Study of Cold-Formed Steel Flexural Member under Monotonic and Reversed Cyclic Loading

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Abstract - *The purpose of this paper is to provide details about the strength and behaviour of cold-formed steel build-up beams under monotonic and cyclic loading. The beams are formed by connecting two or more open and closed section by using various fastening method. The tests for build-up beam were carried out under hinged end condition, fixed end conditions and cantilever end conditions were presented. Various failure modes of build-up beams were observed during the tests are mentioned. The strengths obtained from the experiment were validated with strength obtained from finite element analysis for the beams were reviewed.*

Key Words: *Beam, Cold-Formed steel, Cyclic loading, Experimental analysis, Finite element analysis and Monotonic loading.*

1. INTRODUCTION

Steel products are extensively used in building industries, such as bridges, roof trusses, and transmission line towers, multi-storied buildings, space structure etc., because of its high strength, resulting in the reduction of dead weight. The manufacturing process involves to forming the material by either press-braking or cold roll forming to achieve the desired shape. The uses of these cold formed structural elements in seismic zones are slowly increasing. To understanding and characterizing the load reversal behaviour of materials and structures are still major causes of in-service failure (both in components and in structures). The deformation under the cyclic load in inelastic structural members is an important factor in investigating the dynamic response of the members against repeated loading eg. Earthquake and wave motion frequently and it is directly related to the collapse of the overall structure under major dynamic motions. The purpose of this paper is to briefly describe the experimental and numerical investigations of cold-formed build-up beam made of open and closed sections under monotonic and cyclic loading.

2. MONOTONIC AND CYCLIC LOADING

Anbuchzian and Baskar studied the behaviour of cold-formed double angle beams with and without lips subjected to cyclic load reversal. Double angles without lips and double angles with lips connected back to back were used as test specimens [1]. These lipped angles are stiffened by a lip bend at right angles on both legs and over all depth of lips has been provided based on IS 801-1975. Two numbers of 12mm diameter bolts were drilled at the end for connecting to the frame. Additionally 8mm tack bolts were provided at every one-fourth length of the specimen. Total length of the specimen was 2500mm. The gusset plates are provided at the centre of the beam in order to transmit the load to the specimen. Totally twelve specimens were used for experimental tests. The experiments were conducted with two bolted end conditions with central point load. Effects of lip and ultimate moment carrying capacity on cyclic load reversal, hysteresis behaviour, the effect of flat width to thickness ratio, reduction in stiffness with respect to a number of cycles and ductility of failure were studied. Two deflectometer were used to measure the deflections. Deflectometer was fixed one at bottom of loading gusset plate and another at the flanges of the angle section. The jack having 50kN capacity was used to apply the load. For the first half of the cycle, loading jack was placed at the top of the specimen shown in fig-1. The load was gradually applied and for each and every increment of loading was given at a rate of 0.2 kN. The load was applied till the specimen continued to deflect up to without a small increment in load.

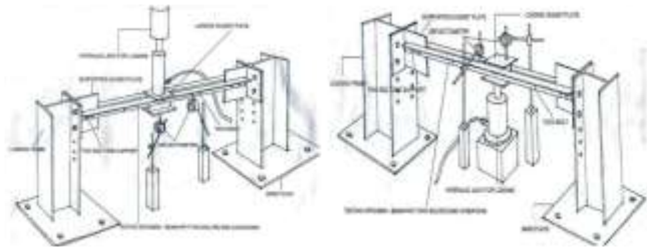

Fig -1: Loading Setup for Upward and Download Loading

While comparing the observed moment with a predicted moment, the observed moment is 6% to 25% higher than the predicted moment except for the first specimen. Similarly while comparing the hysteresis loop of the plain beam and lipped beam of size 50X50X2 and 50X50X15X2, the load carrying capacity of a lipped beam is increased by 10%. It shows the stable behavior of the beam because the flanges were stiffened. In the case of cyclic analysis, the load carrying capacity of each cycle is similar for all cycles and also the loop has been formed very close to cycle-to-cycle. The load carrying capacity of a lipped beam was 60% more than the load carrying capacity of a plain beam. In a cyclic analysis, the load carrying capacity of plain beam was nearly reduced by 50% in the second, third and fourth cycle compared with the first cycle, but the deformation was increased from the second to fourth cycle for the same ultimate load. For lipped beam, the load carrying capacity almost same for all the cycle because of the stiffness but the deformations were increased from cycle to cycle.

