ANALYTICAL AND EXPERIMENTAL STUDY OF THE EFFECT OF BRACING PATTERN IN THE LATERAL LOAD BEARING CAPACITY OF CONCENTRICALLY BRACED STEEL FRAMES

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ABSTRACT :

Seismic design codes give various values for Response Modification Factor (RMF) depending on the lateral load bearing system of the building structure, for example, they give a value like 10 for ordinary moment frames and a value like 6 for Concentrically Braced Frames (CBFs). However, the value of RMF for CBFs in codes does not depend on the number of braced bays and their location, or even the overall pattern of bracing in the building. This is while, at least, the number of braced bays in a frame is important from the redundancy point of view. This paper presents the results of an analytical and experimental investigation performed on a series of 5-story CBFs with three bays of which one or two bays have been braced with different patterns. At first, some Push Over Analyses (POA) have been performed to find out the ultimate capacity of CBFs. Then, some 1/3 scale samples of frames have been built and tested subjected to lateral loads to verify the numerical calculations. Results show that the ultimate capacity of CBFs and their ductility factor strongly depend on the number and location of braced bays and the bracing pattern. The capacity can vary up to 100% from case to case, and the displacement ductility factor can be as low as 3.5 in some cases, which is much lower than the code suggested value. On this basis it can be said that the code suggested values of 'response modification factors' need essential modifications.

KEYWORDS: Response modification factor, Push-over analysis, Bracing pattern, Ultimate capacity

1. INTRODUCTION

The 'Response Modification Factor' (RMF) which has been widely used in most of the seismic design codes all over world, is basically for taking into account the possibility of plastic deformation of the structure, or in other words, the ultimate capacity of the system for withstanding against earthquake effect. Obviously, the ultimate capacity of each structural system, such as a moment frame or a braced frame, depends on its structural configuration and specifications, including type of bracing and size of bracing elements in case of braced frames. Accordingly, the codes give various values for RMFs depending on the lateral load bearing system of the building structure. For example, most of codes suggest a value of 10 for the case of ordinary moment frame (OMF), and a value of 6 for the case of Concentrically Braced Frame (CBF). However, the RMF value in codes does not depend on the number of braced bays and their relative location, or even the overall pattern and form of bracing in a building's frames. This is while, at least, the number of braced bays in a frame is important from the redundancy point of view.

Several analytical and experimental studies have been performed on braced frames since early 70s, of which some experimental works are briefly reviewed here. Shaishmelashvili and Edisherashvili (1973) have done an experimental study on dynamic characteristics of multi-storey steel frame building large-scale models 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. Clearly, in their tests just the linear behavior of building has been considered.

Suzuki and his colleagues (Feb. 1975) have performed an experimental study on the elasto-plastic behavior of tensile braced frames to obtain the restoring force characteristics of low steel structures. Alternating horizontal force was applied at the point of the second floor under a constant vertical load, paying attention to the behavior of two columns subjected to varying axial forces. Test specimens consisted of one-story, one-bay frames with wide flange sections and braces of round steel bars. The relation between shear force and displacement in each column was investigated by numerical analysis. According to the results, the elasto-plastic behavior of the two columns is obviously different; one is subjected to additional tensile force and the other to additional compressive force. From these results, it is found that the restoring force characteristics of braced frames are stable but that the hysteresis loops in each column become unstable because of the additional compressive force.

Inoue and Murakami (1978) performed a study on the plastic design of braced multi-story steel frames by doing some tests on the elastic plastic behavior of 3-story 3-bay braced and un-braced steel frames under monotonic or alternating horizontal forces. Four specimens were tested, two braced frames and two un-braced frames, both designed against the same factored horizontal forces, and their individual members processed to have a net strength. Test frames were subjected to horizontal forces proportional to design forces at each floor level. The force-deflection curves did not differ markedly from test results reported by many investigators. In the case of the braced frames, the bracing members of the lowest story having the smallest ratio of story shear-force shared by the bracings to the total shear force buckles and yielded at the outset, so that the relative story displacement of the lowest story increased before buckling, and yielding developed in all other bracing members. After this happened, however, the relative story displacement of each story increased uniformly. This result suggests that the bracing members should be designed so as to buckle or yield simultaneously against seismic force, and that experimental force-deflection curves are well predicted by the generalized hardening hinge method.

