# EXPERIMENTAL INVESTIGATION OF BEHAVIOR OF Y-SHAPED CONCENTRIC STEEL BRACING

S. Majid Zamani* and M. Rasouli
Department of Civil Engineering,  Building and  Housing  Research Center, Tehran, Iran

## ABSTRACT

Focusing on out of plane buckling of three member y-shaped concentric steel bracings, 5 full scale one story one bay specimens have been tested in BHRC Structural Engineering Lab. The results show that inelastic buckling is the governing mode of failure. The buckling load of bracing increases by moving the convergence point towards the center of bay, while the story drift at buckling decreases by this geometrical change. Hysteretic energy absorption and damping of bracing increases by moving the convergence point towards the corner of frame. By reinforcing the bracing members and increasing the radius of gyration of brace sections, the hysteretic energy absorption and damping of bracing has been slightly increased.

**Keywords:** steel, concentric, bracing, buckling, inelastic, hysteretic

## 1. INTRODUCTION

Y-shaped concentric bracing has been briefly mentioned in technical literature, Figure 1. Taranath [1] introduces it as one of different forms of concentric bracings. Also in Ref.2 this kind of bracing has been introduced and by presenting several perspective drawings, its advantages in providing door and window openings in comparison with other bracing forms has been shown. In Ref.3 which is devoted to industrial steel buildings, y-shaped bracing is introduced and it is stated that this type of bracing is not efficient under compression in its members and so it is necessary to use a symmetrical pair of y-bracing in every plan direction. This idea is also implied in Eurocode 8 [4] for design of buildings against earthquakes. Generally it can be deduced from literature that y-bracing is not so reliable under compression in its members. Maybe this stems from observing the common connections of bracing members including a single steel plate between twin steel sections or beside a single profile. This arrangement which results in severe reduction of radius of gyration in steel plate compared to that of braces themselves is very weak against out of plane buckling and leads to low critical loads .Also due to bending flexibility of a single

---

* Email-address of the corresponding author: majidzamani@bhrc.ac.ir

connection plate, bracing members can not participate in energy absorption of frame through out of plane bending. Under these conditions, it is natural to transfer the lateral loads to the bays that have tensile bracings and are more stable.
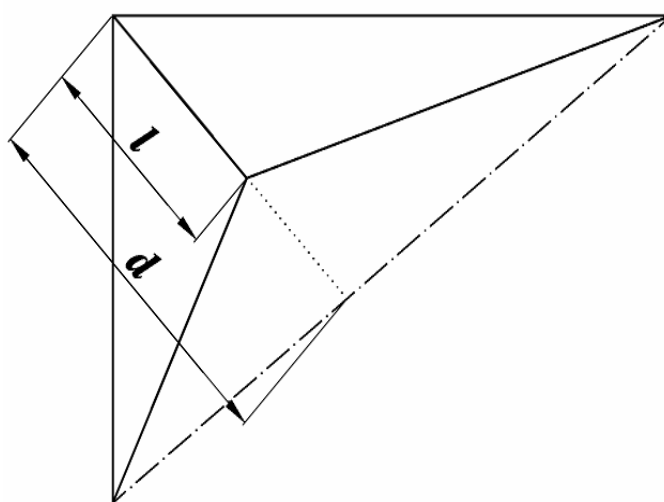


Figure 1. General view of y-bracing

Even if by some measures the load bearing capacity of y-bracing is increased up to acceptable levels, still it is necessary to use symmetrical pairs of this bracing to satisfy the code [5] requirement to resist the earthquake load by a combination of tension and compression in bracing members .This follows from the fact that all three members of y-bracing go into tension or compression simultaneously.

The most scientific document so far on behavior of y-bracing under compression is an MSc thesis [6] in University of Tehran which by assuming full fixity in brace connection to frame and hinge connection in convergence point, has derived relations for computing the critical lateral buckling load. Prismatic frame elements represent bracing members and bending properties have been accounted for. By summing the lateral stiffness of braces and equating it to zero, the relation for computing the critical load is derived. For frames of common dimensions and box sections for braces, it has been shown that critical load of y-bracing is comparable to its allowable tensile load.