In the plain beam, the local buckling was occurred very near to loading point. In case of a lipped beam, buckling occurred at higher load because the beams were stiffened by lip also provided at both flanges. It was observed that the lipped beams were recovered 80% from the effect of local buckling and almost returns back to the original position while unloading the beam.

The variation of ultimate moment carrying capacity with respect to each cycle for tested specimens were evaluated and noted the moment carrying capacity of the beam was reduced cycle to cycle. The moment carrying capacity of lipped beam was more than the plain beam. The moment carrying capacity of equal, unequal plain and lipped beams was gradually reduced from cycle to cycle. For equal lipped angle beams, the moment carrying capacity was almost same for all the cycles. The stiffness factor for all the beams was calculated from the hysteresis loops and the stiffness factor was denoted as 'k'. The lipped beam having larger stiffness factor values compared to plain beams. From the first cycle to the second cycle, the stiffness factor reduced from 1 to between 0.3 to 0.46 for the plain beams. In the case of a lipped beam, stiffness factor reduced from 1 to between 0.63 to 0.77 from the first cycle to the second cycle.

Calderoni et al conduct [2] the experimental investigation to evaluate the performance of a double channel section from the point of deformation capacity and cyclic degradation. Specimens were made up of 1740mm long back to back coupled cold-formed unlipped channel. Two channel sections are linked by a 20mm thick vertical inner plate at mid length and two no's of 15mm thick end plates were welded to both end of the beam as shown in fig-2. The beams were stiffened by means of 4 mm thick vertical plates and that were provided parallel to the web and welded to the flanges at mid-length. Similarly four horizontal 4 mm thick batten plates were welded to the top and bottom of the flanges. By considering the single channel profile, the width to thickness ratio for flange and web was 10.5 and 61.7 respectively. The average yield and ultimate stresses of the specimen was fy=364 MPa and fu=425MPa respectively.

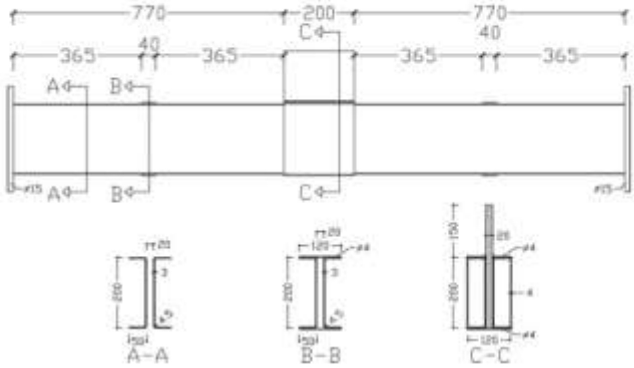

Fig -2: Geometric view of specimen (Front and Transverse View of Specimen)

Totally six beams were tested and that are named as T00, T01, T02, T03, T04 and T05. All the tests were carried out under simply supported end condition under three-point bending. The beam T00 used for preliminary testing. The beam T01 and T02 was used for the monotonic test. In the test on T01 beam, the displacement was progressively increased up to the complete collapse of the specimen at the rate of 120 mm. The little loss of response before reaching the maximum load due to the development of the first yielding in the section. Maximum load under the monotonic test was 77.6 kN and corresponding displacement was 16.2 mm shown in fig-3.

