Structural Performance of Connections in Hollow Structural Steel Modular Buildings

Jothiarun Dhanapal
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Structural Performance of Connections in Hollow Structural Steel Modular Buildings

by

Jothiarun Dhanapal

A Dissertation
Submitted to the Faculty of Graduate Studies through the Department of Civil and Environmental Engineering in Partial Fulfillment of the Requirements for the Degree of Doctor of Philosophy at the University of Windsor

Windsor, Ontario, Canada

2019

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Structural Performance of Connections in Hollow Structural Steel Modular Buildings

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DECLARATION OF CO-AUTHORSHIP AND PREVIOUS PUBLICATION

I. Co-Authorship

I hereby declare that this dissertation incorporates material that is the result of joint research undertaken with Dr. Hossein Ghaednia, of the University of Windsor, Mr. Jonathan Velocci, of the Z-Modular, and my supervisor, Dr. Sreekanta Das, of the University of Windsor. In all cases, the key ideas, the primary contributions, and data analysis and interpretation were performed by the author of this dissertation. The contributions of the co-authors were primarily focused on the provision of the study and suggesting possible directions. Results related to this research are reported in Chapters 2 through 4, inclusive.

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ABSTRACT

Steel modular buildings are on the rise to meet the growing infrastructure demand of big cities. VectorBloc modular construction involves a unique way of constructing steel modular buildings using steel modules made of hollow structural steel (HSS) members. This construction technique involves the use of a state-of-the-art cast-steel connector named VectorBloc connector for the beam-column connections of the steel modular buildings. The primary focus of this research is the VectorBloc beam-column connection. This beam-column connection is going to be used for constructing an assisted living facility in Ontario, Canada. Through full-scale tests and finite element analyses, this study shows that the VectorBloc connection has the ability to safely carry the design loads of the proposed assisted living facility. This study also proposes major design recommendations for the VectorBloc beam-column connection. The secondary focus of this research is the flow drilled connections made on the HSS members. This study discusses the influence of various connection parameters on the connection behavior and highlights the potential of the flow-drill system to increase the ultimate load capacity of the connections subjected to tension load. This study also presents analytical equations to predict the failure behavior of the flow drilled connections.
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CHAPTER 1

GENERAL INTRODUCTION

Modular construction involves completing a significant amount of construction work offsite in a factory compared to traditional onsite construction. Modular construction is often referred to as offsite construction and is defined as the manufacture and preassembly of elements, panels, and modules before installing them onsite [1]. The major advantage of modular construction practices is that it reduces a significant amount of onsite construction time and ensures better quality control. It was reported that modular construction techniques result in reduction and recycling of construction waste [2]. Thus, modular construction is an affordable, time efficient, and sustainable alternative to traditional construction practices. Modular construction was often adopted for low rise buildings. However, due to the growing infrastructure demands of mega cities, modular construction is increasingly adopted in high rise buildings [3]. Hence, the use of modular construction is on the rise in North America, Japan, and parts of Europe [4].

The term modular construction encompasses a wide range of construction procedures and involves the use of various materials such as steel, concrete, and timber. Among these, steel based modular construction is more popular especially in high rise buildings. The modular steel building concept employs varying degrees of modular technologies such as use of prefabricated structural elements, panels, and modules. Examples of steel based high rise modular buildings are 57 storeys high ‘J57 mini sky city’ in China [5] and 32 storeys high modular residential building constructed in New York, USA [6]. The steel modular construction can be classified as either light or heavy steel construction based on the type
of structural member used in the steel modules. Modular construction which uses light steel sections such as cold formed channel sections are called light steel modular construction. Whereas, the modular construction which use heavy steel sections such as hot rolled I-sections and structural tubes are called heavy steel modular construction. VectorBloc Corp. [7] developed a unique steel modular construction technique which involves steel modules made of hollow structural steel (HSS) structural members. This technique involves the use of a state-of-the-art cast steel connector named VectorBloc connector. The structural connections made in the HSS modules of VectorBloc modular construction are the focus of this research.

1.1 Summary of literature review

Modular steel construction is a relatively new and evolving field when compared with traditional steel construction. Thus, research on the steel modular construction especially the steel module based volumetric modular construction is limited. In this section a comprehensive review of steel based modular construction is presented with emphasis on modular connection techniques involving HSS members.

1.1.1 Steel modular construction

Steel modules are the building blocks of volumetric modular construction. They are manufactured in a factory and are fitted with all the electric and plumbing utilities. The finished modules are transported to the site and then installed vertically and horizontally to form the building space. Based on the structural design of the modules the load transfer between them occurs through its walls or through the corner columns. Lawson and Ogden [8] identified different types of steel modules and varying degrees of prefabrication used
in them. It was shown that ‘hybrid’ modular construction, which combines volumetric (3D) and panel (2D) modular construction, is more economical in medium-rise constructions.

Construction tolerances associated with steel modular construction are significantly different from traditional steel construction. The maximum tolerance for out-of-verticactivity of vertically staked modules in comparison with traditional steel frames was presented by Lawson and Richards [9]. Apart from installation procedures, the structural design of steel modular construction requires special consideration when compared with traditional steel frames. Lawson et al. [10] presented the structural design of modules in light steel based modular construction.

Annan et al. [11] highlighted the limitations of using light steel sections in modular steel buildings and proposed the use of heavy steel sections such as I-sections. The connection design and seismic performance of heavy steel modular buildings were presented by Annan et al. [11–14].

1.1.2 Modular steel connections

The success and performance of a steel based modular construction largely depend on the performance of its connections. The connections made in the modular steel buildings must have the ability to safely carry the design loads, as well, it should enable easier and precision installation of modules on site. Thus, a number of new connections were proposed by several researchers in recent years. Liu et al. [15] proposed a bolted-welded joint between a flanged column and a truss beam for prefabricated steel structures. Lee et al. [16] proposed a modular connection consisting of channel beams and tubular columns and studied its potential use as a rigid connection in modular steel buildings. Chen et al.
[17] proposed an interior connection in modular steel buildings made between steel modules made of HSS members. The research on modular steel construction using HSS sections for all the structural members is limited to this study. It is worth noting that in all these proposed modular connections, joints between the structural members are made directly through bolting and welds. Special structural connectors were not used between the beams and the columns in these connections.

Researchers at the University of Arizona at Tucson highlighted the limitations imposed by the shape of commonly available structural steel sections on the design of connections [18–20]. Thus, they developed a cast steel connector for use between hot rolled I-sections and presented its improved seismic performance. However, these connectors are developed for traditional steel moment frames.

1.1.3 Flow drilled connections

The bolted connections made on the HSS members are often made from only one side of the connection without the use of a nut. Special access holes are required on the HSS members to access the other side of the connection to connect nuts to the bolts. The bolted connections made from only one side of the connection are called single sided or one-sided connections. Some commonly used single sided bolting systems are flow-drill, Lindapter HolloBolt, blind bolt, and Huck ultra twist bolt.

Flow-drill system involves flow or friction drilling of screw holes on thin walled structural members. In the flow drilling process, a fast rotating drill bit displaces the work metal to form a bush (Figure 1.1). The resulting bush will have a thickness of two to four times the thickness of the work metal [21]. A special drill bit is used to form the threads in the flow
formed bush without cutting the bush metal. This process is called flow tapping. The flow-drill technique can be adopted for thin walled members of thicknesses up to 12.7 mm. Thus, the flow-drill system offers more number of threads for screw engagement in thin walled members when compared to standard drilling and tapping techniques.

![Socket head cap screw](image1)

**Figure 1.1:** Flow drilled hole with a socket head cap screw

Extensive research has been conducted on the mechanics of flow drilling process and on the application of flow drilling on different metals [21–26]. In recent years, interest in application of flow drilled screw connections in the automotive industry has increased. This is due to the potential of flow drilling techniques to replace the conventional connection techniques such as spot welding. Sonstabo et al. [27–30] studied the behavior of flow drilled screw connections between thin metal plates, usually used in the frame of automobiles. Macroscopic finite element models of these flow drilled connections were also developed. Graf et al. [31] presented macroscopic finite element models of flow drilled connections made between thin walled aluminium and steel plates. However, research on the application of the flow-drill system to structural steel connections is limited. France et
al. [32–34] conducted experimental studies on flow drilled connections between open section beams and tubular columns to demonstrate its application in simple and moment resisting connections in structural frames.

1.1.4 Connections on HSS members

The design philosophy of moment resisting connections made of HSS members is available in the design guidelines of CIDECT [35,36], AISC [37], and Eurocode 3 [38]. Fadden et al. studied the design of HSS-to-HSS moment connections subjected to seismic loads [39,40]. However, in these studies direct connections made between the HSS members are considered and thus no connector was used.

Guidelines are not readily available for the design of flow drilled screw connections subjected to tension and shear loads. However, some recommendations are provided in the CIDECT design guide 9 [35] regarding the selection of HSS wall thickness and the screw diameter for flow drilled connections. SCI P358 [41] provides the tension resistance of flow drilled connections made on two specific grades of hot rolled structural steel thicknesses. However, no information is available on the strength and failure behavior of flow drilled connections made on the HSS members that are commonly used in North America such as ASTM A500 [42] and A1085 [43] grade HSS members.
1.2 VectorBloc modular construction

As mentioned earlier, VectorBloc Corp. [7] developed a unique modular construction technology which involves the state-of-the-art cast steel connector named the VectorBloc connector and steel modules made of HSS members. The steel modules are fabricated in a factory and fitted with all the electric and plumbing utilities. The finished modules are then transported to the construction site and staked horizontally and vertically to form the building space. The structural frame of the VectorBloc construction is shown in Figure 1.2(a). A typical corner connection in this frame with the VectorBloc connector is shown in Figure 1.2(b). The VectorBloc connector with its primary components the upper and lower blocs is shown in Figure 1.2(c). The lower bloc connects the HSS top column to the HSS floor beams and the upper bloc connects the HSS ceiling beams to the HSS bottom column (Figure 1.2(b)). The connection between the HSS members and the blocs are made by all around full-penetration welds. During installation of the modules on site, a gusset plate is attached to the upper bloc using two flat head cap screws (FHCS) as shown in Figure 1.2(c). The gusset plate shape varies between L, T, and cruciform and it depends on the location of the connection in the building such as corner, edge, and interior, respectively. A registration pin is then connected to the lower bloc. This registration pin guides the top module to be properly positioned over the bottom module during vertical staking of the modules. The final connection between the lower and upper blocs are made through two high strength socket head cap screws (SHCS) where the threaded end of the SHCS connects to the threaded hole of the upper bloc (Figure 1.2(c)). Thus, the vertical connection between the upper and bottom modules are secured.
Figure 1.2: VectorBloc modular construction. (a) Structural frame. (b) Corner connection. (c) VectorBloc connector.
1.3 Objectives

The VectorBloc beam-column connection is going to be used for constructing an assisted living facility in Ontario, Canada. Thus, the primary objective of this research is to study the structural performance of the corner VectorBloc beam-column connection under the design loads of the assisted living facility. The design loads to be considered are axial compression load, axial tension load, uniaxial and biaxial bending loads, and combination of bending and axial loads. The secondary objective of this research is to study the structural behavior of single flow drilled connections made on HSS members under tension and shear loads. The other objectives of this research are to propose design recommendations for the corner VectorBloc beam-column connection and single flow drilled connections.

1.4 Methodology

The structural behavior of corner VectorBloc beam-column connection was studied through full-scale tests and finite element analyses. Six different load conditions such as axial compression, axial tension, uniaxial bending, biaxial bending, uniaxial bending with axial compression, and biaxial bending with axial compression were considered. One full-scale VectorBloc beam-column specimen was tested for each of these load conditions and thus, a total of six VectorBloc beam-column specimens were tested.

A unique test setup was designed and fabricated for testing the VectorBloc beam-column specimens under different loading conditions. The picture and schematic of this test setup with the specimen is shown in Figures 1.3(a) and (b). The test specimen was mounted between the supports through a ball-joint assembly provided at the ends of the columns as
shown in Figure 1.3(b). The ball-joint assembly provided at the top column was connected to load actuator 3 which was connected to the support. These ball-joints provided a pin-roller boundary condition at the column ends. The bottom column was restrained to translate in all directions. However, the top column end was restrained from translation in the x- and z-directions and was allowed to translate vertically (y-direction) with the movement of the load actuator. The test setup had three load actuators to apply the axial and bending loads on the connection. Actuators 1 and 2 applied bending loads on the floor beams and actuator 3 applied axial loads on the column (Figure 1.3(b)). Linear variable differential transducers (LVDT) were used to measure the bending deflection of the floor beams and axial deformation of the columns. A number of strain gauges were attached on the specimen at different locations where high strain concentrations were anticipated.

![Figure 1.3: VectorBloc connection test setup. (a) Photo. (b) Schematic.](image)
VectorBloc connection specimens without the floor and ceiling beams were fabricated and tested under the axial tension and compression loads. These specimens were called VectorBloc column specimens and different test setups were used to test these specimens. The digital image correlation technique was used in some of these tests to measure the displacements of the specimen components. Apart from the connection tests, coupon tests were also conducted to determine the mechanical properties of the VectorBloc connection components.

Three dimensional nonlinear finite element (FE) models of the VectorBloc connection were developed in commercially available finite element code, ABAQUS/Standard version 6.13, distributed by SIMULIA Inc. [44]. These models adequately considered the material, geometric, and contact nonlinearities. The adequacy of the element density in the developed FE models was checked through a mesh convergence study. The developed FE models were validated using the test results. A comprehensive parametric study was conducted using the validated FE models to study the influence of different connection parameters such as VectorBloc connector weight, HSS member geometry, and location of the SHCS.

The structural behavior of the flow drilled connections made on the HSS members was studied through experimental tests and analytical analyses. Fifteen different connections were considered in this study by varying four different parameters. These parameters are HSS geometry, screw hole drilling technique, tapping technique, and number of threads available per unit length of the screw. The connections were tested under tensile and shear loads. Five specimens were fabricated and tested for each connection. Thus, 150 specimens (15 connections × 5 specimens × 2 types of tests) were tested in this study. The test
12 specimens were designed to apply the desired tension and shear loads on the connections. The test specimens for the tension and shear tests along with the test setup are shown in Figures 1.4(a) and (b). Analytical equations were presented to determine the failure behavior and the connection capacity under tension and shear loads.

![Figure 1.4: Test setup. (a) Tension test. (b) Shear test.](image)

1.5 Organization of the dissertation

This dissertation consists of five chapters. The first chapter provides the general introduction with a comprehensive literature review and the fifth chapter presents the discussions and conclusions of this research.

The second chapter presents the behavior of corner VectorBloc connection under axial compression and tension loads. This chapter presents two major design recommendations
regarding VectorBloc connector weight and SHCS location to improve the performance of the VectorBloc connection under axial loads.

The third chapter presents the behavior of the VectorBloc connection under bending loads. This chapter discusses how different connection parameters influence the bending behavior of the connection. This chapter also proposes some key design recommendations.

The fourth chapter presents the behavior of single flow drilled connections under tension and shear loads. This chapter discusses different failure modes of the flow drilled connections and provides analytical equations to predict the failure behavior of the connections.