Another aspect of using y-bracing as a means of reducing earthquake effects has been proposed by Moghaddam and Estekanchi [7].They have studied the behavior of y-bracing under tension in its members and concluded that due to geometrical non-linearity of this bracing ,it can act like a base isolator with low stiffness for small to moderate excitations while under high lateral disturbances ,its high stiffness helps to prohibit building collapse.

The above background shows the value of more thorough study of y-bracing. In this paper the results of a series of experimental tests conducted at BHRC is reported and the results elaborated.

## 2. DESCRIPTION OF SPECIMENS

All specimens are one bay one story frames with 4.2m span and 3.1m height. Specimen No.1 which is also called Model 1600 has l/d ratio equal to 0.75 while for specimens No.2 to No.5 (Model 1100-1 to Model 1100-4), l/d is equal to 0.54 (Figure 2&3).
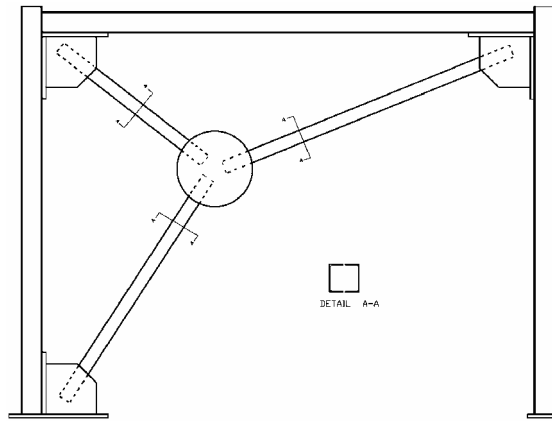


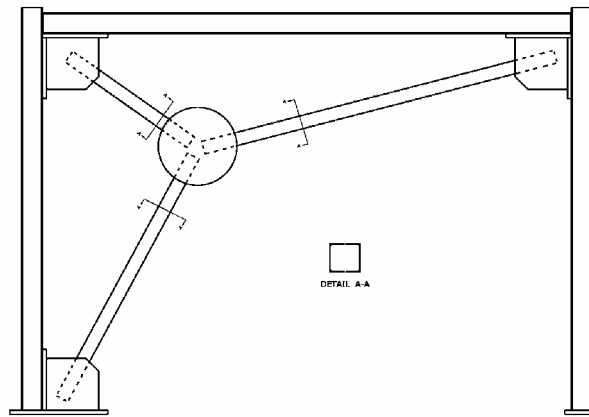Figure 2. Geometry of Model 1600 (Specimen No.1)



Figure 3. Geometry of Models 1100-1 to 1100-4 (Specimens No.2 to No.5)

Beam and columns are comprised of twin IPE180 sections joined with intermittent welding. Braces are generally of box section composed of two U100 sections joined by stitch welding their flanges together on top and bottom of section. Connections are made by double 10mm thickness plates welded on two sides of members. In specimens No.1100-2 to 1100-4 the longer brace which is starting point for buckling, has been strengthened by welding 70x9 mm steel strips on vertical sides of box section.

Lateral support by sag rods at the level of beam centroid was provided to simulate the effect of rigid floor restraining the beam from horizontal deflection or buckling.

Cyclic loading by hydraulic jacks was applied to the specimens at the level of beam centroid. Equal load increments were used up to first sign of yielding or buckling of members. After that, equal drift increments were applied up to failure.

## 3. TEST RESULTS

*3.1 Buckling Load*

From the experimental point of view, buckling load is the load level at which the out of plane lateral deflection of bracing starts to increase sharply without noticeable increase of applied load.

In Figure 4 through Figure 7 the applied load is plotted against the out of plane deflection of the longest brace member. Specimen 1600 shows buckling at 30500 kgf. By moving the convergence point towards the corner of frame, the buckling load is reduced to 21345 kgf for specimen 1100-1.

In specimen 1100-2 the longest brace member has been strengthened against out of plane buckling by welding full length steel strips to its vertical sides. These strips are also welded to the connection plates of bracing. In this specimen the tensile fracture of fillet welds at the end of reinforcing strips, destroyed the tensile capacity of bracing while the compressive capacity was not affected. This can be seen in Figure 10 where no buckling is evident in left branch of skeleton curve which reaches 28400 kgf. The tensile branch goes up to 29000 kgf and then drops to 27000 kgf.