Wakabayashi and his colleagues (1980) did some experimental studies on the elastic-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 are made of built-up H-shapes and whose columns and beams are made of rolled H-shapes. Hysteretic behavior and transition and change of load-carrying capacity of each component member of a frame, i.e., braces, columns, and beams under repeated horizontal load, were examined individually, and the hysteretic behavior of a braced frame as a whole was investigated. Interaction behavior between the braces built into a frame and the components of the surrounding frame was also discussed. It was found that the effective slenderness ratio for buckling of the braces built into a frame could be estimated by the slope-deflection method, taking the rotational rigidity of the members of the surrounding frame into account, and that the effective slenderness ratio for the estimation of post-buckling and hysteretic behavior could be approximated by the assumption that the braces would be rigidly fixed at the ends. Those tests also showed that as the columns are subjected to repeated large axial load due to the deformation of the brace, and the load-carrying capacity of the column decreases substantially when the axial load is large, the behavior of the column is largely affected by that of the brace, and that the load-carrying capacity and the ductility of the brace are reduced and exhausted when cracks are initiated as well.

Lee and Bruneau (2005) studied the energy dissipation of compression members in concentrically braced frames: by reviewing the experimental data. Expressing that 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, they claim 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 as significant a role as what has been typically assumed explicitly by the design provisions. In that study, the existing experimental data were reviewed to quantify the extent of hysteretic energy achieved by bracing members in compression in past tests, and the extent of degradation of the compression force upon repeated cycling loading. The focus of that study was mostly on quantifying energy dissipation in compression and its effectiveness on seismic performance. Based on the experimental data reviewed from previous tests, they found that the normalized energy dissipation of braces having moderate KL/r (80-120) do not have significantly more normalized energy dissipation in compression than those having a slenderness in excess of 120, and that the normalized degradation of the compression force envelope depends on KL/r and is particularly severe for W-shaped braces.

Fahnestock and his colleagues (2006) performed an experimental study on a large-scale buckling-restrained braced frame using the pseudo-dynamic testing method. As part of an integrated analytical and experimental research program on buckling-restrained braced frames (BRBFs), a large-scale BRBF was subjected to multiple earthquake simulations using the explicit Newmark algorithm. A hybrid testing approach was implemented to account for the P-Delta effects associated with the gravity load carried by the prototype building's gravity frames. The test frame sustained significant drift demands with almost no damage. Story drifts of nearly 5% and buckling-restrained brace maximum ductility demands of over 25 were observed in the maximum considered earthquake simulation. No stiffness or strength degradation was observed. Although residual drifts were large, the testing program demonstrated that the BRBF system can withstand significant seismic input and retain full lateral load-carrying capacity. Non-conventional brace-gusset and beam-column connections demonstrated excellent performance under very large drift demands.

It is seen that although several experimental studies have been performed, none of them have been focused on the study of so-called ‘response modification factors’ of CBFs. This paper presents the results of analytical and experimental investigations performed on a series of CBFs with various numbers and locations of X-braced bays. At fist some Push Over Analyses (POA) have been performed to find out the ultimate capacity of frames. Then, some 1/3 scale samples of CBFs with various numbers and locations of X bracings have been tested by actuators. The details of the study are explained briefly in the next sections of this paper.

2. INTRODUCING THE CONSIDERED FRAMES FOR THE STUDY

Regarding that the majority of residential steel buildings in large cities of Iran have around five stories and three bays, it was decided that samples of 5-story 3-bay frames are made for this study, and considerations with regard to the space limitations of the laboratory, in which the tests were supposed to be performed, resulted in making 1/3 scale frames. Also considering common number of braced bays in conventional constructions of the three bays of frames one or two bays have been braced, by three different patterns, including: 1) bracings in the middle bay, called here type I, 2) bracings in the first and the third bays, called here type II, and 3) bracings in the second and the third bays, namely two adjacent bays, called here type III. All of these frames have been modeled in a computer program and have been designed for a lateral force at their highest level, based on the tensile capacity of their bracing element(s), which has been chosen based on the smallest section appropriate for welding. The schematic geometry and the profile types obtained based on the design for these three types of braced frames are shown in Figures 1 to 3.

