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THOMSON REUTERS

[Lenin * *et al.*, 7(5): May, 2018]
IC™ Value: 3.00

ISSN: 2277-9655
Impact Factor: 5.164
CODEN: IJESS7

IJESRT

INTERNATIONAL JOURNAL OF ENGINEERING SCIENCES & RESEARCH TECHNOLOGY

EXPERIMENTAL ANALYSIS ON THE BEHAVIOUR OF COLD FORMED STEEL BEAMS FILLED WITH CONCRETE

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DOI: 10.5281/zenodo.1247111

ABSTRACT

The overall view of this project is to study the behavior of cold formed steel channel section beams with an infill of concrete. The flexural behavior of cold formed steel in filled and hollow beams is investigated by conducting experimental study using two point load test. Totally twelve beams are tested, out of which six are in filled with concrete and remaining six are hollow sections. Depth to thickness ratio is varied from 45 to 75 for all sections. For comparison, theoretical values are calculated based on Rigid Plastic Analysis for in filled beams and Australian/ New Zealand code 4600-2005 for hollow sections. Tremendous increase in load carrying capacity of in filled sections is observed. Using same amount of steel, high strength was attained by filling the sections with concrete.Experimentally it is observed that the load carrying capacity for in filled beams is not affected with a flat width ratio up to 71 for depth of section. The objective of the study is to investigate cold-formed steel rectangular hollow sections beams with respect to strength, stiffness and ductility and load deflection behavior and to predict the behavior of beams in full-scale structures. The entire study is divided in to two distinct phases of activity. In the first phase, the Properties of materials were studied and experimental were conducted on ingredients and also to analyses the mix design of concrete.

I. INTRODUCTION

General

Higher strength materials and a wider range of structural applications have caused a remarkable growth in cold-rolled steel structural members. Hot rolled members, who are subjected to residual cooling stresses, are is in contrast to the cold – formed sections which are subjected to strain hardening caused by cold working. This may significantly affect the structural performance. Even more important is that while the width to thickness of the component plate elements are limited in hot rolled sections due to manufacturing process in hot rolled sections only standard weight, depth and thickness are available. The cold – formed process can be used for practical large width to thickness ratios compared to the hot rolled sections.

As cold – formed sections are lighter in weight and since unusual section configuration can be obtained using them, they are economical in medium scale construction where lighter loads are involved these cold-formed products are used almost in every area of construction. The economy achieved for relatively light loads and shorter spans and their unusual sectional configurations are the two main reasons for their increasing use in building industry the uses are many and range from small tins to structural piling. It can be shaped in to any structural member, which is fabricated from steel sheets. For light loaded roofs this cold formed products are used as structural members like purlins. These walled cold – formed steel members have wide applications in building structures. They can be used as individual structural framing members or as panels and decks. Cold-formed steel members are used in residential construction throughout the world and especially in earth quake affected areas in India (Bhuj). Cold-formed open section steel channel and Zed sections are the most commonly used as flexural members but they are more likely to undergo torsional deformation due to their low torsional rigidity resulting from their thin walls. Cold-formed steel Rectangular Hollow Section members if used can perform better as flexural members when compared to open section members because of their high torsional rigidity. Composite construction refers to two load-carrying structural members that are integrally connected and deflect as a single unit. Slab and beam constructions are commonly used in bridges and buildings. Slab beam bonding is possible through the use of shear connectors welded. When the flange is in compression, the bond between the shear connector and the slab is assumed to be perfect. Cold formed steel members are cold

ISSN: 2277-9655
Impact Factor: 5.164
CODEN: IJESS7

formed in rolls or press brakes from flat steel which is not thicker than 12.5mm. Cold formed steel goods are made by working of steel sheet using stamping rolling or pressing to deform the sheet into a useable product CFS sections are composed of thin plate elements which are highly prone to local buckling and this necessitates more sophisticated analysis and design procedures. In addition, more theoretical and experimental studies being conducted to capture the complex behavior when used in combination with concrete. The composite construction consists of providing monolithic action between prefabricated units like steel beams so that the two will act as one unit. At the initial stages there will be natural bond between concrete and the steel. Shear connectors are provided to help the steel and concrete element to act in a composite manner ignoring the contribution made by the inherent natural bond towards this effect. Here the flexural behavior of cold formed steel channel section beams with an infill of concrete and hollow beams are investigated by conducting experimental study using two point load test.

