

**UNIVERSITI TEKNOLOGI MARA**

**DEVELOPMENT OF BENT-UP  
TRIANGULAR TAB SHEAR  
TRANSFER (BTTST) ENHANCEMENT  
IN COLD-FORMED STEEL (CFS)-  
CONCRETE COMPOSITE BEAMS**

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## ABSTRACT

Cold-formed steel (CFS) sections, have been recognised as an important contributor to environmentally responsible and sustainable structures in developed countries, and CFS framing is considered as a sustainable 'green' construction material for low rise residential and commercial buildings. However, there is still lacking of data and information on the behaviour and performance of CFS beam in composite construction. The use of CFS has been limited to structural roof trusses and a host of non-structural applications. One of the limiting features of CFS is the thinness of its section (usually between 1.2 and 3.2 mm thick) that makes it susceptible to torsional, distortional, lateral-torsional, lateral-distortional and local buckling. Hence, a reasonable solution is resorting to a composite construction of structural CFS section and reinforced concrete deck slab, which minimises the distance from the neutral-axis to the top of the deck and reduces the compressive bending stress in the CFS sections. Also, by arranging two CFS channel sections back-to-back restores symmetry and suppresses lateral-torsional and to a lesser extent, lateral-distortional buckling. The two-fold advantages promised by the system, promote the use of CFS sections in a wider range of structural applications. An efficient and innovative floor system of built-up CFS sections acting compositely with a concrete deck slab was developed to provide an alternative composite system for floors and roofs in buildings. The system, called Precast Cold-Formed Steel-Concrete Composite System, is designed to rely on composite actions between the CFS sections and a reinforced concrete deck where shear forces between them are effectively transmitted via another innovative shear transfer enhancement mechanism called a bent-up triangular tab shear transfer (BTTST). The study mainly comprises two major components, i.e. experimental and theoretical work. Experimental work involved small-scale and large-scale testing of laboratory tests. Sixty eight push-out test specimens and fifteen large-scale CFS-concrete composite beams specimens were tested in this program. In the small-scale test, a push-out test was carried out to determine the strength and behaviour of the shear transfer enhancement between the CFS and concrete. Four major parameters were studied, which include compressive strength of concrete, CFS strength, dimensions (size and angle) of BTTST and CFS thickness. The results from push-out test were used to develop an expression in order to predict the shear capacity of innovative shear transfer enhancement mechanism, BTTST in CFS-concrete composite beams. The value of shear capacity was used to calculate the theoretical moment capacity of CFS-concrete composite beams. The theoretical moment capacities were used to validate the large-scale test results. The large-scale test specimens were tested by using four-point load bending test. The results in push-out tests show that specimens employed with BTTST achieved higher shear capacities compared to those that rely only on a natural bond between cold-formed steel and concrete and specimens with Lakkavalli and Liu bent-up tab (LYLB). Load capacities for push-out test specimens with BTTST are

relatively higher as compared to the equivalent control specimen, i.e. by 91% to 135%. When compared to LYLB specimens the increment is 12% to 16%. In addition, shear capacities of BTTST also increase with the increase in dimensions (size and angle) of BTTST, thickness of CFS and concrete compressive strength. An equation was developed to determine the shear capacity of BTTST and the value is in good agreement with the observed test values. The average absolute difference between the test values and predicted values was found to be 8.07%. The average arithmetic mean of the test/predicted ratio ( $n$ ) of this equation is 0.9954. The standard deviation ( $a$ ) and the coefficient of variation (CV) for the proposed equation were 0.09682 and 9.7%, respectively. The proposed equation is recommended for the design of BTTST in CFS-concrete composite beams. In large-scale testing, specimens employed with BTTST increased the strength capacities and reduced the deflection of the specimens. The moment capacities,  $M_{U,exp}$  for all specimens are above  $M_{U,theory}$  and show good agreement with the calculated ratio ( $>1.00$ ). It is also found that, strength capacities of CFS-concrete composite beams also increase with the increase in dimensions (size and angle) of BTTST, thickness of CFS and concrete compressive strength and a CFS-concrete composite beam are practically designed with partial shear connection for equal moment capacity by reducing number of BTTST. It is concluded that the proposed BTTST shear transfer enhancement in CFS-concrete composite beams has sufficient strength and is also feasible. Finally, a standard table of characteristic resistance,  $P_{t,a,b}$  of BTTST in normal weight concrete, was also developed to simplify the design calculation of CFS-concrete composite beams.

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## LIST OF ABBREVIATIONS

CFS	cold-formed steel
BTTST	bent-up triangular tab shear transfer
CIDB	Construction Industry Development Board
IBS	Industrialized Building System
AISI	American Iron and Steel Institute
DSM	Direct Strength Method
NAS	North American Specification
PNA	plastic neutral axis
EC4	Eurocode 4
LYLB	Lakkavalli and Liu bent-up tab
LVDT	displacement transducers
CDAS	Control and Data Acquisition System
ENA	elastic neutral axis
NA	neutral axis
FE	finite element

## LIST OF SYMBOLS

$\lambda$	slenderness ratio
$p_y$ or $f_y$	strength of the steel section
$L_e$	effective length
$r$	radius of gyration
$L_f$	collar length of BTTST (Figure 4.10 and Figure 4.29)
$L_s$	span length of BTTST (Figure 4.10 and Figure 4.29)
$K$	local buckling coefficient which depends on the element type, section geometry and is detailed for various cases in Appendix B of BS5950: Part 5.
$t$	is the material thickness
$b$	width of the element
$L$	span of beam
$M_{nd}$	inelastic distortional moment capacity
$F_{shear}$	connection force.
$F_c$	longitudinal resultants in the concrete element
$F_s$	longitudinal resultants in the steel element
$M$	total resisting moment
$z$	distance between the centroids of the concrete and steel element
$\epsilon_c$	strain of the concrete
$\epsilon_s$	strain of the steel

$\epsilon_{slip}$	slip strain
$E_c$	modulus of elasticity of the concrete element
$E_s$	modulus of elasticity of the steel element
$A_c$	cross-sectional areas of the concrete element
$A_s$	cross-sectional areas of the steel element
$I_c$	moments of inertia of the concrete element
$I_s$	moments of inertia of the steel element
$y_c$	distances of the lowest fiber and uppermost fiber of the concrete element, measured from the neutral axis
$y_s$	distances of the lowest fiber and uppermost fiber of the steel element, measured from the neutral axis
$s$	longitudinal slip
$u$	longitudinal displacement component
$A_c$	concrete area
$n$	modular ratio, $E_s / E_c$
$E_s$	elastic modulus of structural steel
$E_c$	elastic modulus of structural concrete
$\tau(x)$	shear force transmitted per unit length of the beam
$K_{si}$	initial modulus
$f_{cu}$	Cube compressive strength of concrete
$R_s$	axial strength of the steel element
$R_c$	axial strength of the concrete element
$R_q$	longitudinal shear strength of the shear connector
$\eta$	degree of shear connection

