

## Behaviour of Cold-Formed Steel Beams Under Cyclic Load Reversal

Anbuchejian .A <sup>1</sup>, Dr. Baskar.G <sup>2</sup>

<sup>1</sup> Research Scholar in Civil Engineering, Vinayaka Mission's Research Foundation, Salem, India

<sup>2</sup> Associate Professor in Civil Engineering, Institute of Road and Transport Technology, Erode, India

---

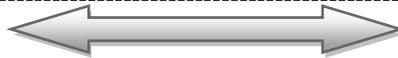
### Abstract

The Cyclic load deformation of inelastic structural members is an important factor in investigating the dynamic response of the members against repeated loading ex. Earthquake and wave motion frequently; it is directly related to the collapse of overall structures under major dynamic motions. Totally 6 No's of cold-formed plain angle beams and 6 No's of cold-formed lipped angle beams are tested under cyclic loading. The hysteresis loops for the tested beams are obtained and the moment carrying capacity, local buckling behaviors are studied. The deformation ductility factors and stiffness factor are calculated. It is observed that the hysteresis behavior depends on the geometry of section. Provision of lips increases moment carrying capacity but reduces the ductility factor. Cold-formed steel structural elements good ductility behaviors and may be effectively used to resist seismic forces.

**Keywords:** Hysteresis behavior, Moment carrying capacity, Cold-formed steel, Ductility,

---

Date of Submission: 10 April 2013



Date Of Publication: 25, April.2013

### I. INTRODUCTION

In spite of the great advances that have been made in understanding and characterizing the load reversal behavior of materials and structures, load reversal are still major causes of in service failure, both in components and in structures. It is still not widely recognized in the Engineering community that load reversal failures are far more common in welded steel and concrete structures. These problems are extremely critical in terms of safety and economy. The range of structures and components, which have been or could be, affected by load reversal is large and diverse. In effect, anything that is subjected to repeated loading, arising for example, from waves, wind, live loading, pressure and temperature fluctuations, vibrations etc., is potentially at risk. Among the major structures, which have been acknowledge as having load reversal, problems are bridges offshore platforms, ships and rails of railway tracks. Building, bridges and other Civil Engineering Structures must resist hundreds of loading cycle's causes by strong earthquakes, while major damage can occur with as few as thirty cycles of large deformations. Past studies using experiments on steel structural members including columns, beams and braces, commonly report that global buckling of steel members can easily trigger local buckling of these plate elements.

The local buckling cause concentrate large plastic deformations, which within five to twenty loading cycles, induces cracking that eventually leads to rupture of members. Also, it has been observed that seismic loading of steel members has caused ruptures at locally buckled locations. Some of the earlier researchers have classified this type of failure caused by crack propagation without macroscopic deformations. During a severe earthquake, structures behave inelastically, and the relationship between the applied forces and the resulting displacements becomes nonlinear. Such behavior is best described with the aid of force displacement hysteretic diagram for cyclic loadings. Many diagrams of this kind have been generated in various laboratories throughout the world, the evaluation of these results as they pertain to seismic design of structures is an important task faced by engineers.

In studying the behavior of structural framing in either material, the extent to, or the manner in which the inelastic action can take place is particularly important. The ability of a structure to sustain its load carrying capacity into the inelastic range is very important in seismic design. The extent of this ability is identified with ductility, which may be either that of the material itself, of a member or of a structure. In experiments the behavior of structure can be reasonably well simulated by a sub assemblage. A clear distinction among these kinds of ductility's is essential.

## II. OBJECTIVE OF INVESTIGATION

The aim of this investigation is to study the behavior of cold formed double angle beams with and without lips subjected to load cyclic reversal. Totally twelve experiments were conducted with two bolted end conditions and central point loading. Figure 1 shows the details of experiments.

The following effects are studied.

- [1] The effect of lip and the ultimate moment carrying capacity on load reversal.
- [2] The hysteresis behavior
- [3] The effect of flat width to thickness ratio.
- [4] The reduction in stiffness with respect to number of cycles.
- [5] The ductility of the material.