1.6 References


CHAPTER 2

STRUCTURAL PERFORMANCE OF STATE-OF-THE-ART VECTORBLOC MODULAR CONNECTOR UNDER AXIAL LOADS

2.1 Introduction

Modular construction is often referred to as offsite construction and defined as the manufacture and preassembly of elements, panels, and modules offsite in a factory before installing them onsite [1]. This construction method industrialises the construction sector to provide an efficient, a much cheaper, and a greener alternative to traditional onsite construction methods. Modular construction also offers a better quality control. Hence, the use of modular construction is on the rise in North America, Japan, and part of Europe [2,3].

Steel structural element based modular constructions are more popular in high-rise buildings and it employs varying degrees of modular technologies such as prefabricated structural elements, panels, and modules. Various light steel structural modular constructions were presented by Lawson and Ogden [4,5]. Annan et al. [6,7] highlighted the limitations of light steel based modular construction and proposed the use of high strength steel sections, such as flanged I-sections, in modular steel buildings (MSB). The success and performance of a steel modular construction largely depends on the design of the connectors or connections that are used to stack various modules and also to build a module. As a result, a number of new modular connection techniques were proposed by
researchers. Liu et al.[8] developed a new bolted-welded joint between flanged column and truss beam in modularised prefabricated steel structure. Lee et al. [9] proposed a bolted connection between the modules involving ceiling bracket and presented its seismic performance. Chen et al. [10] presented an interior connection of MSB where inter modular connection was established by bolted connections. Researchers at the University of Arizona at Tucson studied a cast-steel connector for beam-column connections in traditional onsite steel construction which uses hot-rolled flanged I-sections [11–13].

Research on steel modular construction using steel tubular sections or hollow structural steel (HSS) sections has been limited to only one recent study [10]. In this study, HSS sections are directly connected to each other using bolts. VectorBloc Corp.[14] has been developing a very different and efficient technique for constructing HSS element based modular buildings using their innovative cast-steel connector called VectorBloc connectors. These connectors are connected to the HSS members by welding. The aim of this construction approach is to build high precision steel modular buildings using HSS sections only. The steel modules in this construction are manufactured and subsequently fitted with all the electrical and plumbing utilities in the factory or fabrication plant. The complete modules are then transported for installation in the field. The novelty of the VectorBloc connector is that it accommodates both beam-column and inter modular connections and enables faster and easier onsite installation. However, the VectorBloc connector has a complex geometry varying in all the three dimensional axes. Hence, it is imperative to study the structural behavior of the VectorBloc connector under various load combinations to ensure its safe structural performance under service limit state as well as under ultimate limit state loadings.
In this research, the performance of a typical corner vertical connection made of VectorBloc connector, subjected to axial tension and axial compression loads was studied experimentally and numerically. The connection studied in this research will soon be used in an eight-storey high assisted living facility located near Cambridge, ON, Canada. Hence, the primary objective of this research was to determine the structural behavior and structural performance of various components of a typical corner connection of the proposed assisted living facility. This study also completed a comprehensive parametric study for possible improvements in the VectorBloc connector.

### 2.2 VectorBloc connection

The HSS modular framing system being developed by VectorBloc Corp. is shown in Figure 2.1(a). In this system, the modules are stacked horizontally and vertically and the modules are connected at the corners through VectorBloc connectors which is also referred to as bloc or bloc connector (Figure 2.1(b)). The VectorBloc or bloc connector and its components are shown in Figure 2.1(c). The primary components of the connector are the upper and lower blocs. The upper bloc connects the ceiling beams to the column whereas the lower bloc connects the column to the floor beams as shown in Figure 2.1(b). All the column, ceiling, and floor beams are made of HSS sections. The connection between the blocs and the HSS sections are made by all around full-penetration welds. When the modules are installed on site, a gusset plate is attached to the upper bloc using flat head cap screws (FHCS) as shown in Figure 2.1(c). The location of the connection in the frame such as corner, edge, and interior controls the shape of the gusset plate such as L, T, and cruciform, respectively. The gusset plate extends over and connects to the upper blocs of horizontally adjacent modules thereby establishing horizontal connection between the
modules. Then a registration pin is connected to the upper bloc as shown in Figure 2.1(c). This pin helps in aligning the top module over the bottom module in the vertical staking of modules. After proper alignment of the top and bottom modules the upper and lower blocs are connected by means of two high strength socket head cap screws (SHCS) as shown in Figure 2.1(c) where, the threaded end of a SHCS connects to the threaded hole of the upper bloc. Thus, the vertical connection between the modules is established and properly secured.
Figure 2.1: Details of VectorBloc modular construction. (a) Framing system. (b) Corner connection. (c) VectorBloc connector.
2.3 Test specimen and setup

2.3.1 VectorBloc beam-column specimens

Two identical full-scale (beam-column) connections (specimens) were built and tested. These specimens represent a typical corner connection of a residential building. One specimen was subjected to axial compression load and other one was subjected to axial tension (Figure 2.2). The connection between the blocs and the HSS sections were made by all around full-penetration welds. A 12.7 mm (0.5 inch) thick gusset plate was fastened to the upper bloc using two 19.05 mm (0.75 inch) diameter FHCSs and then a 50.8 mm (2 inch) diameter registration pin was connected to the upper bloc for the assembly of the specimen. Final connection between the upper and lower blocs was made using two 25.4 mm (1 inch) diameter SHCS. The schematic of the VectorBloc connector with its outer dimensions is presented in Figure 2.3. Table 2.1 presents the geometric details of the HSS sections used in these two specimens and Table 2.2 presents the material properties of various components obtained from coupon tests [15].

The test setup was designed and built for the application of design axial compression and axial tension loads of a typical corner beam-column connection in the proposed eight-storey high assisted living facility. A pin-roller boundary conditions at the column ends was provided. The end of the bottom column was restrained to translate in all the three x-, y- and z-axes (Figure 2.2(b)). The end of the top column was allowed to translate with the loading jack movement in y-axis but it was restrained to translate in x-and z- axes.
Figure 2.2: Test setup of beam-column corner connection. (a) Photo of test setup. (b) Test setup schematic.

Figure 2.3: Dimensions of lower and upper blocs ([inch] mm).
Table 2.1 Geometric properties of the HSS sections

<table>
<thead>
<tr>
<th>HSS section</th>
<th>Width× Depth× Thickness mm × mm × mm (inch × inch × inch)</th>
<th>Length mm (inch)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Floor beam</td>
<td>76.2×203.2×12.7 (3×8×0.5)</td>
<td>1270 (50)</td>
</tr>
<tr>
<td>Ceiling beam</td>
<td>76.2×76.2×9.5 (3×3×0.375)</td>
<td>1219.2 (48)</td>
</tr>
<tr>
<td>Column</td>
<td>101.6×101.6×9.5 (4×4×0.375)</td>
<td>457.2 (18)</td>
</tr>
</tbody>
</table>

Table 2.2 Mechanical properties of the specimen components

<table>
<thead>
<tr>
<th>Component</th>
<th>Young’s modulus (GPa)</th>
<th>Yield strength (MPa)</th>
<th>Ultimate strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Upper and lower blocs</td>
<td>197.5</td>
<td>336.5</td>
<td>558.6</td>
</tr>
<tr>
<td>Gusset plate</td>
<td>197.2</td>
<td>355.3</td>
<td>554.9</td>
</tr>
<tr>
<td>SHCS</td>
<td>203</td>
<td>1055.2</td>
<td>1290.9</td>
</tr>
<tr>
<td>HSS sections</td>
<td>200</td>
<td>359.3</td>
<td>552.9</td>
</tr>
</tbody>
</table>

Two linear variable differential transducers (LVDT) were connected to the specimen as shown in Figure 2.4. LVDT 1 was connected between the top column and the lower bloc and LVDT 2 was connected between the upper bloc and the bottom column. These two LVDTs measured the relative movement of the sections to which they were connected. As shown in Figure 2.5 a total of 22 strain gauges were attached at various locations of the specimen where high strain concentrations were anticipated. It should be noted that strain gauge numbers 21 and 22 were situated on the shank of the SHCS 1 and SHCS 2, respectively as shown in Figure 2.5(c).
Figure 2.4: Location of LVDTs 1 and 2.
Figure 2.5: Strain gauge layout (mm). (a) Elevation. (b) Plan. (c) SHCS 1,2.
2.3.2 VectorBloc column specimens

Full-scale (beam-column) specimens were tested under factored design loads and no visible failure occurred in those test specimens. Hence, the finite element (FE) model was developed and validated using these test results. The FE analyses indicated that this connection is expected to fail at loads higher than the design loads applied to the full-scale beam-column specimens. The FE analyses showed that under axial compression, the failure is expected to occur in the columns due to inelastic global buckling followed by local buckling of column walls. However, in axial tension, the SHCS are expected to fail by rupture. The full-scale beam-column test setup (Figure 2.2) did not have the capacity to apply these failure loads. Hence, additional four specimens of VectorBloc connector with top and bottom columns were fabricated and tested in axial compression and tension. These specimens did not have any beams attached and these four specimens are called column specimens (Figures 2.6 and 2.7). The lengths of the columns in these column specimens are 457.2 mm (18 inch) and 215.9 mm (8.5 inch).

The test setup with the column specimen tested in axial compression is shown in the Figure 2.6(a) and the schematic of the test specimen is shown in Figure 2.6(b). The test specimen was supported through the top and bottom support plates and this created a fixed support condition at both the top and bottom column ends as shown in Figure 2.6(a). The loading actuator used in the compression test as shown in this figure has a capacity of 3000 kN. The LVDT attached to this loading actuator measured axial deformation of the specimen. Two strain gauges were attached on the specimen at locations corresponding to numbers 3 and 8 in Figure 2.5(a) where maximum strains were anticipated. It can be observed in this figure that strain gauge number 3 was on the lower bloc and 8 was on the HSS column.
The axial tension tests on column specimens were conducted in a universal testing frame which has the axial load capacity of ±800 kN. The test setup along with the test specimen for axial tension tests is shown in Figure 2.7(a). The schematic of the test specimen is shown in Figure 2.7(b). The full-scale (beam-column) test specimen in axial tension showed a significant separation between upper and lower blocs which was difficult to measure because of the complexity in the test setup. Hence, digital image correlation (DIC)

Figure 2.6: Test setup of column connection in compression. (a) Test setup photo. (b) Test specimen schematic.
technique was used in column specimens to measure the separation between the upper and lower blocs under axial tensile loading.

2.4 Test procedure and test results

2.4.1 VectorBloc beam-column specimens

The design axial compression and axial tension loads for a typical corner beam-column connection in the assisted living facility are 400 kN and 200 kN, respectively. These loads were applied by an universal loading jack through the top column as shown in Figure 2.2(b). The load control method was used for the application of the loads. The load data was acquired through a universal loadcell attached to this loading jack. The deformation data were acquired through LVDTs attached (Figure 2.2).

Figure 2.7: Test setup of column specimen in tension. (a) Test setup. (b) Test specimen schematic.
The first specimen was tested in axial compression and maximum load (design load) applied to this specimen was 400 kN. The axial load-deformation behavior of this specimen is presented in Figure 2.8(a). This figure also shows axial load-deformation behavior obtained from finite element analysis (FEA) and this will be discussed later. It can be observed from this figure that the behavior of the connection under the applied load is linear. The difference in cross-sectional areas and geometries between the columns and the blocs created an eccentricity of the resultant load at the lower bloc-gusset plate contact interface. Thus, the columns and the blocs cross sections were subjected to axial compression with bending. The associated non-uniform compressive deformation of the column and the bloc cross-section resulted in a small relative rotation of the upper and lower blocs about the axis shown in Figure 2.9. This relative rotation due to non-uniform cross sectional deformation of the columns and blocs was successfully resisted by SHCS.

The strain data recorded at the maximum axial compression load of 400 kN is presented in Figure 2.8(b). It can be observed from this figure that the specimen locations where the strain readings were obtained remained within the elastic limit of 2000 micro strains. It is also observed that the strain values obtained from strain gauges 8 and 10 which were located on columns were much higher than the strain values recorded from other strain gauges located on the blocs (Figure 2.8(c)). This is due to relatively smaller wall thickness of the columns compared to the thickness of the bloc (connector) sections. The strain values on the floor beams obtained from strain gauges 14-16, 19, and 20 are also shown in Figure 2.8(b). It is found that these strain values are negligible which indicates that the beams were not engaged in sharing the loads.
Figure 2.8: Axial compression test results. (a) Axial deformation. (b) Strain data. (c) Strain variation across the specimen.
The second beam-column specimen was tested in axial tension and maximum load (design load) applied was 200 kN. Figure 2.10(a) presents the axial load-deformation behavior of specimen 2. In this figure, load-deformation behavior obtained from FEA is also presented. It can be observed from this figure that the behavior of the VectorBloc connection is linear under the applied load which is the design load of the assisted living facility. In axial tension the loads are primarily resisted by the SHCS. There was a relative rotation between the upper and lower blocs about the axis shown in Figure 2.9 due to the eccentricity of the SHCS with the axis of the applied load. This rotation caused separation between blocs on one side of the axis and bearing between the blocs on the other side of the axis as shown in Figure 2.9.

The strain readings obtained from the strain gauges at the axial tension load of 200 kN is presented in Figure 2.10(b). It can be observed that the strain values are much higher at strain gauges 21 and 22 than the strain values recorded from other strain gauges. These two strain gauges (21 and 22) were located on the shank of the SHCS 1 and SHCS 2,
respectively. The eccentricity of the SHCS with the axial load results in the non-uniform tensile strain around the shank. Hence, for the same distance from the head of the SHCS the strain data measured by the strain gauges depends on the orientation of the strain gauges in plan (xz-plane in Figure 2.5(b)). This difference in plan orientation of strain gauges 21 and 22 caused the difference in strain values recorded by them. It can also be noted that the strain values at other components of the specimen are much smaller. All the recorded strain values are within the elastic limit of 2000 micro strains. However, the strain value recorded by strain gauge 22 is 1757 micro strains and is very close to the yield strain of 2000 micro strains. This indicates in the axial tension the SHCS are the critical components.
Figure 2.10: Axial tension test results. (a) Axial deformation. (b) Strain data.
2.4.2 VectorBloc column specimens

Axial compression tests were conducted on the two identical column specimens. The compression load was applied in displacement control and the specimens were loaded until a significant drop in load was observed after reaching the ultimate load. The load-deformation behavior of the specimens is presented in Figure 2.11(a). It can be observed in this figure that the maximum load reached in both the specimens are nearly same at about 1830 kN. In specimen 1 the test was discontinued when the load capacity dropped to 1396 kN. However, for specimen 2, the test was continued until the load capacity dropped to 1636 kN. Figure 2.11(b) shows the strain values measured in specimen 1 at the top column and lower bloc. This figure shows at the ultimate load the rate of increase in strain is much higher in the top column compared to that of the lower bloc. Thus, in both the specimens inelastic global buckling of the columns was observed indicating that if the test was continued the columns might locally buckle. Deformed Specimen 1 is shown in Figure 2.12 where the buckling of the column can be observed.
Figure 2.11: Axial compression test results obtained from column specimens. 
(a) Load-deformation behavior. (b) Strain data.
Two identical column specimens were tested in axial tension. The axial tensile load was applied using displacement control method and the loading was applied until connection failed in rupture of the SHCS. The eccentricity of the SHCS caused the blocs to rotate relative to one another causing bloc separation on one side. This bloc separation increased with the increase in the applied tension load and this was accompanied by the deformation of the SHCS and local deformation of the lower bloc and gusset plate interface on the bearing side. The bloc separation under applied load was obtained from the DIC data as shown in Figure 2.13. The bloc separation versus the applied axial tension load for specimens 1 and 2 are presented in Figure 2.14. In specimen 1, the SHCS 1 failed suddenly in rupture followed by the rupture failure of SHCS 2. However, both SHCS in specimen 2 failed simultaneously in sudden rupture. As the SHCS ruptured, the load capacity of the connection suddenly dropped to zero. The maximum load applied to the specimens 1 and

**Figure 2.12:** Compression column specimen 1 after failure.
2 are 516 kN and 526 kN, respectively. The SHCS of specimen 1 after failure is shown in Figure 2.15(a) and the local crushing deformation of the gusset plate is shown in Figure 2.15(b).