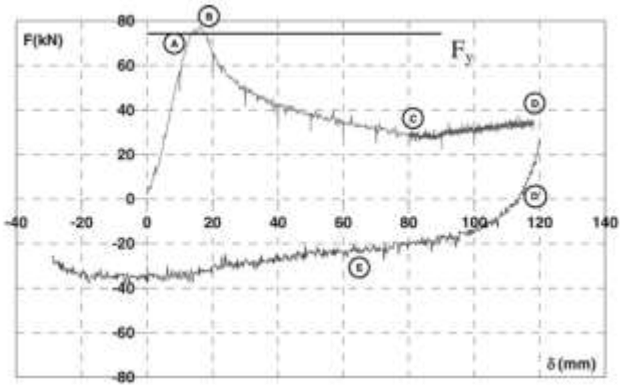

Fig -3: Load-Displacement Curve for Test T01

Once this deflection exceeds, the specimen experienced an unstable behaviour up to the collapse and the respective displacement was 80mm. During this stage, the flanges were deformed and went in contact with the central stiffening plates of the specimen. Further increasing the deflection led to growing of reaction force. Torsional flexural failure was observed when the download deflection reached at 29mm.

For T02 beam, the maximum displacement fixed to at 20mm, in order to investigate the response of unloading phase immediately after attainment of local buckling. For the first loading phase, the behaviour of beam was similar to the beam T01 and the maximum load was recorded as 72.4 kN and the corresponding displacement was 11.7mm.

Local buckling was observed at bottom flanges and just after the web. When the displacement was 40 mm, the local buckling occurred at the half wave length. Subsequently, the reaction force also reduced about 55kN with 20mm deflection as shown in the fig-4. Once reached the null value of the load for a residual displacement of 14 mm, the reloading phase started. The beam has been pushed down until the occurrence of global flexural-torsional instability and this process continued until reaching its original position.

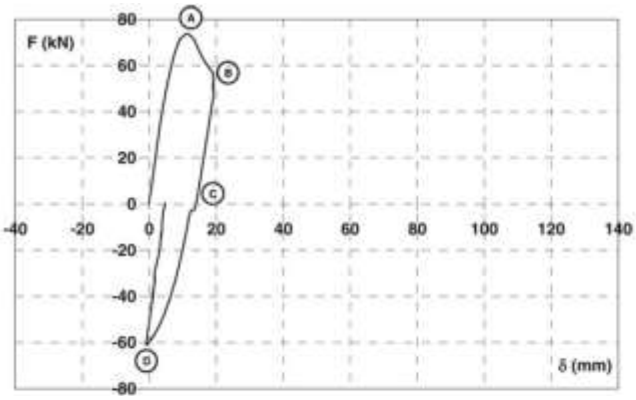

Fig -4: Load Displacement Curve for Test T02

The cyclic tests on beam T03, T04 and T05 are exhibited a similar behavior, characterized by progressive deterioration of stiffness and strength and typical steel members when affected by local buckling. Lateral bracings were provided to prevent the flexural and Torsional buckling shown in fig-5.

Fig-5: Testing Configuration

During the cyclic test, if the number of cycle increased, the crack spread the flanges and to a web connection. These cracks developed towards free edges of the flanges. The beam lost its entire load bearing capacity when the whole length of the flange involved by crack. For beam T03, the amplitude of displacement kept constant as 15mm for 47 loading cycles. The maximum load was recorded 72.3 kN while the deflection of 13.1 mm in the first cycle when the local buckling already developed in the top of flange. The first crack initiated in fifteenth cycle and fully developed through the whole flange at the last cycle i.e. 47th cycle and is shown in fig-6

Fig -6: Starting and Fully Developed Crack

For beam T04, the displacement amplitude has been 30mm kept constant in each of the 21 cycles applied to the specimen. The maximum measured reaction force was equal to 76.7 kN while the applied deflection of 14.5 mm, corresponding also to the occurrence of local buckling in the bottom flange. When the beam has been unloaded and reloaded in the reverse side, it experienced that local buckling developed at the top flanges too. The cracks started at the seventh cycles and developed more and more during the test and it interesting also the web, until they reached the free edges of the flanges during the 21st cycle. The load-displacement curve as shown in Fig-7.