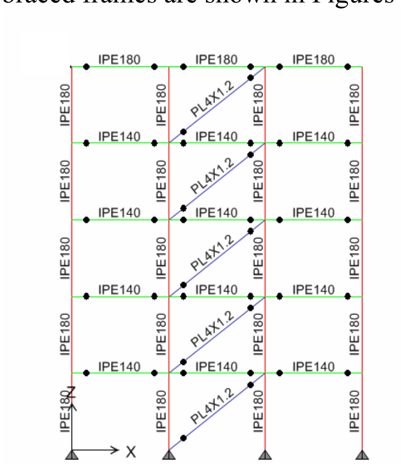

Figure 1. Frame type I

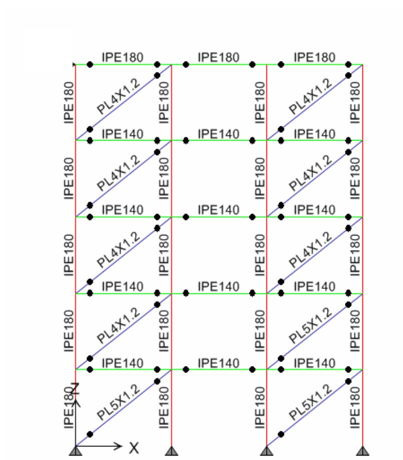

Figure 2. Frame type II

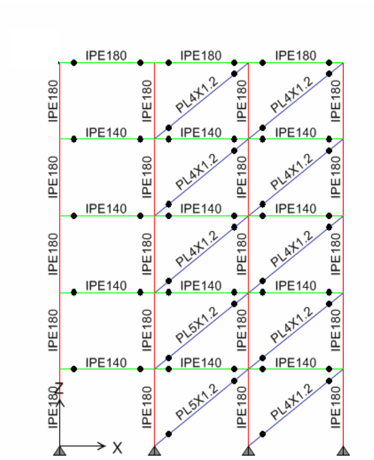

Figure 3. Frame type III

The locations of potential plastic hinges considered for nonlinear analyses are also shown in Figures 1 to 3. More detailed information about these frames can not be given here because of lack of space and can be found in the main report of the study (Heravi, 2008).

3. POSH OVER ANALYSES (POA)

The three considered frames have been analyzed subjected an increasing concentrated load at their highest level and in each case the push has been continued till the occurrence of instability in the frame. As a sample of POA results Figure 4 shows the load-displacement curve of frames of type I. It is seen that the yielding and ultimate displacement of this frame are respectively around 20 mm and 220 mm, and its yielding and ultimate strengths are respectively around 9 tonf and 11.8 tonf.