$A_{c1}$ to $A_{c3}$	concrete area (refer to Figure 2.10)
$A_{s1}$ to $A_{s4}$	steel area (refer to Figure 2.10)
$Q_u$	ultimate shear capacity of the stud connector
$f_c$ or $f_{ck}$	concrete cylinder compressive strength
$f_u$	tensile strength of stud material
$V_c$	shear strength due to concrete pull-out failure
$\lambda_1$	factor dependent upon type of concrete (1.0 for normal density concrete, 0.85 for semi-low density concrete, 0.75 for structural low density concrete)
$P_{RD}$	strength of the stud connector
$\gamma_v$	partial safety factor, taken as 1.25 at ultimate
$\alpha$	$0.2 \left( \frac{h}{d} + 1 \right) < 1.0$ , h is the overall length of stud
$Q_n$	nominal strength of one channel shear connector
$t_f$	flange thickness of channel shear connector
$t_w$	web thickness of channel shear connector
$L_c$	length of channel shear connector
$q_{rs}$	strength resistance of channel shear connector embedded in a solid concrete slab
$\phi_{sc}$	resistance factor of shear connectors
H	height of the channel
$A_{f1}$	area of the front surface (refer to Figure 2.15)
$A_{f2}$	area of the front surface enlarged at a slope of 1:5 to the rear surface of the adjacent connector (refer to Figure 2.15)
$\eta_1$	$\frac{\sqrt{A_{f2}}}{\sqrt{A_{f1}}}$ , but not greater than 2.5 for normal density concrete or 2 for lightweight aggregate concrete

$b$	length of the angle
$h$	width of the upstanding leg of the angle
$q$	shear capacity per perfobond rib connector
$A_{cc}$	shear area of concrete per connector
$A_{tr}$	total area of transverse reinforcement
$d$	diameter of the perfobond rib holes
$h_1$	rib height,
$t_r$	thickness of the rib
$b_f$	slab thickness
$h_2$	slab height downward the connector
$L_{c1}$	contact length between the concrete and flange of the steel section
$B_e$	effective width of composite beam
$P_u$	ultimate shear resistance
$P_{Rk}$	characteristic shear resistance
$\delta_i$	initial slip
$\delta_u$	slip capacity
$\delta_{Pu}$	slip at ultimate load
$P_{tab}$	ultimate load per shear tab
$M_u$	moment capacity
$\beta_1$	constant to be refer to section Regression Analysis, Chapter 5
$A_b$	bearing area of BTTST (refer Figure 4.10 and Figure 4.29)
$A_s$	shear area of BTTST, $L_f t$
$\beta_2$	constant to be refer to section Regression Analysis, Chapter 5

$\theta$	angle of BTTST (degree)
$\mu$	average arithmetic mean
$\sigma$	standard deviation
CV	coefficient of variation
$M_{u,theory}$	predicted plastic moment capacity
I	second moment of area
$\delta$	deflection of the CFS-concrete composite beams
$b_c$	effective breadth of concrete slab
$h_c$	depth of concrete slab
$h_{top}$	depth of slab from upper steel flange
$D_s$	depth of CFS
$t_f$	thickness of CFS
$t_l$	lip length of CFS
$b_f$	width of CFS
$R_{shear}$	longitudinal shear resistance of the shear connectors
$R_{CFS}$	resistance of the CFS beam
$R_{conc}$	resistance of the concrete
$\delta_c$	deflection of the composite beam with full shear connection
$\delta_o$	deflection of the steel beam acting alone
$I_{comp}$	second moment of area of the composite section
$I_g$	second moment of area of uncracked section
$I_p$	second moment of area of cracked section
P	load

$P_{p,exp}$	experimental elastic load
$M_{u,exp}$	experimental ultimate moment
$M_{e,exp}$	experimental elastic moment
$M_{u,theory}$	theoretical ultimate moment
$d_p$	depth of PNA
$d_{p,conc}$	depth of PNA in the concrete element
$d_{p,CFS}$	depth of PNA in the CFS element
$y_b$	depth of ENA
$\epsilon_y$	yield strain
$f_{cu,mean}$	average concrete strength
$f_{y,mean}$	average CFS strength
$L/d$	span to depth ratio
$Q_k$	characteristic resistance of headed studs in normal weight concrete

# CHAPTER 1

## INTRODUCTION

### 1.1 Introduction

Cold-formed steel (CFS) and hot-rolled steel are two main steel material types that are being used in the steel industry. Although hot-rolled steel is more familiar among structural engineers, the use and importance of CFS are expanding in the present building constructions. The usage of CFS sections started in the United States for at least a century, mainly for non-structural purposes only. It has become more and more popular in the 20<sup>th</sup> century. However, the structural use of CFS sections began in the mid-20<sup>th</sup> century especially for industrial and commercial buildings [1]. Recently, the use of CFS sections as an alternative material for timber roof structure keep increasing due to the quality assurance of steel structures.

CFS sections are made by bending a flat sheet of steel at room temperature. CFS structural members are normally used in roof and wall systems of industrial, commercial and agricultural buildings. The typical sections widely used as purlins and truss members, are “Z” and “C” sections (Figure 1.1). For typical applications of cold-formed steel sections, there are a number of codes of practices [1]–[3] available.

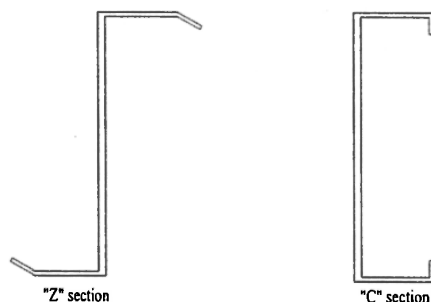


Figure 1.1. Typical cold-formed steel sections

The term “composite construction” in civil engineering structures is always referred to the use of steel and concrete formed together into a component in such a way that the resulting arrangement can function as a single item [4]. Buckling is one of the major weaknesses for steel structures which will reduce the strength especially when used as compression members. Table 24(c) in BS5950: Part 1 (2000) gives the reduction strength of CFS section under compression which depends on the slenderness ratio,  $\lambda$  and strength of the steel section,  $p_y$  [5]. Steel beams without lateral restraint are subjected to lateral torsional buckling and twisting. Thus, whenever concrete is used to restrain the steel beam by encasing the compression steel flange, such arrangement is called a composite section. Figure 1.2 shows two common types of composite sections of restrained beams.

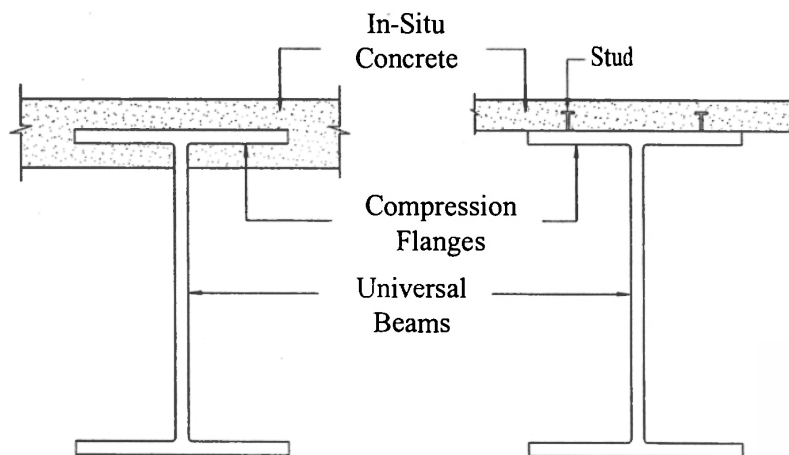


Figure 1.2. Composite Sections

## 1.2 Problem Statement

In steel industry hot rolled steel is more familiar and popular to structural engineers and contractors compared with CFS. The used of cold-formed steel is limited in construction industry and currently application of CFS mainly in roof trusses and host of non-structural application [6].