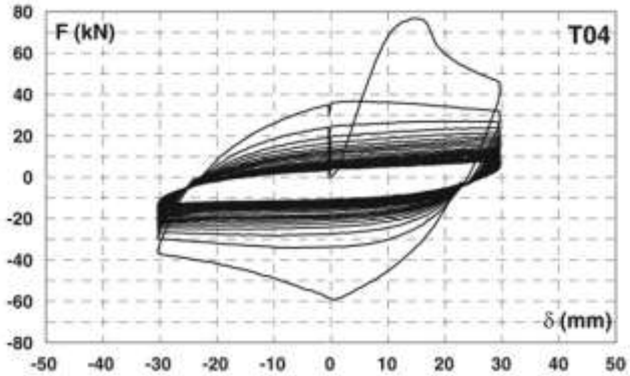

Fig -7: Load Displacement Curve for T04

For beam T05, the displacement amplitude curve kept constant as 45 mm. The maximum reaction force experienced was equal to 72.9 kN when the deflection was 13.5 mm. it was experienced just before the occurrence of local buckling in the bottom flange. Local buckling developed at the top of the flanges too when the beam was loaded in the reversal side. The cracks were started in the fourth cycle and fully developed in the 9th cycles. The tests were shown good uniformity in term of global behaviour, load-bearing capacity and displacement corresponding to maximum load. From the entire test, the maximum load varied in the range of 72.3 to 77.6 kN, while the displacements in the range of 12 to 16 m. which makes the reliability and the correctness of testing.

Corrato Chisari et al conduct monotonic and cyclic tests on a square hollow section member is investigated [4]. Stiffness and strength degradation of square hollow section under different end condition were carried out. The hydraulic actuator having a maximum load capacity of 365kN in compression and 204 kN in tension were fixed to the rigid braced frame and the column base was bolted with the rigid base. The actuator was also connected to a central unit for controlling the displacement to apply to the specimen. The main scope was to investigate the flexural behaviour of steel beams under monotonic and cyclic loads. No axial load was applied on the column. The clear length i.e. the distance between the load application points to the column base is equal to 1865 mm.

The monotonic test carried out under displacement control up to a value corresponding to a rotation equal to 0.21 rad and displacement applied at a constant rate of 0.25mm/s. The maximum moment achieved during the experimental test was equal to 374.95 kN. The quasi-static cyclic test was performed as per the AICI loading protocol [2]. This procedure is characterized by the control of the inter-storey drift angle, imposed on the test specimen as specified six cycles with chord rotation of 0.00375 rad, 0.005 rad and 0.0075 rad, four cycles with chord rotation of 0,01 rad and two cycles with chord rotation of 0.015rad, 0.02 rad, 0.03 rad and 0.04 rad. The moment-rotation plot and failure mechanism for the cyclic test were shown in fig-7. The maximum moment during the test was equal to 362 kN. The resistance reduction by cyclic test compared to the monotonic test and the value of rotations are the same. Fig-8 showed the failure of the column.

Fig -8: Failure Mechanism during Cyclic Test

In order to apply actual earthquake action, the specimen was subjected to a pseudo-dynamic test. By combining an online computer simulation of the dynamic problem damping and inertial effects are fielded into the system with the experimental response providing the actual restoring forces. This testing method provides a realistic dynamic response even in the case of nonlinear behaviour of severely. The specimen is analyzed by the pseudo-dynamic testing method with two-degree of freedom corresponding to cantilever end condition. The diagonal mass matrix was adopted for this method to characterize a mass value equal to 40t. The displacement of the structure corresponding to the load application points was measured by means of two external MTS Temposonic Transducers.