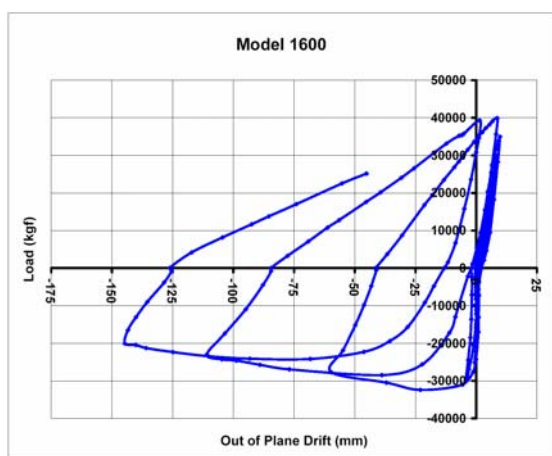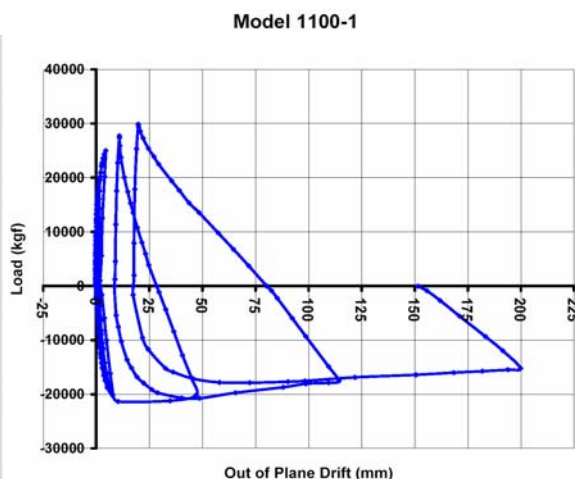Figure 4.                                                                      Figure 5.
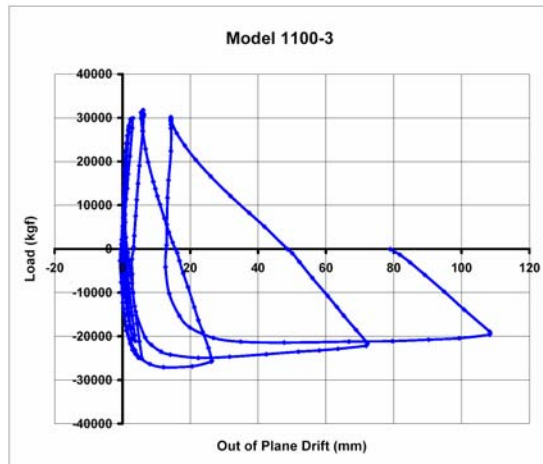
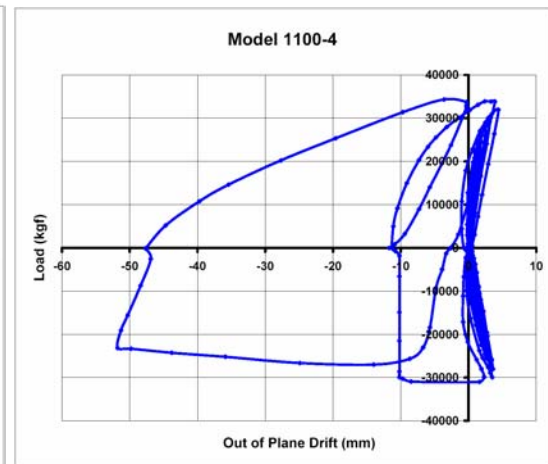Figure 6.                                              Figure 7.

To alleviate the problem of fragile fillet welds in specimen 1100-2, the length of reinforcing plates were reduced by 100 mm at each end in specimen 1100-3 so as direct contact between reinforcing and connection plates was deleted. In this specimen the local buckling of unreinforced part of brace preceded the general out of plane buckling of bracing. The buckling load is determined to be 25075 kgf which is almost %17 higher than that of basic 1100-1 specimen.