Square and rectangular hollow sections

The use of square and rectangular hollow sections manufactured by cold forming has increased worldwide as research in to their strengths and acceptability has continued. Square and rectangular hollow sections with slender geometry minimize the amount of material used and hence maximize the structural economy. They are also superior to the conventional structural elements in tension, compression, bending, fatigue, torsion and shear. It has additional functional advantages because of lesser susceptibility to corrosion, lower drag co-efficient, and ease of fabrication, transportation, maintenance and aesthetic appearance.

Cold-formed steel tubular in filled concrete sections are widely used in many countries, because the properties of steel and concrete are effectively used to their maximum advantage. The performance of sections with laterally confined concrete with respect to strength, ductility and stiffness is better than that of sections with unconfined concrete. The steel casing in a concrete filled tubular section confines the core triaxially and the in filled concrete prevents local buckling of the shell. Any structure should behave in a ductile manner. It should not collapse in brittle failure without any warning. When a ductile structure is subjected to overloading it will tend to deform in elastically and will redistribute the excess load to the elastic parts of the structure. These essentially different materials are completely compatible and complementary to each other; they have almost the same thermal expansion; they have an ideal combination of strengths with the concrete efficient in compression and the steel in tension; concrete also gives corrosion protection and thermal insulation to the steel at elevated temperatures and additionally can restrain slender steel sections from local or lateral-torsional buckling.

Cold-formed steel concrete composite slabs are common in building construction. The use of concrete steel composite beams and columns is very limited. The conventional reinforced concrete beams can be replaced by cold-formed steel concrete in filled beams. The cold-formed steel outer cover will act as formwork before concrete attains the strength and later as reinforcing steel. The construction cost can be significantly reduced because of the absence of formwork. As a result of confinement of concrete, the ductility of concrete core will be improved and the failure of steel shell due to buckling will be delayed. Concrete in filled cold-formed steel tubes have better fire resistance, earthquake impact resistance, high ductility, high strength, ease in construction procedure and overall economy compared to similar reinforced concrete members.

Scope of investigation

The aim of this investigation is to study the behavior of cold-formed Rectangular hollow section beams with and without concrete infill.

It is clear from the review of literature that much work has not been done on the behavior of cold formed steel Rectangular hollow section in filled beams. Hence experimental were conducted on cold-formed steel hollow and concrete in filled beams to theirbehavior and effect of infill. Eleven bending test were performed on cold-formed steel Rectangular hollow section (RHS) and concrete in filled beams of different sections and spans to assess the suitability of such section as flexural members. The experimental behavior should be predicted using a standard software package and comparison made whether the analytical behaviors is closed to the experimental behavior.

The theoretical studies related to concrete in-filled beams and hollow channel sections are carried out based on the rigid plastic analysis and Australian /New-Zealand code (AS/NZS 4600-2005) respectively. The ultimate loads are found out from the known moment carrying capacity.

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For analysis of in filled beams the followingconsiderations are made.
Fy = yield strength of the steel
l = lip length
d = web of the section
b = bottom flange
t = thickness of the steel
r = radius
E = Young's modulus of elasticity=2x10^5MPa
L = span of the beam.
For a beam,
Pc = force due to concrete
Pb = bond force
bc = width of the concrete
Nc and Ns = distances of neutral axis from the end of compression face for concrete and steel.
M = moment carrying capacity of the composite beam
ɣ = the reduction factor
d = the depth of the web
l = lip length
t = thickness of steel
fy = yield stress of the steel
fck = characteristic strength of concrete.
The equilibrium of forces in concrete, the distance from the neutral axis is obtained from the equation.