**Figure 2.13:** Bloc separation of specimen 1 at 516 kN from DIC analysis.
Figure 2.14: Bloc separation with applied tension loads.

Figure 2.15: Tension column specimen 1 after failure. (a) SHCS. (b) Local crushing of gusset plate.
2.5 Finite element model development and validation

The physical testing of the specimens provided key information about the axial behavior of the VectorBloc connection and the assembly. However, the physical tests could not provide many important information such as region of maximum stress, stress variation (stress contour) across the specimen, and load level for plastic strain initiation. Hence, three-dimensional finite element (FE) models of beam-column and column specimens tested in this study were developed using commercially available general-purpose FE code, ABAQUS/Standard version 6.13, distributed by SIMULIA Inc.[16] to fully understand the structural behavior of the connection under axial loads. In these FE models, all three nonlinearities namely, material, geometric, and contact nonlinearity were modeled. Elastic-plastic material model with isotropic strain hardening was adopted in these FE models. The element density in these models were decided based on mesh convergence study conducted.

2.5.1 VectorBloc beam-column model

The beam-column specimens were modelled using continuum elements C3D10 and C3D20R [16]. The welded joints in the connection were modelled using surface-to-surface tie constraints. Tie constrains were also used to model the screw connection between the SHCS and upper bloc and connection between gusset plate and upper bloc. The interfaces in the specimen where contact occurs were modelled as surface-to-surface frictionless hard contact interactions. The ends of the columns were kinematically coupled to points representing the centers of the column end fixture and support conditions were applied to these points.
The FE model was first analysed in axial compression load of 400 kN. The axial load-deformation behavior obtained from the finite element analysis (FEA) and the tests are
presented in Figure 2.8(a). The strain data obtained from the FEA results corresponding to strain data obtained from the test is presented in Figure 2.8(b). It can be observed from Figures 2.8(a) and (b) that the FEA results are in good agreement with the test results. von Mises stress distribution in the model at the maximum compressive load is shown in Figure 2.16. It can be observed from this figure that the maximum stress occurs at the columns. At the maximum applied compressive load the equivalent plastic strain measure (PEEQ) is zero. Thus, the FE analysis confirmed that the beam-column specimen under axial load of 400 kN remained well within the elastic limit and the failure would probably occur at the column if the load was increased well beyond 400 kN.

The FE model was also analysed in axial tension load and a maximum load of 200 kN was applied. The axial load-deformation behavior obtained from the FEA is compared with the test data in Figure 2.10(a). The strain data at the maximum applied tension load of 200 kN is presented in Figure 2.10(b). From these figures, it can be found that the FEA results agree well with the test results. Figure 2.17 shows the von Mises stress distribution in the SHCS at the maximum applied load of 200 kN. This figure shows that the maximum stress occurred at the section representing the threaded region of the screws. The PEEQ measure is zero on the SHCS at the maximum load of 200 kN and hence, FEA results shows that the SHCS remained within the elastic limit under the applied (design) tensile load and the specimen would probably fail due to rupture in the SHCS if the axial tension load was increased.

2.5.2 VectorBloc column model

The column specimen tested in axial tension was modelled using quadratic tetrahedral elements of type C3D10 [16]. The welded connections of the specimen were modelled
using surface-to-surface tie constraints. The contact interfaces of the specimen were modelled as surface-to-surface frictionless hard contact interactions. The FE model developed in this study is shown in Figure 2.18(a). Figure 2.14 presents bloc separation data obtained from this FE analysis and comparison with the data obtained from the test specimens using DIC. It can be observed from this figure that the model simulated the behavior of the specimen with reasonably good accuracy. From the FEA results it can be found that initiation of plastic strain in the SHCS occurred at 215 kN. The maximum PEEQ measure on SHCS in the FEA results corresponding to failure loads (516 kN and 526 kN) in the test specimens 1 and 2 are 17.9% and 22.2%, respectively. Figure 2.18(b) shows the distribution of PEEQ in the socket head cap screws (SHCS) of specimen 1 when this specimen fails. The column specimens tested in axial compression had a fixed end boundary condition. A FE model was developed with fixed end boundary condition and validated using the test data. The FE model showed the compression load capacity is 1822 kN which is very close to the test value of 1830 kN. The boundary condition of the FE model was changed to pin-roller and the maximum load capacity was found to be 1439 kN.

The validated FE model was then used for the parametric study and the parameters chosen are the locations (positions) of SHCS and the weight of the VectorBloc connector. These two parameters affect the structural performance and cost of the connector or the connection. It was observed from the test and FEA results that in axial tension the SHCS fail in rupture due to the combined action of axial and bending stresses. Thus, location of the SHCS plays a significant role in determining the connection stiffness and capacity. Also, reducing the weight of the blocs reduces the material cost of the connector. Hence, weight reduction was considered and it was changed by changing the thickness of the blocs.
The failure of SHCS in FE models were detected by tracking the PEEQ. The lowest failure load obtained from the tests is 516 kN. However, in the FE model, the failure load was conservatively considered at 490 kN which is 95% of 516 kN. The FE models shows the value of PEEQ at 490 kN is 8.8%. Hence, PEEQ value of 8.8% was considered as the failure in the SHCS in the FE models used in the parametric study. A similar approach for identifying a rupture in FE models was proposed by Das et al [17] and subsequently used by other researchers [18–21].

2.6 Parametric study

The structural performance of the VectorBloc connector under axial loads is influenced by parameters such as weight of the upper and lower blocs and location of the SHCS in the specimen. Table 2.3 shows the values of these parameters chosen in the parametric study.
The percentage weight reduction (W) presented in this table is calculated with respect to the VectorBloc specimen tested in this study which is referred to as the reference model. The reduction in the weight was achieved by reducing the thickness of upper and lower blocs. Three different locations of the SHCS were considered with respect to the position of the SHCS in the reference model as shown in Figure 2.19. The values presented in Table 2.3 represents the change in the hole location of SHCS 1 in x-axis and SHCS 2 in z-axis. Therefore, 15 (= 5 weights x 3 locations for SHCS) FE models for VectorBloc connectors were considered in the parametric study. The models are identified with the notations presented in Table 2.3. For example, the model with 10% weight reduction in its blocs is represented by 10W. Three different locations of SHCS are 0 mm (reference model), -12.7 mm, and +12.7 mm and they are represented by ZH, PH, and NH, respectively. Hence, the FE model 10WPH refers to 10% weight reduction in the VectorBloc connector with SHCS holes positioned at -12.7 mm with respect to the reference model.

**Table 2.3 Values of the parameters**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Values</th>
<th>Notation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Weight reduction (W)</td>
<td>0%, 5%, 10%, 15%, and 20%</td>
<td>0W, 5W, 10W, 15W, and 20W</td>
</tr>
<tr>
<td>Location of SHCS holes</td>
<td>0, -12.7 mm (-0.5 in), and +12.7 mm (+0.5 in)</td>
<td>ZH, PH, and NH</td>
</tr>
</tbody>
</table>
2.6.1 Axial compression

The parametric study undertaken on beam-column specimen showed that the change in the locations of the SHCS does not affect the axial compressive behavior of the connection if the weight of the connector is kept unchanged. Figure 2.20 presents the axial compressive load versus the axial displacement at the end of the top column of the ZH model where the weight of the model varies.
In this study, the ultimate load is considered as the maximum load capacity that the FE model exhibited and the yield load is the load at the inception of plastic strain in the FE model. The load in the FE model was applied by applying axial displacement. The blocks and the columns experienced axial deformation with the increase in axial load and after reaching the maximum load inelastic buckling of the columns occurred. Further increase in the axial deformation caused local buckling in the HSS column walls. In all FE models with axial compression load, the plastic strains first occurred at the HSS columns. Figure 2.21(a) presents the yield and ultimate loads of the ZH model when the weight reduction varies within the range of 5% to 20%. The beam-column specimen was subjected to a maximum load of 400 kN and Figure 2.21(a) confirms that this specimen was loaded well within the elastic limit. This figure also shows that the test specimen needed to be loaded with about 1439 kN for its complete failure. Figure 2.21(b) shows the relationship between the weight reduction and reductions in yield and ultimate load capacities. The reduction in yield and ultimate load capacities were calculated with respect to the reference model. It
can be observed from this figure that the weight reduction in the range of 5% to 20% reduces the ultimate load capacity of the connection from 2% to 21%. Figure 2.21(a) suggests that the reduction in the weight up to 20% may be acceptable since the design load for the connection is only 400 kN. Such reduction in the weight will make the VectorBloc connection cheaper and lighter.
Figure 2.21: Effect of weight reduction on axial compression capacity. (a) Yield and ultimate loads. (b) Percentage reduction in load capacity.
At the ultimate load the von Mises stress and PEEQ values were obtained from the FEA results with various weight reductions at the reference points (C1, L1, L2, L3, U1, U2, and C2) shown in Figure 2.22(a). Figures 2.22(b) and 2.22(c) show the distributions of von Mises stress and PEEQ across the connection. It can be observed from the Figure 2.22(b) that as the VectorBloc connector becomes lighter, the stresses in the columns reduce and, the stresses in the blocs increase. Nonetheless, the stresses at the columns always remained higher than the stresses at the blocs. However, Figure 2.22(c) indicates that the trend reverses for PEEQ and this is due to the difference in the post-yielding behaviors of these two materials. Thus, as the weight of the blocs reduces, inelastic buckling in the columns occurs early and reduces the yield and ultimate load capacities of the connection.
Figure 2.22: von Mises stress and PEEQ distributions. (a) Reference points. (b) von Mises stress distribution. (c) PEEQ distribution.
2.6.2 Axial Tension

The FE model was also used to simulate the behavior of the column specimen in axial tension. It was found that the axial tensile behavior of the model is not affected by the weight of the blocs (Figure 2.23). However, Figure 2.23 shows that the locations (positions) of the SHCS affect the stiffness and the load carrying capacities of the connection. This is due to the eccentricity of the axial load with SHCS. The eccentricity of the axial load decreases with the reduction in the distance between the HSS columns and the SHCS.

Figure 2.23: Bloc separation behavior in axial tension.

Figure 2.24(a) presents the yield and ultimate load capacities obtained from the FE models when the location (position) of the SHCS were varied keeping the weight unchanged at 0% (0W). This figure confirms that the beam-column specimen in axial tension was within the elastic limit since a maximum load of 200 kN was applied to this test specimen. This figure indicates that an axial tension load of about 490 kN was required to fail the beam-column
specimen in tension. The percentage variation in load capacities for FE models with various SHCS locations is presented in Figure 2.24(b). This figure shows that PH alignment (-12.7 mm) of the SHCS increases the yield and ultimate load capacities by 25% and 11% compared to the ZH alignment. Thus, the parametric study suggests moving the SHCS towards the HSS columns is required to increase the stiffness and load carrying capacities of the VectorBloc connection under axial tension.
Figure 2.24: Effect of SHCS location on load capacities in axial tension. (a) Yield and ultimate loads for various locations of SHCS. (b) Percentage variation of load capacity with SHCS locations.
2.7 Conclusions

This paper presents an innovative modular construction which uses state-of-the-art cast-steel connector namely, VectorBloc connector and HSS members. The structural performance of a typical corner connection of a proposed assisted living facility when subjected to design axial compression and axial tension loads was studied. The study was completed using full-scale tests and finite element method. The study found that the connection remains within its elastic limit under the design compression and tension loads. Thus, this study concludes that the VectorBloc connection to be incorporated in the proposed assisted living facility is able to carry design axial loads.

A detailed parametric study was completed using finite element method. It was found that weight of the cast-steel connector affects the compressive strength of the connection and the location of the SHCS (screws) affects the strength of the connection in tension. This study recommends two improvements in the design of the VectorBloc connector. The first recommendation is that the weight of the cast-steel connector can be reduced up to 20% of its current weight making the modular construction lighter and more cost-effective. Reduction in weight can be achieved by thinning the walls of the cast-steel connector. The second recommendation is that the location of two SHCS can be moved up to 12.7 mm towards the columns and this will result in increase in the stiffness of the connection considerably and yield and ultimate strength by 25% and 11%, respectively when the connection is subjected to tension load.

The failure mode of this connection when subjected to increasing axial compression load is inelastic global buckling which is ductile in nature, however, the failure mode is brittle due to sudden rupture of SHCS when the connection is subjected to increasing axial tension.
load. However, these failure loads are much higher than the factored design loads of the proposed assisted living facility. Nonetheless, the design of this connection need to be improved to ensure a ductile failure of the connection when subjected to tension.

2.8 Acknowledgements

This study was completed with the financial and technical assistance of VectorBloc Corp. located in Toronto, ON, Canada. Technical assistance was also received from Z-Modular located in Harrow, ON, Canada.

2.9 References


CHAPTER 3

PERFORMANCE OF VECTORBLOC MODULAR BEAM-COLUMN CONNECTION SUBJECT TO BENDING LOAD

3.1 Introduction

Modular construction involves a significant part of the construction work completed offsite in a factory in comparison with the traditional onsite construction. Hence, it is also referred to as offsite construction and is defined as manufacture and preassembly of elements, panels and modules offsite before installing them onsite [1]. Modular construction has the potential to provide a cheaper, faster, and greener alternative to tradition construction practices by reducing the construction time, cost, and waste [2]. Modular construction also ensures a better quality control. Thus, the use of modular construction is on the rise in North America, Japan, and parts of Europe [3].

Modular constructions using steel structural elements are becoming popular since it offers multiple levels of modularisation such as prefabricated structural elements, panels, and modules. Possibilities of various light steel modular constructions were discussed by Lawson et al. [4,5]. Annan et al. [6,7] highlighted the limitations of using light steel sections in modular construction and hence, proposed the use of hot-rolled sections such as I-sections in modular steel buildings. The choice of connectors or connections in building a module and stacking the modules largely decides the success and performance of a modular steel building. This led to the development of a number of modular connection techniques [8–10]. These techniques, however, involve a direct connection between the
beam and column members by welding and bolting. As a result, the connection elements in these techniques are made of combination of hot-rolled members and rectangular tubes.