### 2.1 Ductility

Usually building are designed on an elastic basis for code specified force which are smaller than the actual ones likely to occur during a major earthquake, if the structure were to remain linearly elastic. Since, however, the elastic capacity is limited, inelastic behavior in the critically stressed elements of a structure can be anticipated. For a structure to survive, it must behave in a ductile manner, and for analysis, ductility must be quantified. In this regard it is important to differentiate between at least three types of ductility's. One of these references to the ductility of the material itself. It can be defined as the absolute maximum strain ( $\epsilon_m$ ) at maximum stress to the strain ( $\epsilon_y$ ) at yield.

Material ductility factor  $\mu_e = \epsilon_m / \epsilon_y$  (1)

Secondly the behavior of individual members their rotation ductility factor  $\mu_\theta$  is an important parameter. It is defined as an analogous manner, the ration between the member's ultimate or maximum rotation capacity  $\theta_m$  to be rotation  $\theta_y$  at yield.

$\mu_\theta = \theta_m / \theta_y$  (2)

Lastly, the displacement ductility factor  $\mu_\delta$  which is related to the displacements of a frame or a sub assemblage has particular relevance in seismic design. Again, formally  $\mu_\delta$  is defined as the ration of the ultimate (or) maximum deflection  $\delta_m$  of a structure at a selected story to the deflection  $\delta_y$  at the same point at yield.

$\mu_\delta = \delta_m / \delta_y$  (3)

For cyclic loading  $\delta_m$ , just as  $\epsilon_m$  and  $\theta_m$ , will be taken as the total inelastic excursion during a half cycle. In many instances the determination of  $\delta_y$  cannot be made precisely, and may vary among the different investigators. One can call  $\delta_y$ , the beginning of yield at any one point of a structure. Alternatively, as second definition of  $\delta_y$  One can point of a structure and note the significant deviation from linearly elastic behavior of the whole system to define  $\delta_y$  Still another possible approach for defining the value of  $\delta_y$  consists finding the intersection of the asymptotes to the elastic and plastic ranges; If either one of the latter two approaches is used for defining  $\mu_\delta$  the ductility ratios may be considerably smaller than that which would result when using the first definition of  $\delta_y$  The ductility factors used in this project conform with the second definition of  $\delta_y$

### 2.2 Significance of Hysteresis Loops.

The intensity of earthquake for which a structure needs to be designed is highly uncertain. The effect of earthquake on a building is strongly dependent not only on the ground motion, but also on the mechanical properties of the structural elements. Experimental data in the form of hysteresis loops provide direct evidence in the material behavior. The area enclosed by the loops is a measure of energy dissipation capacity of a structure, which plays a dominant role in damping its vibration. Also much is learned from the experiments regarding the correct