$$Nc = \frac{P_b}{(0.85\gamma fckbc)}$$

The equilibrium of forces in the steel, the distance from the neutral axis is obtained from the equation.

$$Ns = \frac{(tfy(2d + bc - 2l) - P_b)}{4tfy}$$

The moment of forces on the top fibre, the moment carrying capacity M of the composite sections can be determined from expression.

$$M = t\,f\,y\,(d^2+dbc-2Ns^2)-0.425\gamma^2 Nc^2 fckbc-t^2 fy$$

For complete interaction between steel and concrete $Nc = Ns = N$ and

$$P_b = \frac{0.85\gamma f_c^{'} b_c tfy(2d + bc - 2l)}{0.85\gamma f_c^{'} b_c + 4tfy}$$

For the analysis of hollow beams Australian/ New Zealand code (AS/NZS 4600-2005) is referred for determining the moment carrying capacities. The following considerations are made for a hollow beam,
i. Based on the initiation of yielding and
ii. Nominal section moment capacity
Fy = yield strength of the steel
l = lip length
d = web of the section
b = bottom flange
t = thickness of the steel
r = radius
E = Young's modulus of elasticity=2x10^5MPa
L = span of the beam.

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II. LITERATURE REVIEW

Rolf Baehre (1993) investigated the steel design and research in cold formed steel. In the field of light – gauge structures, rules for the design of sheeting and members are approved in Germany were worked out parallel to the appropriate ECCS recommendations and to Eurocode 3. Ongoing research activities on Germany demonstrate the unbroken expansion of light – weight building techniques.

Zhong –Quan Zhang (1993) briefly described the limit state design method for cold-formed steel structures in China. Some basic factors affecting the structural safety, such as the reliability index, partial coefficients for loads and resistances etc. And design general expression were systematically discussed and presented. Besides, the design method for various cold-formed steel structural members and connections.

Clarke and Hancock (1994) proposed and validated a simple design procedure for the cold-formed tubular frames. The possible provisions off the American Iron and steel Institute load and Resistance Factor Design (AISILRFD) specification are proposed.

Batista (1994) investigated the local-global buckling interaction. The research focuses on the local-global buckling interaction and deals with steel structures in which cold-formed profiles are used. In this context. Three main aspects were discussed; the collapse load of an isolated thin-walled member, its post-critical nonlinear behavior and the complex behavior of thin-walled plane-framed structures under local-global mode interactions. All these aspects are interconnected for structural behavior and therefore for code prescription.

Wei-Wen Yu and LaBoube (1997) presented cold-formed steel structures during the numerous research projects have been conducted at the University of Missouri-Rolla. The purpose of these investigations had been tostudy the structural strength of cold-formed steel members, connections, and structural systems. Some of the research findings have been used in the development of the AISI specification for the design of cold-formed steel structural members. ASCE specification for the Design of cold-formed stainless steel structural members.

Dissanayakea, Davisonb and Burgess (1998) worked on a numerical study has been conducted to investigate the influence of composite beam-to-steel column joints on the behavior of composite beams, for a number of sub frames representing the spans and loading arrangements of current commercial buildings in the UK. a two-stage analysis has been performed on each of these subframes, varying the steelwork connection, the percentage of reinforcement over the support and the degree of shear connection between the steel beam and the concrete slab. The studies use a computer model which has been developed to simulate the behavior of steel-framed buildings with composite floor decks. The program is capable of simulating the behavior of two-dimensional subframes, and considers their two-stage behavior, both during unprompted construction and as fully composite beams. It was also capable of taking in to account the partial interaction between the steel beam and the composite slab, the orientation of the profiled metal deck, the effect of additional reinforcement over supports and the semi-rigid nature of the joint between the composite beam and steel column. In contrast to the usual observations made in isolated joint tests, the study indicates very low values of strains in reinforcing bars at the composite beam-to- steel column joint at the ultimate limit state. The results also indicate that the common types of composite joints available are capable of providing the rotation capacity required to sustain the ultimate load with about 1% of reinforcement over the support, without the use of expensive column web stiffeners.

