Behavior of Vectorbloc Beam-Column Connections

L. Kalam, J. Dhanapal, S. Das, and H. Ghaednia

1 Introduction

Modular construction is on the rise due to the increased speed, efficiency, and quality control it provides in comparison to traditional construction practices. Modular construction involves the offsite construction of elements, panels, and modules in a factory before onsite installation [1]. Among the many types of modular construction, structural steel-based modular construction is of particular interest in high-rise buildings [2]. A key component in modular construction is the connection between the individual modules, which hold the building together against applied loads such as gravity, snow, and wind.

A state-of-the-art cast steel connector, named the VectorBloc connector, is presently used in the connections between steel modules of modular buildings. These modules are made using hollow structural steel (HSS) members. The novelty of this connector is that it provides both beam-column connection in a module and intermodular connection between modules with a construction tolerance of 1/16 inches. This study focuses on the VectorBloc connector used in the corner connection of these modules under the design loads of a mid-rise building that was in development.

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2 Test Specimen

Four full-scale specimens were manufactured for experimental testing. Two of the specimens were used for axial tension and compression testing and the remaining two were used for bending testing. The specimens were composed of the VectorBloc corner connector, two 18 in long HSS 4″ × 4″ × 3/8″ upper and lower columns, two 50 in long HSS 3″ × 8″ × 1/5″ floor beams, and two 48 in long HSS 3″ × 3″ × 3/8″ ceiling beams. The columns, floor, and ceiling beams were connected to the connector via full penetration fillet welds all around.

2.1 Vectorbloc Connector

The VectorBloc corner connector comprises a upperbloc, lowerbloc, two 0.75 in diameter flat head cap screws (FHCS), two 1 in diameter socket head cap screws (SHCS), a 0.5 in thick gusset plate, and a 2 in diameter registration pin. The lower bloc connects the top HSS column to the HSS floor beam and the upper block connects the bottom HSS column to the HSS ceiling beam. The gusset plate is connected to the upper bloc with the FHCS, and the registration pin is threaded into the lower bloc. The upper bloc and lower bloc are vertically connected via the SHCSs, which thread into holes in the upper bloc. Details of the corner VectorBloc connector can be seen in Fig. 1.

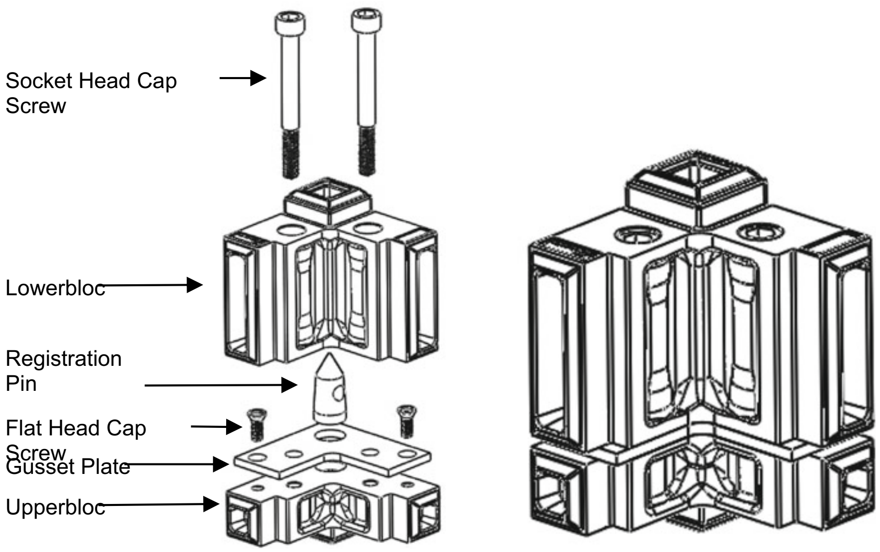

Fig. 1 Vectorbloc connector

3 Test Overview and Setup

3.1 Experimental Test

Three types of tests were conducted on the specimen: axial tension, axial compression, and bending. The experimental test setup for the axial tension and compression tests were identical, while the setup for the bending test differed. The maximum loads applied to the specimen were determined based on the maximum design loads expected on a corner connection of a mid-rise building that was in development. Loading the specimens until failure would cause damage to the loadcells, loading jacks, and potentially other parts of the test setup. As the development of the test setup was very expensive and time-consuming, it was determined that loading until failure should be avoided to protect the equipment. Loads to the specimen for all of the tests were applied using a displacement control method until the desired design load was reached.