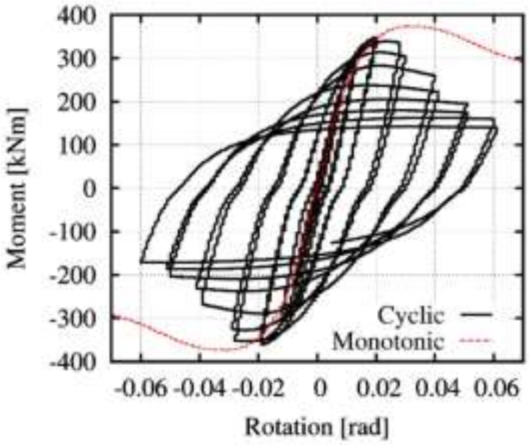

Fig -9: Moment-Rotation Plot for Monotonic and Cyclic Loading

This test was carried out by assuming no discuss damping and applying loading velocity equal to 0.1mm/s. The test was carried out with reference to Spitak (Armenia) earthquake and as recorded at a station of Gukasyan (Armenia). This earthquake record (magnitude 6.8 Ms, depth 5 km, duration 19.89 s) was characterized by low amplitude cycles during the first ten seconds and followed by few cycles with high amplitude and then a fast decay. Further research is needed to validate or improve the strategy. and Fig-9 shows the moment- rotation plot for monotonic and cyclic loading.

The behaviour metallic cold-formed thin-walled structures i.e. members, connections as well as structural systems subjected as a whole to monotonic and cyclic loads were investigated by Landolfo et al [8]. The specimens were made of 3000 mm long back-to-back coupled profiles which had lipped C-sections defined, according to the Euro code 3 classifications [7] and shown in Fig-9. The beam has a span of 2800 mm with a simply supported end condition and externally loaded by two forces applied at a distance of 500 mm from the beam mid-span. The beams were braced against lateral-torsional buckling at both the supports and at the point of loading and fig-10 shown loading and cross section of geometry.

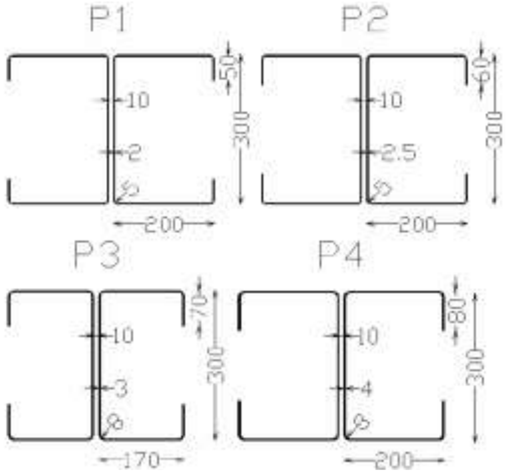

Fig -10: Cross Section of the Specimen

The ABAQUS Version 6.2 used as finite element program for the numerical analyses and each beam has been meshed by means of quadrilateral shell element with four nodes. The beams were investigated under two-point bending as shown in Fig-11. By comparing the numerical and experimental results, the plastic failure mechanism is different from the elastic buckling pattern and the members subject to elastic local buckling were failed because of coupled local, lateral-torsional and distortion mechanism type. The geometric imperfections pattern along the beam length could affect the computed plastic collapse mechanism and collapse load.