Schafer and Pekoz (1998) presented a computational modeling of cold-form steel thin-walled, cold-formed steel members exhibiting a complicated post-buckling behavior. Today advanced computational modeling supplements experimental investigation. Accuracy of computational models relies significantly on the characterization of selected inputs. Distributions or magnitudes to be used for modeling geometric imperfections and for modeling residual stresses of cold-formed steel members were not found. In order to provide additional information existing data was collected and analyzed and new experiments were performed. Simple rules of thumb and probabilistic concepts were advanced for characterization of both quantities. The importance of the modeling assumptions was shown in the examples. The ideas were summarized in a preliminary set of guidelines for computational modeling of imperfections and residual stresses.

Put et al (1999) reported the results of ten biaxial binding tests on unraced, simply supported, cold-formed steel Z-beams loaded at midspan in planes inclined to the major principal plane. These results and those of six beams loaded in the major principal plane were compared with analytical predictions and with simple design approximations. The tests showed that the beam strengths decreased as the load inclination to the major principal plane increased, and that the strength was higher when the load inclination was toward the web that when it was toward the flange lip. Good agreement was demonstrated between the test results and analytical

ISSN: 2277-9655
Impact Factor: 5.164
CODEN: IJESS7

predictions of the strengths. An extended series of analytical predictions was used to develop simple interaction equations that can be used in the design of eccentrically loaded cold – formed Z-beams.

Put et al (1999) reported the results of 34 bending and torsion tests on unbraced simply supported cold-formed steel channel beams loaded eccentrically at midspan. These results and those of 10 concentrically loaded beams were compared with analytical predictions and with simple design approximations. The tests show that the beam strengths decreased as the load eccentricity increased and that the strength was higher when the load acted on the centroid side of the shear center. Good agreement was demonstrated between the test results and analytical predictions of the strengths. An extended series of analytical predictions was used to develop simple interaction equations that can be used in the design of eccentrically loaded cold-formed channel beams.

Put et al (1999) studied the lateral buckling capacities of cold-formed lipped Z-beams. The design code formulations based on those for hot-rolled I-beams are inappropriate for cold formed lipped Z beams, because of the very different cross- sectional shapes and methods of manufacture. There was a need for test data on the lateral buckling tests on simply supported, unbraced beams, and a series of subsidiary tests, including tension tests, a torsion test, and measurements of initial crookedness and twist were done. The test beams failed catastrophically by local or distortional buckling of the most compressed element of the cross section. The moments at failure were lower when the beam lateral deflections increased the compression in the compression flange lip, and higher when they increased the compression in the compression flange lip, and higher when they increased the compression in the flange-web junction. The lateral bucking test results were compared with theoretical predictions and with capacities calculated using existing design codes AS4100 and AS/NZS4600. Agreement between the test results and theoretical predictions was reasonably good. The single design curve of AS/NZS4600 was unduly optimistic for near-uniform bending and unnecessarily conservative for high-moment gradients in beams of intermediate slenderness. Improvements of design method of AS4100 were suggested for future design code formulations.

III. MATERIAL PROPERTIES

Specific gravity of cement

Specific gravity of cement is obtained as 3.05.

Specific gravity of coarse aggregate

Specific gravity of coarse aggregate is 2.81.

Specific gravity of fine aggregate

Specific gravity of sand is 2.63.

Water Absorption of coarse aggregates

Water absorbing capacity of coarse aggregate is 0.5%.

Abrasion valueof coarse aggregates

Abrasion value of coarse aggregate is 8.4%

IV. THEORETICAL INVESTIGATION

Bending behavior of structural steel beams and the plastic hinge

Bending tests were performed on cold-formed RHS to examine the behavior of beams. When a transverse load (*P*) is applied to a steel beam, there is corresponding deformation, which includes curvature induced in the beam. Internal forces, such as bending moments *(M*), occur within the beam.

