types of 18 stories with different number of spans and its naming based on naming convention used in this study. The naming convention for six and 12-story frames are similar to 18-story frame.

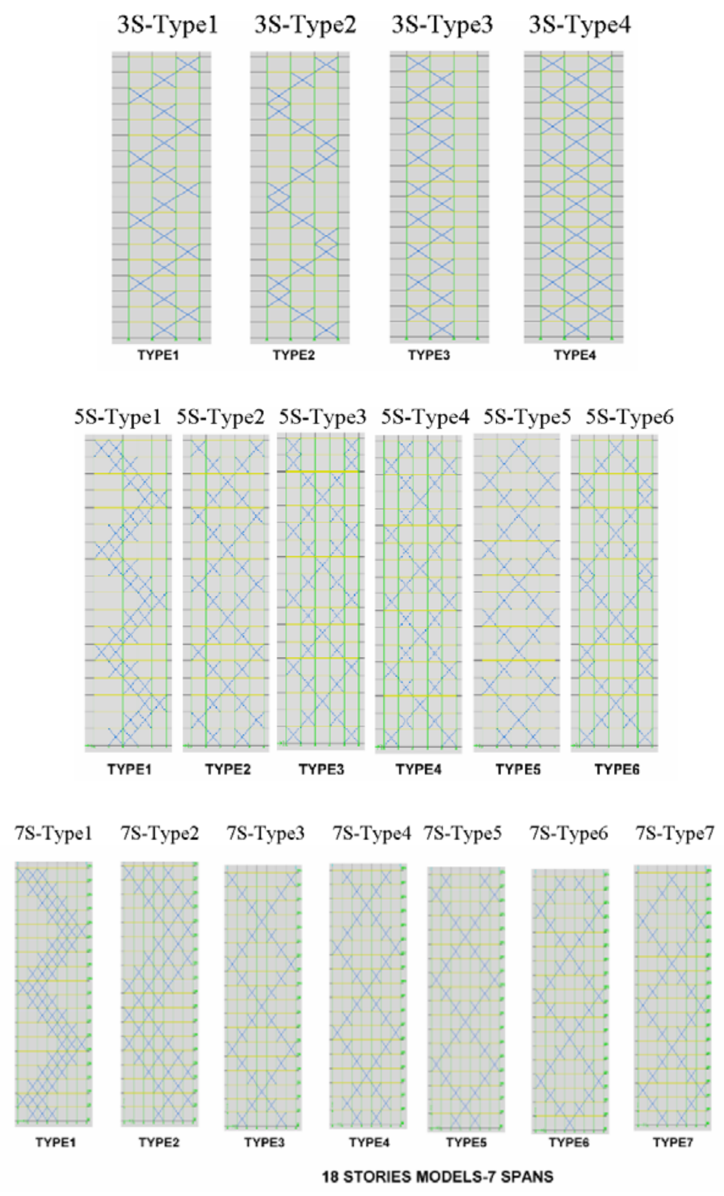

Fig. 1 View of analytical Models with different bracing configurations 18-story CBF frame

Gravity loads were applied according to Iranian loading code and composite floor system was assumed, seismic loading were applied according to Iranian seismic design code (fourth edition) and soil type II was assigned for the site effect. Computation procedure of base shear is shown in the Table I; moreover, the response spectrum analysis was used for designing the models.

TABLE I
BASE SHEAR AND SLASH FORCE COMPUTATION

2800 Standard	6 STORY	12 STORY	18 STORY
$T=0.05\times H^{3/4}$ (sec)	0.7	1.176	1.594
Soil Type 2 $B=2.5\times(0.5/T)^{2/3}$, $B\leq 2.5$	1.9	1.176	1.154
C=ABI/R	0.1166	0.0825	0.0673
V=CW (ton)	74.8	107.1	133.5
F_t=0.07TV (ton)	0	8.82	14.91

Steel frames were designed according to AISC-ASD 89 since it is compatible with Iranian steel design code. The effective length factor of braces for out of plane buckling is considered equal to 0.67; this value for in plane is 0.5. IPE sections were used for beams and IPB section were provided for columns and also double L sections were employed for braces. Material properties assumed compatible with ST-37 steel grade.

III. Pushover Analysis: Load Pattern and Hinge Specifications

FEMA-356 [15] was used to conduct displacement controlled pushover analyses. The reverse triangular loading pattern or first mode compatible pattern was applied according to the problem at hand. During the analysis, the location of plastic hinges and the analysis termination criteria were controlled. Properties of hinges in each element were defined according to geometry, material mechanical properties and applied forces in the elements. Axial - moment interaction hinges (P-M hinge) [15] were used for columns and axial hinges was assigned to brace elements (P hinge) [15].

