

# CONNECTION MODEL OF CONCRETE FILLED STEEL TUBE (CFT) COLUMN TO STEEL BEAM UNDER CYCLIC

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**Abstract:** Various types of structural technology began to develop rapidly, one of which was composite steel. Composite steel (Concrete Filled Steel Tube) is a structure consisting of two or more materials with different material properties and form one unit so as to produce better combined properties. Compared to conventional steel, this column has many advantages such as convenient formwork for concrete cores provided by steel tubes, increased strength and good ductility. This study uses the CFT column as the main variable by considering variations in the CFT column on the connection using ABAQUS. Modeling was carried out with a test object of  $203 \times 133 \times 7.8 \times 5.8$  mm and the CFT column dimensions of rectangular  $220 \times 220 \times 6$  mm, and circular  $\varnothing 220$  mm  $t = 8$  mm. The research method uses two steps static-general method with static-risk. The results of these two research methods have the results of approaching and having the same behavior. Modeling has fulfilled the requirements of modeling the column-beam joints that can be used in a system of earthquake-resistant reinforced steel composite concrete frame structures. Based on the modeling results that have been done, modeling has fulfilled the connection that has sufficient ductility capability.

**Keywords:** CFT Column, Connection, Displacement, Failure, Steel Beam.

## INTRODUCTION

Composite steel or steel filled with concrete (Concrete Filled Steel Tube) illustrates a wide interest as one of the steel structures in the column. Compared to conventional steel or other composite columns, this column has many advantages such as convenient formwork for concrete cores provided by steel tubes. Another advantage of Concrete Filled Steel Tube (CFT) is to increase strength and have good ductility<sup>[1]</sup>. While many advantages exist, their use in building construction has been limited due to lack of construction experience and connection complexity.

Some improvement strategies can be applied in composite steel connections (CFT), such as using different cross section columns with various configuring configurations. Regarding the connection of composite steel (CFT), the easiest connection is to install a steel beam through an external connection stiffener. This is an efficient method for both concrete manufacturing and casting in columns. The stiffener reduces the stress concentration on the steel wall column preventing it from failing. Therefore, recent investigations have focused on developing various forms of recognition. Many recent research works have been conducted to study the relationship between WF steel beams and CFT columns. Details of CFT connections can have different levels of difficulty in fabrication and stiffness properties. In general, welding the beams directly into the tube skin is flexible enough, while welding through the CFT column is quite rigid<sup>[2]</sup>.

Some testing of beam connections with CFT columns has been carried out. On the wing beam WF the tensile force is transferred through the horizontal plate element, the vertical plate element, and through the beam body of the rectangular CFT. The aim is to move the plasticity away from the column and joint<sup>[3]</sup>. However, the load path is not direct and in most test failures occurs by fracturing the welding for steel columns.

Previous research<sup>[4]</sup> conducted an experimental test on composite column joints with steel beams with external stiffener resulting in a stable deviation angle of more than 5%. Plastic joints of test objects occur on the beam and away from the face of the column.

From the experimental results<sup>[4]</sup>, were verified by finite element program (ABAQUS). In this thesis focuses on the connection between composite steel columns or Concrete Filled Steel Tube (CFT) and steel beams with the main parameters considering two variations of column shape, namely rectangular and circular. Modeling using ABAQUS 6.14 software.

## Concrete Filled Steel Tube Column

In general, the research method is done by modeling 3D test objects with the ABAQUS program and validating the results of the analysis obtained experimentally.

In the first step a literature study was carried out to better understand the connection of CFT columns with steel beams and to recognize several types of connections and CFT columns. Literature study is done by reading, studying and retrieving some reference data and conclusions from several sources such as journals, proceedings, and other related sources. Reference sources are mostly taken from international books and journals regarding connection configuration.

In steel and composite concrete construction, two integrated materials in structural members can combine the advantages of each material. Steel structure has high strength, ductility, and fast for the manufacturing process. Reinforced concrete provides high rigidity and is economical, fireproof and durable. Different composite members provide different advantages through the wise use of the material. There are several types of composite

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columns, the most basic and common steel wrapped concrete (SRC) where the steel form is wrapped in a concrete column and a full concrete steel tube (CFT) is an outer tube filled with concrete. Full Concrete Tubes are generally designated by steel tube shapes, namely, rectangular and square (RCFT) or circular (CCFT)<sup>[5]</sup>.