Consider the RHS in bending, the cross-section of which is shown in Figure 2.5. According to simple engineering bending theory, the distribution of strain across the section is linear regardless of the stress state, and the value of the strain at the extreme fibers is proportional to the curvature. Initially, in the elastic range, the stress distributions linear. As the curvature increases, the extreme fibers reach the yield stress at the yield moment (My), where My = fyZ and Z is the elastic section modules. At larger curvatures and strains, yielding spreads inwards toward the neutral axis. For the elastic – plastic – strain hardening material, the section yields almost completely and is fully plastic at high values of curvature (theoretically full plasticity can only occur at infinite curvature). The moment at which full yielding occurs is termed the plastic moment (Mpy), where M p = fyS and S is the plastic section modulus. Strain hardening is initiated at even higher curvatures, and the stress can exceed the yield stress. In the case of cold-formed RHS, there is no plastic plateau as strain hardening occurs immediately after yielding, and the stress increases beyond fy at lower values of curvature compared to the idealized case.

ISSN: 2277-9655
Impact Factor: 5.164
CODEN: IJESS7

For the idealized case, the curve includes a linear range and a transition from the yield moment to the plastic moment. Once the cross-section is fully plastic, increases in curvature can occur without a corresponding moment increase. Not only does the moment reach Mp but the beam maintains Mp as the curvature increases. The increasing curvature at constant moment Mp is termed a plastic hinge and demonstrates the ductility of the steel beam. The moment may raise above the plastic moment due to strain hardening but the increase in moment is sometimes ignored. The behavior can be idealized as "rigid plastic", in which no deformation occurs until the plastic moment is reached.

For an RHS , yielding occurs before the yield moment is reached due to residual stresses. The moment rises above the plastic moment due to the lack of a plastic plateau and early strain hardening. The rigid-plastic assumption is an approximation of the true behavior of an RHS beam.

Rotation capacity

A steel beam cannot sustain infinite curvature, and at some curvature failure occurs. The most common mode of failure is local instability (buckling) of the plate elements in the section, although material fracture is another possible failure mode. It is assumed that there is adequate lateral restraint to ensure that no failure occurs due to (out-of-plane) lateral buckling. Some beams may fail before reaching the yield moment or the plastic moment. If the beam can teach the plastic hinge can rotate before failure occurs. To calculate R the moment curvature graph is normalized with respect to the plastic moment and plastic curvature (Mp/E1), (where E is Young's modules of elasticity, or elastic modulus, and I1 is the second moment of area of the section).

Sections are classified into groups depending on their behavior under bending. (Their rotation capacity and maximum moment, **M**

Class 1 sections can attain the plastic moment and have plastic rotation capacity sufficient for plastic design. Such sections are sometimes referred to as plastic sections or compact sections.

Class 2 sections can develop the plastic moment but have limited rotation capacity and are considered unsuitable for plastic hinge formation. Class 2 sections may be known as compact elastic sections.

Class 3 sections can reach the yield moment, but cannot reach the plastic moment due to local buckling. Such sections are sometimes called semi-compact or non-compact.

Class 4 sections cannot reach the yield moment due to local buckling. They are also known as slender sections.

The behavior of a simply supported beam

Consider a simply supported beam of length L, subject to a point load, P, at the center. The bending moment distribution is triangular, and is not affected by yielding of the steel beam. As the load increases, the stress reaches the yield stress at the most stressed point. Further increase of the load causes yielding to spread through the depth and along the length of the beam until the plastic moment is reached at a load of P u=4Mp/L. A plastic hinge has now been created and no extra load can be sustained. A mechanism has formed, since there are the two hinges (pins) at the supports and the plastic hinge at the center. A mechanism is formed when an increase in deflection can occur in any part of the structure, without the addition of any extra load. At collapse, most of the beam remains elastic and hence has small curvatures. If the curvature in the elastic parts of the beam is ignored, it is an example of the rigid-plastic concept, whereby all the curvature and rotation occur at the plastic hinge. The curvature is assumed to be infinite at the plastic hinge where discontinuities occur in the beam rotation.