Researchers at University of Arizona introduced the concept of a cast-steel connector in traditional onsite steel construction. The study found that this cast-steel connector improved the seismic performance of the steel frame [11–13]. Study on possible use of hollow structural steel (HSS) structural members in modular construction is limited to only one study [10]. In this study, direct connection between the HSS members was made using bolts.

VectorBloc Corp. [14] has developed a novel technique for building modular construction using HSS members. VectorBloc Corp. has developed an unique cast steel connector namely VectorBloc Connector. This connector has a complex geometry with the cross-sectional shape varying in all the three dimensional axes. The HSS beams and HSS columns are connected to the cast steel connector by welding. The novelty of this connector is that it houses both beam-column connection and the inter modular connection enabling high precision and easier installation of modules on site. However, since such modular connection is new and unique; it has not yet been used in large-scale constructions. It is important to study the structural performance of this connector and the beam-column connection under various load combinations before using in large-scale building structures.

The design philosophy of moment resisting connections made of HSS members is available in existing design guidelines of CIDECT [15,16], AISC [17], and Eurocode 3 [18]. The design of HSS-to-HSS moment connections to be used in regular (onsite) construction subjected to seismic load was studied by Fadden et al. [19,20]. However, these studies considered direct connections made between the HSS members and hence, in these studies
no connector was used. The modular connection involving the VectorBloc connector is unique and much different from the traditional HSS-to-HSS beam-column connections since multiple HSS beams of various sizes are connected to the HSS columns through the VectorBloc connector (Figure 3.1).

In this research the bending behavior of a typical corner connection made of VectorBloc connector with and without column axial load was studied experimentally and numerically. The connection considered in this study is planned to be used in an eight-storey high assisted living facility near Cambridge, Ontario, Canada. Hence, the primary objective of this research is to study the structural behavior of this beam-column connection under the design bending load with and without axial load on the columns. Finite element model of this connection was developed and validated using the test results. The moment-rotation relationship of this modular connection was developed and a detailed parametric study was undertaken. The results were then used in developing design recommendations for this modular connection. As well, possible improvements in the shape and design of the current VectorBloc connector are also recommended.

3.2 VectorBloc connection

The modular construction technique developed by VectorBloc Corp. involves building a HSS module with VecorBloc connectors and fitting those modules with all the plumbing and electrical utilities in a factory. The finished modules are then transported to the construction site and stacked horizontally and vertically to form a building space. A typical structural frame of this modular system with vertically and horizontally staked modules is shown in Figure 3.1(a) and the vertical corner connection considered in this research is shown in Figure 3.1(b). This figure shows all the components of the connection namely the
top and bottom columns, floor and ceiling beams, and the VectorBloc connector. The primary components of the VectorBloc connector are the upper and lower blocs (Figure 3.1(c)). The HSS floor beams are connected to the top HSS column through the lower bloc. The upper bloc provides connection between the HSS ceiling beams and the bottom HSS column. Connections between a bloc and the HSS members are made by full-penetration weld. During installation of the modules on site, a gusset plate is attached to the upper bloc using flat head cap screws (FHCS) to provide horizontal connection between the modules (Figure 3.1(c)). The shape of this gusset plate varies between L, T, and cruciform and it depends on the location of the building such as corner, edge, and interior, respectively. A registration pin (Figure 3.1(c)) is connected to the upper bloc and it guides the top module to be properly positioned over the bottom module during vertical staking of the modules. Finally, the vertical connection between the upper and bottom modules are secured through high strength socket head cap screws (SHCS) where the threaded end of the screws is connected to the threaded holes of the upper bloc (Figure 3.1(c)).
Figure 3.1: VectorBloc modular construction. (a) Frame system. (b) Corner connection. (c) VectorBloc connector.
3.3 Test setup

Four identical full-scale test specimens of VectorBloc corner connection were fabricated for testing under bending loads. The test setup with a test specimen is shown in Figure 3.2. Each test specimen had a VectorBloc connector with its upper and lower blocs. The major dimensions of these blocs are shown in Figure 3.3. The geometry of the HSS members are provided in Table 3.1. The connection between the upper and lower blocs were made by two 25.4 mm diameter SHCS. Prior to this, a 12.7 mm gusset plate was connected to the upper bloc using two 19.05 mm FHCS. As well, a 50.8 mm diameter registration pin was connected to the upper bloc. It should be noted that in all the four specimens the free ends of the floor and ceiling beams were touching each other (Figure 3.2 (b)). The material properties of the connection elements obtained from coupon tests are provided in Table 3.2 [21].

![Test setup](image)

**Figure 3.2:** Test setup. (a) Photo. (b) Schematic.
Table 3.1 HSS member properties

<table>
<thead>
<tr>
<th>HSS members</th>
<th>Length mm (inch)</th>
<th>Width× Depth× Thickness mm × mm × mm (inch × inch × inch)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Floor beam</td>
<td>1270 (50)</td>
<td>76.2×203.2×12.7 (3×8×0.5)</td>
</tr>
<tr>
<td>Ceiling beam</td>
<td>1219.2 (48)</td>
<td>76.2×76.2×9.5 (3×3×0.375)</td>
</tr>
<tr>
<td>Column</td>
<td>457.2 (18)</td>
<td>101.6×101.6×9.5 (4×4×0.375)</td>
</tr>
</tbody>
</table>

Table 3.2 Mechanical properties of the specimen components

<table>
<thead>
<tr>
<th>Component</th>
<th>Young’s modulus (GPa)</th>
<th>Yield strength (MPa)</th>
<th>Ultimate strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Upper and lower blocs</td>
<td>197.5</td>
<td>336.5</td>
<td>558.6</td>
</tr>
<tr>
<td>Gusset plate</td>
<td>197.2</td>
<td>355.3</td>
<td>554.9</td>
</tr>
<tr>
<td>SHCS</td>
<td>203</td>
<td>1055.2</td>
<td>1290.9</td>
</tr>
<tr>
<td>HSS sections</td>
<td>200</td>
<td>359.3</td>
<td>552.9</td>
</tr>
</tbody>
</table>

The test specimens were mounted between the supports through ball-joint assembly provided at the ends of the top and bottom columns (Figure 3.2(b)). The ball-joint assembly provided at the top column was connected to the loading actuator (actuator 3) which was connected to the support. These ball-joints created a pin-roller boundary conditions at the
ends of the columns. The bottom column end was restrained to translate in all three directions. However, the top column end was only restrained to translate along x-and z-directions and was allowed to translate vertically (y-direction) with the movement of the load actuator 3. The test setup consisted of three load actuators to apply bending and axial loads on the specimen. Loading actuators 1 and 2 applied bending loads to the connection and it was done by applying vertical loads on the floor beams 1 and 2 in xy- and zy- plane of the specimen as shown in Figure 3.4. Load actuator 3 applied axial compressive load at the top HSS column (Figure 3.2(b)). Four linear variable differential transducers (LVDT) were used to measure the deflection of the floor beams under bending loads. LVDT 1 and 2 measured the deflection of floor beam 1 and LVDT 3 and 4 measured the deflection of floor beam 2. The location of these LVDT are shown in Figure 3.4. A total of 22 strain gauges were installed on each specimen in the regions were high strain concentration was anticipated. The layout of the attached strain gauges is presented in Figure 3.5. In order to monitor the behavior of the SHCS during testing a strain gauge was attached to each SHCS as shown in Figure 3.5(c).
Figure 3.4: Location of loadcells and LVDT.
Figure 3.5: Strain gauge layout. (a) Elevation. (b) Plan. (c) SHCS 1 and 2.
3.4 Test procedure

The test specimens were tested under the design loads expected in a typical corner connection of the proposed assisted living facility. Four different load cases as shown in Table 3.3 were applied to these four specimens. The load cases involved applying bending moments about one axis or both axes of the HSS columns with or without the presence of axial loads. The moment values shown in Table 3.3 were calculated using the lever arm of 1.25 m of the bending loads with the face of the HSS column as shown in Figure 3.4.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Description</th>
<th>Bending in xy-plane</th>
<th>Bending in zy-plane</th>
<th>Axial compression</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Actuator-1 (kN)</td>
<td>Moment (kN-m)</td>
<td>Actuator-2 (kN)</td>
<td>Moment (kN-m)</td>
</tr>
<tr>
<td>UB</td>
<td>Uniaxial bending</td>
<td>38</td>
<td>48</td>
<td>-</td>
</tr>
<tr>
<td>BB</td>
<td>Biaxial bending</td>
<td>38</td>
<td>48</td>
<td>38</td>
</tr>
<tr>
<td>UBAC</td>
<td>Uniaxial bending + Axial compression</td>
<td>38</td>
<td>48</td>
<td>-</td>
</tr>
<tr>
<td>BBAC</td>
<td>Biaxial bending + Axial compression</td>
<td>38</td>
<td>48</td>
<td>38</td>
</tr>
</tbody>
</table>

In Table 3.3, specimens are given unique names as explained in column 2 of this table. Hence, specimen UB indicates that it was a specimen where only uniaxial bending load was applied through the loading actuator 1 and hence, no axial load was applied to this specimen. Specimen UBAC was subjected to same bending load, however, a maximum axial load of 200 kN was also applied to this specimen. Similarly, specimens BB and
BBAC were subjected to same biaxial bending moments, however, specimen BBAC was subjected to axial load whereas, specimen BB had no axial load on it. For specimens UB and BB the bending loads were applied monotonically in one step. However, for specimens UBAC and BBAC an axial compressive load of 50 kN was first applied by actuator 3 followed by application of 10 kN bending loads by actuator 1 and/or 2 and this sequence of load application was repeated to obtain the load combination presented in Table 3.3. It should be noted that in biaxial bending load cases (for specimens BB and BBAC) the bending loads were applied simultaneously on both the floor beams at a constant rate.

3.5 Test results

The bending loads applied on the floor beams caused flexural deflections of the floor and ceiling beams. These deflections of the beams caused bending deformation of the columns. The flexural deformation of the columns tried to separate the upper and lower blocs, however, it was resisted by SHCS. The results of uniaxial bending test (specimen UB) is presented in Figure 3.6. This figure also shows the data obtained from the finite element analysis (FEA) which will be discussed later. In this figure, the deflections refer to the deflections measured from LVDT 1 and LVDT 2 in Figure 3.4. Figure 3.6(a) shows that under the applied design load, the behavior of the connection was linear. The strain values measured at different locations of the specimen at the maximum bending load of 38 kN and corresponding bending moment of 48 kN-m remained within the elastic limit of 2000 micro-strain (Figure 3.6(b)). It can be observed in Figure 3.6(b) that the maximum strain occurred at the HSS columns at strain gauges 9 and 10 which were located on the column surface near the intersection of column and VectorBloc connector. This indicates that the columns are the critical components when VectorBloc beam-column connection is
subjected to an uniaxial bending. It was anticipated that among all components in the beam-column connection, the column member would fail first if the moment in the test could be increased well beyond the design moment of 48 kN-m which was applied to the test specimen. The high strain at the column is due to its smaller section modulus compared to other elements. The location of the SHCS with respect to the bending loads are such that in uniaxial bending the SHCS 2 has to carry a higher load compared to the SHCS 1 to prevent the separation of the upper and lower blocs. Thus, it can be observed in Figure 3.6(b) the strain at SHCS 2 (S22) is much higher than the strain observed in SHCS 1 (S21).
Figure 3.6: UB test results. (a) Bending behavior. (b) Strain data.
The results of the biaxial bending test (specimen BB) is presented in Figure 3.7. The displacements presented in Figure 3.7(a) were recorded through four LVDT (LVDT 1, LVDT 2, LVDT 3, and LVDT 4 in Figure 3.4). The deflections of the floor beams under the design bending loads (Figure 3.7(a)) show that the connection behaved linearly when subjected to design biaxial bending moments. The bending loads were applied through loading actuators 1 and 2 in Figure 3.4. Both loading actuators applied same load (±0.1 kN) at any given time. The strain data obtained at different locations of the specimen at maximum bending load of 38 kN which corresponds to a moment of 48 kN-m are shown in Figure 3.7(b). This figure shows that the specimen regions where the strain data was measured remained within the elastic limit of the material. The maximum strain values were recorded from the strain gauges 8, 9, and 10 which were on the column (Figure 3.7(b)). This indicates that the columns are the critical components when this VectorBloc beam-column connection is subjected to a biaxial bending moment and this connection may have failed due to plastic deformation of the column wall if the bending moments were increased well beyond the design bending moment of 48 kN-m. The strain values measured in the SHCS 1 (S21) and SHCS 2 (S22) indicate that both these screws were highly strained when transferring the load between the upper and lower blocs.
Figure 3.7: BB test results. (a) Bending behavior. (b) Strain data.
The bending behaviors of the VectorBloc connection in uniaxial bending with axial compression (specimen UBAC) and biaxial bending with axial compression (specimen BBAC) are presented in Figures 3.8(a) and 3.9(a), respectively. As can be found in these figures, the behavior of the connection in both specimens is linear. A small drop in the bending load at points A, B, and C can be noted in these figures. This is due to the application of incremental axial compressive load at these points. Figures 3.8(b) and 3.9(b) present the strain data measured when an axial load of 200 kN and a moment of 48 kN-m were applied. These figures show that the specimens remained within the elastic limit even after the introduction of axial compression loads of 200 kN. However, the introduction of axial loads in the connection increased the strain values in the VectorBloc connector. For example, Figure 3.8(b) for specimen UBAC shows that the strain gauge S5 (Figure 3.5(a)) had a strain of -962 micro strains whereas, the same strain gauge in Figure 3.6(b) for specimen UB shows a much smaller strain of -685 micro strains.
Figure 3.8: UBAC test results. (a) Bending behavior. (b) Strain data.
Figure 3.9: BBAC test results. (a) Bending behavior. (b) Strain data.
As can be found from Figures 3.10(a) and 3.10(b), the VectorBloc connection in all the four specimens was subjected to flexural deformation resulting in tensile and compressive strains on the surface of the HSS columns and the VectorBloc connector. As well, it is evident from these figures that the presence of axial compressive loads in specimens UBAC and BBAC increases the compressive strains in both HSS columns as well as in the VectorBloc connector. However, the axial compressive loads did not have significant effect on the floor beam behavior as can be observed in Figure 3.10(c). It can also be observed from this figure that the strain on floor beam 2 (strain gauges S19 and S20) is zero when uniaxial bending load was applied in specimens UB and UBAC, since in these specimens the bending loads were applied only on floor beam 1. This shows that under uniaxial bending, the floor beam 2 did not participate in load transfer or load sharing.