Currently, the design of CFS structures is provided in BS 5950: Part 5 [3], whilst the design of composite steel and concrete structures is given in BS 5950:Part3 [20].

Studies on the CFS composite beams from several countries had pointed the potential of CFS for beam structures [9] – [14]. Figure 1.3 shows that all the researchers agree on the usage of CFS beams and composites with concrete and detail discussion on related studies conducted are discussed in Section 2.4, Chapter 2.

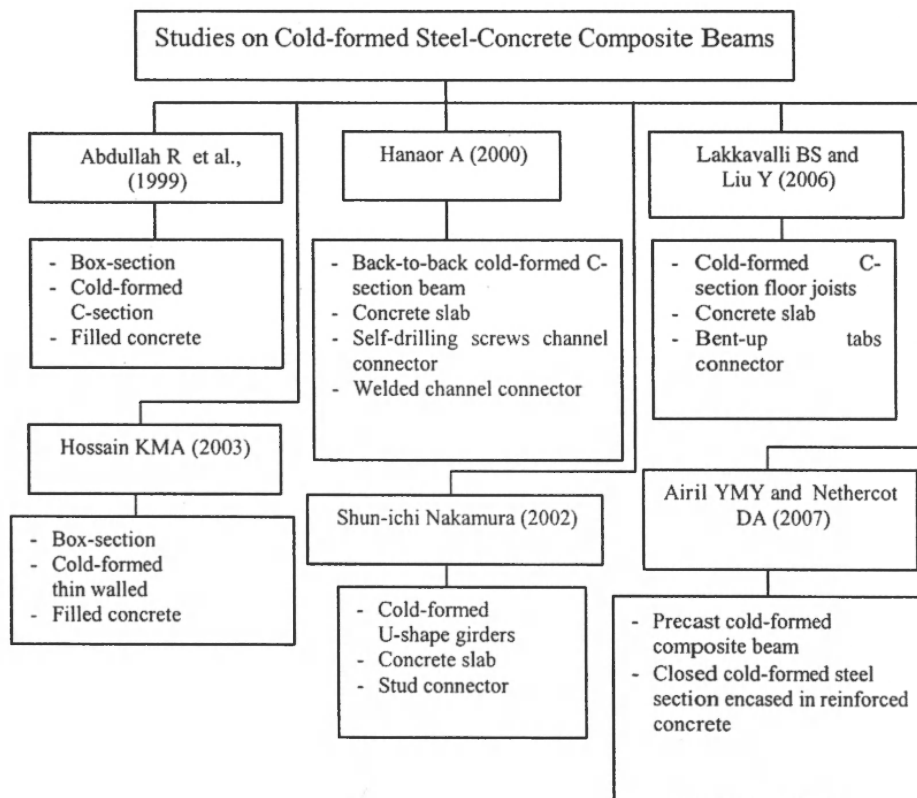


Figure 1.3. Previous studies on cold-formed steel-concrete composite beams

Some of hot-rolled section shapes usually used for joist, universal columns and universal beams are not available with CFS sections. This limits the structural use of CFS sections especially for long span structures and heavily loaded structures.

The main limiting feature of CFS is the thinness of its section that makes it susceptible to torsional, distortional, lateral-torsional, lateral-distortional and local buckling. The thinness of CFS is also inapplicable for CFS-concrete composite beam on the welding of shear studs [11][13]. Typical range of thickness for CFS sections is from 1.2 to 3.2 mm [7]. As stated in BS5950 [5], compression strength and buckling resistance of steel sections are influenced by the slenderness ratio ( $\lambda$ ).  $\lambda$  is equal to  $L_e / r$  where  $L_e$  is effective length and  $r$  is radius of gyration. Radius of gyration,  $r$  of a CFS section is small due to the thinness of the section. High value of  $\lambda$  can reduce the compression strength and buckling resistance.

It is clearly seen that the design of CFS and concrete composite structures has not been provided in any standard. Clause 1.1 of BS 5950: Part 3 recommended only the design of composite beams, comprising hot-rolled steel sections, plate girders and hollow sections. Clause 3.1 mentioned that in the design of a composite beam, the design strength,  $p_y$  for structural steel should not be taken as being more than 355 N/mm<sup>2</sup>. The limit of 355 N/mm<sup>2</sup> is due to lack of test evidence using higher-strength steel [20].

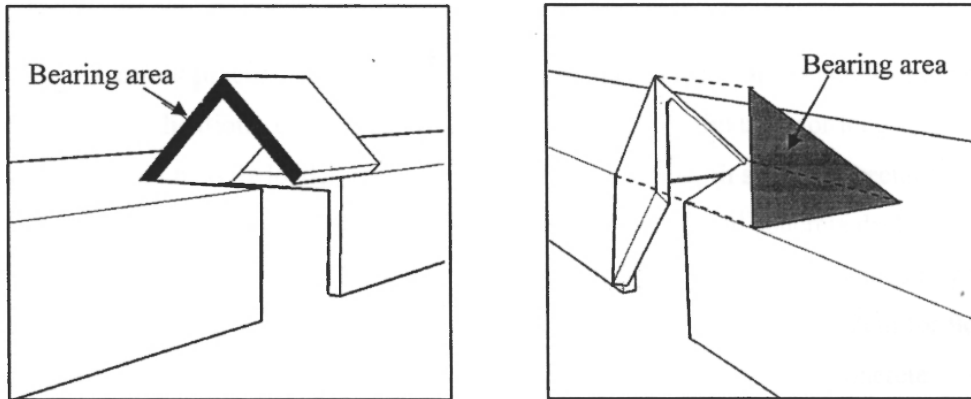
Previous studies showed that there is very little work and lack of technical literature such as codes of practices in CFS-concrete composite beams. As illustrated in Figure 1.3 and several studies as in [15]-[18] had pointed out that these studies only focused on the use of CFS for beam structures. Thus, these studies had limited discussion on the development processes of shear connector.

Hence, a reasonable solution is resorting to a composite construction of structural CFS section and reinforced concrete deck slab [8], which minimises the distance from the neutral-axis to the top of the deck and reduces the compressive bending stress in the CFS sections. Also, by arranging two CFS channel sections back-to-back restores symmetry and suppresses lateral-torsional and to a lesser extent, lateral-distortional buckling. These two-fold advantages promised by the system, promote the use of CFS sections in a wider range of structural applications.

There are several reasons for combining steel and concrete elements to form a composite member. Concrete has good sound and fire insulation properties. The applied flexural force in un-bonded system is therefore resisted solely by the CFS beam, with the top half being subjected to compression with the possibility of buckling and with the bottom half being in tension. When the CFS/concrete interface is bonded together, the two elements act as one, where CFS is subjected to tension and concrete element resists the compressive force. The depth of the CFS beam resisting flexure has now increased from that of the CFS beam acting by itself to that of the composite beam, and this can double both the flexural strength and stiffness of the beam and consequently lead to reduce span to depth ratio [19].

Therefore, an efficient and innovative floor system of built-up CFS sections acting compositely with a concrete deck slab was developed to provide an alternative composite system for floors and roofs in buildings. The system, called Precast Cold-Formed Steel-Concrete Composite System, is designed to rely on composite action between the CFS sections and a reinforced concrete deck where shear forces between them are effectively transmitted via another innovative shear transfer enhancement mechanism called a bent-up triangular tab shear transfer (BTTST). BTTST was proposed to give a solution on welding problem of shear stud on CFS.