Plastic analysis

A statically indeterminate beam does not necessarily fail when a single plastic hinge forms. Load is transferred to other less highly stressed parts of the structure. Additional load is carried, and the first (and any subsequent) hinges maintain the plastic moment and undergo rotation. A collapse mechanism forms (for the entire structure or part of the structure) when there is a sufficient number of plastic hinges. The process of transferring load to other parts of the structure is known as moment redistribution.

At the formation of the final hinge at B, at point A, the beam must be able to sustain the plastic moment for the required rotation, or moment redistribution cannot take place. If the beam is unable to achieve the necessary rotation at A, the beam is unsuitable for plastic design.

Rotation requirements

Plastic hinges must be able to rotate a certain amount to redistribute the bending moment and eventually form a plastic collapse mechanism. The required hinge rotation depends on the nature of the loading, the properties of

ISSN: 2277-9655
Impact Factor: 5.164
CODEN: IJESS7

the member, and the size of the frame or beam. Hinge rotation is dimensionless quantity, and refers to the rotation in radians. The required hinge rotation is the angle through which an idealised plastic hinge must rotate. In reality, there is a plastic zone of high curvature which acts like a hinge.

The Eurocode 3 summaries the rotation requirements for a variety of frames and multi-span beams constructed from I – sections and concluded that a value of R = 3 was a suitable value to ensure that a plastic collapse mechanism could form. The value of R = 3 is used in Eurocode 3.It was found that R = 3 was an acceptable rotation capacity requirement for continuous beams. To utilize plastic design, the rotation capacity of the member must exceed the rotation requirement at the hinge for the given frame and distribution of loading. Not all members can achieve the required rotation.

Local buckling

Most structural sections can be idealized as being comprised of individual flat plate elements. An RHS can be considered as four plates joined to form the hollow section. Plate elements are susceptible to local buckling. In plastic design, the plate elements are required to achieve substantial deformations once they have yielded before local buckling occurs, in order for moment redistribution to take place. Most RHS exhibited a “bow-out” or “bow-in” on each flat face of the section that was reasonably constant along the length of the RHS. It was most common for there to be a bow-out on each web, and bow-in on each flange.

V. METHODOLOGY

Selection Of W/c Ratio

From figure-1 IS 10262-1982
W/c ratio = 0.5

Selection of water and sand content

From table-4 IS 10262-1982 for 20mm nominal maximum size aggregate and sand conforming to grading zone water content/ cubic meter of concrete = 186Kg and sand content as percentage of total aggregate by absolute volume = 35 % for change in value in W/c, compacting factor and sand belonging to Zone III, the following adjustment is required.

Water content
= 186 + 3 % of 186
= 186 + 5.58
= 191.6 lit/m^3

Determination of Cement Content
W/c ratio = 0.50
Water = 191.6 lit/m^3
Cement = 191.6/0.5
= 380 Kg/m^3

Determination of Coarse and Fine Aggregate Content
From table-3 for the specified maximum size of aggregate of 20 mm, the amount of entrapped air in the wet concrete is 3 %.

(i) Fine aggregate:

$V = [W + C/S_c + 1/p * fa/S_{fa}] * 1/ 1000$

V = 100-2

V = 98 %

$0.97 = [191.6 + 383/3.05 + 1/0.315 * 0.315* f_a/2.63] * 1/1000$

$f_a = 540$ Kg/ m^3

(ii) Coarse aggregate:

$V = [W + C/ S_c + 1/1-P * C_a/ S_{ca}] * 1/1000$

$0.98 = 191.6 + 383/3.05 + 1/1-0.315 * C_a/2.81] * 1/1000$

$C_a = 1282.80$ Kg/ m^3

Result

M_{20} = 1: 1.44: 3.40
W/c = 0.50

VI. TEST FOR CONCRETE

Load deflection behavior of infilled and hollow beams are compared and discussed as shown in Fig .2 and Fig 3.There is linearity in the curves of in filled sections almost up to 55kN, later the curve followed a non-linear path up to the failure point. Whereas, the curves of hollow sections AH1, AH2, AH3 followed a linear path almost up to 15kN and then showed a non-linear path. The sections AH4, AH5, AH6 showed a linear path up to the failure point since these sections failed to resist the loads.