A. Numerical Results

The results of two-dimensional nonlinear analyses were depicted as base shear versus roof displacement. Some information was derived from the curves that are important for computation of strength factor and response reduction factor, such as yield displacement, yield base shear, ultimate base shear and ultimate displacement. Fig. 2 shows base shear versus roof displacement for 6 story 7-span frame and different types of X-bracing configuration as a sample.

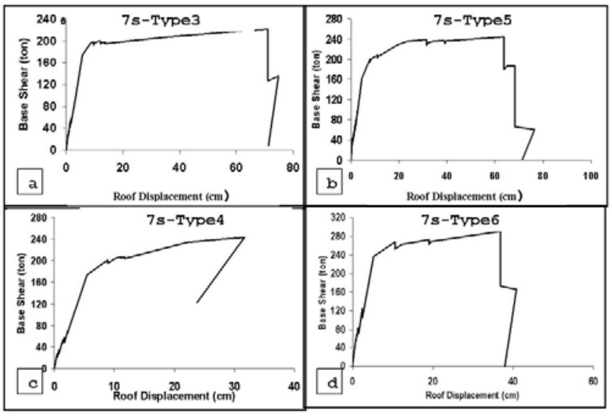

Fig. 2 Base shear vs. roof displacement for 6-story frame and different configuration of X-bracing

In previous research effects of bracing configurations were not studied beyond elastic response as presented here. Computation of R can be carried out using the following method but there are some essential values to be derived first. These values include: yield and ultimate displacements shown respectively by D_y and D_u; also yield force and elastic strength demand force which would be presented by F_y and F_{ed} notations respectively. R estimation can be performed by defining two factors: strength demand reduction factor, R_d as (1) and over strength factor, Ω which is presented in (2). Fig. 3 shows parameter derived for evaluation of R-values [16], [17]. Then R can be computed as (3):

$$R_d = \frac{Elastic\ Strength\ Demand}{Real\ Strength} \tag{1}$$

$$\Omega = \frac{Real\ \ Strength}{Design\ Strength} \tag{2}$$

$$R = R_d.\Omega \tag{3}$$

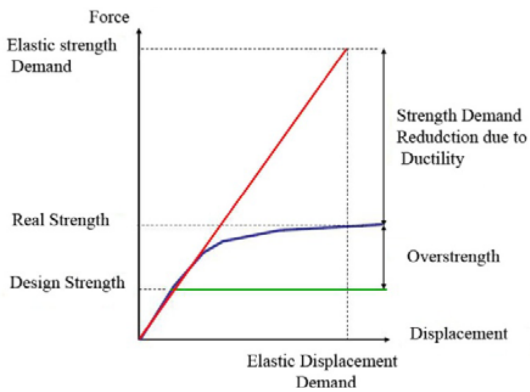

Fig. 3 Parameters used in R evaluation

Effects of different bracing configurations on response reduction factors in line with over strength factor have been summarized as Table II. Comparison of response reduction factors for the models studied in this research are displayed in Figs. 4 to 6. Response reduction factor for each configuration of CBFs is different and strongly depends on bracing configurations. The discussion and conclusion will be presented in next section.

IV. Discussion and Conclusion

Analytical study of the configuration influence on concentrically braced frame nonlinear behavior has been performed**.** The results show that there are meaningful differences between seismic behavior of concentric braced frame for different bracing configuration and the following conclusion can be drawn:

1- According to Fig. 4, generally in low rise frames (Six stories) with low number of spans, type2 demonstrates better nonlinear response (i.e. 3S-Type 2 and 5s-Type 2), when the number of spans increases in (5-span), also type 2 has larger response reduction values than other configurations. In the frame with 7 spans with 6 stories, types2, 3, 4 (7S-type 2, 3 and 4) are better choices.

2- According to Fig. 5, it seems in middle rise frames (12 stories) with varieties of number of spans, type2, 3 and type4 behave acceptably (3, 5 and 7S-type 2, 3)

3- According to Fig. 6 for the structures with 18 stories, the results are scattered, but in less number of spans, 3S-Type2 and 3 behave more acceptable and they have larger R-value compared to other configurations.

4- This study showed that the response reduction factor (R) for this kind of structural system represented in current codes is underestimation, for example for 18 stories frame with 7 spans nearly all configurations, present Response reduction factor of "8" or larger value, while IBC represent value of 6. This study suggest the CBFs in high rise frames must categorized according to their configurations and on the basis of this finding, n selection of the response reduction factor "R" configuration should also be taken into account.