The idea of this proposed precast CFS-concrete composite beam and the use of BTTST as shear transfer enhancement are based on the combination research done by Hanaor [11] and Lakkavalli and Liu [13]. Shear connector functions as the holder between the concrete slab and steel beam due to the inadequate bonding between concrete slab and steel beam. In addition, it also need to resist horizontal shear between concrete slab and steel beam where the bottom part of the shear connector transfers the horizontal shear and the top part of the shear connector holds the concrete slab from the steel beam. Lakkavalli and Liu [13] were using bent-up tab (LYLB) as shear transfer enhancement. Based on their observation, bearing area,  $A_b$  ofLYLB is small, where  $A_b$  forLYLB is equal to bent-up length multiplied with CFS thickness as show in Figure 1.14(a). BTTST was proposed in order to get better shear transfer enhancement due to the larger of  $A_b$  compared than  $A_b$  ofLYLB as show in Figure 1.14(b). It is expected that BTTST archives higher shear capacity as compared toLYLB.



(a) Bearing area of LYLB

(b) Bearing area of BTTST

Figure 1.14. Bearing area of shear transfer enhancement

The advantages of the CFS-concrete composite beams are [16]:

- i) The partial encasement of CFS eliminates any bearing problem due to load introduction.
- ii) The use of CFS instead of hot-rolled steel leads to a lighter beam section for a given ultimate performance since the former has smaller area for a given material performance as compared to the latter. Any disadvantages of using very thin and high-strength steel sections i.e. local buckling and bearing, have been largely removed by the concrete encasement
- iii) The prefabricated nature of the beam as well as the slab allows for rapid construction.
- iv) Theoretically, the CFS-concrete composite beam ensures better transfer of stress between individual units as compared with slim floor transverse reinforcement.

Research on the structural behaviour and performance of composite CFS and concrete structures is important to develop an expression in order to predict the shear capacity of innovative BTTST as the solution in predicting and designing composite CFS and concrete structures in addition of using the existing code, BS 5950: Part3: 1990.

### **1.3 Significance of the Research**

Current practise of cold-formed steel is mainly in roof trusses and host of non-structural application. Lack of studies in CFS especially in the used for long span structures and heavily loaded structures. Therefore, an in depth studies especially on composite construction of structural CFS section and reinforced concrete deck slab need to be formed. Studies on floor system of built-up CFS sections acting compositely with a concrete deck slab was developed to provide an alternative composite system for floors and roofs in buildings. The system, called Precast Cold-Formed Steel-Concrete Composite System. Application of precast is one of group of the Industrialised Building System (IBS).

The application of IBS is fully supported by the government. The government, through Construction Industry Development Board (CIDB), has promoted industrialisation and modernisation of our local design and construction industry and laid out the Industrialised Building System (IBS) Roadmap 2003-2010, evolving phased reduction on unskilled foreign workers and encouraging technologies, techniques and site-assembling-process of construction where construction elements prepared outside of a construction site are transported to the site, and later launched and locked into position [21].

Furthermore, a recently completed survey conducted by CIDB, called IBS Survey 2003, on the application of IBS by Malaysian contractors shows that the usage of IBS in local construction projects is only about 30% and most of the respondents cited only roof truss systems that have been used [6]. However IBS is fully supported by the government. In this case CIDB and Ministry of Public Work had started implementing this system and it will be fully used in 2015 [22], Based on previous researches, IBS managed to minimise 50% of manpower needed, assurance of work quality, limited duration of work, minimise cost and excess of building material. Besides that, it can protect the environment with cleaner site [23]. The Government had also announced to apply at least 50% of IBS technology in RMK-9 development projects [24], Based on the above advantages, the Prime Minister had suggested the use of IBS in low and medium

cost housing projects to ensure that the projects run smoothly and they produce quality houses [25].

This study also proposed innovative shear transfer enhancement mechanism called a bent-up triangular tab shear transfer (BTTST). The finding from this research may eventually lead to the development or improvement of the existing database on the welding problem of shear stud on CFS due to the thinness of CFS.

Therefore, this research is to investigate the possibility of using CFS-concrete composite beams for structures. It is expected that the outcome of this research will contribute in promoting the proposed shear transfer enhancement, BTTST for possible industry implementation and also the use of CFS as one of the alternative materials for building construction in Malaysia. The findings from this research will provide important technical knowledge which can be used as a design guideline for composite CFS and concrete structures. Consequently, the findings will encourage the use of local CFS structures.

#### **1.4 Objectives**

The overall goal of this research is to study the proposed shear transfer mechanisms (BTTST) in cold-formed steel (CFS)-concrete composite beam. The research is also aimed at providing a better understanding on the development processes and mathematical equation for BTTST. The equation established will be useful in determining the ultimate strength of CFS-concrete composite beam.

The five specific objectives of this study are:

1. To evaluate the mode of failures, shear capacities and effects of studied parameters of proposed innovative shear transfer enhancement, bent-up triangular tab shear transfer (BTTST) for CFS-concrete composite beams through experimental and analytical works.
2. To develop and produce a new equation which can be used to calculate the predicted shear capacity of BTTST in CFS-concrete composite beams.

3. To evaluate the structural behaviour namely, mode of failure, slip behaviour, deflection behaviour and stress distribution of CFS-concrete composite beams through complete composite beam tests and analysis.
4. To determine the flexural capacity of CFS-concrete composite beams through analytical and experimental works.
5. To establish and prepare standard tables of characteristic resistance of BTTST in normal weight concrete and design recommendation of CFS-concrete composite beam.

### **1.5 Scope and Methodology of Study**

A new type of composite beam comprising CFS sections embedded in precast concrete plank, called Precast Cold-Formed Steel-Concrete Composite System, is suggested. Three different configurations of precast composite beams are proposed, as shown in Figure 1.5, which show the incorporation of precast composite beams in the floor system. The slab, in Figure 1.5(a), is made of in-situ concrete, while Figure 1.5(b) and (c) used half-slab system. In the latter system, nibs are provided in the beam cross-section for the placement of the half-slab planks prior to the casting of concrete.

This study, however, only focuses on the strength capacity of the bent-up triangular tab shear transfer (BTTST) enhancement and the performance of CFS-concrete composite beams. Experimental work was carried out on small-scale and large-scale specimens in order to investigate the contribution of the proposed innovative shear transfer enhancement. The scope of the study covers two different topics on CFS-concrete composite action. The first topic is related to the performance of shear transfer enhancement and the second topic is related to the performance of CFS-concrete composite beams. There are four major parameters studied, which include compressive strength of concrete, CFS strength, dimensions (size and angle) of BTTST and CFS thickness.