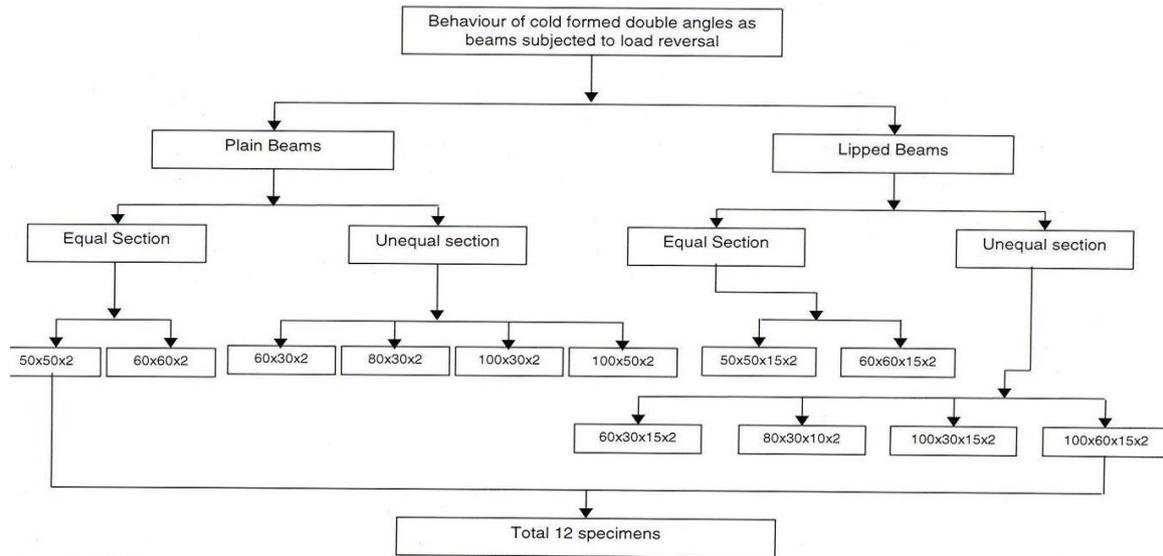


Figure 1: Details of specimens Tested

detailing of members and connections for resisting cyclic loading. The hysteresis curves also provide required information on the load- deformation paths of the members for analysis. With the aid of a computer this will enable one to make calculations on the dynamic response of a structure to a prescribed earthquake using the deformation spectra.

### III. FABRICATION OF TEST – RIG

In the conventional UTM's and the Loading Frame available in the laboratory, only unidirectional loading can be applied. Hence to facilitate the application of reversal loading a separate test-rig is fabricated for the present study. Two simple supports were made using I section of one meter height as Columns. These two I sections are welded centrally at bottom with the base plate of size 600mm x 600 mm. Then 2 nos of separate 600mm x 600mm of earth pit up to a depth of 600 mm was dug out to provide the foundation bed for the columns at a distance of 3.0 mts apart. While pouring 1:2:4 concrete for foundation bed 4 nos of 25 mm bolt was embedded into the foundation concrete itself for each foundation unit. Then the I section columns along with base plates were fitted above the foundation bed using 4 No's of 25 mm bolt and nuts at a centre to centre distance of 3.0 mts. The I section columns were placed such a manner that to face the flanges on opposite side. In order to adjust the vertical distance of the specimen for loading and unloading, 5 nos of 16mm bolt holes are provided on flanges of both I sections at a pitch of 30mm apart. To connect the specimens separate gusset plates of 5 mm thickness were attached to the flange to the column with an end plate. In that gusset plate 12 mm bolt holes are provided to fix the specimen. The schematic diagram of entire test setup with specimen is shown in figure 2 & 3.