For specimens UB and BB, relatively higher strain values in the SHCS (strain gauges S21-S22) were observed as shown in Figure 3.10(d). This figure indicates that a separation between the upper and lower blocs occurred in UB and BB specimens due to the deformation of the SHCS. This was due to the fact that the bloc separation caused by the flexural deformation of the columns were resisted by the SHCS in the absence of axial compressive load on the column. It should be noted that none of the four test specimens showed any visible failure or drop in the load capacity. Thus, it can be concluded that the VectorBloc connection has the ability to safely carry the design loads that can develop in a typical corner connection of the proposed assisted living facility.
Finite element model development and validation

The full-scale tests provided key information on the behavior of the VectorBloc connector and the beam-column connection under the applied bending and axial loads. However, such full-scale tests are expensive and very time consuming. Further, full-scale tests are not able to provide other important information such as the stress and strain profiles on the connection that are useful for better understanding of its structural behavior. Hence, three-dimensional finite element (FE) models were developed and analysed using a commercially available finite element code, ABAQUS/Standard version 6.13, distributed

Figure 3.10: Strain at design loads. (a) HSS columns. (b) VectorBloc connector. (c) Floor beams. (d) SHCS.
by SIMULIA Inc. [22]. The geometry of the connection was modelled using continuum elements C3D10 and C3D20R. Elastic-plastic material model with isotropic strain hardening was adopted in these FE models. In the FE models, all three nonlinearities namely material, geometric, and contact were considered. The interfaces in the connection where contact occurs were modelled as surface-to-surface frictionless hard contact interactions. Tie constraints were used to model the welded joints between the VectorBloc connector and the HSS members. The free ends of the top and bottom columns were kinematically coupled to reference points representing the centres of the ball joint assembly. Then, support conditions were applied to these reference points. The adequacy of the element density in the FE models was checked through a mesh convergence study.

All the four tests were simulated using the FE method and the results obtained from FE analysis for each specimen are presented in Figures 3.6 to 3.9. It is evident from these figures that the FE models predict the global bending behavior and local strain state of the connection with a good accuracy.

The FE analyses confirmed that the HSS columns which were identified as the critical components in the uniaxial bending tests (specimens UB and UBAC) did remain in the elastic limit when subjected to design load. However, FE analyses revealed that for biaxial bending specimens (specimens BB and BBAC), localised plastic deformation occurred at the columns near the intersection of column and VectorBloc connector when subjected to design biaxial bending. No strain gauges were present at these locations and hence, these localised plastic deformations were not detected in the test specimens. The von Mises stress distribution in the connection under the design load for specimen BB (subjected to biaxial bending) is presented in Figure 3.11. This figure shows the high stress concentration
occurred near the intersection of column and the VectorBloc connector. However, occurrence of local stress concentration and plastic strain in the columns of specimens BB and BBAC were not large enough to cause a nonlinearity in the global bending behavior of the connection (Figures 3.7(a) and 3.9(a)). Hence, global load-deformation behaviors obtained from the tests did not show any non-linearity.

![Figure 3.11: von Mises stress distribution in the connection of specimen BB.](image)

**3.7 Moment-rotation behavior**

The test specimens were subjected to design loads of the assisted living facility and the test data showed that the specimens remained within the elastic limit (Table 3.3). However, FE models were used to determine the maximum load capacity and a possible failure. The moment-rotation relationship of the connection was developed for each specimen using the results obtained from FE analyses. Figure 3.12 shows the schematic of the model adopted to define the moment-rotation relationship of the connection. As shown in this figure, this connection has two different beam-column joints, one in the top module (floor beam-top column) and other one in the bottom module (ceiling beam-bottom column). Hence, it is
necessary to define the moment-rotation relationship of each joint in this beam-column connection. Thus, for the applied moment, rotation of joints in the top module (TM) and in the bottom module (BM) was calculated separately to plot moment-rotation curves of these two joints in the connection.

The moment-rotation behaviors of the connection in uniaxial bending (for specimens UB and UBAC) are shown in Figure 3.13. The bending load was applied on floor beam 1 and the moment was calculated at the face of the column by considering the lever arm of 1.25 m (Figure 3.4). Thus, the connection was subjected to bending in xy-plane only. Figure 3.13 shows that the initial part of the moment-rotational behavior of both joints (TM and BM) in the connection is linear. However, the ceiling beam-bottom column joint in the bottom module (represented by BM in Figure 3.13) shows a non-linear rotational behavior prior to reaching the maximum moment capacity of the connection. This, non-linear behavior of the bottom module joint is due to the plastification of the bottom column walls.

**Figure 3.12: Moment-rotation relationship.**

![Diagram of moment-rotation relationship](image)

Top column  
TM = φ₁₀ - φ₁₂  
BM = φ₁₂ - φ₁₀  

Floor beam  
Ceiling beam  
Bottom column

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and upper bloc connector regions near the upper bloc-bottom column intersection. Increasing the flexural deformation of the beams beyond the maximum capacity of the connection increases the plastic deformation of the column walls and results in the local buckling in the wall of the bottom column as shown in Figure 3.14. It should be noted that the axial compressive load applied on the top of the columns in specimens UB and UBAC are 0 kN and 200 kN, respectively. At small bending moment, the presence of axial compressive load on the columns in specimen UBAC helps in resisting the separation between the upper and lower blocs caused by the flexural deformation of the columns. Thus, the stresses in the SHCS are much less at this stage. However, the walls of the columns and VectorBloc connector start to plastically deform as moment increases. At this stage, the compressive stress resulted from the application of the axial load increases the plastic strain in the columns and the connector and thus, it reduces the capacity of the connection as can be observed in Figure 3.13. It can also be observed in this figure, for specimen UBAC, soon after reaching the maximum capacity of the connection, the moment capacity dropped at a faster rate if compared with the specimen without any axial load (specimen UB).
The moment-rotation behaviors of the beam-column connection subjected to biaxial bending moments (specimens BB and BBAC) are shown in Figure 3.15. Bending loads of equal magnitude was applied on both floor beams and thus, moments were calculated in...
xy and zy-planes at the face of the HSS columns by considering a moment lever arm of 1.25 m (Figure 3.4). Hence, the VectorBloc connection was subjected to equal moments in both the planes (xy and zy-planes). The behavior of the connection in biaxial bending (specimens BB and BBAC) is similar to that of the connection subjected to uniaxial bending (specimens UB and UBAC). However, the capacity of the connection is less when subjected to biaxial bending. This is due to presence of bending moments in both planes. Similar to the connection subjected to uniaxial bending (specimens UB and UBAC) the initial behavior of the connection subjected to biaxial bending (specimens BB and BBAC) is linear (Figure 3.15). At higher bending moment, the plastification of the bottom column walls and the upper bloc caused the non-linear behavior in the ceiling beam-bottom column joint (BM). Increasing the moments beyond the maximum capacity caused local buckling of the bottom column as shown in Figure 3.16.
Figure 3.15: Moment-rotation behavior of the connection in biaxial bending.

Figure 3.16: Local buckling of bottom column in BBAC specimen.
Eurocode 3 [18] provides guidelines for calculating the limits for classifying a connection in steel construction as a nominally pinned connection, rigid connection or a semi-rigid connection. These limits are based on the initial rotational stiffness of the connection. These limits are calculated by considering the geometry of the connection elements and the bracing conditions of the structural frame in which the connection is used. In this study, Eurocode 3 [18] was used to determine the limits for the VectorBloc connection considering the floor beam with a span length of 4.4 m. These limits are presented in Figures 3.13 and 3.15 by three inclined straight lines: nominally pinned, rigid (braced), and rigid (unbraced). If the initial rotational behavior of the connection lies below the limit for nominally pinned, then the connection can be classified as a nominally pinned connection. However, to classify a connection as a rigid connection, the initial rotational behavior should be above the limit line for rigid connection. The limit for rigid connection is based on bracing condition of the structural frame in which the connection is used. If the frame is braced against lateral displacements, then the rigid (braced) line can be taken as the limit for rigid connection. Otherwise, rigid (unbraced) line should be taken as the limit for rigid connection. If the initial rotational behavior of the connection lies between two different limit lines for rigid connection and pin connection, then the connection can be classified as a semi-rigid connection. It is obvious from Figures 3.13 and 3.15 that the VectorBloc beam-column connection studied in this research can be classified as a rigid connection, if the structural frame in which it is being used is effectively braced against horizontal displacements. This shows that the VectorBloc beam-column connection has the ability to transfer moments from the beams to the columns and thus, it can be used as a moment resisting connection.
3.8 Parametric study

Important parameters that influence the bending behavior of the VectorBloc beam-column connection are amount of axial load applied on the columns, geometry of the VectorBloc connector, and the geometrical properties of the columns. The influence of these parameters on the bending behavior of the VectorBloc beam-column connection was studied using FE method. The FE models for the full-scale beam-column connection specimens tested in the lab are referred to as the reference models. Many more FE models were developed for the parametric study. Table 3.4 shows all the parameters and their values chosen for the parametric study. Both test and FE analysis (FEA) results revealed that the columns are the critical components in controlling the bending capacity of the VectorBloc beam-column connection. The cross-sectional dimensions of the column except its wall thickness cannot be changed due to the geometry of the VectorBloc connector. The column section (101.6 mm × 101.6 mm) used in the test specimens is available with six different wall thicknesses and these are 12.7 mm, 9.525 mm, 7.938 mm, 6.35 mm, 4.763 mm, and 3.2 mm. Among these the column wall thicknesses of 9.525 mm and 12.7 mm were considered in the parametric study. Column wall thicknesses less than 9.525 mm was not considered since this will result in a much lesser bending capacity of the connection. As shown in Table 3.4 the axial compressive load in the FE model was varied as a percentage of yield load capacity of the HSS column cross-section (= yield stress × cross-sectional area of the column). The yield load for columns with wall thickness of 12.7 mm and 9.525 mm are 1474 kN and 1178 kN, respectively. The feasibility of reducing the weight of the VectorBloc connector to make the connection cheaper and lighter was also studied by considering a range of weight reductions (Table 3.4). The
weight reduction was accomplished by reducing the thickness of the VectorBloc connector walls.

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Values</th>
</tr>
</thead>
<tbody>
<tr>
<td>Column thickness (t_c)</td>
<td>9.525 mm and 12.7 mm</td>
</tr>
<tr>
<td>Axial compressive load</td>
<td>0%, 17%, 34%, 50%, and 75% of yield load of column cross-section</td>
</tr>
<tr>
<td>Weight reduction</td>
<td>0%, 5%, 10%, 15%, and 20% of reference model</td>
</tr>
</tbody>
</table>

The effect of axial load on the bending behavior of the connection can be observed in Figures 3.17(a) and (b). These figures present the deflection of floor beam 1 at the point where bending load was applied (Figure 3.4). Column wall thickness of 9.525 mm is considered in these analyses. In biaxial bending equal moment magnitude were applied on both floor beams. As discussed earlier the presence of axial load helps in reducing the stress in the SHCS when bending loads are not high. However, at higher bending loads, the axial compressive load results in a negative effect. The presence of axial load causes an increase in the compressive strain in the columns and the VectorBloc connector and thus, reduces the capacity of the beam-column connection. This effect of axial load is evident in both uniaxial and biaxial bending models as presented in Figures 3.17(a) and (b). These figures show that an increase in axial load from 0 kN to 880 kN (0 to 75% of the yield load) reduces the bending capacity by about 45%.

The interaction between the axial compressive load and the maximum bending capacity of the connection is presented in Figure 3.18 for models with two different wall thicknesses
of the column. It can be observed that increasing the thickness of the column wall increases the capacity of the connection when subjected to uniaxial and biaxial bending moments. The design axial load and bending moment of the connection must lie within the interaction curves presented in Figure 3.18. The design loads of the assisted living facility are also shown in this figure and it can be observed that the design loads (maximum loads applied to the test specimens) are well within the capacity of the connection. Thus, it can be concluded that the VectorBloc connection has a much higher capacity than the expected design loads of the assisted living facility.
Figure 3.17: Effect of axial loads on connections with $t_x=9.525$ mm. (a) Uniaxial bending. (b) Biaxial bending.
Figures 3.19(a) and (b) show the effect of reduction in the weight of the VectorBloc connector on the bending behavior of the beam-column connection. The results presented in these figures, correspond to the column wall thickness of 9.525 mm ($t_c = 9.525$ mm). An axial load of 600 kN (50% of yield load) was applied to all the models. Deflection in these figures represent the vertical deflection of floor beam 1 at loading point (Figure 3.4). It can be observed that reducing the weight of the connector reduces the connection capacity and also causes early initiation of the non-linearity in the bending load-deflection behavior.
Figure 3.19: Effect of VectorBloc connector weight reduction on connections with $t_c=9.525$ mm. (a) Uniaxial bending. (b) Biaxial bending.
FE analysis showed that connection failure in all the models subjected to uniaxial and biaxial bending was due to the plastic deformation of the HSS columns and the VectorBloc connector walls. However, the location of plastification or failure depend on the amount of weight reduction of the VectorBloc connector as well as the thickness of the column wall ($t_c$) chosen. As mentioned earlier the weight reduction of the VectorBloc connector was achieved by reducing the connector’s wall thickness. For the column with wall thickness of 9.525 mm, two different failure (plastification) locations were observed in the FE models depending on the magnitude of weight reduction of the VectorBloc connector. Up to 5% weight reduction, the location of failure or plastification occurred in the bottom column and upper bloc walls. Above 5% weight reduction, the location of plastification occurred at the top column and lower bloc walls. This is due to the reduction in the thickness of the VectorBloc connector wall which increased the stress and plastic strain in the lower bloc. To illustrate this, von Mises stress on the connector and the columns were obtained at the reference points shown in Figure 3.20. The von Mises stress obtained at the maximum biaxial bending capacity are presented in Figure 3.21(a). This figure shows that up to 5% reduction of connector weight the maximum stress occurs at the bottom column and above 5% reduction the maximum stress occurs on the top column and the lower bloc. For FE models with column thickness of 12.7 mm the connection failure was due to the plastic deformation of top column and lower bloc walls. Figure 3.21(b) shows the von Mises stress distribution on the connector and the columns at the maximum biaxial bending capacity for column wall thickness of 12.7 mm. This figure (Figure 3.21(b)) shows that the maximum stress occurs at the top column and the lower bloc walls when the weight reduction was varied between 0% and 20%.
Figure 3.20: Reference points.
The maximum moment capacity of the VectorBloc connection with various column wall thicknesses and weight reductions of the connector is presented in Figure 3.22. The moment capacity in this figure is the maximum moment that the beam-column connection...
can carry. This figure shows that a higher column thickness (12.7 mm) provided a higher capacity for the beam-column connection. However, this beneficial effect diminishes when the weight reduction is higher than ~7% as shown by a vertical line in Figure 3.22. Weight reduction higher than ~7% makes the connector wall thickness smaller than the thickness of the column wall (12.7 mm). Thus, high plastic strain occurs at walls of the VetcorBloc connector when its weight reduction is above ~7% resulting in a reduction in the moment capacity. The moment capacity presented in Figure 3.22 is obtained by applying an axial load of 50% of yield load of column cross-section. However, the design axial compression load which was applied to the test specimen was 17% of the yield load. Hence, from Figure 3.22 it is evident that the weight of the current VectorBloc connector can be reduced to 80% of its current weight without having to compromise the design moment requirements of the assisted living facility. However, if the weight of the VectorBloc connector is reduced beyond ~7% then column wall thickness of 9.525 mm is needed as this offers a higher capacity compared to connection made of columns with wall thickness of 12.7 mm.
This paper introduced an innovative modular construction technique using the state-of-the-art cast-steel VectorBloc connector and all HSS members. The primary objective of this study was to determine the structural performance of a typical corner beam-column connection when subjected to bending moments. This beam-column connection will soon be used in an assisted living facility. This research was completed using full-scale tests and non-linear finite element analyses. The following conclusions are made based on the outcomes of this study. However, these conclusions may be limited to the scope of this study.