3.1.1 Axial Tension and Compression

Two full-scale specimens representing a typical corner connection were used: one for the axial tension test and one for the axial compression test. The axial loads were applied with a load jack and loadcell located at the end of the top column. The column ends were given a pin-roller boundary condition. The bottom column was restrained in translation in all three axes and the top column was restrained in translation in the horizontal axes and free to translate in the vertical axes to allow for movement of the load jack. Two linear variable transducers (LVDTs) were placed on the specimen to measure the displacement between the sections of the specimen to which they were connected. The first LVDT was connected to the top column and lower bloc, while the second column was connected to the bottom column and upper bloc. The details of the axial tension and compression setup can be seen in Fig. 2a.

3.1.2 Bending

Two full-scale specimens representing a typical corner connection were used for the bending tests. The free ends of the floor beams were reinforced with 0.5″ plates to prevent local deformation under the applied loads. The reinforced ends of the floor beams were beared on the ceiling beams to allow the load to be transferred between them. The top and bottom columns were given a pin-roller boundary condition and restrained in translation in all directions. Two load cells were placed at the end of each floor beam in this test setup. Bending loads were applied to the connection by applying vertical loads on the ends of the floor beams. Four LVDTs were placed on the floor beams to measure its deflection at different points along its length. The labeling and location of these LVTDs can be seen in the schematic of the test

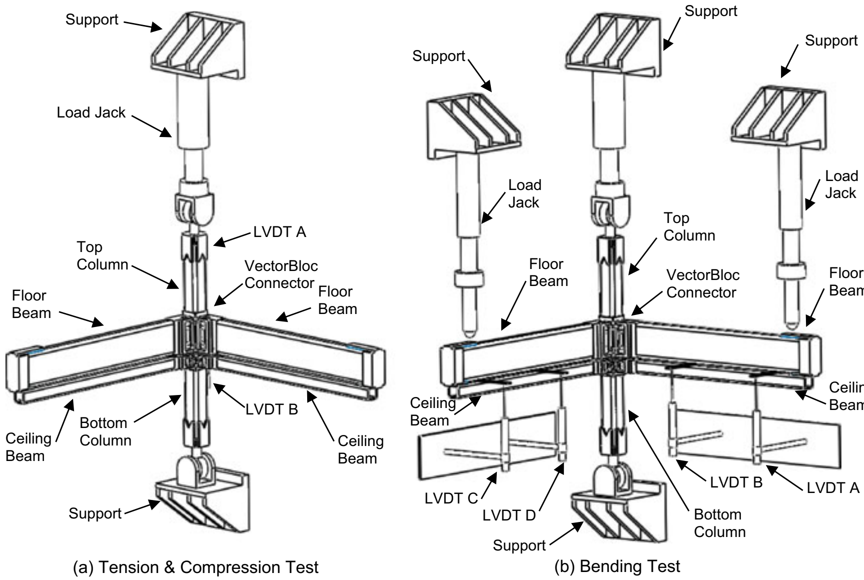

Fig. 2 Test schematics

specimen in Fig. 2b. As the floor beam beared on the ceiling beam, it was assumed that the deflection for both would be the same, hence only the deflection of the floor beam was measured.

3.2 Finite Element Analysis

Full-scale experimental tests are useful in obtaining information on the overall behavior of the connection under axial and bending loads. However, they are very expensive and do not provide certain information, such as stress and strain distribution, that is important to understand the structural behavior of the connection. Finite element (FE) models of the specimen were developed using a commercially available finite element code, ABAQUS/Standard version 6.13 [3]. Three models were developed to simulate the axial tension, axial compression, and bending tests that were experimentally conducted. These models were used to conduct further tests on the connection and collect information that could not be obtained from the experimental testing.