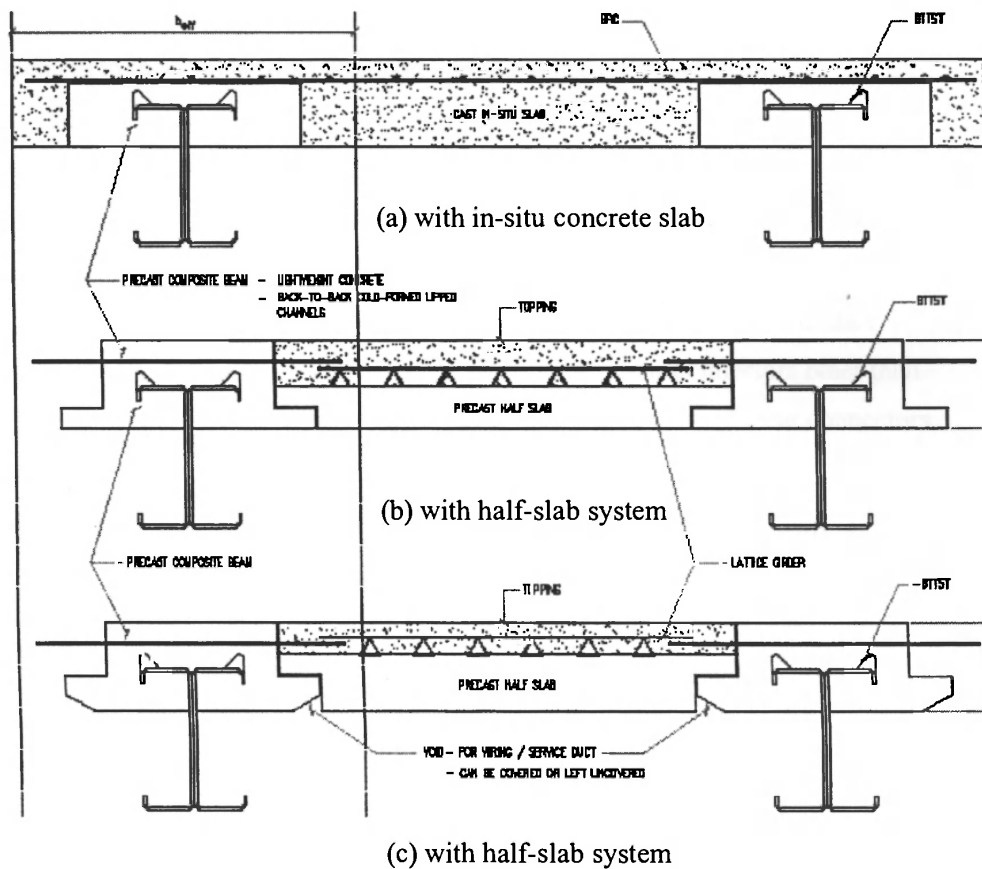


Figure 1.5. New types of cold-formed beams embedded in precast plank incorporation with floor system.

The first topic is related to the performance of shear transfer enhancement. Thus it was conducted to achieve the first objective. It involves 48 (Phase 1) and 20 (Phase 2) small-scale specimens. The studies are as follow:

1. Push-out test method was adopted to study the mode of failure, shear capacity, ductility and effect of difference parameters of innovative shear transfer enhancement mechanism, bent-up triangular tab shear transfer (BTTST) to the interface shear strength. The strength and ductility of shear connectors are always determined experimentally due to the complexity of the dowel action [19]. The behaviour of shear connectors must be determined from the test in which the connectors are loaded

directly. In this case, push-out test is suitable and the easiness of placement of specimens. Specimens are then loaded directly by applying a vertical downward displacement to the CFS section. The load applied to the CFS section is transmitted through the dowel action of the shear connectors in the concrete element. Shear connector strength, are normally obtained from this test, in which a load-slip curve is determined using a standard test arrangement. The detail arrangement is discussed in Chapter 3.

2. Clause 5.4.3 of BS 5950: Part 3 mentioned that since the characteristic resistance values are not presently given in the code for all types of shear connectors other than headed studs, therefore the characteristic resistance of other types of shear connectors should be determined from push-out test [20],
3. The beam section for the embedded connections consists of two lipped channels connected back-to-back by 6 mm diameter bolts. This CFS I-section beam formed by back-to-back lipped channels is used with the flanges cast into the concrete. The detail of the specimen description and parameters studied are discussed in Chapter 3.
4. Load-slip curves are obtained from push-out test. Later, based on the curves, the shear strength and ductility of shear transfer enhancement is determined.

Data from push-out tests will be used to develop a new shear capacity equation of BTTST in order to achieve the second objective. Regression analysis method was used to analyse data from push-out test for development of a new shear capacity equation of BTTST. The value of shear capacity is used to calculate the theoretical moment capacity of CFS-concrete composite beams, which is discussed in Chapter 6. The theoretical moment capacities are used to validate the large-scale test results.

The second topic conducted to achieve objective 3 and 4, involves a study on the structural behaviour of cold-formed steel (CFS)-concrete composite beams. The study is as follows:

1. CFS-concrete composite beams are physically tested and the results are used to evaluate the design methodology for this type of beams. CFS I-section beams formed by back-to-back lipped channels are used. There were 15 specimens tested. Detail of specimen description and parameters studied are discussed in Chapter 3.

2. The flexural test obtained is to validate the theoretical analysis based on push out test. These large-scale specimens were tested using four-point bending experimental work. The loading system consisted of two point loads produce region of constant moment between the loads. In this region, it can be assumed that at any cross section, the vertical shear is small and it can be neglected. Thus, the beam would be subjected only to pure bending moment. The purpose of the experimental work is to determine the ultimate strength of the CFS-concrete composite beams and also modes of failure. Results from these large-scale tests are used to verify the theoretical results obtained from the small-scale study.
3. The validity and accuracy of the design calculations and proposed equations in order to predict the shear capacity of BTTST were evaluated by comparing the results of theoretical analysis and the experimental results.

Finally, all of finding results was used to developed standard tables of characteristic resistance of BTTST in normal weight concrete and design recommendation of CFS-concrete composite beam. Figure 1.6 is the flowchart illustrating the scope and methodology of work for this study.