In an attempt to reach the optimum configuration of reinforcing plates, their length was increased in specimen 1100-4 so as the unreinforced length of brace at each end was reduced to 50 mm. In this case the buckling mode was the same as specimen 1100-3 but the buckling load was increased to 30000 kgf which is %41 higher than that of basic 1100-1 specimen.

*3.2 Elastic Stiffness*
Stiffness derived from in plane drift–load curve before tensile yield or before compressive buckling of bracing is studied. Refer to Figure 8 to Figure 12.

In specimen 1600, the tensile stiffness is Kt=1050 kgf/mm while compressive stiffness is computed as Kc=1450 kgf/mm. The ratio is $K_c/K_t$=1450/1050=1.4. For symmetric bracings such as X bracing , this ratio is equal to unity.If y-bracing is to have an acceptable distribution of base shear between tensile and compressive braces , then the ratio Kc/Kt should be between 0.7 to 1.43.[5]

In specimen 1100-1 which differs from 1600 in having its convergence point closer to corner of frame, Kt=500 kgf/mm and Kc=721 kgf/mm. The ratio is $K_c/K_t$=1.4.

Comparing specimens 1600 with l/d=0.75 and 1100-1 with l/d=0.54, it is deduced that stiffness has been halved by reducing l/d as much as %28.But the ratio between compressive and tensile stiffnesses has not changed which can be a clue to the basically same behavior of these two specimens.
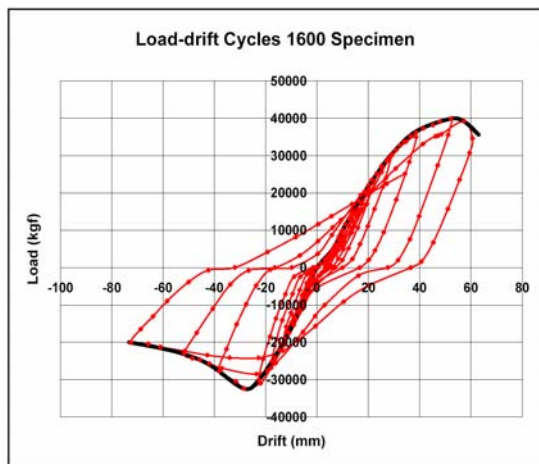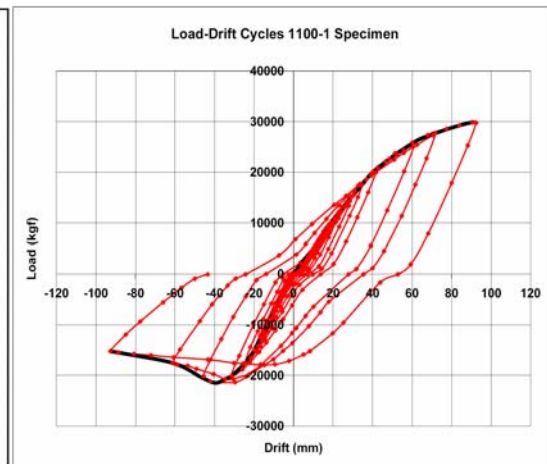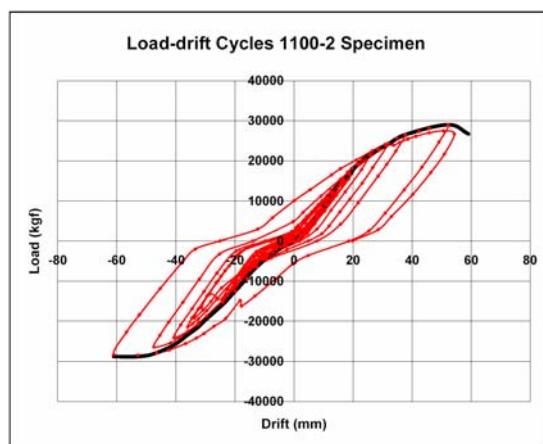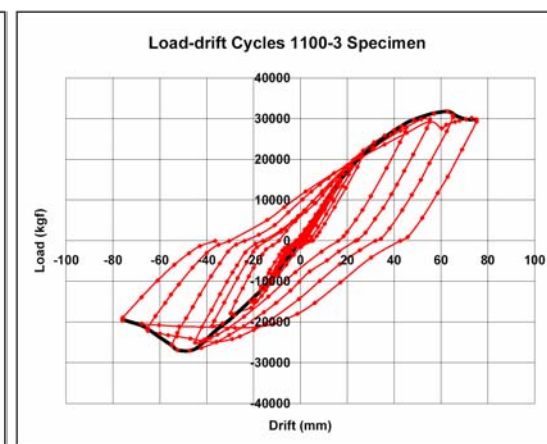
Figure 8.