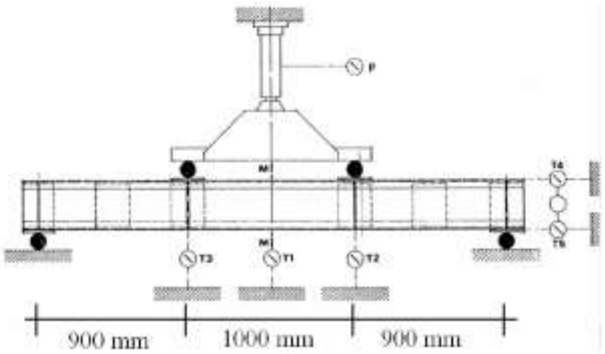

Fig -11: Loading of the Specimen

Corrado Chisari and Gianvittorio Rizzano carried out three sets of tests on hot-rolled I and H cross-sections, cold-formed rectangular hollow cross-sections and cold-formed square hollow cross-sections [3]. For every beam one monotonic and one cyclic loading were performed under cantilever end condition and the length of the beam was 1865mm. In particular, a simple hysteretic model (Hyst), the modified Ibarra-Medina-Krawinkler model (IMK), the Bouc-Wen model (BW) and the Sivaselvan-Reinhorn model (SR) were calibrated against the results coming from an experimental programme and which involving monotonic and cyclic testing of different types of steel sections. For each beam, one monotonic and one cyclic bending test were carried out under cantilever end condition where the length of the beam is 1865 mm. The displacement was applied at 0.25mm/s constant rate. The cyclic test was performed as per the AISC loading protocol [2]. In the monotonic test, the open profiles are less pronounced than for the hollow sections. Furthermore, for the hollow sections, the cyclic maximum moment is always less than the monotonic maximum moment, which indicates the effect of low-cycle fatigue. In the case of open cross-sections, the cyclic maximum moment is similar or sometimes larger than the monotonic one. All models are able to simulate energy dissipation occurring in a quasi-static cyclic test until the collapse.

While comparing to cyclic energy response, the monotonic behaviour is more difficult to match, and errors are generally bigger. Only SR model showed errors less than 5% for all profile sets, while open cross-section responses were matched best by Hyst and IMK models. BW model generally provided results comparable or equal to the worst in each set. The multi-objective optimization methodology may enrich the additional objectives. The cyclic behaviour is represented by the dissipated energy plot may not be robust and it may provide solutions which do not capture the corresponding moment-rotation plot accurately. This issue may be addressed by adding one objective related to the experimental and numerical envelope curve were mismatched.

David et al investigate an experimental program of the cyclic behaviour and energy dissipation of CFS flexural members with a focus on thin-walled buckling limit states[5]. The experimental program was conducted on cold-formed lipped channels with a length of 3048mm and 1626mm. The beam was subjected to four-point bending with a constant moment region under simply supported end condition. Lipped channel sections were used and specimens were subjected to four-point bending with a constant moment region of length. The span between the end supports is 4877 mm. and the beam was loaded through an adjustable spreader beam that accommodates variation in loading point location to change the unbraced length. This allows the rotation about the cross-section major bending axis. Slotted holes were provided at the supports and at the loading points, to prevent axial forces from crack developing in the specimen. Lateral bracing was placed along the shear spans of a length, to develop longitudinal warping and that provide fixed conditions at the minor axis of loading points that defined the unbraced length.

The cyclic test is carried out while the displacements kept constant as 0.002 mm/min. Initial geometric imperfections of cold-formed steel members reduce flexural strength and initial stiffness. Global, distortion and local buckling are responses experienced in monotonic tests. The pre-peak responses were linear past 50% of the peak moment for all the members and that became nonlinear when the buckling deformations appeared. In all the members buckling occurred before the peak moment and cross-section failure happened close to the mid-span. In the case of cyclic tests, for the entire specimen, the moment-rotation response started as linear elastic. The first six cycles with similar stiffness in both loading directions. The buckling deformations started after the tenth cycle and a pattern was similar to the monotonic members. Specimens are generally buckled at different locations during positive and negative flexure due to the strain hardening and redistribution of stresses around the first buckled cross-section of the specimen. The cyclic moment-rotation response was symmetric for most of the members with three exceptions and that are global buckling, distortional buckling and local buckling. Flexural strength decreased rapidly after the peak moment reached but some residual load carrying capacity remained. Cyclic strength degradation, cyclic stiffness degradation and cyclic pinching of the hysteretic response were varied for different buckling modes. The unloading and reloading were affected the amount of pinching observed, which was larger in members and that experiencing lateral-torsional buckling and reflected in the lowest energy dissipation compared those members experiencing local and distortional buckling.