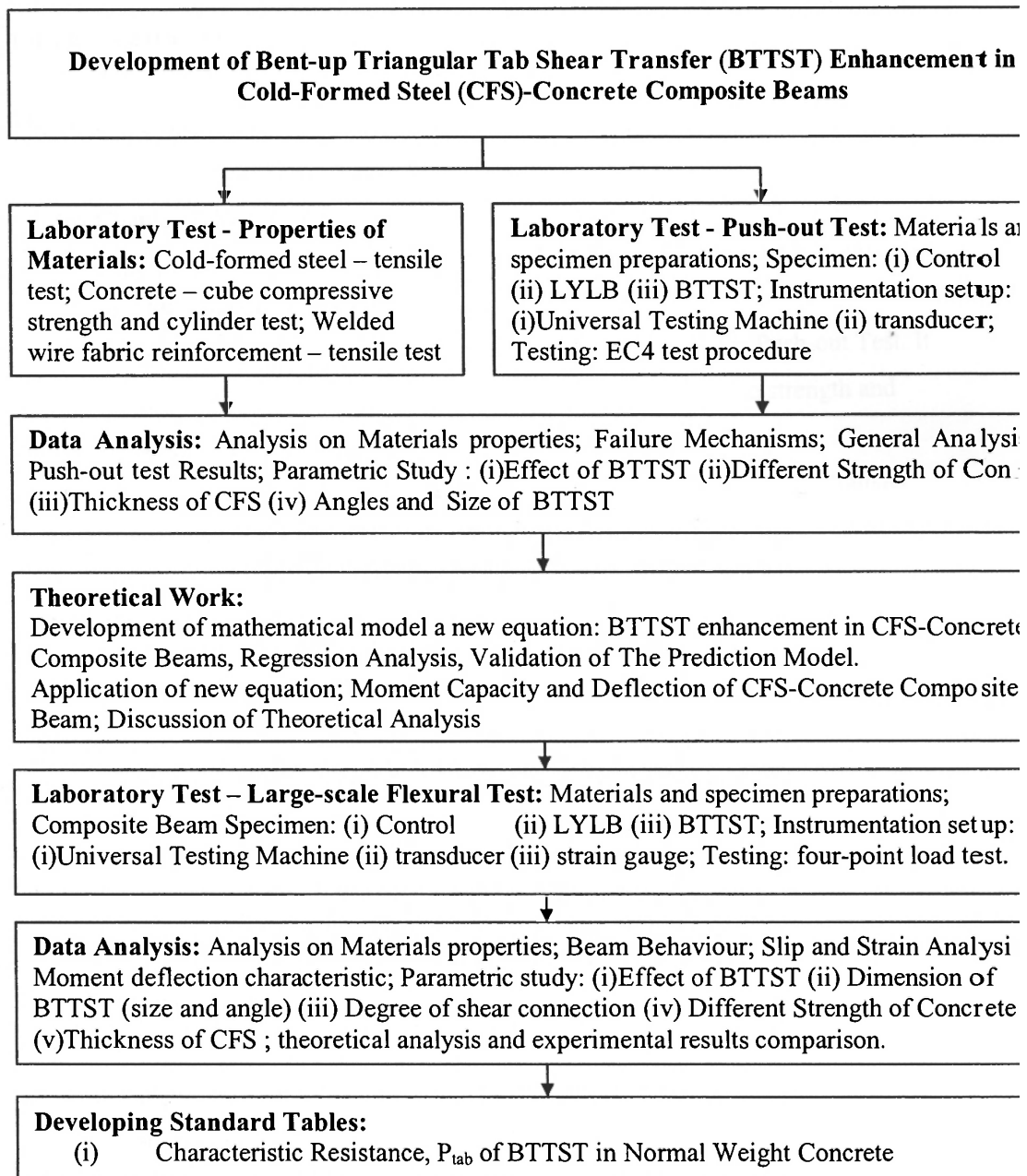


Figure 1.6. Scope and methodology of work for the study

## 1.6 Thesis Structure

The structure of this thesis is shown below:

- (a) Chapter 2 describes a review of the literature on the subject of this thesis.
- (b) Chapter 3 describes the specimen, test setup and instrumentation used in the experimental works for small-scale, push-out test and the large-scale flexural test of CFS-concrete composite beams.
- (c) Chapter 4 describes the results of the experimental works for Push-out Test. It includes analysis of the push-out test results and evaluates the strength and behaviour of a shear transfer enhancement.
- (d) Chapter 5 predicts analytically the shear capacity of proposed shear transfer enhancement in CFS-concrete composite beams, call Bent-up Triangular Tab Shear Transfer (BTTST). A new equation (Equation 5.7) has been developed for the design of shear capacity of BTTST in precast CFS-concrete composite beams. This equation includes all important parameters, i.e. compressive strength of concrete, modulus of elasticity, CFS strength, dimensions of BTTST ( $L_f$ ,  $L_s$ , angle) and CFS thickness.
- (e) Chapter 6 presents theoretical works on CFS-concrete composite beams, which includes the use of the new proposed equation as in Chapter 5, i.e. on how to calculate the shear capacity of BTTST and to determine the moment capacity of CFS-concrete composite beam.
- (f) Chapter 7 describes the results and analysis of the large-scale flexural test of CFS-concrete composite beams.
- (g) Chapter 8 presents the discussion and comparison of all the test results, conclusions and the recommendations for further works.

## CHAPTER 2

### LITERATURE REVIEW

#### 2.1 Cold-Formed Steel Structures

Cold-formed Steel (CFS) members are another type of steel structural members beside the more familiar and popular group of hot-rolled steel members. CFS members are the products made by bending a flat sheet of steel at room temperature into a shape that will support more loads than the flat sheet itself [1]. They are less familiar but tend to gain more popularity and importance in building construction throughout the years in future development. Generally, CFS members are the steel products manufactured from steel sheet, strip or plate into different types of desired shapes, by roll-forming machines, press brake or folding operations at room temperature. For all these manufacturing processes, no heat is required to form the shapes unlike hot-rolled steel which formed at elevated temperature, and thus it is called as CFS.

CFS sections are lightweight material and suitable for building construction owing to their high structural performance. Conventionally, they are used as purlins and side rails in the building envelopes of industrial buildings. The most common sections are lipped C and Z sections, and the thickness typically ranges from 1.2 to 3.2 mm, and sections with yield strengths at 280, 350 and 450 N/mm are commonly available [7]. Yu [26] mentioned that the thickness of cold-formed material ranges from 1.2 to about 6.4mm and the depth of members ranges from 51mm to 305 mm. In Malaysia manufacturing [27 - 29], the thickness is ranging from 1.5mm to 6.0mm and the depth is ranging from 50 mm to 254 mm. The sections yield strengths is a minimum of 250 N/mm<sup>2</sup> and up to 550 N/mm<sup>2</sup>.

During the years of 1850s, the use of CFS members in building construction began in both the United States and Great Britain. However, CFS members were not widely used in buildings construction until around 1940. At around 1930s, the acceptance and the development of CFS construction in the United States faced

difficulties due to the lack of an appropriate design specification. Various building codes made no provision for CFS construction at that time [26]

Meanwhile, the first documented research into the behaviour of CFS sections was carried out by George Winter at Cornell University in 1939. According to Ghersi et al., [30], Winter is a pioneer in this field. Since 1946, the use and development of CFS construction in United States have been accelerated by the issuance of various editions of the "Specification for the Design of Cold-Formed Steel Structural Members" of the American Iron and Steel Institute (AISI). The editions of the specification were based largely on the research sponsored by AISI at Cornell University, directed by George Winter in 1939 [26],

### **2.1.1 Advantages of CFS Sections**

CFS sections used in building construction have several advantages over hot-rolled steel sections and other construction materials, such as timber and concrete. These advantages helped to promote the use of CFS in the past, and also tend to further develop and increase the usage of CFS in the future. The major advantages of CFS sections are described below [26][31]:

i. Consistency and accuracy of profile:

Cold-formed sections can be formed to very close tolerances. The nature of the process is that when the desired profile is achieved then this can be maintained and repeated for as long as it is required. There is very little tool used and the cold rolling process is ideally suited to computerized operation which can assist in the maintenance of accuracy.

ii. Versatility of profile shape:

Cold rolling process can be used to produce almost any desired cross-sectional shape. The members can be designed from the viewpoint of efficiency, beauty, economy, or any combination of the above.

iii. Variety of materials :

Any material which is sufficiently ductile can be cold rolled. For structural use of steel, cold forming can be used with variety of steels, from low yield mild steel to high tensile steel.

iv. Pre-galvanized or pre-coated materials:

Cold-formed sections can be manufactured from pre-galvanized materials to ensure high resistance to corrosion and can obtain an attractive surface finish.

v. Variety of connections or jointing methods:

Cold-formed sections use most of the conventional connection methods, such as riveting, bolting, welding, and adhesive. Besides these, there are various special and new types of connections which have been developed to consider the thinness of CFS, such as lockseaming, press joint and rosette-joint.

vi. Increase in yield strength due to cold forming:

The cold forming process introduces local work hardening in the strip being formed in the vicinity of the formed corners. This local work hardening results in the yield strength and ultimate strength being increased at the corners. Due to the local increase in yield strength, the average yield strength of the formed section can be as much as 25% greater than the virgin yield strength of the flat strip. Consequently, this condition leads to the increasing load-carrying capacity of cold-formed sections.