The study found that the beam-column modular connection used in this study can be classified as a rigid connection provided the structure in which it is going to be used is effectively braced against horizontal displacements. This connection can safely carry the
design uniaxial and biaxial moments of the assisted living facility. The columns are the critical members and the failure of the connection usually occurs due to plastification in the column walls.

The study found that the presence of axial compressive load reduces the bending capacity of the connection. The bending capacity of the connection reduces by 45% when the axial load in the column is increased from 0% to 75% of the yield capacity of the column cross-section. Thus, to aid the design of the VectorBloc beam-column connection an interaction diagram between axial load and bending moment capacity of the connection was developed.

The feasibility of reduction in the weight of the VectorBloc connector was studied. This study concluded that the weight of the current VectorBloc cast-steel connector can be reduced to 80% without compromising the design load requirements of the assisted living facility. However, the thinner column wall thickness (9.525 mm) needs to be considered if the connector weight is reduced beyond ~7% since a larger column thickness (12.7 mm) adversely affects the moment capacity.

3.10 Acknowledgements

This study was completed with the financial and technical assistance of VectorBloc Corp. located in Toronto, ON, Canada and NSERC located in Ottawa, Canada. Technical assistance was also received from Z-Modular located in Harrow, ON, Canada.

3.11 References


CHAPTER 4

STRUCTURAL PERFORMANCE OF SINGLE FLOW DRILLED CONNECTIONS IN HOLLOW STRUCTURAL STEEL MEMBERS

4.1 Introduction

Hollow structural steel (HSS) members are extensively used in steel construction as truss members and as columns since it offers better strength and architectural appearance over the open section structural members. In recent years the HSS members are increasingly used as beam and column members in modular steel construction. However, the bolted connections on these HSS members are often made from only one side of the connection without the use of a nut. Special access holes on the HSS members are needed to connect the nuts to the bolts on other side of the connection (interior of the HSS members) which is why one sided connections are preferred. Some commonly used one sided or single sided bolting systems are flow-drill, Lindapter HolloBolt, blind bolt, and Huck ultra twist bolt.

The flow-drill system involves flow or friction drilling of screw holes. In the flow drilling process, a fast rotating drill bit displaces the work metal to form a bush (Figure 4.1(a)). This process does not generate any waste metal chips and the bush formed has a thickness of two to four times the thickness of the work metal [1]. However, the material properties may be altered near the screw hole region due to the generation of frictional heat. The threads on the bush are usually formed by a special drill bit without cutting the bush metal in a process called flow tapping. However, threads can also be made on the flow formed
bush by traditional tapping methods by threading the bush metal. The flow-drill technique can be adopted on thin wall sections of thicknesses up to 12.7 mm. Thus the flow drilling process offers a larger number of threads for screw engagement on thin wall thicknesses when compared to standard drilling and tapping techniques.

Extensive research work is available on the application of flow drilling on different metals and the mechanics of the flow drilling process [1–6]. In recent years, interest in the application of flow drilled screw connections in the automotive industry to replace conventional connection techniques such as spot welding has increased. Thus many research works studied the behavior of flow-drill screw connections between thin sheets of metals usually used in the frames of automobiles. Attempts were also made to develop macroscopic finite element models of these screw connections [7–11]. However, research on the application of flow drilled screws in structural steel connections is limited. France et al. conducted experimental studies on flow drilled connections between open section beams and tubular columns to demonstrate its application in simple and moment resisting connections in structural frames [12–14].

The guidelines for the tension and shear design of screw connections made with flow drilled holes on HSS members are not readily available. However, certain design recommendations are provided in the CIDECT design guide 9 [15] regarding the selection of HSS wall thickness and the screw diameter for flow drilled connections. An appendix in SCI P358 [16] provides the tension resistance of flow-drill connections made on two specific grades of hot rolled structural steel thicknesses. However, to the best of the authors knowledge no information is available regarding the strength of flow drilled screw connections made on hollow structural sections commonly used in North America such as
ASTM A500 [17], and A1085 [18] grade HSS sections. Also, no information is available on the possible failure modes of the flow-drill screw connections made on the walls of HSS members.

In this paper single flow-drill screw connections made on the walls of HSS members are studied under pure tension and shear loads through experimental tests. The effect of flow drilling and flow tapping on the ultimate strength and failure behavior of the connection was studied. The results of this study were compared with the results of the connections made using standard (traditional) drilling and tapping techniques. The influence of number of threads per inch (TPI) in the connection was also investigated in this study. This paper also attempted to develop and synthesize analytical equations to predict the mode of failure of the single flow-drill connections in tension and shear.

4.2 Test specimen and test setup

Fifteen different connections were considered in this study as shown in Table 4.1. As can be observed in this table, this study considered four different parameters namely HSS member geometry, number of threads per inch (TPI), drilling technique, and tapping technique. Based on the value of the parameter chosen, unique names were given to each connection as shown in column five of Table 4.1. The first part of the name before the hyphenation symbol represents the cross section geometry (depth × width × thickness) in inches. The second part of the name after hyphenation represents, in order, number of threads per inch of the screw (TPI-10 or 16), drilling technique (Flow drilling-FD or Standard drilling-SD), and tapping technique (Flow tapping-FT or Standard tapping-ST). The diameter of screw hole in all the connections was kept constant with a value of 19.05 mm (0.75 inch).
The connections presented in Table 4.1 were studied under both tensile and shear loads. Hence, a minimum of five specimens were fabricated for each connection in tension and in shear to achieve good statistical results. Thus, a total of 150 specimens (15 connections × 5 specimens × 2 types of tests) were tested. The HSS members used in this study conform to the specifications of ASTM A500 Grade C [17] and the socket head cap screws (SHCS) used in the connections conform to the specifications of ASTM A574 [19]. The minimum requirements of mechanical properties provided in these standards are presented in Table 4.2.

<table>
<thead>
<tr>
<th>HSS section d mm × b mm × t mm (inches × inches × inches)</th>
<th>Screw hole</th>
<th>Drillling technique</th>
<th>Tapping technique</th>
<th>Connection</th>
</tr>
</thead>
<tbody>
<tr>
<td>152 x 76 x 3.18 (6×3×1/8)</td>
<td>10</td>
<td>FD</td>
<td>ST</td>
<td>6×3×1/8-10FDST</td>
</tr>
<tr>
<td></td>
<td></td>
<td>FT</td>
<td>6×3×1/8-10FDFT</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>SD</td>
<td>6×3×1/8-10SDST</td>
<td></td>
</tr>
<tr>
<td>152 x 76 x 4.76 (6×3×3/16)</td>
<td>10</td>
<td>FD</td>
<td>ST</td>
<td>6×3×3/16-10FDST</td>
</tr>
<tr>
<td></td>
<td></td>
<td>FT</td>
<td>6×3×3/16-10FDFT</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>SD</td>
<td>6×3×3/16-10SDST</td>
<td></td>
</tr>
<tr>
<td></td>
<td>16</td>
<td>FD</td>
<td>ST</td>
<td>6×3×3/16-16FDST</td>
</tr>
<tr>
<td></td>
<td></td>
<td>FT</td>
<td>6×3×3/16-16FDFT</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>SD</td>
<td>6×3×3/16-16SDST</td>
<td></td>
</tr>
<tr>
<td>127 x 51 x 6.35 (5×2×1/4)</td>
<td>10</td>
<td>FD</td>
<td>ST</td>
<td>5×2×1/4-10FDST</td>
</tr>
<tr>
<td></td>
<td></td>
<td>FT</td>
<td>5×2×1/4-10FDFT</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>SD</td>
<td>5×2×1/4-10SDST</td>
<td></td>
</tr>
<tr>
<td></td>
<td>16</td>
<td>FD</td>
<td>ST</td>
<td>5×2×1/4-16FDST</td>
</tr>
<tr>
<td></td>
<td></td>
<td>FT</td>
<td>5×2×1/4-16FDFT</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>SD</td>
<td>5×2×1/4-16SDST</td>
<td></td>
</tr>
</tbody>
</table>
Table 4.2 Minimum requirements of mechanical properties

<table>
<thead>
<tr>
<th>Component name</th>
<th>Standard</th>
<th>Yield strength (N/mm²)</th>
<th>Ultimate strength (N/mm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SHCS</td>
<td>ASTM A574 [19]</td>
<td>1055</td>
<td>1172</td>
</tr>
<tr>
<td>HSS</td>
<td>ASTM A500 [17]</td>
<td>345</td>
<td>425</td>
</tr>
</tbody>
</table>

4.2.1 Tension test specimen and test setup

The schematic of the test specimen used for the tension tests is shown in Figure 4.1(a). The test specimen consisted of a hollow structural steel section with a single screw hole on its wall. The bottom surface of this HSS section was welded to a steel T-section as shown in Figure 4.1(a). A SHCS of diameter 19.05 mm (0.75 inch) with its head and shank removed was connected to the screw hole located on the top face of the HSS section. The other end of the SHCS was connected to a steel support rod (Figure 4.1(a)). The specimen was mounted in a universal testing machine by clamping the ends of the support rod and the web of the T-section as shown in Figure 4.1(b). The tension load was applied using the displacement control method at a rate of 2.5 mm/min. Thus, the test setup configuration applied pure tension load to the screw connection in the HSS section.
4.2.2 Shear test specimen and test setup

The schematic of the test specimen used in the shear tests is shown in Figure 4.2(a). Similar to the tension test specimens the shear specimens consisted of a HSS section, with a screw hole in its wall, welded to a T-section. However, the alignment of the screw hole with the support rod is different in this case. The support rod was designed to apply the desired shear force. A 19.05 mm (0.75 inch) diameter SHCS passing through the clearance hole in the support rod connected it to the HSS section (Figure 4.2(a)). Since the shear capacity of the connections were anticipated to be higher than its tension capacity, the T-section was strengthened by two 6.35 mm (0.25 inch) brackets as shown in Figure 4.2(a). The test specimens were supported in the same universal testing machine by clamping the ends of
the support rod and the web of the T-section as shown in Figure 4.2(b). The shear load was also applied using the displacement control method at a rate of 2.5 mm/min.

**Figure 4.2:** Shear test. (a) Specimen (mm). (b) Test setup.
4.3 Analytical design equations for different failure modes

The possible modes of failure for the connections presented in Table 4.1 under tension and shear were identified. Four different failure modes were possible in a connection under tension load. These failure modes were identified as: fracture of the SHCS, thread stripping in the screw hole of HSS section wall, yielding of the HSS section wall, and shear rupture of the flow formed bush. Two different failure modes namely, bearing-tearing failure of HSS wall and shear rupture of the SHCS were identified in the connections subjected to shear load. Analytical design equations available in the scientific literature and design codes were used to calculate the load capacity corresponding to the individual failure modes. However, for failure modes that are unique to the connections studied in this research, analytical equations were developed to predict the maximum load capacity. The geometry of the SHCS considered in computing the load capacity was based on the thread profile provided in ASME B1.1-2003 [20] shown in Figure 4.3.

Figure 4.3. Thread profile of the SHCS.
4.3.1 Failure modes in tension

(i) Rupture of SHCS

In this mode of failure, the SHCS in the connection will rupture as shown in Figure 4.4(a). The load capacity corresponding to this failure mode was calculated based on the design equation provided in the Canadian standard, CSA S16-14 [21].

\[ T_{r,1} = 0.75 \varphi_b A_b F_u \] (1)

In Equation (1), \( \varphi_b \) is the resistance factor with a value of 0.8, \( A_b \) is the nominal area of the bolt, and \( F_u \) is the ultimate tension strength of the bolt material.

(ii) Thread stripping in the hole of HSS section wall

The failure mode corresponding to the stripping of threads in the screw hole of the HSS section wall is shown in Figure 4.4(b). Thread stripping occurs due to the shear rupture or bending deformation of the threads in the screw hole. The tension load required to cause thread stripping was calculated using the analytical equation developed by Zhu et al. [22,23]. This equation is,

\[ T_{r,2} = \min (\sum F_{s,1,Rd}, \sum F_{s,2,Rd}) \] (2)

In Equation (2) \( F_{s,1,Rd} \) and \( F_{s,2,Rd} \) are the shear and bending strength of one thread. These quantities were calculated as follows,

\[ F_{s,1,Rd} = \frac{\pi D_h F_{y,p}}{\sqrt{3}} \] (3)

\[ F_{s,2,Rd} = \frac{\pi D_h^2 F_{y,p}}{4b} \] (4)
In Equations (3) and (4) $F_{y,p}$ is the yield strength of the HSS material, $D$ is the outer diameter of the SHCS, $h$ and $b$ are the thickness and height of a thread in the screw hole of the HSS section. The quantities $h$ and $b$ were calculated based on the screw profile of the SHCS shown in Figure 4.3. Thus, no distinction was made in the calculations between flow formed thread and standard tapped thread.

The summation in Equation (2) represents the total number of threads in the screw hole. To determine the number of threads in the flow formed bush, the thickness of the bush was assumed as $3t$ for HSS wall thicknesses 3.18 mm and 4.76 mm. However, the thickness of the bush made on HSS wall thickness 6.35 mm was assumed as $2.3t$ ($t$ is the thickness of the HSS section). This assumption of flow drilled bush thickness was based on the measurements made on the flow drilled bushes of test specimens.

(iii) Yielding of the HSS section wall

The plastic yielding of the wall of the HSS section surrounding the screw hole is shown in Figure 4.4(c). The circular yield pattern shown in this figure is comparable to the circular yield pattern of a concrete slab around a column. Thus the yield line analysis results of the circular yield pattern of slabs provided by Macgregor [24] was adopted to determine the tension load required to cause the circular yielding in the HSS wall. If $m_1$ and $m_2$ are the plastic moment capacity per unit length along the circumferential and radial direction, then the load required ($T_{r,3}$) to cause the circular yield pattern is [24],

$$T_{r,3}=2\pi(m_1+m_2)$$

(6)

Since $m_1$ and $m_2$ are equal for an isotropic material like steel, the plastic moment capacity per unit width is,
Substituting Equation (7) in Equation (6) the tension load corresponding to the yielding of HSS section was obtained as,

\[ T_{r,3} = \pi t^2 F_{y,p} \]  

(iv) Shear rupture of the flow formed bush

This mode of failure is possible in the flow drilled connections where the flow formed bush ruptures as shown in Figure 4.4(d). The rupture of this bush occurs along the shear plane shown in this figure. The tension load required to cause the shear rupture of the flow formed bush is,

\[ T_{r,4} = \frac{\pi D t F_{y,p}}{\sqrt{3}} \]
4.3.2 Failure modes in shear

(i) Bearing-tearing failure of HSS wall

The bearing-tearing failure of HSS wall under the shear load is shown in Figure 4.5(a). The shear load on the connections was transverse to the screw hole. Hence, the bearing capacity of the flow drilled connections was anticipated to be same as the standard drilled
connections. The shear load required to cause the bearing failure was calculated using the design equation provided in Canadian standard, CSA S16-14 [21].

\[
S_{r,1} = 3 \phi_{br} t_D F_{u,p} \tag{10}
\]

In Equation (10) \( \phi_{br} \) is a resistance factor with a value of 0.8 and \( F_{u,p} \) is the tensile strength of the HSS material.