Feng-Xiang Li et al investigate [8] an analytical study of local instability behaviour in a reduced beam section RBS beam section subjected to cyclic loading and finite-element method is used for numerical analysis. The beam was loaded by cyclically with the increasing beam chord angle amplitudes of 0.0037 rad, 0.0056 rad, 0.0075 rad, 0.015rad, 0.0224 rad, 0.03 rad, and 0.037 rad. Three cyclic tests were performed for the amplitudes of 0.0037 rad, 0.0056 rad, 0.0075 rad, 0.015 rad, and 0.0224 rad and two cycles were performed for the amplitudes of 0.03 rad and 0.037 rad. The reduced beam section (RBS) was modelled as a cantilever beam of half-span length (L/2) with a concentric force applied at the free end of the beam as shown in fig-12.

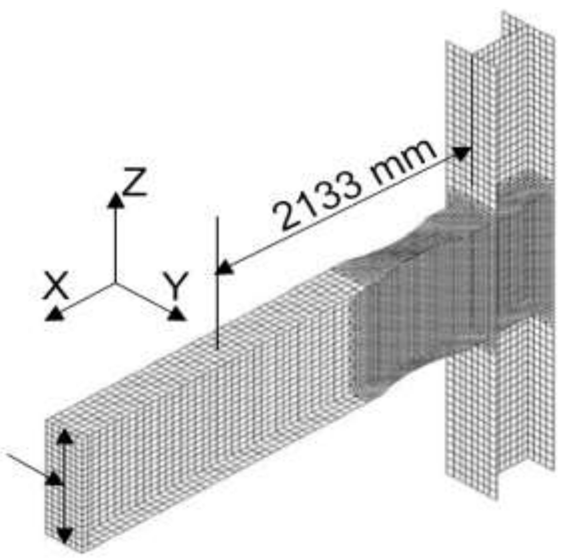

Fig -12: Geometrical view of modeling

The stress-strain relation followed a bilinear model in which considered the second stiffness was as E/100 and the yield stress was 235 MPa. The analysis was repeated by changing the yield stress of 345 MPa. There is no significant lateral buckling in beams with an unbraced length of 73.2ry. Similarly, strength deterioration caused by local buckling and did occur under cyclic loading. This suggests that strength deterioration caused by local buckling when the lateral buckling is minimized. Similarly, the stiffeners placed in the RBS portion of the beam can effectively delay local buckling and increase beam strength. The effect of thickness of stiffener on the strength was small which causes the stiffener thickness required to provide a sufficient strength is the same thickness as the web too. To withstand the amplitude of 0.045rad, recommend using three pairs of stiffeners placed and that divide the RBS beam portion into four equal regions. The maximum stress at the fixed end of RBS beams with stiffeners is almost the same as that of the standard RBS beams. The maximum stress of RBS beams with stiffeners in the RBS portion is larger than the maximum stress of standard RBS beams i.e. the maximum stress value of RBS beams with stiffeners is 1.25 times that of standard RBS beams. In addition, the RBS zone is sensitive and RBS beams with stiffeners can undergo brittle fracture in the welded parts.

3. CONCLUSIONS

The research of cold-formed steel flexural members performed by various authors has been described in this paper. The experimental and numerical investigations of open and closed sections under monotonic and cyclic loading have been investigated. The following conclusions were observed

- The monotonic and cyclic tests have shown a quite good uniformity in terms of global behaviour, load-bearing capacity and displacement corresponding to the maximum load
- The strengths obtained from FEM reasonably good agreement in with the experimental test results
- The hysteresis behaviour of cold-formed steel flexural member depends on the shape and symmetric of cross section of the beam
- Increase of the number of cycles is indirectly proportional to the strength and stiffness of the flexural member

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