Some of the advantages of the CFT column system<sup>[6]</sup>:

1. Local buckling of steel tubes is delayed and degradation of strength due to the holding effect of concrete.
2. Concrete can develop a higher compressive strength because it can be limited to the effects of steel tubes.
3. Degradation of concrete strength is not so severe due to spalling prevented by steel tubes.
4. Creep and shrinkage of concrete fillers are smaller than conventional exposed concrete.
5. Concrete improves fire resistance from steel tubes.
6. There is no concrete formwork or reinforcement needed. Therefore labor, construction time and costs are reduced.
7. The construction site is cleaner and produces less waste.

The effect of the steel tube providing confinement can increase the flexural strength of the CFT member. Therefore the composite flexural strength is greater than the combined strength of individual materials combined. A circular CFT column also benefits from confinement to a greater level and has better ductility than a rectangular shape column<sup>[7]</sup>.

Previous research conducted an experimental study on the portal frame experiencing constant axial loads and cyclic lateral loads. The frame consists of steel beams into a square CFT column with a diaphragm connection type. The specimen is designed so that the bending occurs first in the column and the beam remains in the elastic range. Different width-to-thickness ratios ( $D/t = 21, 39, 54$ ) are used together with various levels of axial load (15%, 30%, and 50% of nominal axial strength). Some pinch behavior is seen in the dominant modeling more concretely, however, the performance of most modeling is good and after observing produces a stable hysteresis rotation<sup>[5]</sup>.

### Beam column connection

Several methods have been used to connect beams to columns in the frame CFT connection details have different levels of difficulty, fabrication and stiffness properties. In general, welding the beams directly into the skin of the tube is quite flexible, while the stiffener passing through the beam and through the CFT column is quite rigid. Concrete filling usually increases the strength of the joint panel zone compared to steel columns only. This allows CFT to be designed to remain elastic during extreme seismic shocks. It is also possible to find the location of the non-linear beam action away from the joint<sup>[7]</sup>.

In Japan, there are three beam connections to the conventional column for the MRF system in the CFT column. Conventional connection uses an internal plain diaphragm, an internal diaphragm with an extended

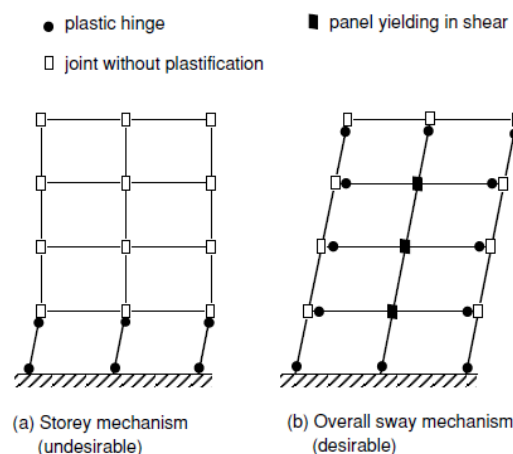
flange, and an external diaphragm. Internal diaphragms are mainly used for welding columns and tall buildings. Although this connection has good by not interfering with finishing material installed outside the column, much care is needed in filling the concrete into the tube to prevent cavities under the internal diaphragm. Connections with internal diaphragms with extended flanges can be used for simple buildings or buildings that are not too high (medium). This connection is almost the same as the internal plain diaphragm connection in terms of concrete filling and finishing material. However, in the process of work and welding, it is very complex because manufacturing requires cutting the steel tube so that the diaphragm can be installed and welded to the steel tube. Whereas in the external diaphragm is located outside the column and welded on the outer side of the steel column so that the concrete filling process is easier but can interfere with the outer side of the column<sup>[1]</sup>.

Innovation of steel beam connections to the CFT column was developed to overcome some of the above problems and can increase manufacturing productivity, erection and concrete filling.

The presence of stiffeners on the outer column perimeter can simplify fabrication, and reduce stress concentration. Therefore, the final moment capacity of the part connection through square and circular columns increases rapidly. Fill the cross section of the concrete column by increasing the final moment in the joint up to 33% and 39% for unstiffened and rigid columns. Filling of concrete in the column can delay local bending of steel wall columns<sup>[8]</sup>.

### Strong Column Weak Beam

Hinge formation in columns, as opposed to beams, is not desirable, because this can result in the formation of a multilevel mechanism (see **Figure 1**), where damage concentrates on several floors, and relatively few elements participate in energy dissipation. In addition, this mechanism can cause local damage to columns which are critical gravity load elements<sup>[9]</sup>.