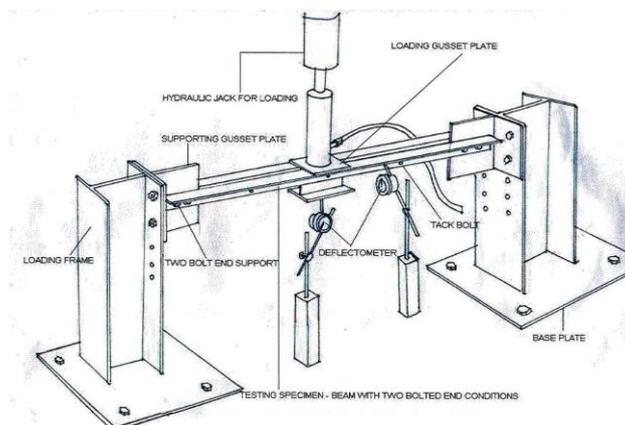


Figure 2: Schematic diagram Loading Setup-Downward loading

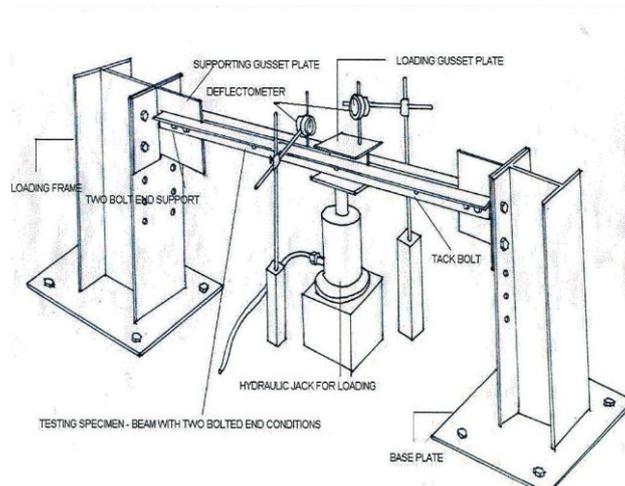


Figure 3: Schematic diagram Loading Setup-Upward loading

### 3.1 Test Specimen Details

All the specimens are fabricated by cutting and bending of cold rolled steel sheets of St-32-1079 grade to a required size. Six numbers of double angles without lips and set of six double angles with lips connected back to back were used as test specimens. Lipped angles are stiffened by a lip bent at right angles on both legs. The overall depth of such lips has been provided in accordance with IS 801-1975. 2 Nos of 12 mm diameter holes were drilled at the ends for the connection. In addition to the end bolts, 8 mm tack bolts were provided at every  $1/4^{\text{th}}$  length of the specimen. Length of the specimen used was 2.5mts. At centre of the specimen, i.e the point at which load was applied a separate gusset plate was provided in order to transmit the load to the specimen.

### 3.2 Test Set Up

All the tests were conducted in the loading frame fabricated as described. The loading frame was firmly bolted with 25 mm bolts above the foundation bed. The end gusset plates were firmly fixed on the loading frame with 2 nos of 16 mm bolt at both the ends. The specimens were firmly fixed at end supports by using 2 nos of 12mm bolts at each end. At centre the loading plate was fitted with the specimen to transmit the load from the jack to the specimen using 8mm bolts.

### 3.3 Testing Procedure

The specimen of double angle back to back was held in position by means of 2 nos of 12mm bolts at either end. Two deflectometers were fixed one at bottom of the loading gusset plate at centre and another at flange of the angle section, which is very near to the loading point. The deflectometer fitted at bottom of the loading gusset plate is used to measure the deflection of the specimen under which where the load is applied. Loading of specimen was done by 50 KN capacity hydraulic jack. Least count of the hydraulic jack is 0.2 KN and least count of the deflectometers is 0.01mm. For the first half cycle loading the jack was placed at the top of the specimen. Loading was applied to the specimen gradually by means of placing the jack at centre of the loading gusset plate which was fitted at exact center of the testing specimen. Loading was gradually applied and the corresponding deflections were noted from the two deflectometers. For each and every increment of loading was given at a rate of 0.2 KN. The loading was applied till the specimen continued to deflect even without applying the small increment of load. Then the dials of deflectometers were readjusted to Zero. Then the unloading was done and also corresponding deflections were noted. At the time of unloading the specimen is coming back to its original position because of elastic nature.

For apply the reversal loading the jack position was changed from top to bottom. Then the jack is placed at the bottom of the specimen exactly at centre point. The deflectometers are fitted such that one at exact centre of the loading gusset plates which was directly under the loading point also and another at flange of the specimen. The loading and unloading of the specimen was applied and corresponding deflections are noted. This completes one full cycle of loading. The sequence of operation as mentioned above was repeated. Totally four cycles of loading were applied on each specimen.

#### IV. RESULTS AND DISCUSSIONS

Totally twelve experiments were conducted on cold formed double angles as beams with and without lips for two bolted end conditions. Four reversed static cyclic loading was applied on each specimen.

##### 4.1 Moment Carrying Capacity

Table 1 presents a comparison of moment carrying capacity of all the specimens tested for the first half cyclic loading. The observed moment is calculated by the formula  $M = PL/6$  assuming partially fixed end condition.

Where  $M$  = Observed Moment Carrying Capacity in KN.m.

$P$  = Ultimate load during first half cycle of loading in KN.

$L$  = Effective span of the beams in meters.

The observed moment is calculated for partially fixed condition because of two bolted end condition. The predicted moment carrying capacity ( $M$ ) is calculated by multiplying the plastic section modulus ( $Z_p$ ) of the section with the yield stress ( $f_y$ ) of the material used. While comparing the observed moment with predicted moment, the observed moment is on higher side which vary from 6% to 25% except for the first specimen. For this specimen EASI (50x50x2) tested as first specimen, the difference in moment carrying capacity is 30%. This may be due to the fact that the end bolts were not fully tightened.