The welded joints of the connection, the threaded screw connection between the SHCSs and upperbloc, and the connection between the gusset plate and upperbloc were modeled using surface-to-surface tie constraints. Surface-to-surface frictionless hard contact was used on all areas where contact occurred between the components of the connection. The ends of the upper and lower columns were kinematically coupled to points corresponding to the center of the column end fixture. Boundary conditions of the corresponding supports were applied to these points. Coupon testing of the components of the specimen was performed and the results were used to determine the material properties used in the models.

4 Test Results

4.1 Axial Compression

The maximum compression load applied to the specimen was 400 kN. The load was gradually increased using the displacement control method until the maximum design load of 400 kN was reached. The axial load-deformation results of this test are shown in Fig. 3. It can be seen that the behavior of the connection is linear under the applied load, indicating that it is still in the elastic range and the connection is safe under the design load. The load was eccentric at the lower bloc and gusset plate contact due to the difference in the cross-sections of the blocs and columns. This resulted in the columns and cross-section of the bloc being subjected to both axial compression and bending, causing a small relative rotation of the blocs which was resisted by the SHCSs.

Figure 3 also shows the axial load-deformation results of the finite element analysis (FEA) that was conducted on the specimen. From this figure, it can be observed that the FEA results are in good alignment with the experimental test