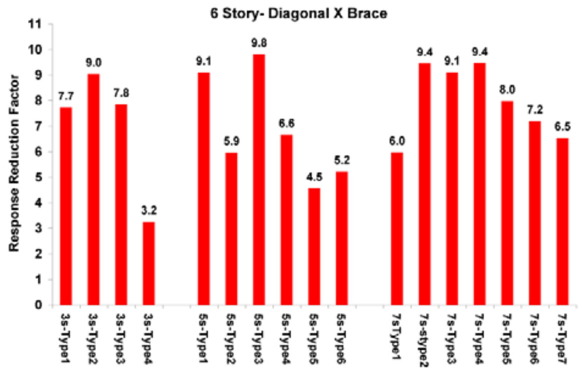

Fig. 4 Computed R-Values for six-story frame and different braced bays

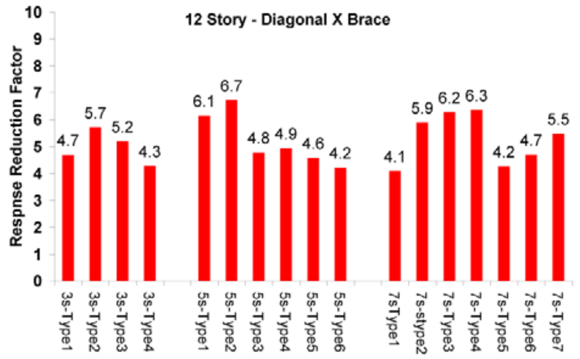

Fig. 5 Computed R-Values for 12-story frame and different braced bays

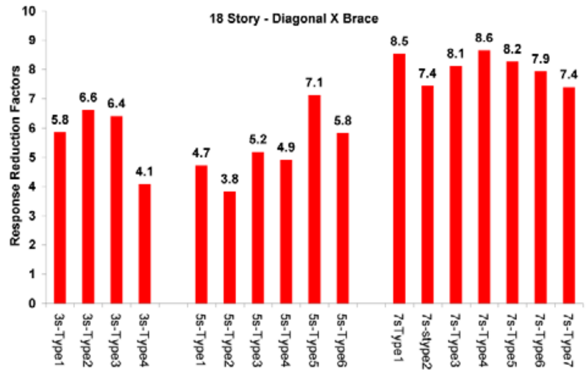

Fig. 6 Computed R-Values for 18-story frame and different braced bay

TABLE II
COMPUTED R-VALUES AND OVER STRENGTH FACTORS FOR DIFFERENT BRACING CONFIGURATIONS (TON, CM)

	6 story-Diagonal X brace			12 story-Diagonal X brace			18 story-Diagonal X brace		
span	initial stiffness	R	O.S	initial stiffness	R	O.S	initial stiffness	R	O.S
3S-Type1	15.23	7.7	1.13	9.01	4.67	1.02	4.18	5.84	1.06
3S-Type2	16.71	9.01	1.15	8.96	5.69	1.06	4.51	6.6	0.5
3S-Type3	15.07	7.83	1.14	7.09	5.17	1.11	3.94	6.39	1.03
3S-Type4	22.17	3.23	1.29	10.65	4.26	1.13	7.76	4.07	1.4
5s-Type1	36.33	9.07	1.1	19.66	6.13	1.12	14.57	4.69	1.32
5s-Type2	35.84	5.95	1.07	17.24	6.7	1.29	14.49	3.81	1.23
5s-Type3	27.45	9.8	1.12	20.38	4.76	1.37	14.6	5.16	1.34
5s-Type4	36.89	6.64	1.38	21.45	4.92	1.35	14.36	4.89	1.37
5s-Type5	36.77	4.55	1.41	21.75	4.56	1.13	14.95	7.11	1.52
5s-Type6	27.14	5.21	1.16	19.38	4.2	1.36	13.96	5.8	1.2
7s-Type1	59.75	5.96	1.3	30.05	4.07	1.69	21.81	8.52	1.18
7s-Type2	48.85	9.42	1.18	29.34	5.87	1.74	20.05	7.42	1.02
7s-Type3	26.94	9.08	1.11	23.4	6.25	1.08	16.88	8.1	1.21
7s-Type4	23.81	9.44	1.1	24.29	6.34	1.31	16.86	8.63	1.01
7s-Type5	23.82	7.96	1.29	26.22	4.25	1.75	13.84	8.24	0.98
7s-Type6	44.58	7.17	1.27	23.81	4.68	1.54	16.85	7.91	1.32
7s-Type7	45.09	6.52	1.26	23.05	5.45	1.19	14.12	7.38	1.13

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