**Figure 1** Comparison of collapsing mechanisms between "undesirable and desirable"<sup>[9]</sup>.

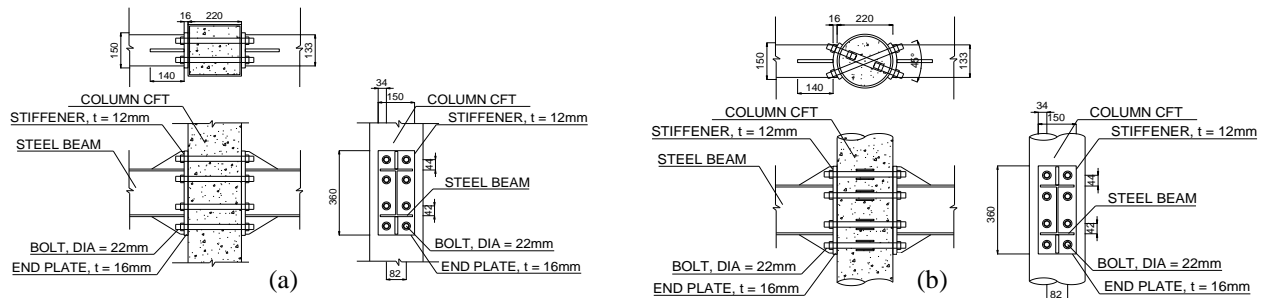


Figure 2 Modelling in Abaqus : (a) specimen A1 (b) specimen A2

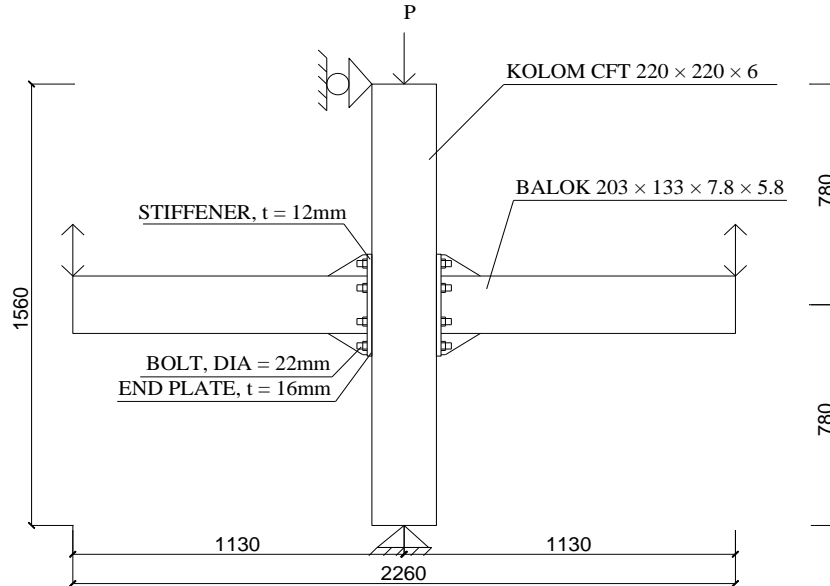


Figure 3 Modeling scheme for Abaqus

The Japanese design guide for cold formed columns (BCJ 1996) recommends that the amount of plastic moment capacity of a column must be 1.5 times greater than the amount of plastic momentary capacity of the column, both of which are calculated using nominal melting strength, at each connection. The 1.5 ratio is the result of an engineering assessment based on an examination of the following factors affecting the condition of strong weak column noise and found to be about the same as the ratio given by the FEMA Design Criteria. The Interim Guidelines by SAC (1999) and the FEMA Design Criteria (2000) recommend a more detailed formula to ensure the condition of strong column weak beams. The formula reflects the increase in the likelihood of melting strength of the beam material and the location of the plastic joint reinforced in the connection of the strong column weak beam. [9].

1. When horizontal seismic loads act diagonally to the main axis of the building, the beam in two directions participates in carrying the bending moment in the column. Thus, the column must be 1.4 times stronger than the beam.
2. Beams are often designed as composite elements with concrete.
3. Variability in melting strength in beams and material columns gives a certain probability of columns that are weaker than beams.
4. Higher modes of vibration during earthquake response can force the concentration of bending moments on one side of the column.