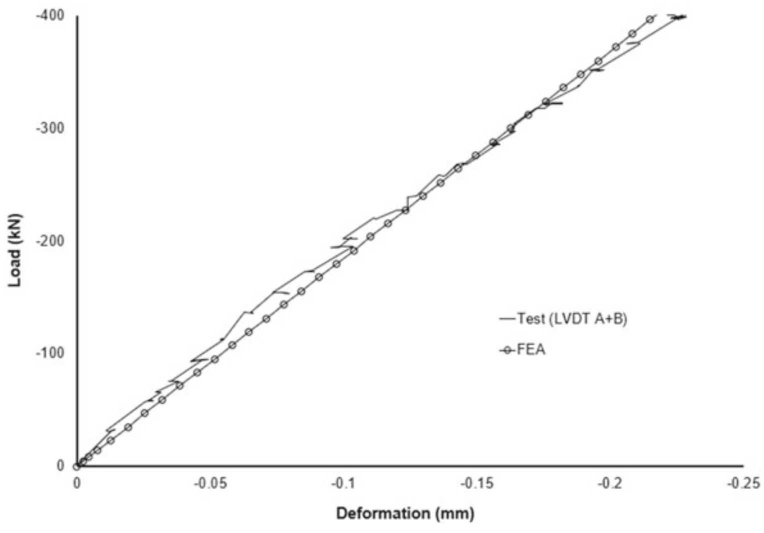

Fig. 3 Axial compression load deformation

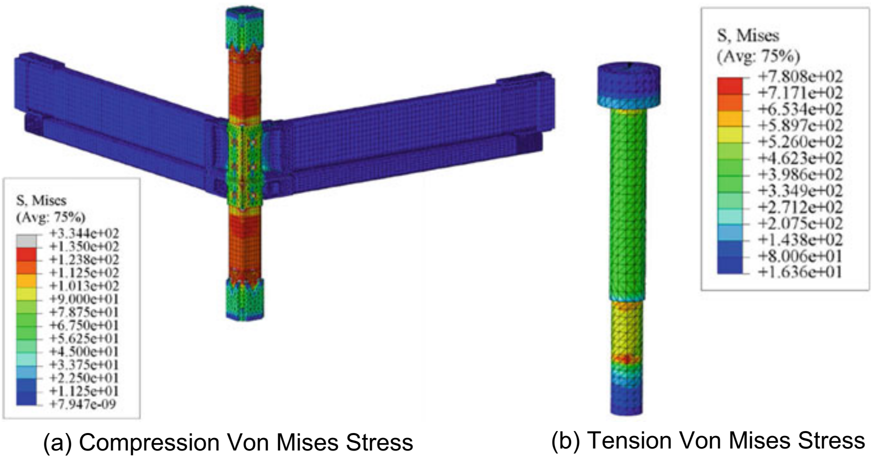

Fig. 4 Von Mises Stress Distribution

results. Observing the von Mises stress distribution of the specimen in Fig. 4a, the maximum stress occurred at the columns. At the maximum applied compressive load, the equivalent plastic strain (PEEQ) is zero. This confirms that the specimen under an axial compressive load of 400 kN remains within the elastic limit and if the specimen were loaded to failure, the failure would most probably occur at the columns.

4.2 Axial Tension

The maximum tension load applied to the specimen was 200 kN. The displacement control method was used to gradually apply tension load to the specimen until the maximum design load was reached. The axial load-deformation results of this test are shown in Fig. 5, which also shows the results of the FEA that was conducted on the specimen. Like the compression test results, the behavior of the specimen under the design load was linear. The SHCSs primarily resist the loads under axial tension. Due to the eccentricity of the SHCSs with the center of the applied load, there was a relative rotation between the upper and lower blocs which caused a separation between the blocs on one side and bearing on the other.

Comparing the FEA data with the data collected from the experimental testing, it can be seen that they are in good agreement. Observing the von Mises stress distribution of the SHCS shown in Fig. 4b, it can be seen that the maximum stress occurred at the threaded region of the SHCSs. The PEEQ measure on the SHCSs is zero at the maximum design load of 200 kN, which shows that the SHCSs stayed within the elastic limit. If the load was increased beyond 200 kN until failure, the ultimate failure would likely occur due to the rupture of the SHCSs.

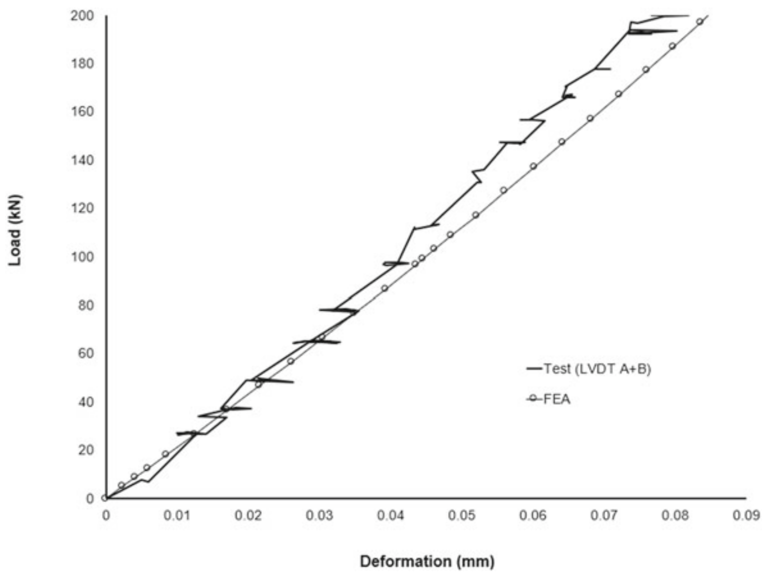

Fig. 5 Axial tension load deformation

Table 1 Bending load summary

Specimen	Applied load			
	Left Plane (kN)	Moment (kNm)	Right plane (kN)	Moment (kNm)
Uniaxial bending	40	50	–	–
Biaxial bending	40	50	40	50

4.3 Bending

Two types of bending tests were conducted, uniaxial bending and biaxial bending. The bending load was applied to the connector by applying a vertical load to the end of one floor beam for the uniaxial test and the ends of both floor beams for the biaxial bending test. The maximum vertical load applied to the ends of the floor beams was 40 kN. A summary of the applied vertical loads and resulting moments can be seen in Table 1. The moment values in this table were calculated using a moment arm of 1.25 m, which is the distance from the location of the applied load to the face of the column.