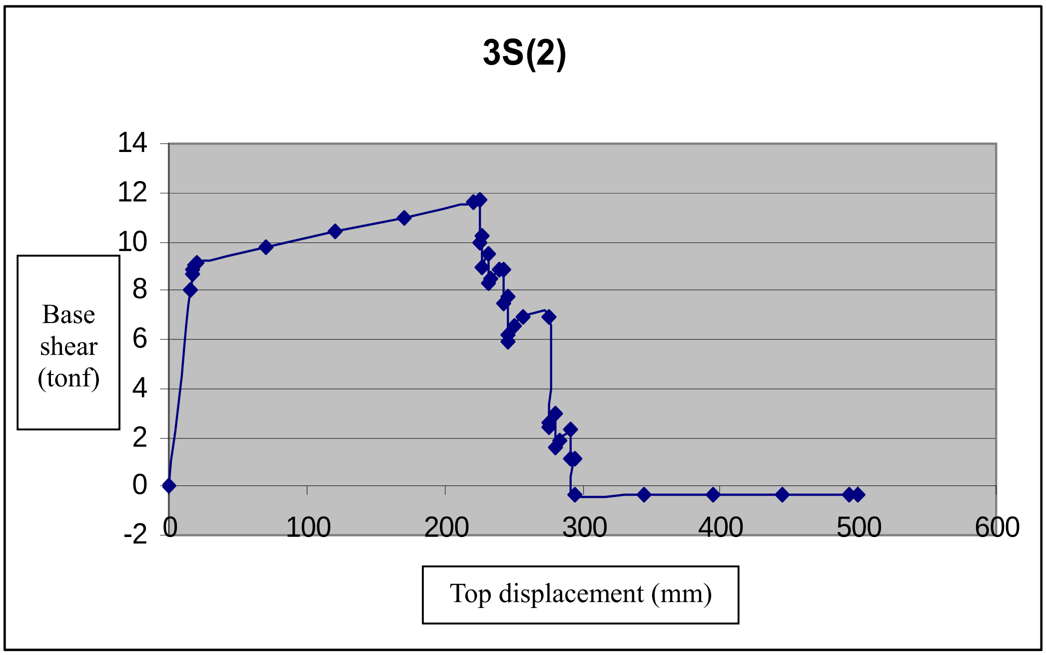

Figure 4. The POA result for frames of type I

Similar curves have been obtained for the other two frames and their results of the POA can be found in the main report of the study (Heravi, 2008).

4. CONSTRUCTING THE SAMPLES AND SETTING UP THE TESTS

For obtaining more reliable results from the experiments two identical samples of each three types of considered frames were constructed. It was tried to make the beam-to-column connections as much similar as the conventional connections in the current practice of steel structures construction in Iran. Since welded connection is the most popular type in Iran all connection were considered as that type. Of course, the required quality control was applied. Regarding that columns and beams of the scaled samples had been chosen stronger than the those in conventional steel CBFs, to make sure that all bracing element would yield before any beam or column, the sections of beams and columns were oversized to some extent and this could result in some additional moment resistance in beam-to-column connections if they were constructed using the conventional construction method. Therefore, a triangular part of each connection plate was cut off to let the frames connections act as a hinged connection by bending the angles used at the top and bottom of beam profiles, as shown in figure 5. Also a relatively rigid beam was considered to act as the foundation of frames as shown in Figure 6.

Figure 5. Connection plate with cut off corner to let the connections act as hinge connections

Figure 6. The relatively rigid bean used at the bottom of frames to act as their foundation

Figures 7 and 8 show respectively a frame of type I before test, and installed on supports to be tested. In Figure 8 the lateral support beams which have been installed to prevent the frame from out-of-plane buckling are also seen in dark green. Three pairs of these beams were used in each test, one pair at the top level of the frame to prevent the top beam from out-of-plane buckling, another pair at one level lower than the top, and the third pair almost at the mid-height of the frame.

Figure 7. A frame of type I before test

Figure 8. A frame of type I ready for test

Figure 9. The used actuator in the tests and details of its support and connection to samples

The location of actuator is also seen in Figure 8. The capacity of employed actuator is 25 tonf which is higher than the ultimate strength of considered frames, however, the frames of type III showed much higher strength than what the calculations predicted, and for their tests an actuator with capacity of 50 tonf was employed. Figure 9 shows the actuator from two different views, in which the details of actuator support and also the details of connection of actuator to samples can be seen. One important point with regard to shipment of samples was their low out-of-plane bending resistance, which could lead in large deformations and even damage to connections. Therefore, it was necessary to pick them up by getting a location of them very close to their center of gravity, when they were supposed to be moved as a horizontal plane, and their top at the middle, when they were supposed to be carried as a vertical plane, as shown in Figure 10.

Figure 10. A framed of type III during shipment to the lab by lifer (left), and ready for test (right)

Another point which is worth mentioning with regard to tests is about preventing them from out-of-plane buckling. Actually, at the beginning some pairs of rolling rods, as shown in Figure 11, were built and were supposed to be sufficient for preventing frames from buckling. However, during the first test of a frame to type III, which is stiffer than the frames of other two types, surprisingly the frame buckling happened, of course not in its first mode of buckling but the second mode as shown if Figure 12, and the test was stopped.

Figure 11. The rolling supports used at the beginning for preventing the frames from out-of-plane buckling

Figure 12. Buckling of a frame of type III in its second mode because of insufficient prevention

On the basis, other type of supports made of Channel Profiles, as mentioned before, were built and used (see Figures 8 and 10).