Figure 9.



Figure10.

Figure11.
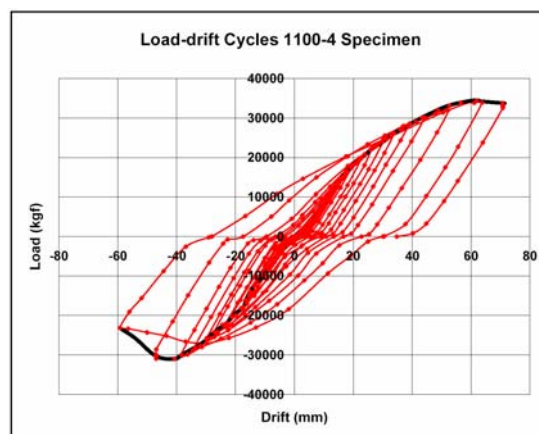


Figure 12.

In specimen 1100-2 the stiffness values are Kt=880 kgf/mm and Kc=712 kgf/mm. The ratio is $K_c/K_t$=0.81 which is a result of %76 increase in tensile stiffness while compressive stiffness has not changed in comparison with specimen 1100-1.

Specimen 1100-3 has Kt=880 kgf/mm and Kc=667 kgf/mm. The ratio is $K_c/K_t$=0.76.

For specimen 1100-4 stiffness values are Kt=733 kgf/mm and Kc=760 kgf/mm and their ratio is $K_c/K_t$=1.04.

Comparing different versions of specimen 1100 which all have the same geometry but differ in unreinforced length of longest brace, one can say that specimen 1100-4 shows the highest compressive stiffness and also the most favorable ratio of compressive to tensile stiffness of almost equal to unity.

*3.3 Post buckling stiffness*

Slope of skeleton curve or compressive stiffness after inelastic buckling is an indicator of severity of reduction in load bearing capacity. Post-buckling compressive stiffness generally is a negative value which is close to zero for structures which retain their load bearing capacity almost intact and have a stable behavior. Structures that sharply lose their load bearing , have lower post-buckling stiffness.So we can adopt the name "index of post-buckling stability" for this stiffness.

Starting with model 1600, post buckling stiffness Kp is equal to −490 kgf/mm just after buckling and increases to −110kgf/mm after two cycles.

In specimen 1100-1 Kp is equal to -190 kgf/mm just after buckling and increase to −75 kgf/mm after two cycles. In comparison with specimen 1600 stability of behavior has increased by moving the convergence point towards the corner of frame.

Specimen 1100-2 failed under premature tension and no buckling was evident. For specimen 1100-3 just after buckling Kp= −395 kgf/mm which increases to −240 kgf/mm after two cycles.

Finally, in specimen 1100-4 post buckling stiffness is −445kg/mm. Despite the higher buckling load of specimen 1100-4 its stability after buckling is less than specimen 1100-3.

*3.4 Damping*

Inelastic deformations under cyclic excitations results in hysteretic energy absorption. Damping obtained from hysteretic behavior of structure is proportional to the amount of energy absorbed in each cycle and also reversely proportional to the stiffness of structure. The relation used to compute the damping ratio is [8] :

$$D = \frac{W_d}{4\pi W_s}$$

In which $W_d$ is the area under one complete cycle of load-drift curve and $W_s$ is the work done by spring force during loading the structure to the maximum force in the same cycle. In the following table the damping ratio has been computed for tensile and compressive parts of load-drift cycles separately and also for the entire cycle.