vii. High strength to weight ratio:

Cold-formed sections are thinner and hence lighter than hot-rolled sections. This special feature contributes to the higher strength to weight ratio of CFS sections.

viii. Economic building material in terms of cost:

Reduction in CFS weight produces economies not only in the steel cost, but also in the costs of handling, transportation and erection. This is due to the ease of prefabrication and mass production and also easy erection and installation of CFS sections.

ix. Load-carrying panels and decks:

CFS sections used as load-carrying panels and decks can provide useful surfaces for floor, roof, and wall construction, and in other cases they can also provide

enclosed cells for electrical and other conduits. Besides that, they do not only withstand loads normal to their surfaces, but they can also act as shear diaphragms to resist forces in their own planes if they are adequately interconnected to each other and to supporting members,

x. Others benefits:

There are several other benefits of usage of CFS in building construction compared with other building materials such as timber and concrete. Some of the benefits include substantial elimination of delays due to weather, more accurate detailing, non-shrinking and non-creeping at ambient temperature, formwork unneeded, termite-proof and rot-proof, uniform quality and noncombustibility.

### **2.1.2 Cold-Formed Steel (CFS) Beams**

A beam is a structural member subjected to loads that acts laterally to the longitudinal axis. Lateral loads acting on a beam cause the beam to bend, or flex, thereby deforming the axis of the beam into a curve. When the beam is loaded, strain and stresses will occur due to bending moment and shear force.

The usual shapes of the cold-formed section used in beams structural are C-section with or without lips, Z-sections and I-sections. CFS I-sections are usually used in pairs so that higher moment stability and bearing performance can be acquired. In relation with this, symmetrical I-shape formed by pair of channels bolted back-to-back, as shown in Figure 2.1(a) are usually used in construction. Hsu *et al.*, [32] proposed the I-shaped formed by rolling thin-walled plates into the desired double symmetrical geometry and clamping the plate edges to form closed loops. This design philosophy can be achieved by forming the I-section with plate edges clamped at the flange location, as shown in Figure 2.1(b). However, this type of beam is difficult to form and not practical to be used in construction.

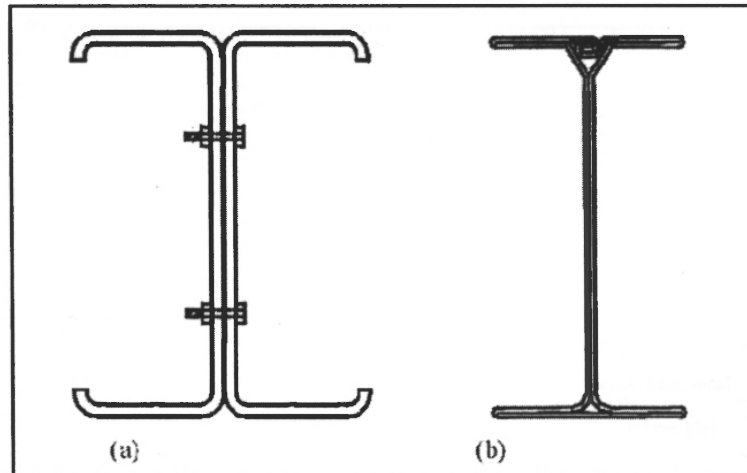


Figure 2.1. Symmetrical cold-formed I-sections: (a) two channels using bolting; (b) section using clamping [32]

### 2.1.3 Structural Behaviour - Modes of Failure Due To Bending Buckling

There are three basic buckling modes for CFS members such as local buckling, distortional buckling and lateral or flexural torsional buckling, as shown in Figure 2.2 [1][8][33]. Local buckling is a mode involving plate flexure alone without transverse deformation of the line or lines of intersection of adjoining plates. Distortional buckling is a mode of buckling involving change in cross-sectional shape excluding local buckling. While lateral-torsional buckling is a mode in which the bending member can bend or twist to the lateral from the plan view without change of cross-sectional shape. Davies [34] stated that there are four types of buckling modes, and the other one is shear buckling. Shear buckling may also be either local or global. Local shear buckling can occur in the slender webs of members subject to bending or in the wide flanges of sheeting and decking profiles subject to diaphragm action. Global shear buckling generally only arises when relatively lightweight sheeting or lining profiles are subject to diaphragm action.

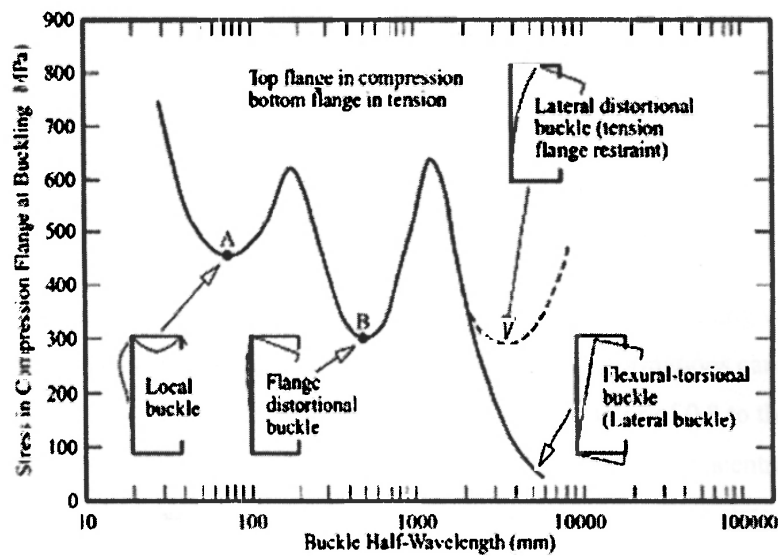


Figure 2.2. Buckling modes of lipped channel in bending [26]

### 2.1.3.1 Local buckling and post-buckling strength of thin plate elements

Since the individual components of CFS members are usually so thin with respect to their widths, these thin elements may buckle at stress levels less than the yield point if they are subjected to compression, shear, bending, or bearing. For the CFS I-beam, the thin flange which subjected to load is possible to buckle locally too. Local buckling of such elements is therefore one of the major design consideration.

However, the presence of local buckling of an element does not mean that its loading capacity has been reached when its buckling stress has already reached. But, they often will continue to carry increasing load in excess of that at which local buckling first appears. Hence, it possesses still greater strength, called post-buckling strength. Local buckling is expected in most cold-formed sections and often ensures greater economy than a heavier section that does not buckle locally.

According to BS5950: Part 5 [3], the formula for calculating the local buckling stress of element is given below:

$$p_{cr} = 18500K \left( \frac{t}{b} \right) \quad (2.1)$$

where  $K$  is the local buckling coefficient which depends on the element type, section geometry and is detailed for various cases in Appendix B of BS5950: Part 5 [3]. In Equation 2.1,  $t$  is the material thickness and  $b$  is the width of the element.

Research carried out by Dundu *et al.*, [35] shows that local buckling was made more critical by stress concentrations, shear lag and bearing deformations caused by back-to-back channels bolted connections. They proposed a factor of 0.8 to the yield moment and the buckling moment of resistance to account for stress concentrations, shear lag and bearing deformations of back-to-back channels. Design bending strength predicted using American Specification, Australian/New Zealand Standard and North American Specification are generally conservative for cold-formed stainless steel tubular sections subjected to bending [36].