5. TEST RESULTS

The results obtained from the tests include: yielding and ultimate displacements, shown respectively by D_y and D_u, yielding and ultimate lateral forces, shown respectively by F_y and F_u, initial stiffness of frame, which is calculated as F_y/D_y, the displacement ductility factor, shown by $\mu=D_u/D_y$, frame over-strength (OS), calculated as F_u/F_y, and finally the RMF calculated as 1.4 times μ times the OS value. A sample of curves obtained from the tests is shown in Figure, which shows the force-displacement curve related to a frame of type I. By comparing this figure with Figure 4, one can realize that the yield displacement obtained by the test is almost 1.5 times of the value obtained from POA, however, the ultimate strength obtained by the test is close to the value predicted by POA.

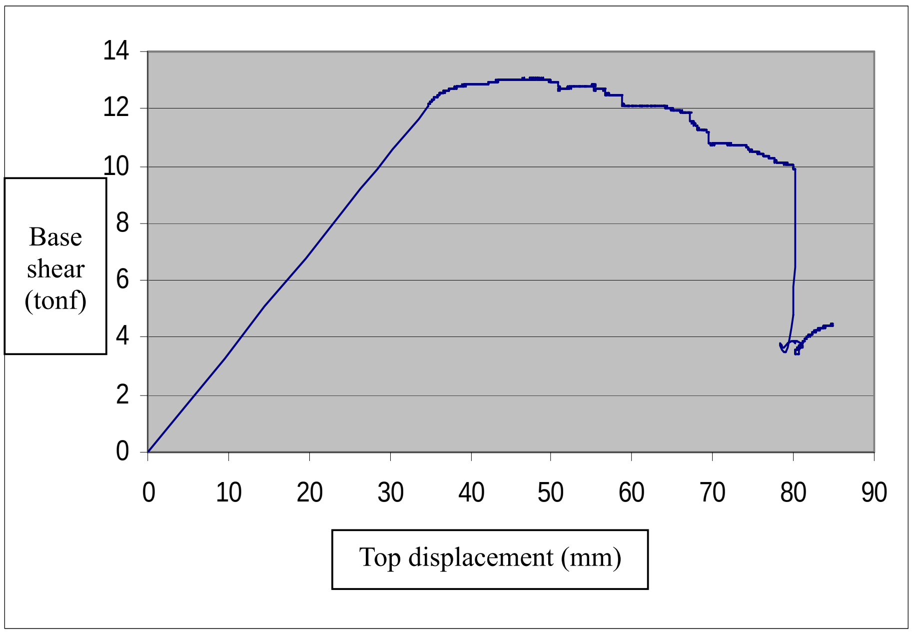

Figure 13. Force-displacement curve related to a frame of type I, obtained form the test

Table 1 shows the average results obtained from two series of tests (which were close enough to use their average) for the three frame types.

Table 1. Comparison of ductility factor obtained by tests for frames with different locations of braced bays

Frame type	D_y (mm)	F_y (tonf)	D_u (mm)	F_u (tonf)	Relative Initial Stiffness	$\mu=D_u/D_y$	Over-strength= F_u/F_y	RMF
I	29	10.7	151	12.7	0.369	5.223	1.187	5.78
II	30	17.3	195	10.5	0.578	6.51	0.606	3.68
III	30.5	30.5	228	48.8	1.000	7.425	1.600	11.08

It can be seen that although the number and location(s) of braced bays does not have a significant effect on the yielding displacement of the frame, the yielding force and particularly the ultimate force, as well as the ultimate displacement are quite different for different pattern of braced bays, and therefore the values of RMFs for are also different for different patterns of braced bays in CBFs. This fact has not been taken into consideration in any of the existing seismic design codes.

6. CONCLUSIONS

The numerical results of POA and also the results obtained form the tests show that the ultimate capacity of CBFs and their ductility factor, and therefore, their response modification factor strongly depend on the number and location of braced bays. The capacity can vary up to 100% from case to case, and the ductility factor can be as low as 3.5 in some cases, which is much lower than the code suggested value to more that 10 which is much higher than the code suggested values for CBFs. On this basis it can be said that the code suggested values of ‘response modification factors’ need essential modifications. Finally, it can be suggested that to get higher ductility factor the bracing are considered in adjacent bays.

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