(ii) Shear rupture of the SHCS

The failure mode corresponding to the shear rupture of the SHCS is shown in Figure 4.5(b). The shear load corresponding to this failure mode was calculated using the guidelines provided in Canadian standard, CSA S16-14 [21].

\[
S_{r,2} = 0.42 \phi_h A_b F_u \tag{11}
\]

Equation (11) consider that the shear rupture occurs at the thread of the SHCS. Thus the actual area of the SHCS at the root of its threads for different TPI was not considered while calculating \( A_b \) in Equation 11.

**Figure 4.5:** Failure modes in shear. (a) Bearing-tearing of HSS section wall. (b) SHCS rupture.
4.4 Test results

4.4.1 Tension tests

The maximum load carried by a specimen in the tension test was considered as the ultimate load. This ultimate load corresponds to the peak load in the load-displacement behavior of the specimen. As mentioned earlier five specimens were tested for each connection presented in Table 4.1. The average value of ultimate load of specimens tested for each connection is presented in Table 4.3. The standard deviation and coefficient of variation of ultimate load values of individual specimens is also presented in Table 4.3. As can be observed in this table the coefficient of variation is less than 5% for all the connections indicating a good statistical result and the repeatability of the specimen test results. One sample t-test was conducted on the ultimate load values of individual specimens and a confidence interval corresponding to a confidence level of 95% was obtained for the ultimate load of the connections (Table 4.3). The percentage increase in the average ultimate load capacity of the flow drilled connections (FDST and FDFT) with respect to the standard drilled connections (SDST) is presented in Table 4.3 and Figure 4.6.
Table 4.3 Tension test results

<table>
<thead>
<tr>
<th>Connection</th>
<th>Ultimate load</th>
<th>Percentage increase in ultimate load over SDST connection (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Average (kN)</td>
<td>Standard deviation (kN)</td>
</tr>
<tr>
<td>6×3×1/8-10FDST</td>
<td>28.4</td>
<td>0.7</td>
</tr>
<tr>
<td>6×3×1/8-10FDFT</td>
<td>30.8</td>
<td>1.1</td>
</tr>
<tr>
<td>6×3×1/8-10SDST</td>
<td>15.8</td>
<td>0.5</td>
</tr>
<tr>
<td>6×3×3/16-10FDST</td>
<td>52.3</td>
<td>1.9</td>
</tr>
<tr>
<td>6×3×3/16-10FDFT</td>
<td>57.3</td>
<td>1.9</td>
</tr>
<tr>
<td>6×3×3/16-10SDST</td>
<td>33.5</td>
<td>1.2</td>
</tr>
<tr>
<td>6×3×3/16-16FDST</td>
<td>53.1</td>
<td>1.9</td>
</tr>
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<td>6×3×3/16-16FDFT</td>
<td>57.2</td>
<td>1.4</td>
</tr>
<tr>
<td>6×3×3/16-16SDST</td>
<td>30.6</td>
<td>0.4</td>
</tr>
<tr>
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<td>98.4</td>
<td>5.1</td>
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<td>1.9</td>
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<td>0.9</td>
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<td>1.8</td>
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<td>1.8</td>
</tr>
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<td>5×2×1/4-16SDST</td>
<td>80.5</td>
<td>1.2</td>
</tr>
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</table>
The representative load-displacement behavior of connections made on HSS section with geometry 152 mm x 76 mm x 3.18 mm (6 inch x 3 inch x 1/8 inch) is shown in Figure 4.7. This figure shows the behavior of one specimen of each connection which is representative of the behavior of all the five specimens tested for that connection. It can be observed in Figure 4.7 that flow drilled connections (FD) had a higher stiffness and ultimate load capacity than the connection made on standard drilled holes (SD). The increase in ultimate capacity can be appreciated by observing Figure 4.6 where the increase in ultimate load is about 95% for FDFT connection when compared with SDST connection. This figure also shows that flow tapped connection (FDFT) had relatively higher capacity than the standard tapped connection (FDST).

It can be observed in Figure 4.7 that the behavior of the flow drilled (FD) connections were linear initially and at around 10 kN the wall of the HSS section started to yield causing a non-linear behavior. However, in standard drilled (SD) connections this non-linear
behavior started early at 8 kN. Thus in all the connections yielding of the HSS wall was observed. However, the load continued to increase in all the connections after the yielding of the HSS wall due to the clamping effect of the SHCS. The additional threads available in the bush of the flow drilled connections increased the ultimate load capacity when compared with the standard drilled connection (Figure 4.7). Upon reaching the maximum or ultimate load capacity the bush of the flow drilled connections ruptured as shown in Figures 4.8(a) and (b). However, in standard drilled connections thread stripping was observed at ultimate load as shown in Figure 4.8(c).

**Figure 4.7:** Representative load-displacement behavior of connections on 152 mm x 76 mm x 3.18 mm HSS section.
Figure 4.8: Connections after failure. (a) 6×3×1/8-10FDST. (b) 6×3×1/8-10FDF. (c) 6×3×1/8-10SDST.
The representative load-displacement behavior of connections made on HSS section geometry 152 mm x 76 mm x 4.76 mm (6 inch × 3 inch × 3/16 inch) is shown in Figure 4.9. This figure shows that the flow drilled connections had a relatively higher stiffness and a much higher ultimate load capacity than the standard drilled connections. However, the percentage increase in load capacity obtained with flow drilling for the 6×3×3/16-10FDFT connection (71%) was much lower than the percentage increase obtained with the 6×3×1/8-10FDFT connection (95%) (Figure 4.6). This shows that the improvement in ultimate load capacity obtained from using flow drilled connections reduces with an increase in HSS wall thickness. It can be observed in Figure 4.9 that the behavior of connections with 10 TPI and 16 TPI screw holes was nearly the same. However, in standard drilled connections the ultimate load of the connection with a screw hole of 16 TPI (16SDST) was 9% lower than the connection with 10 TPI (10SDST) screw hole.

**Figure 4.9:** Representative load-displacement behavior of connections on 152 mm x 76 mm x 4.76 mm HSS section.
As can be observed in Figure 4.9 the behavior of all the connections is linear up to 20 kN. At this load level the HSS wall containing the screw hole started to plastically deform. However, due to the clamping effect of the SHCS, the load continued to increase even after the yielding of the HSS wall. In the flow drilled connections with standard tapped screw holes (FDST), circular yielding of the HSS wall was observed at the ultimate load as shown in Figures 4.10(a) and (b). Examination of the FDST connections after failure indicate that the SHCS was still tightly clamped to the flow bush. This shows that at ultimate load the failure might have occurred due to the onset of bush rupture. However, in connections with flow tapped screw holes (FDFT) bush rupture was observed at the ultimate load as shown in Figures 4.10(c) and (d). In connections with standard drilled screw holes (SDST) thread stripping occurred at the ultimate load as shown in Figures 4.10(e) and (f).
Figure 4.10: Connections after failure. (a) 6x3x3/16-10FDST. (b) 6x3x3/16-16FDST. (c) 6x3x3/16-10FDFT. (d) 6x3x3/16-16FDFT. (e) 6x3x3/16-16SDST. (f) 6x3x3/16-10SDST.
The representative load-displacement behavior of the connections made on the HSS section 127 mm x 51 mm x 6.35 mm (5 inch x 2 inch x 1/4 inch) is shown in Figure 4.11. It can be observed in this figure that all the connections had the same behavior up to 60 kN. However, the flow drilled connections had a relatively higher ultimate load capacity than the standard drilled connections. Figure 4.11 shows that the ultimate capacity of the connection with flow tapped screw hole with 10 threads per inch (10FDFT) was much higher than the other flow drilled connections. However, the behavior of flow drilled connections other than 10FDFT connection was nearly the same in the non-linear part of the load-displacement curve (Figure 4.11).

![Figure 4.11: Representative load-displacement behavior of connections on 127 mm x 51 mm x 6.35 mm HSS section.](image)

The circular yielding of the HSS wall containing the screw hole was observed in all the flow drilled connections. This plastic deformation of the HSS wall increased with the increase in load. In the flow drilled connections 10FDST and 16FDFT bush rupture occurred at the ultimate load as shown in Figures 4.12(a) and (b). Plastic deformation of
the HSS wall with the SHCS tightly clamped to the bush was observed in 16FDST connection as shown in Figure 4.12(c). This indicates that the connection load capacity could have dropped after reaching the ultimate load due to the onset of bush rupture. In the 10FDFT connection the edges of the HSS wall containing the screw hole suddenly fractured at ultimate load as shown in Figure 4.12(d). This fracture in the HSS section caused the sudden load drop observed in Figure 4.11 at point A. However, after the fracture of the HSS wall in 10FDFT connection, the load dropped to the load level of other flow drilled connections (Figure 4.11). In the case of standard drilled connections, the plastic deformation of the HSS wall was not significantly observable. However, the standard drilled connections exhibited a significant portion of nonlinear load-deformation behavior in Figure 4.11 indicating small plastic deformation of the HSS wall. This plastic deformation of the HSS wall was followed by stripping of screw hole threads as shown in Figures 4.12(e) and (f).
Figure 4.12: Connections after failure. (a) 5×2×1/4-10FDST. (b) 5×2×1/4-16FDFT. (c) 5×2×1/4-16FDST. (d) 5×2×1/4-10FDFT. (e) 5×2×1/4-10SDST. (f) 5×2×1/4-16SDST.
4.4.2 Shear tests

The maximum load carried by a connection specimen under shear load was considered as the ultimate load. This ultimate load corresponds to the peak point of the load-displacement curve. The average value of the ultimate load of all the five specimens tested for each connection considered in Table 4.1 is presented in Table 4.4. This table also presents the standard deviation and coefficient of variation of the ultimate load values of the individual specimens of each connection. As can be observed in Table 4.4 the standard deviation values of all the connections, except for the 5×2×1/4-10FDT connection, are less than 5% indicating a good statistical sample set and repeatability of the tests. The confidence interval of the ultimate load of the connections corresponding to 95% confidence level is presented in Table 4.4. The confidence interval was obtained by conducting a one sample t-test on the ultimate load values of individual specimens tested for each connection. The percentage difference in ultimate load values of flow drilled connections with respect to standard drilled connections (SDST) was calculated and is presented in Table 4.4 and Figure 4.13.
<table>
<thead>
<tr>
<th>Connection</th>
<th>Ultimate load</th>
<th></th>
<th></th>
<th>Percentage difference in ultimate load over SDST connection (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Average (kN)</td>
<td>Standard deviation (kN)</td>
<td>Coefficient of variation (%)</td>
<td>Confidence interval (kN)</td>
</tr>
<tr>
<td>6×3×1/8-10FDST</td>
<td>55.8</td>
<td>0.9</td>
<td>2</td>
<td>(54.3, 57.3)</td>
</tr>
<tr>
<td>6×3×1/8-10FDFT</td>
<td>54.9</td>
<td>1.9</td>
<td>3</td>
<td>(52.6, 57.3)</td>
</tr>
<tr>
<td>6×3×1/8-10SDST</td>
<td>50.4</td>
<td>2.5</td>
<td>5</td>
<td>(47.3, 53.6)</td>
</tr>
<tr>
<td>6×3×3/16-10FDST</td>
<td>96.4</td>
<td>2.7</td>
<td>3</td>
<td>(93, 99.8)</td>
</tr>
<tr>
<td>6×3×3/16-10FDFT</td>
<td>96</td>
<td>2.4</td>
<td>3</td>
<td>(93, 99)</td>
</tr>
<tr>
<td>6×3×3/16-10SDST</td>
<td>97.9</td>
<td>1.6</td>
<td>2</td>
<td>(95.9, 99.9)</td>
</tr>
<tr>
<td>6×3×3/16-16FDST</td>
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<td>2.2</td>
<td>2</td>
<td>(100.8, 106.2)</td>
</tr>
<tr>
<td>6×3×3/16-16FDFT</td>
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<td>2.7</td>
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</tr>
<tr>
<td>5×2×1/4-10FDST</td>
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<td>3</td>
<td>(116.2, 123.9)</td>
</tr>
<tr>
<td>5×2×1/4-10FDFT</td>
<td>126.6</td>
<td>17.3</td>
<td>14</td>
<td>(108.4, 144.7)</td>
</tr>
<tr>
<td>5×2×1/4-10SDST</td>
<td>120.4</td>
<td>2.0</td>
<td>2</td>
<td>(117.9, 122.9)</td>
</tr>
<tr>
<td>5×2×1/4-16FDST</td>
<td>122.5</td>
<td>3.0</td>
<td>2</td>
<td>(119.4, 125.6)</td>
</tr>
<tr>
<td>5×2×1/4-16FDFT</td>
<td>124.7</td>
<td>2.0</td>
<td>2</td>
<td>(122.3, 127.2)</td>
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<tr>
<td>5×2×1/4-16SDST</td>
<td>120.5</td>
<td>1.1</td>
<td>1</td>
<td>(117.8, 123.1)</td>
</tr>
</tbody>
</table>
The results of shear tests conducted on connections made on HSS sections 152 mm x 76 mm x 3.18 mm (6 inch x 3 inch x 1/8 inch) and 152 mm x 76 mm x 4.76 mm (6 inch x 3 inch x 3/16 inch) are shown in Figures 4.14 and 4.15. These figures present the load-deformation behavior of a specimen which is representative of all the five specimens tested for a connection. It can be observed in Figures 4.14 and 4.15 that the behavior of the flow drilled connections in terms of initial stiffness and nonlinear behavior are almost similar to the standard drilled connections. It can also be observed in these figures that the type of tapping (FT or ST) and the number of threads available (10 TPI or 16 TPI) in the screw hole did not have any significant influence on the behavior of the connections under the shear load. Figure 4.13 indicates that flow drilling did not result in a significant difference in ultimate load of the connections made on HSS section 152 mm x 76 mm x 4.76 mm (6 inch x 3 inch x 3/16 inch). However, a 11% increase in ultimate load was observed for
the FDST connection over the SDST connection made on HSS section 152 mm x 76 mm x 3.18 mm (6 inch × 3 inch × 1/8 inch) (Figure 4.13).