From this table it can be deduced that:

1) Total damping ratio $\xi$ increases due to buckling between %17 to %98.

2) Compressive damping ratio $\xi_c$ increases between %36 to %128 due to buckling.

3) Tensile damping ratio $\xi_t$ may increase or decrease due to buckling. In two cases it has increased by %67 while in two other cases it has decreased by %10.

Table 1. Damping ratios before and after buckling

| specimen | cycle | Drift ratio $\Delta/h$ | Compressive $\xi_c$ | Tensile $\xi_t$ | Total $\xi$ | $\xi_c / \xi$ | $\xi_t / \xi$ |
|---|---|---|---|---|---|---|---|
| 1600 | Before buckling | 0.007 | %4.6 | %4.4 | %9.0 | 0.51 | 0.49 |
| | 1st cycle after buckling | 0.012 | %10.5 | %7.4 | %17.9 | 0.59 | 0.41 |
| | 2nd cycle after buckling | 0.017 | %14.9 | %8.9 | %23.8 | 0.63 | 0.37 |
| 1100-1 | Before buckling | 0.01 | %6.1 | %10.5 | %16.6 | 0.37 | 0.63 |
| | 1st cycle after buckling | 0.015 | %12.4 | %9.1 | %23.5 | 0.53 | 0.47 |
| | 2nd cycle after buckling | 0.02 | %16 | %11.6 | %27.6 | 0.58 | 0.42 |
| 1100-2 | No buckling Last cycle | 0.02 | %8.4 | %9.6 | %18 | 0.47 | 0.53 |
| 1100-3 | Before buckling | 0.0145 | %7.7 | %7.6 | %15 | 0.51 | 0.49 |
| | 1st cycle after buckling | 0.018 | %10.5 | %7 | %17.5 | 0.6 | 0.4 |
| | 2nd cycle after buckling | 0.021 | %15.1 | %9.2 | %24.3 | 0.62 | 0.38 |
| | 3rd cycle after buckling | 0.0245 | %17.6 | %10.5 | %28.1 | 0.63 | 0.37 |
| 1100-4 | Before buckling | 0.0145 | %6.5 | %5.4 | %12 | 0.54 | 0.46 |
| | 1st cycle after buckling | 0.017 | %11 | %9 | %20 | 0.55 | 0.45 |
| | 2nd cycle after buckling | 0.021 | %16.6 | %10.7 | %27.4 | 0.61 | 0.39 |

1) Compressive damping accounts for more than half of total post buckling damping.

2) By increasing drift in post buckling cycles, total damping ratio increases. Increase in compressive damping is generally higher than that of tensile damping.

3) Comparing specimens 1600 and 1100-1 it is observed that decreasing l/d ratio has resulted in higher damping ratio.

4) Strengthening and increasing radius of gyration of longest brace in specimens 1100-3 and 1100-4 has no meaningful effect on damping of basic 1100-1 specimen.