4.3.1 Uniaxial Bending

As shown in Table 1, the applied bending load to the floor beam end for the uniaxial bending test was 40 kN. The results of the uniaxial bending test are shown in Fig. 6.

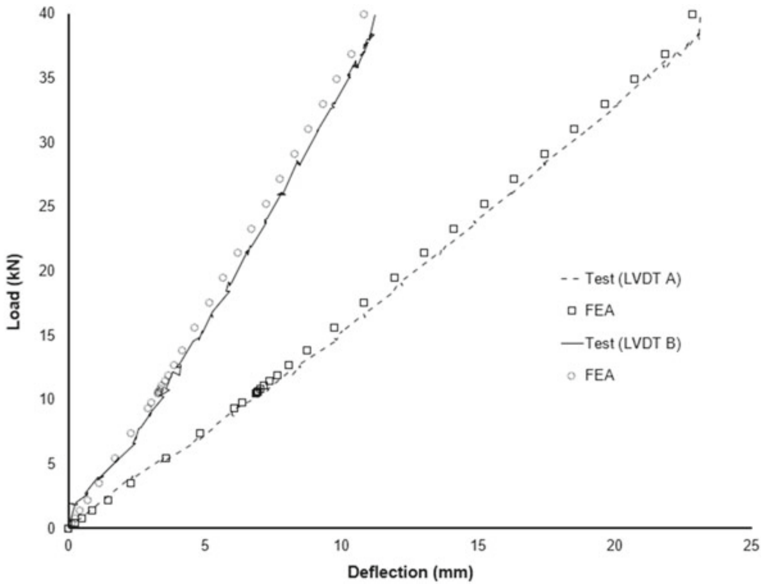

Fig. 6 Uniaxial bending load deformation

which also shows the results of the FEA that was conducted. The applied bending load caused the floor and ceiling beams to flexural defect, causing deformation on the columns. These deformations caused a separation between the upper and lower blocs, but this was resisted by the SHCSs which held them together. The deflections in Fig. 6 are the deflections measured from the LVDTs A and B shown in Fig. 2 (b). From the load-deformation plot, it can be observed that the behavior of the connection is linear under the applied load.

It can be seen from Fig. 6 that the FE analysis results are in good agreement with the results obtained from experimental testing. The von Mises stress distribution of the specimen showed that the columns experienced the maximum stress under the applied design load. The FE analysis also confirmed that the specimen remained well within the elastic limit, which indicates that the connection is safe under the maximum bending design load.

4.3.2 Biaxial Bending

The bending load applied to both floor beams was 40 kN. Both loads were applied at the same time to the two beams. The load-deformation curve of the biaxial bending test can be seen in Fig. 7. The deflections in Fig. 7 are the deflections measured from LVDTs A, B, C, and D, as shown in Fig. 2b. The deflections of the floor beams under the applied loads show that the behavior of the connection under the design bending loads was linear.