Table 1: Comparison of Ultimate moment Carrying Capacity

SI No	Specimen Name	Ultimate Moment		% Difference
		Observed $M_o = P_o L/6$ kN.m	Predicated $M_p = Z_p \cdot f_y$ kN m	
1	EAS1	1.66	1.12	32.81
2	EAS2	1.83	1.46	20.05
3	UEAS1	1.83	1.44	21.31
4	UEAS2	3.08	2.56	16.88
5	UEAS3	3.54	4.02	11.94
6	UEAS4	4.37	4.07	6.86
7	EALS1	1.83	1.61	12.02
8	EALS2	2.92	2.17	25.41
9	UEALS1	---	---	---
10	UEALS2	3.75	3.10	17.33
11	UEALS3	4.79	5.16	7.17
12	UEALS4	5.83	5.21	10.63

##### 4.2 Behavior of Hysteresis Loops

The load versus deformation hysteresis loops for all the tested specimens were drawn by using the readings taken from the experiments as shown in figure 4 & 5 separately for each specimen. In general, it is observed from the hysteresis loops the ultimate load carrying capacity increases with the increase in sectional area of the beams. And also if the beams are stiffened by means of lips at ends, the load carrying capacity is further increasing when comparing plain beams of same section.

When comparing the hysteresis loops of the specimens EASI and EALS1, i.e., the plain beam and lipped beam of size 50x50x2 and 50x50x15x2, it is seen that the load carrying capacity of lipped beam is increased by 10%. It seems that the lipped beam EALS1 shows a stable behavior. Since the flanges are stiffened. The load carrying capacity for each cycle is almost same for all cycles and also the loop is forming very close to cycle to cycle. The load carrying capacity of the lipped beam EALS2 is nearly 60% more than the plain beam of EAS2. In the plain beam EAS2 the load carrying capacity if nearly 50% reduced in the plain beam EAS2, the load carrying capacity if nearly 50% reduced in the second, third and fourth cycle compared with first cycle, but the deformation is increased from the second to fourth cycle for same ultimate loads. For the lipped beam EALS2, load carrying capacity is almost same for all the cycle because of stiffness but the deformation is increased from cycle to cycle. From the hysteresis loops of UEAS2 and UEALS2, the load carrying capacity if 22% more for the lipped beam than the plain beam. For the plain beam UEAS2. The load carrying capacity is reduced to nearly 35% from the first cycle to the second, third and fourth cycle. But the deformation has not changed. This exhibits un-stable behavior. For the lipped beam UEALS2, the load carrying capacity is almost same for all the cycle, but the deformation is increased for every cycle and also the hysteresis loops have become stable. While comparing the hysteresis loops of UEAS 3 and UEALS 3 it is observed that the load carrying capacity of the lipped beam is 35% more than that of the plain beam. 30% of load is decreased from

the first cycle to the Second, third and fourth cycles for the lipped beam. The hysteresis loops are stable in the cycles for lipped beam and also the load carrying capacity remains unchanged.

From the hysteresis loops of the specimens the load carrying capacity is 33% more for the specimen and UEALS4 than the UEAS4, for the specimen UEAS4, the load carrying capacity is linearly reduced by cycle to cycle/ But for the specimen UEALS4 the load carrying capacity and the deformation is almost same for all the cycles and also the loops are stable because of the stiffness offered by the lips. From the hysteresis loops of the specimen 3 the load carrying capacity is reduced from cycle to cycle hysteresis loops are formed very close to each other.