### 3.5 Bare Frame Characteristics

While every attempt was made to reduce the fixity of beam to column connections, some moment transfer is possible due to presence of double gusset plates.To asses the participation of frame in total results presented so far , the cyclic loading was conducted on bare frame of above specimens.Load-drift cycles are shown in Figure 13 below. Regarding in-plane horizontal displacemens, three cycles of frame which correspond to buckling , 1st and 2nd post-buckling cycles of complete braced frame are extracted and shown in Figures 14 to 16 , respectively.
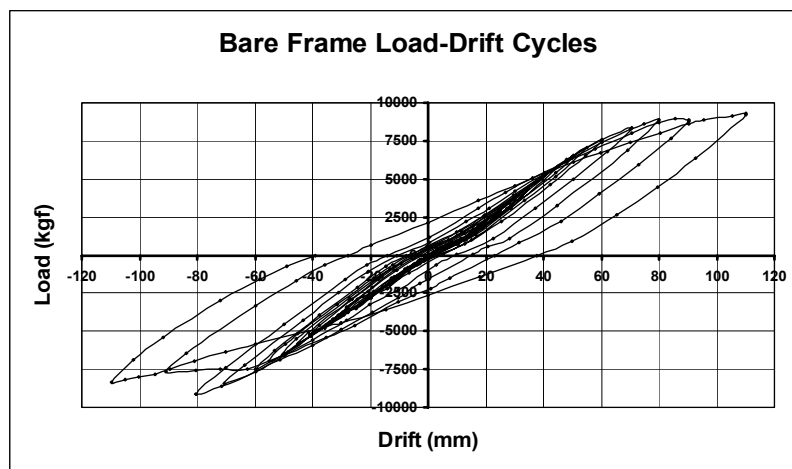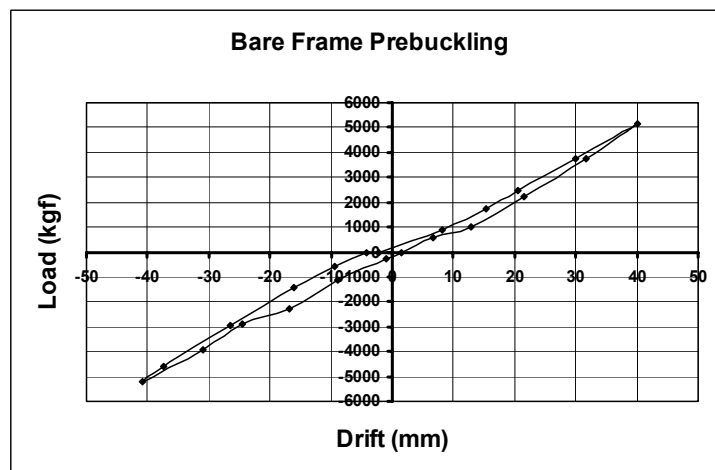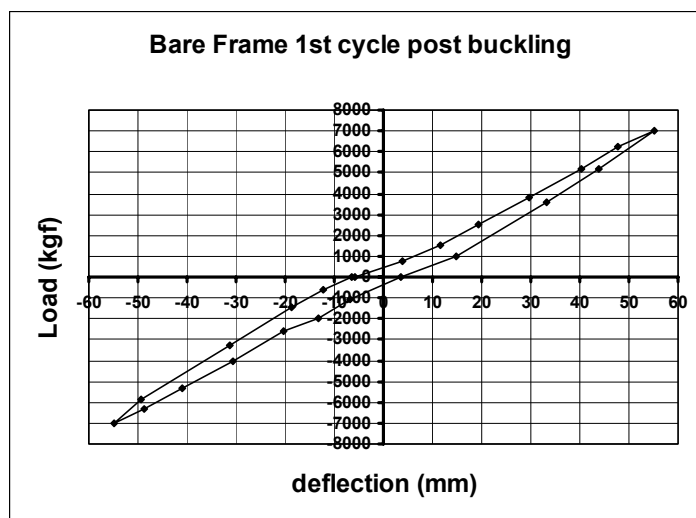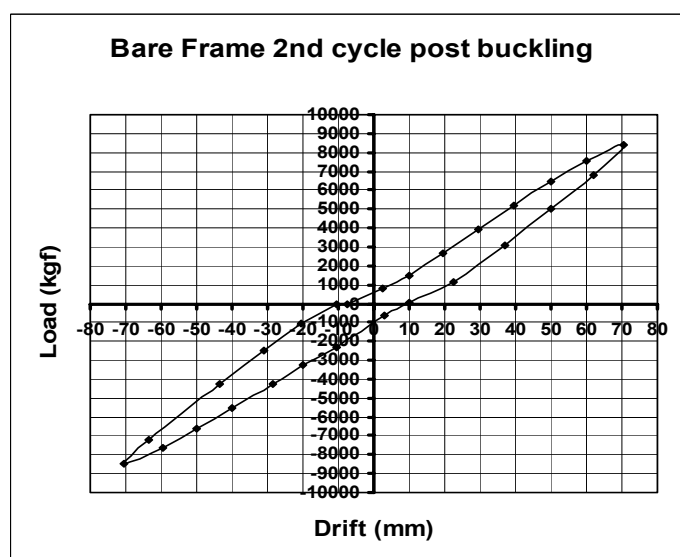


Figure 13.