![Figure 4.14](image)

**Figure 4.14:** Representative load-displacement behavior of connections on 152 mm x 76 mm x 3.18 mm HSS section.

![Figure 4.15](image)

**Figure 4.15:** Representative load-displacement behavior of connections on 152 mm x 76 mm x 4.76 mm HSS section.
As can be observed in Figures 4.14 and 4.15, the behavior of the connections was initially linear. At the end of the linear behavior, the wall of the HSS section starts to plastically deform resulting in the nonlinear behavior of the load-displacement curve. The plastic deformation of the wall continued to increase with the increase in load. This plastic deformation of the HSS wall resulted in the bearing-tearing failure as shown in Figures 4.16(a) and (b). A small drop in load capacity can be observed at the ultimate load in all the flow drilled connections in Figures 4.14 and 4.15. This drop may be a result of the rupture of the flow formed bush at the ultimate load (Figure 4.16(a)). This effect was more visible in the behavior of connections with thickness 3.18 mm as shown in Figure 4.14. In this figure the ultimate load of the flow drilled connections is relatively higher than the standard drilled connection. At ultimate load, the fracture of the flow formed bush caused a small load drop. However, the rupture of the flow formed bush did not have any significant effect on the connection behavior presented in Figure 4.15, where the connection thickness is 4.76 mm.
The representative load-displacement behavior of the connections made on the HSS section 127 mm x 51 mm x 6.35 mm (5 inch × 2 inch × 1/4 inch) is shown in Figure 4.17. As can be observed in this figure, the behavior of the connections up to the ultimate load is nearly same for all the connections except for the 10FDFT connection. The behavior of all the connections was linear up to the point where plastic deformation of the HSS wall started. This plastic deformation increased with the increase in load and resulted in the nonlinear connection behavior (Figure 4.17). After reaching the ultimate load, all the connections with 16 TPI screw holes underwent a bearing-tearing failure as shown in Figure 4.18(a). However, two different failure modes were observed among the specimens of connections with 10 TPI screw holes. The first mode was shear rupture of SHCS as shown in Figure 4.18(b). The 10 TPI connection specimens chosen for Figure 4.17 underwent this mode of

![Figure 4.16: Connections after failure. (a) 6×3×1/8-10FDST. (b) 6×3×1/8-10SDST.](image)
failure and thus sudden load drops can be observed at points A, B, and C where the SHCS ruptured suddenly. The second mode was the bearing-tearing failure similar to the failure observed in the connections with 16 TPI screw holes (Figure 4.18(a)). Figures 4.19(a), (b), and (c) present the behavior of specimens of connections with 10 TPI screw holes. In these figures specimen 1 underwent a rupture failure of SHCS and specimen 2 failed by bearing-tearing of the HSS wall. As can be observed in Figures 4.19(a), (b), and (c), specimen 1 and specimen 2 had a relatively similar initial behavior. However, specimen 1 associated with the rupture failure of SHCS had a relatively higher ultimate load capacity than the specimen 2. The difference in the ultimate load capacity between the two types of failure modes was higher in the 10FDFT connection (Figure 4.19(b)) and this resulted in a very high coefficient of variation of 14% for the 5x2x1/4-10FDFT connection (Table 4.4).

Figure 4.17: Representative load-displacement behavior of connections on 127 mm x 51 mm x 6.35 mm HSS section.
Figure 4.18: Connections after failure. (a) 5×2×1/4-16FDST. (b) 5×2×1/4-10SDST.
Figure 4.19: Connection load-displacement behavior. (a) 5×2×1/4-10FDST. (b) 5×2×1/4-10FDFT. (c) 5×2×1/4-10SDST.
4.5 Comparison between analytical and test results

4.5.1 Tension connections

The load capacity of the connections corresponding to four different failure modes was calculated using Equations (1), (2), (8), and (9) and is presented in Table 4.5. In calculating these failure loads the HSS wall thickness (t) was taken as 90% of the nominal wall thickness. This reduction of HSS wall thickness was considered in all the analytical equations to account for variation in HSS wall thickness permitted in ASTM A500 [17]. The failure mode corresponding to the smallest load was predicted to occur as a primary failure in a connection. As can be observed in Table 4.5, the predicted mode of primary failure and observed mode of primary failure in tests is the same in all the connections. This primary failure mode is the circular yielding of the HSS wall. In test results, it was found that plastic deformation of the HSS wall caused the nonlinear behavior in the connection. However, the ultimate load was governed by the type of secondary failure which occurred at the ultimate load of the connection. Table 4.5 presents the mode of failure observed at the ultimate load in all the connections. As can be observed in this table, all the flow drilled connections except the 5×2×1/4-10FDFT connection underwent bush rupture at ultimate load. However, thread stripping was the failure mode in all the standard drilled connections. The failure mode corresponding to the second least calculated failure load is highlighted in bold fonts in Table 4.5. This predicted secondary mode corresponds to the secondary failure observed in the tests. Thus, the analytical equations presented in section 4.3.1 was able to predict the sequence of failure modes possible in both the flow drilled and standard drilled connections.
It can be observed in Figures 4.7 and 4.9 that the calculated load to cause the walls of HSS to plastically deform ($T_{r,3}$) is nearly at the onset of nonlinear behavior of all the connections. This shows a good prediction of primary failure load by the analytical equation. The calculated load for secondary failure mode which is thread stripping for standard drilled (SD) connections ($T_{r,2}$) and bush rupture for flow drilled (FD) connections ($T_{r,4}$) is also presented in Figures 4.7 and 4.9. Figure 4.7 shows that the calculated secondary loads ($T_{r,2}$ and $T_{r,4}$) overestimates the connection capacity for all the connections made on HSS section 152 mm x 76 mm x 3.18 mm. Similar overestimation of secondary loads ($T_{r,2}$) can be observed in Figure 4.9 for standard drilled connections made on HSS section 152 mm x 76 mm x 4.76 mm. However, in this figure a good agreement between calculated secondary loads ($T_{r,4}$) and the test loads of the flow drilled connections can be observed. The discrepancy in estimation of secondary failure load could be due to the influence of primary failure mode on the secondary failure mode which was not considered in the analytical calculations. Nonetheless, safety factors could be used in analytical equations to calculate the secondary failure loads to get a reasonable estimate of ultimate load of the connections. The primary ($T_{r,3}$) and secondary ($T_{r,2}$ and $T_{r,4}$) failure loads calculated for the connections made on HSS section 127 mm x 51 mm x 6.35 mm is shown in Figure 4.11. This figure shows that the predicted failure loads are conservative in comparison with the test loads.
Table 4.5 Comparison between analytical and test results in tension

<table>
<thead>
<tr>
<th>Connection</th>
<th>Analytical results</th>
<th>Test results</th>
<th>Average ultimate load (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Mode 1 Bolt rupture$^{1}$ $T_{r,1}$ (kN)</td>
<td>Mode 2 Thread stripping $T_{r,2}$ (kN)</td>
<td>Mode 3 HSS wall yielding $T_{r,3}$ (kN)</td>
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<tr>
<td>6x3x1/8-10FDST</td>
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<td>70</td>
<td>9</td>
</tr>
<tr>
<td>6x3x1/8-10FDFT</td>
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<td>70</td>
<td>9</td>
</tr>
<tr>
<td>6x3x1/8-10SDST</td>
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<td>5x2x1/4-16SDST</td>
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<td>45</td>
<td>35</td>
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</table>

$^{1}$ Note- Load $T_{r,1}$ was calculated by considering resistance factor
4.5.2 Shear connections

The load capacity of the connections corresponding to two different failure modes calculated using Equations 10 and 11 is presented in Table 4.6. The failure mode corresponding to the smallest load was predicted to occur in the connections. As can be observed in Table 4.6 the predicted mode of failure and the failure mode observed in tests is the same for all the connections except for connections made on HSS section 127 mm x 51 mm x 6.35 mm (5 inch × 2 inch × 1/4 inch) with 10 TPI screw holes. The bearing-tearing capacity of this HSS section (111 kN) is very close to the rupture capacity of the SHCS (112 kN). Thus shear rupture of SHCS was observed in some specimens of 10 TPI connections. However, rupture of SHCS was not observed on connections with 16 TPI screw holes (Table 4.6). This may be due to the smaller root diameter of the 10 TPI SHCS (15.99 mm) compared with the root diameter of 16 TPI SHCS (17.22 mm). The root diameter of the SHCS for different TPI was calculated using Figure 4.3.

The load to cause bearing-tearing failure ($S_{r,1}$), calculated using Equation 10, is presented in Figures 4.14, 4.15, and 4.17. As can be observed in Figure 4.14, the analytical result overestimates the ultimate load value of nearly all the connections made on HSS section 152 mm x 76 mm x 3.18 mm. However, the analytical results provide a conservative estimate of ultimate load capacity of the connections made on HSS sections 152 mm x 76 mm x 4.76 mm and 127 mm x 51 mm x 6.35 mm (Figures 4.15 and 4.17). Thus, the design equations (Equations 10 and 11) of Canadian standard, CSA S16-14 [21] can be used to determine the connection capacity in shear regardless of the type of drilling technique and tapping technique used in the connection. However, for thin section
additional safety factor need to be used to get a reasonable estimate of ultimate load capacity.
Table 4.6 Comparison between analytical and test results in shear

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<thead>
<tr>
<th>Connection</th>
<th>Analytical results</th>
<th>Test results</th>
<th>Average Ultimate load</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Mode 1: Bearing-tearing failure</td>
<td>Mode 2: Shear rupture</td>
<td>Failure mode</td>
</tr>
<tr>
<td></td>
<td>$S_{r,1}$ (kN)</td>
<td>$S_{r,2}$ (kN)</td>
<td></td>
</tr>
<tr>
<td>6x3x1/8-10FDST</td>
<td>56</td>
<td>112</td>
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</tr>
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<td>5x2x1/4-16SDST</td>
<td>111</td>
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<td>Mode 1</td>
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</table>

Note- Loads $S_{r,1}$ and $S_{r,2}$ were calculated by considering resistance factors
4.6 Conclusions

This paper presented the relative performance of flow drilled connections made on HSS steel sections over standard drilled connections under pure tension and shear load. The influence of four different parameters namely HSS geometry, screw hole drilling technique, tapping technique, and the number of threads per unit length on the behavior of the connections was studied. The possible modes of failure in the connections were identified and analytical equations for prediction of failure load was presented. Based on this study the following conclusions are made.

i. The plastic deformation of the HSS wall containing the screw hole occur in all the connections when subjected to tension load. However, this plastic deformation does not cause complete failure of the connection.

ii. The complete failure of the connection under tension load occurs after reaching the ultimate load capacity. In flow drilled connections the connection failure at ultimate load occurs through bush rupture. However, in standard drilled connections the connection failure occurs through thread stripping.

iii. An increase in ultimate load capacity of 95% can be obtained by using a flow drilled connection over a standard drilled connection in HSS wall thickness 3.18 mm. However, this increase is only about 71% for flow drilled connection made on HSS wall thickness 4.76 mm.

iv. The process of flow tapping offers relatively higher ultimate load capacity than the standard tapping technique when the connection is subjected to tension load.

v. The effect of number of threads per unit length (10 TPI or 16 TPI) does not have significant influence on the connection behavior in tension.
vi. The analytical equations presented for different failure modes in tension has the ability to predict the sequence of failure in the connections. These equations can be used with appropriate safety factors to estimate the ultimate load capacity of the connections.

vii. The process of flow drilling and flow tapping does not have significant influence on the connection behavior under shear load.

viii. The design equations of Canadian standard, CSA S16-14 [21] can be used to estimate the ultimate shear load capacity of the flow drilled connections similar to the standard drilled connections. However, additional safety factors need to be included in these design equations for HSS section thickness 3.18 mm.

4.7 References


CHAPTER 5

GENERAL DISCUSSIONS AND CONCLUSIONS

This research was conducted to study the behavior of connections made on HSS members in the steel modules of VectorBloc modular construction. The primary objective was to study the structural behavior of corner VectorBloc beam-column connection subjected to design loads of the proposed assisted living facility. The secondary objective was to study the behavior of single flow drilled connections made on the HSS members. This chapter presents the main findings of this research and proposes some recommendations for future work.

5.1 Conclusions

The conclusions presented in this section are based on the experimental tests, finite element analyses, and analytical analyses completed in this research. Thus, these conclusions are limited to the findings associated with the test specimens, finite element analyses, and analytical analyses presented in the previous chapters.

1. The tests and the finite element analyses results shows that the VectorBloc beam-column connection has the ability to safely carry the design loads of the proposed assisted living facility.

2. The parametric study has shown that the VectorBloc connector weight can be reduced to 80% of its current weight without compromising the design requirements of the assisted living facility. The weight reduction can be achieved by reducing the wall thickness of the VectorBloc connector.
3. The thinner column wall thickness (9.525 mm) needs to be considered if the VectorBloc connector weight is reduced beyond ~7% since a larger column thickness (12.7 mm) adversely affects the moment capacity.

4. The SHCS location can be moved 12.7 mm towards the columns. This will result in considerable increase in the connection stiffness and increase in yield and ultimate strength by 25% and 11%, respectively when the connection is subjected to axial tension load.

5. The moment-rotation relationships developed for the VectorBloc beam-column connection shows that this connection can be classified as a rigid connection.

6. The axial compression load on the columns reduces the bending capacity of the connection. The bending capacity of the connection reduces by 45% when the axial load in the column is increased from 0% to 75% of the yield capacity of the column cross-section.

7. The interaction diagram between the axial load and the bending capacity of the connection developed in this study can be used in the design of the VectorBloc beam-column connection.

8. The failure of the VectorBloc beam-column connection under increasing axial compression load is due to the global inelastic buckling which is ductile in nature. However, the failure mode of the connection is brittle in axial tension due to the sudden rupture of the SHCS.

9. The failure of the VectorBloc beam-column connection under the action of increasing bending loads is due to the plastification of the column walls.
10. The flow-drill system has the potential to increase the ultimate load capacity of the screw connections under tension load when compared with standard drilled connections. However, this increase in capacity depends on the thickness of the HSS wall. Flow drilled connection made on thinner HSS wall (3.18 mm) results in an increase in ultimate load capacity of 95%. However, this increase is only about 71% for connection made on thicker HSS wall (4.76 mm).

11. The flow tapping process offers relatively higher ultimate load capacity than the standard tapping technique when the connection is subjected to tension load. However, the number of threads per unit length (10 TPI or 16 TPI) does not have a significant influence on the connection behavior.

12. The process of flow drilling and flow tapping does not influence the behavior of the connections made on HSS walls when subjected to shear load.

13. The analytical equations presented in this study can be used to predict the failure behavior of flow drilled connections subjected to tension and shear loads.

5.2 Recommendations for future work

1. The potential for using the VectorBloc beam-column connection as a lateral force resisting system in the modular structural frame can be studied through full-scale tests and finite element analyses.

2. The overall dimensions of the VectorBloc connector can be modified to meet the needs of a seismic moment resisting connection. The beam-column connection with these modified connectors can be studied through finite element analyses to determine the optimum connector geometry. Full-scale tests can be performed on
the beam-column connection with this optimum connector to verify the finite element results.

3. More cross-sectional dimensions of HSS members can be considered for flow drilled connection tests. The screw hole diameter and grade of the HSS material can also be varied in these tests to determine the tension strength of flow drilled connections.

4. The application of flow drilled connections in HSS-to-HSS moment connections can be studied through full scale tests and design guidelines can be developed for such connections.
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