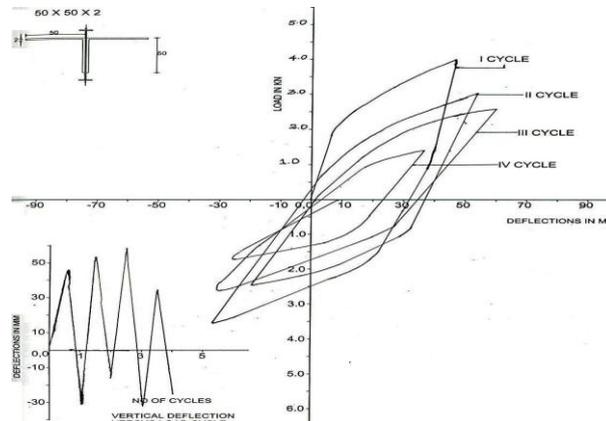


Figure 4: Hysteresis Curve for 50 X 50 X 2 with Two Bolted End Conditions

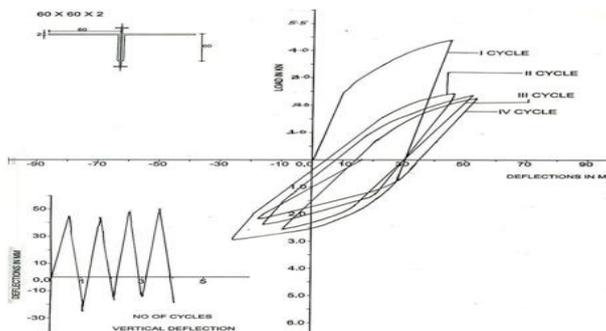


Figure 5: Hysteresis Curve for 60 X 60 X 2 with Two Bolted End Conditions

It is also to note that the hysteresis loops shifts towards the positive deflection direction in the case of the equal plain angle beams. But in the case un-equal plain angle beams, the hysteresis loops have a tendency to shift towards the negative deflection direction for the entire lipped angle tested, the hysteresis loops are stable and symmetrical. This indicates that the hysteresis behaviour depends on the shape and symmetric of the beam cross section.

#### 4.3 LOCAL BUCKLING BEHAVIOUR

Some of the local buckling occurred while conducting the experiments for different sections. While comparing the behavior of plain beams with respect to lipped beams, the local buckling has more pronounced in plain beams. The buckled plain beams in which buckling occurs very near to the load point. In lipped beams the effect of buckling is marginal at higher load. This is because of the beams are stiffened by the way of lips provided at both flanges. Moreover during testing it was observed that the lipped beams recovered from the effect of local buckling and almost (80%) returned back to original position while unloading.

#### 4.4 Ultimate Moment Carrying Capacity

The variation of ultimate moment carrying capacity with respect to each cycle for all the tested specimens. The moment carrying capacity is reduced from cycle to cycle. The moment carrying capacity of the lipped beam is more than the plain beam. Figure 5 shows that both equal and unequal plain angle, unequal lipped beams, and the moment carrying capacity is gradually reduced from cycle to cycle. For the lipped equal angle beams the moment carrying capacity is almost same for the cycles.

#### 4.5 Ductility

Ductility is the important factor for selection material for the construction industry. Cold-formed steel is having good ductility behavior comparing to other building. The ductility factors calculated for the tested specimens are shown in the Table 2. It is seen that the ductility factor of the plain beams is more than the lipped beams because the lipped beam are stiffened by the lips. The average deformation ductility factor for the plain beams is observed to be 5.4 and for the lipped beams it is found to be 3.4