Figure 14.

Figure 15.



Figure 16.

From the above cycles it's deduced that lateral load at buckling is 5000 kgf which is almost %17 of buckling load of braced frames except for specimen 1100-1 which the ratio is %23.

Stiffness of frame in linear phase is 127 kgf/mm which should be compared to results of section (2).Frame stiffness ranges between %10 to %21 of average elastic stiffness of braced frames.The minimum ratio is for specimen 1600 while the maximum value is for specimen 1100-1.

Damping ratio $\xi$ is computed to be %2 , %3 and %4.5 for Figure 14 to 16 respectively.Comparing these values with Table (1) it is seen that frame damping constitutes between %12 to %22 of braced framed pre-buckling damping.The maximum frame effect is in specimen 1600 while specimen 1100-1 is least affected.

After buckling the frame damping ranges between %16 to %19 of total damping in 2nd cycle.For 1st cycle this is between %13 for specimen 1100-1 to %17 for specimen 1600.

From above comparisons one may say that subtracting frame participation from total braced frame behavior always leads to reduction in the buckling load , stiffness and damping . Generally,the amount of this reduction is between %10 to %20 .


# 4. CONCLUSIONS

After conducting cyclic static racking load tests on five samples of concentric y-shaped three member bracings with two different geometries and three strengthening details it was concluded that:

1)  The buckling load increases by moving the convergence point towards the center of bay.
2)  Strengthening the longest brace member against out of plane buckling sharply increases buckling load.
3)  Elastic stiffness of bracing is higher when convergence point is nearer to center of bay.
4)  Elastic stiffness of bracing increases by strengthening the longest brace member.
5)  Stability of behavior that is assessed by the slope change of skeleton curve due to buckling is higher for convergence point nearer to frame corner.
6)  Strengthening the longest brace member tends to decrease post buckling stability of behavior.
7)  Moving the convergence point towards the corner of frame increases damping ratio.
8)  Buckling itself and increasing post buckling drift increase the damping ratio.
9)  While it's not common to consider the effect of non-moment frames on total braced frame behavior, but even by subtracting the frame participation , the results (1) to (8) above are still true.

For further research more specimens with different frame geometries should be tested. Different structural sections should be considered as bracings and frame members. Due to importance of convergence point in y-bracing behavior , other locations for it should be checked.


# REFERENCES

1.  Taranath, S., *Steel, Concrete and Composite Design of tall Buildings* , Secend Edition, McGraw Hill, New York, (1998) 423-428.
2.  Sontag, S., Henn, W., and Hart, F., *Multi Storey Buildings in Steel*, Secend Edition, Collins, London (1985) 252-257.
3.  Yasery, S., Shasb, F.R., *Design and Calculate of Heavy and Light Industrial Steel*

*Buildings*, In Persian, Dehkhoda Press, Tehran, (2001).

4.  Eurocode 8, *Design of Structures for Earthquake Resistance*, Part 3, BSI London December 2001.

5.  Building and Housing Research Center, *Iranian Code of Practice for Seismic Resistant Design of Buildings*, Standard No. 2800, 2nd. Edition, (1999).

6.  Badpar, A., *Investigation of Behavior of y-shaped Bracing*, M.Sc. Thesis, In Persian, Tehran University, (1996).

7.  Moghaddam, H.A. and Estekanchi, H.E., *Seismic Behavior of Off-centre Bracing Systems*, Journal of Constructional Steel Research , No.2, **51**(1999) 177-196.

8.  Clough,R.W. & Penzien,J.,*Dynamics of Structures*, Mc-Graw-Hill, Chapter 4, (1975) pp 75.



Image 1. Set up of tests and buckling of longest member of bracing

Image 2. Plastic hinge due to buckling in model 1600



a) Fillet weld fracture in Model 1100-2

b) Buckling of flange lip in Model 1100-3    c) Local buckling of flange in Model 1100-4

Image 3. Performance of reinforced member in three different conditions of reinforcing in models 1100

Image 4. Bare Frame under lateral cyclic loading