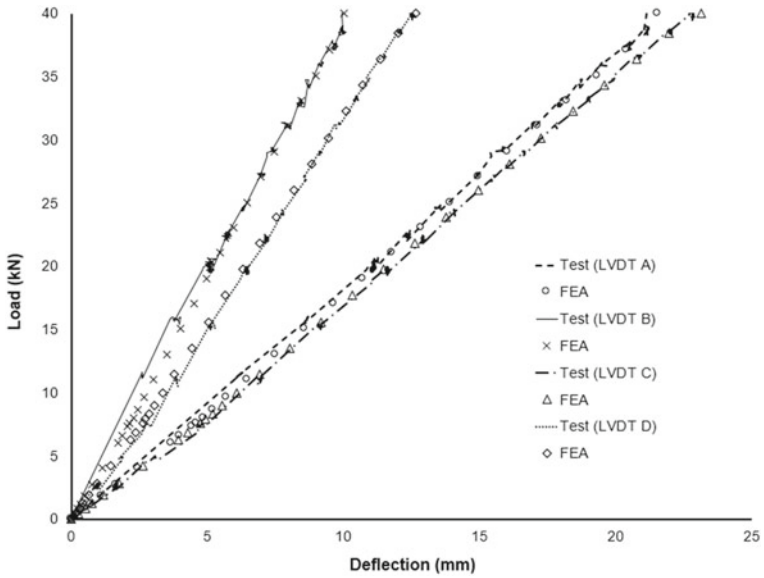

Fig. 7 Biaxial bending load deformation

From Fig. 7, it can be seen that the FEA results are in good agreement with the experimental test results. The von Mises stress distribution of the specimen under the biaxial design loads showed a high concentration of stress in the columns where the column and VectorBloc connector intersect. The FE analysis revealed local plastic deformation in this area. These local stress concentrations and plastic deformation in the columns were not large enough to cause any nonlinearity in the overall global behavior of the specimen under the bending loads, which is in line with what was observed in the experimental tests.

5 Parametric Study

A parametric study was conducted on the specimen using FEA. The parameters considered were the weight of the upper and lower blocs and the location of the SHCSs. The models were tested under axial compression and tension. The results of the parametric study were compared with that of the reference model, which is the model used in the experimental tests in this study. The varied weights of the blocs were calculated based on the percent weight reduction with respect to the reference model. The weight reductions considered were 5%, 10%, 15% and 20%. The locations of the SHCSs varied from that of the reference model by ± 0.5 inches and are shown along with their respective notations in Fig. 8. In this parametric study, the models were loaded until failure, and the ultimate load is considered to be the

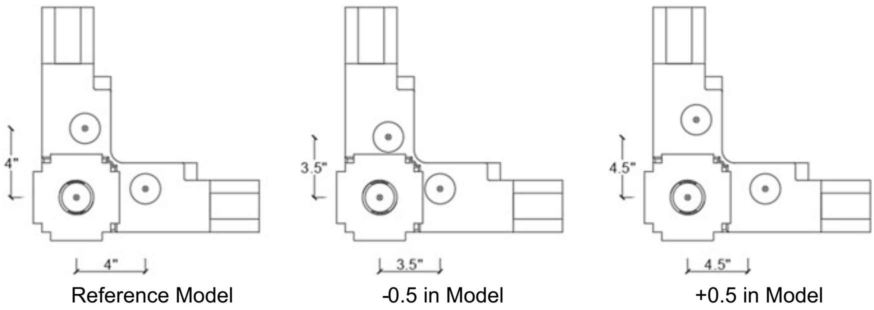

Fig. 8 SHCS locations

maximum load capacity of the model. The yield load is considered to be the load at which the model begins to show plastic strain.

In all of the FE models under axial compression, the plastic strain initially occurred in the columns. The columns underwent axial deformation with increased compressive loading and experienced inelastic bucking after the ultimate load was reached. All the models were also subjected to the maximum compressive design load of 400 kN and remained well within the elastic region at this load.

The yield and ultimate loads of the reference model with varying weights under axial compression are shown in Fig. 9. The specimen had an ultimate compression capacity of approximately 1440 kN. It can be observed that a reduction in the weight reduces the ultimate load capacity of the connection. The reduction in capacity compared to the reference model was calculated to be 2%, 6%, 13%, and 21% for the