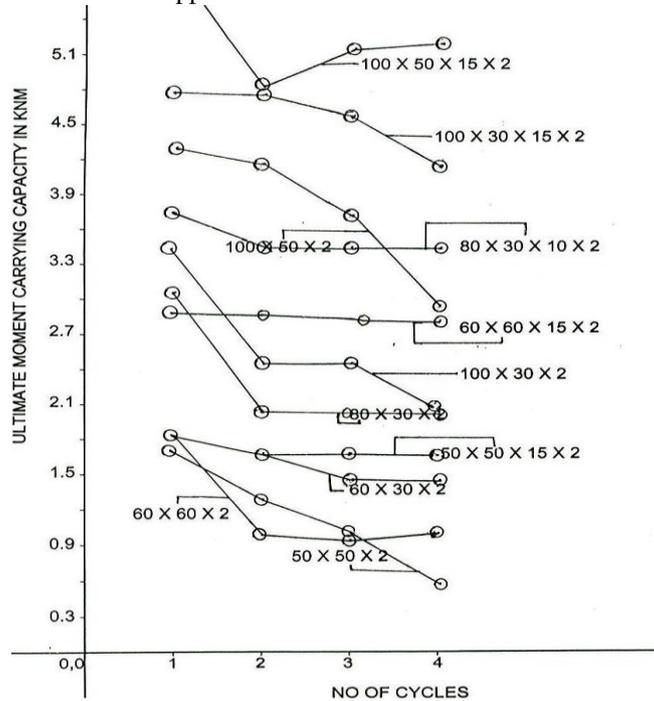


Figure 6: Ultimate Moment Vs Load Cycle for Two Bolted End Conditions

Table 2: Comparison of Ductility Factors

Sl no	Specimen	Ultimate load ( $P_u$ ) kN	Yield load ( $P_y$ ) kN	Max deflection ( $\delta_m$ ) mm	Deflection at the point of yield ( $\delta_y$ ) mm	Ductility $\mu_\delta = \delta_m / \delta_y$
1	EAS1	4	2	47	6.5	7.2
2	EAS2	4.4	2.5	45	9.5	4.7
3	UEAS1	4.4	1.5	37	4	9.2
4	UEAS2	7.4	4.5	39	12.5	3.1
5	UEAS3	8.5	5.0	32	9	3.6
6	UEAS4	10.5	5.0	33	7	4.7
7	EALS1	4.4	2.0	34	11	3.1
8	EALS2	7.0	4.5	46	16.5	2.8
9	UEALS1	----	----	----	----	----
10	UEALS2	9.0	5.0	29	7	4.1
11	UEALS3	11.5	6.0	28	7.5	3.7
12	UEALS4	14.0	7.0	38	11	3.5

#### 4.6 Stiffness Factor

The stiffness factor for all the specimens were calculated from the hysteresis loops. The figure 6 shows the variation of stiffness factor versus number of cycles. As expected the lipped beams are having larger values of 'k' than the plain beam. Stiffness factor reduced from 1 to between 0.3 to 0.46 from first cycle to the second cycle respectively for the plain beams. Stiffness factor reduced from 1 to between 0.63 to 0.77 from first cycle to the second cycle in the case of the lipped beams.

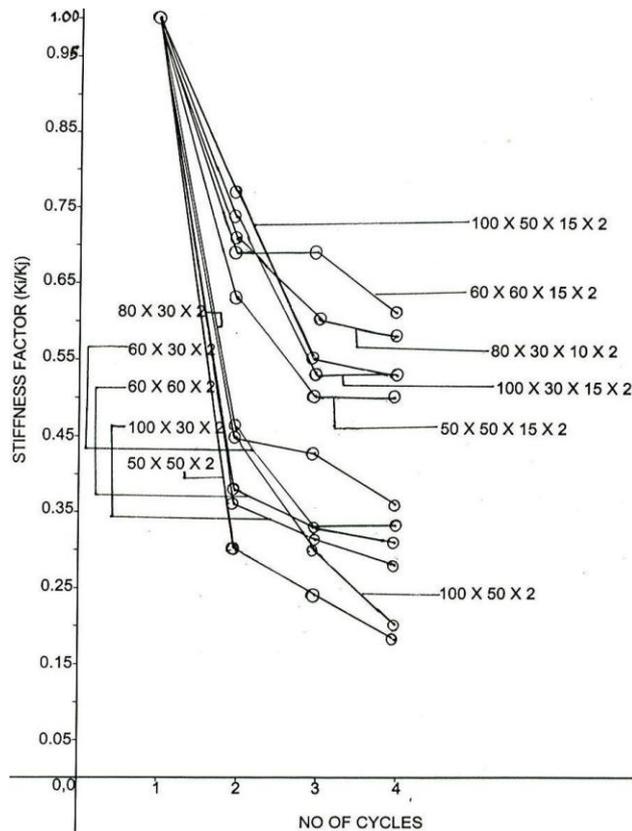


Figure 7: Stiffness Factor Vs Load Cycle for Two Bolted End Conditions

## V. SUMMARY AND CONCLUSION

The study of the behavior of cold formed double angle beams with and without lips subjected to cycles load double reversal is carried out with the help of 12 angle specimens. The experiments are carried out on a separate testing fabricated for this purpose. The test specimens were as per specifications by cutting and bending of plain cold formed sheets and tested under revised cyclic loads. From the experiments results the hysteresis loops were obtained and presented the hysteresis behavior moment carrying capacity, local buckling ductility behavior, stiffness factor are studies the beams are calculated and presented.