Chu *et al.*, [37] investigated about the buckling behaviour of CFS channel section beams when subjected to uniformly distributed load. Their study focused on the local and distortional buckling, for which existing results are only for sections subjected to pure compression or pure bending. CFS member is commonly subjected to uniformly distributed transverse load. Results from their study showed that for local buckling, there is no practical difference in the critical loads between the two loading systems (pure bending and uniformly distributed transverse loading). However, for distortional buckling, significant differences exist. The critical load for uniformly distributed loading is generally higher than that for pure bending. For most practical cases the difference in the critical loads between uniformly distributed loading and pure bending is not negligible and it has been shown to decrease with the beam length.

Through computational and experimental study by Cheng and Benjamm [33], the developed testing plan and details have been shown to adequately restrict distortional buckling and provided a simple repeatable test that generates the local buckling flexural capacity for C and Z sections. Overall the test results indicate that AISI(1996), SI36 (1994) and the new NAS (2001) design methods provide adequate strength predictions. However, the overall agreement is slightly skewed by a number of quite conservative

predictions for non-slender members that had observable inelastic reserve capacity ( $M_{st} / M_y > 1$ ). Among the considered methods, the direct strength method provides the best test-to-predicted ratio for both slender and non-slender specimens. The Direct Strength method is predicated upon the idea that if an engineer determines all of the elastic instabilities for the gross section (i.e., local, distortional, and global buckling) and the load (or moment) that causes the section to yield, then the strength can be directly determined. The test results demonstrate that many improvements in the elastic buckling and effective width calculation of C's and Z's are still possible.

### 2.1.3.2 Lateral buckling and deflection of CFS beams

Lateral buckling, sometimes called lateral-torsional buckling, generally occurs when a beam which is bent about its major axis develops a tendency to displace laterally or vertically, for instance displace perpendicularly to the direction of loading, and twist, if braces are not adequately provided. This behaviour of an I-section beam is shown in Figure 2.3. In addition with this, the deflection history due to lateral-torsional buckling of the I-beam is shown in Figure 2.4. Based on the figure, the beam deflects in the y direction or displace vertically, at distance of v from its original location before the lateral buckling occurred. This type of vertical deflection is considered as in-plane bending. Later, whenever the buckling load reached, the beam buckles laterally, at a distance of u, and the section rotates about the center of rotation **C<sub>lb</sub>** at a degree of rotation,  $\theta$  [38].

Furthermore, most of the CFS beams are restrained against lateral movement, either by the other members connected to the beam or by bracing such as anti-sag bars. Such restraints reduce the potentiality of lateral buckling, but do not necessarily eliminate this problem [31]. In cases of members with continuous restraint the lateral torsional buckling capacity reaches a minimum value over a finite length of beam, whereas in the absence of restraint the buckling capacity decreases indefinitely with length. Most of the design standards for CFS provide criteria for the design of CFS I beams against their lateral buckling behaviour. For example, BS5950: Part 5 provides this design criteria in Section Five (Design of members subject to bending) [3].

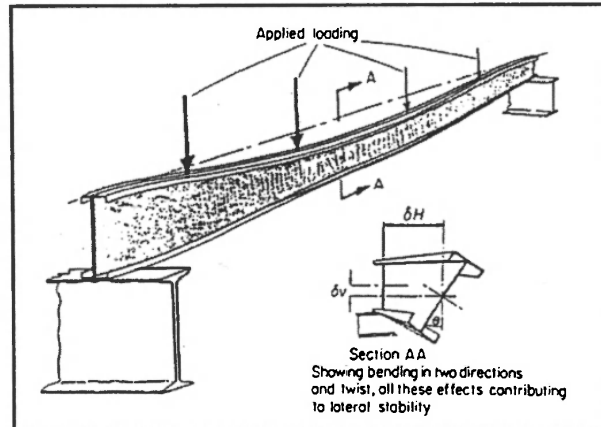


Figure 2.3. Lateral buckling of an I-section beam [31]

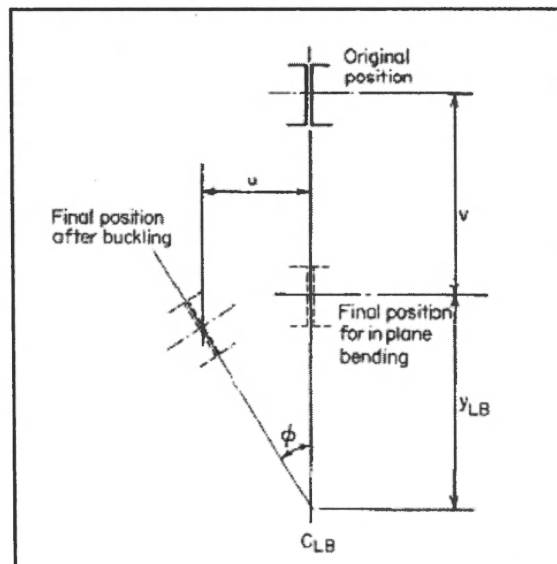


Figure 2.4. Deflection history of I beam due to lateral buckling [26]

In addition, the deflection of beams due to lateral buckling behaviour which tends to displace axially or laterally must conform to the deflection limit recommended in the design standard, such as in Table 2.1 based on BS5950: Part 5. According to BS5950: Part 5, when checking the deflections, the most adverse realistic combination and arrangement of unfactored loads should be assumed or its structure may be assumed to be

elastic. Furthermore, the calculation for deflection of beams under certain loading system should be based on Section 5.7 (BS5950: Part 5) [3].

Table 2.1. Deflection Limit of Beams Due To Unfactored Imposed Loads

Condition of Beams	Deflection Limit
Cantilevers	Length / 180
Beams carrying plaster or other brittle finish	Span / 360
All other beams	Span / 200

According to Put *et al.*, [39], the earliest tests on the lateral buckling of cold-formed channel section (CFC) were attributed to Cornell University (1941) by Hsiao *et al.*, (1988), but the details are not generally available. Winter *et al.*, (1949) carried out biaxial bending and torsion tests on CFCs loaded eccentrically through their webs. Hill (1954) tested four aluminum unlipped channels, but details of the loading and restraint conditions are unclear. Celebi *et al.*, (1972) reported tests on two unlipped CFCs loaded eccentrically through the web, while Lindner and Kurth (1980) reported approximately 160 tests on lipped and unlipped CFCs loaded eccentrically through the web or the centroid. Davies *et al.*, [40] referred to tests on lipped and unlipped CFCs by Lovell (1983) and Leach (1989), but again the details are not generally available. Djugash and Kalyanaraman (1990) tested three brake-pressed cold-formed sigma sections loaded concentrically at the levels of the top flange, shear center, and bottom flange, but full details were not given. Ellifritt *et al.*, (1991, 1992) tested 12 CFCs, but it appears that the failure loads acted eccentrically near the top flange-web junction.

Pi *et al.*, [41] studied about inelastic lateral buckling strengths and finite element model of cold-formed channel section (CFC) beams. The main purposes of the study were to develop a finite element model for the nonlinear inelastic analysis of cold-formed channel section (CFC) beams accounting the effects of web distortion and the asymmetrical nature of the yielded cross section, to investigated the elastic lateral-distortional buckling, inelastic behaviour, and lateral buckling strengths of CFC beams with residual stresses and initial imperfections, to develop improved design rules for the

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