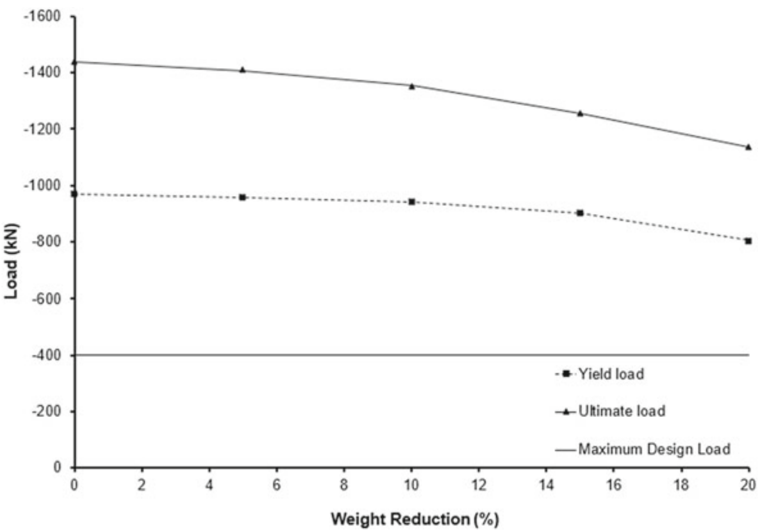

Fig. 9 Ultimate and yield loads under compression

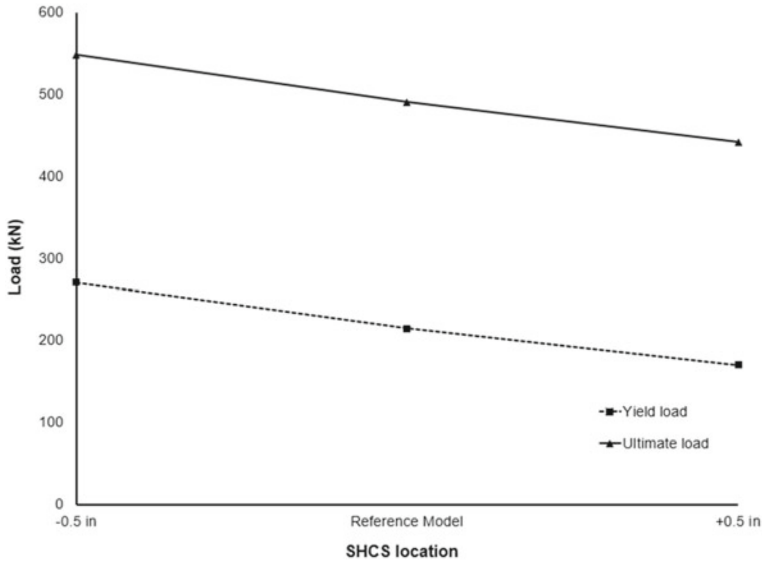

Fig. 10 Ultimate and yield loads under tension

5%, 10%, 15%, and 20% weight reductions, respectively. The reduction in capacity may be acceptable due to the maximum design load of 400 kN being much lower than the yield load, as shown in Fig. 9. Reducing the weight would decrease the cost of the connector and make it lighter. The parametric study showed that the location of the SHCS did not have an effect on the axial compressive behavior of the connection.

The yield and ultimate load capacities of the reference model with varying SHCS locations are shown in Fig. 10. It can be seen from this figure that the locations of the SHCSs affect the capacity of the connection. As the eccentricity of the SHCSs from the column increases, the capacity of the connection decreases. Thus, the highest capacity was seen in the model with 0.5 in eccentricity. The increase in the ultimate load capacity for the −0.5 in SHCS location was calculated to be 25%. It is thus recommended that the SHCSs are moved towards the column to increase the load-carrying capacity of the connection under axial tension. It can also be seen from this figure that the reference specimen has an ultimate tensile capacity of approximately 490 kN.

6 Conclusion

The main objective of this study was to determine the structural performance of the VectorBloc connector under design axial compression, tension, and bending loads. Full-scale experimental testing and finite element analysis were conducted. A parametric study of the connector was also conducted. The following conclusions were made based on the analysis of the results of these tests.

i. The connection is within the elastic limit under the applied axial compression, axial tension, and uniaxial and biaxial design loads.
ii. The failure of the connection occurs due to plastic deformation of the columns, which are the critical members.
iii. The weight of the connector affects the compression capacity of the connection and the location of the socket head cap screws affect the tension capacity of the connection.
iv. Up to 20% of the weight of the current connector can be reduced while still maintaining its ability to safely carry the design loads.
v. The socket head cap screws can be moved up to 0.5 inches towards the column to increase its tensile capacity by 25%.
vi. The failure loads of the connection are much higher than the maximum design loads.

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