The following conclusions are arrived at from the present experimental study.

- [1] The hysteresis behavior of cold-formed steel angle beams depends on the shape and symmetric of the beam cross section.
- [2] Assuming a partial fixity (50%) for two bolted and condition the predicted moment capacity was always less than the observed value. The under estimation of the moment by the theory was within 25%.
- [3] Provision of lip makes the hysteresis loops stable for all the specimens.
- [4] Moment carrying capacity is more in case of lipped angles than the plain angles.
- [5] 5. In case of plain angles, the reduction in the ultimate moment carrying capacity is gradual from first to four cycles the total reduction being between 35 to 40%.
- [6] 6. For cold-formed plain angles sections the reduction in the stiffness factor for the first two cycles is between 65% to 70% and for lipped sections it varies between 23% to 37%
- [7] 7. The deformation ductility factor is more for the plain beams than that of the lipped beams. The higher value of ductility factor coil with stand indicated that cold-formed steel elements seismic forces more effectively than other materials. The local buckling occur under the point of loading itself. Where as it is occurring for the entire beam as single unit is case of lipped beams with more load carrying capacity. This buckling effect is returning back to original position almost 80% by unloading.
- [8] 8. In the case of plain angle beams the flat width to thickness ratio is greater than the limiting value and they are subjected to local buckling.
- [9] 9. For lipped beams the effect of local buckling is only marginal.
- [10] 10. To avoid lateral buckling of beams tack bolts should be provided.

### REFERENCES

- [1] Ashok K.Jain, SubhashC.Goel, Robert D.Hanson, "Hysteretic cycles of axially loaded steel members", J1 of structural division, ASCE, Vol.106, No.8, Aug.1980, pp 1977-1795.
- [2] Brue F.Maison and EgorP.Popov, "Cyclic response prediction for braced steel frames", J1 of Structural Division, ASCE, Vol.106, No.7, July, 1980, pp.1401-1416. Code of practice for the use of cold formed light gauge steel structural members in general building construction IS 801-1975, Bureau of Indian Standards, New Delhi.
- [3] EgorP.Popov, "Seismic behavior of structural sub assemblages", Journal of the Structural Division. ASCE/Vol.106, No.7, July Construct. Steel Research Vol.30, 1994, pp.107-124.
- [4] Indian Standard Specification for cold formed light Gauge structural steel sections IS-811-1087, Bureau of Indian standards.
- [5] Kandasamy S."Cycle Load reversal of cold formed Steel Beams – Elements – Val I (1996) – M.E., Thesis Department of Civil Engineering, Anna University, and Chennai 1996.
- [6] Kandasamy.S. "Fatigue and load Reversal Behavior of cold formed steel structural Elements" Ph.D Thesis, Department of Civil Engineering, Anna University, Chennai August 2000.
- [7] Satish Kumar and Tsutomu Usami, "Damage evaluation in steel box columns by cyclic loading tests", J1 of Structural Engineering, ASCE, Vol.122, No.6, June 1996, pp.626-634.
- [8] Tsutomu Usami and Satish Kumar. "Damage evaluation in steel box columns by pseudodynamic tests", J1 of Structural Engineering ASCE Vol.122, No.6, June 1996, pp.635-642.
- [9] Wei-Wen Yu, "Cold formed steel structures", McGraw Hill Book Company, New York, 1973.
- [10] Yeon-Soo Park, satoishiwi, Hiroyuki Kameda and TaijiroNonaka, "Very low cycle failure process of steel angle members", J1. Of Structural engineering, ASCE, Vol.122, No.2, Feb, 1996, pp.1330141.