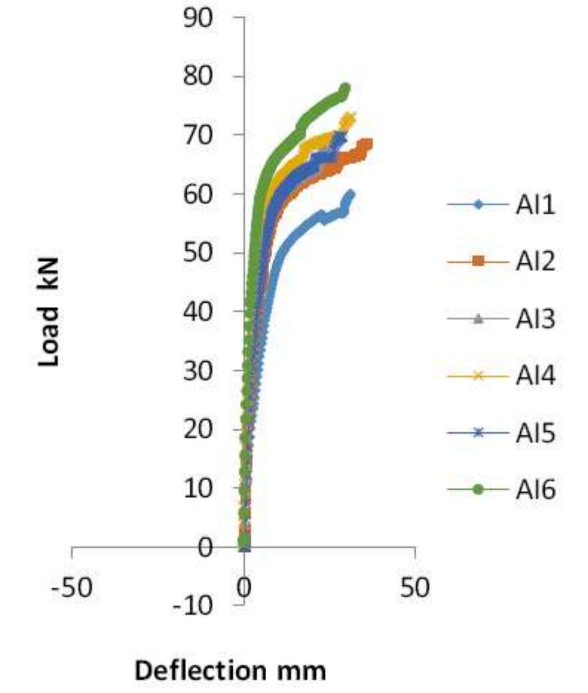

Fig 2. Load-Deflection curve for all the infilled beams

The curves of infilled sections were highly stiff than the curves of hollow sections. The average ductility index for all the infilled sections is 5.33 as shown in Fig 4.The composite interaction between the steel and the concrete increased the ductility index.

ISSN: 2277-9655
Impact Factor: 5.164
CODEN: IJESS7

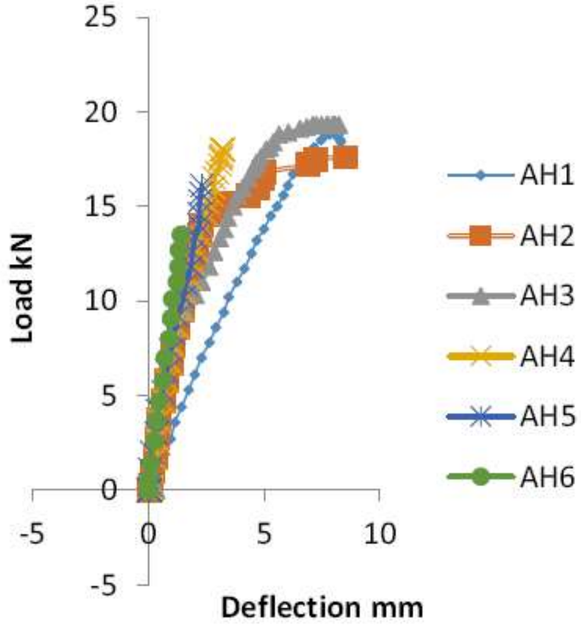

Fig 3. Load-deflection curve for all hollow beams

VII. CONCLUSION

Failure was by buckling of the infilled beams where concrete in filled was easily separated from the steel section. Bending was also observed for all the infilled sections as seen in plate 1 and plate 2.It was noticed that in case of hollow beams, local buckling occurred in the compression zone. As seen in the plate 3 and plate 4 there occurred inward and outward buckling of the hollow beams and therefore it was end by the overall buckling.

ISSN: 2277-9655
Impact Factor: 5.164
CODEN: IJESS7

Plate 1. Buckling of steel in compression zone

Plate 2.Separation of concrete and steel

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CITE AN ARTICLE

Olivu.M, L. M., Ravindra Raj.B, J., Imran, A., & Baja, S. (2018). EXPERIMENTAL ANALYSIS ON THE BEHAVIOUR OF COLD FORMED STEEL BEAMS FILLED WITH CONCRETE. *INTERNATIONAL JOURNAL OF ENGINEERING SCIENCES & RESEARCH TECHNOLOGY*, *7*(5), 360-369.