## MODELLING

Determination of the type of connection between CFT columns with steel beams. The type of connection to be used refers to the previous research<sup>[4]</sup>, where using bolt and weld joints outside the column body. There are 2 types of specimens that will be used in this study by comparing shapes and dimensions. Experimental results from previous studies<sup>[4]</sup>. This experimental result will be used as a validation of this study.

1. Modeling Connections between Concrete Filled Steel Tube columns and WF Steel Beams. In this study using two variations that have been done [4] and using the finite element calculation method. Modeling is done by modeling the variation of the connection stiffener between Concrete Filled Steel Tube columns with WF steel beams in the 3D finite element finite element model program. Modeling can be seen in **Figure 2**. As well as the modeling scheme that will be carried out as in **Figure 3**.
2. The loading is adjusted according to what has been done by previous studies by Sheet et al. Modeling is designed to bear axial loads and cyclic lateral loads. Cyclic loading procedure will be carried out in modelling shown on **Figure 4**.
3. Modeling Analysis. Existing specifications data will be processed using FEM modeling methods using the help of ABAQUS software with the results of experimental verification conducted by previous

research<sup>[4]</sup>. Then the 3D model analysis that must be done is as follows:

1. Analysis of the comparison of lateral loads, displacement and drift ratio joints between Concrete Filled Steel Tube (CFT) columns and WF steel beams.
2. Analyze the connection failure between Concrete Filled Steel Tube columns and WF steel beams effects of various column shapes.
4. After analyzing the results of the modeling, there will be validation between the experimental results by previous reserach<sup>[4]</sup>.

### MATERIAL SPECIMEN

This study uses 2 types of specimens that have different dimensions and column shapes. The dimensions of the test object used are shown in Table 1.

The quality of steel used for rectangular columns of steel and steel beams uses grade 300. For the quality of steel columns circle and stiffener use grade 250. The steel strain stress curve to be inputted in the abaqus program is in accordance with the literature study that has been carried out<sup>[10]</sup>. Concrete uses  $f_c'30$  MPa. For some parameters that will be included in the concrete material in accordance with the concrete damage plasticity default value in accordance with previous studies.<sup>[11]</sup>

### FINITE ELEMENT METHOD

This modeling uses two modeling methods, static-general modeling and static-risk. This method is distinguished in step modules in Abaqus. Where will you differentiate how to load on modeling.

#### Static-general

In this method axial loading will be given to the top face of the column at 490 kN and cyclic load on the left and right beams. Axial load is modeled with the axial loading abaqus program modeled by the pressure load above the column.

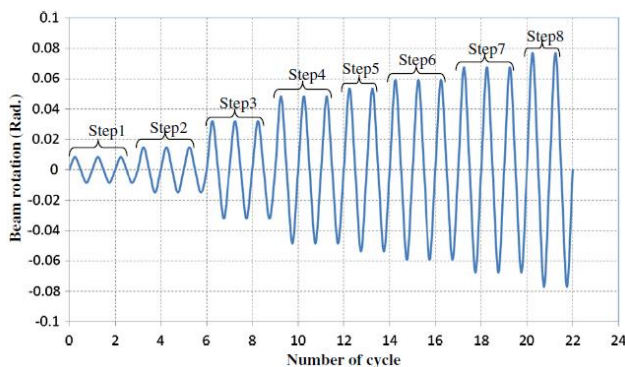


Figure 4 Cyclic prosedure

The cyclic load is modeled with displacement / rotation. For the amplitude to be inputted in accordance with Figure 4.

#### Result of the Static General method Hysteretic Loops

Hysteretic loops are generated from testing with alternating loading which is the relationship between load

and deviation, this relationship shows the capacity and behavior of the structure in receiving and holding the load in each cycle. The results of the modeling hysteresis curve can be seen in Figure 5 and Figure 6.

Figure 5 shows the A1 modeling has a compressive ultimate force of 163.01 kN and a drift ratio of 4.8%. Whereas the ultimate force due to compressive stress is 174.6 kN with a drift ratio of 6.24%. For Figure 6, it is seen that the A2 modeling results with the ultimate force due to compressive stress of 173.93 kN with a drift ratio of 5.34% and the ultimate force due to compressive stress of 173.5 kN with a drift ratio of 5.6%. For comparison, each model will be explained in the next section.

A1 modeling experienced the first yield in step 17 with a displacement of 1.93 mm and a lateral load of 90.08 kN. For A2 modeling, the first yield is in step 15 with a 4.5 mm displacement and a lateral load of 92.23 kN.

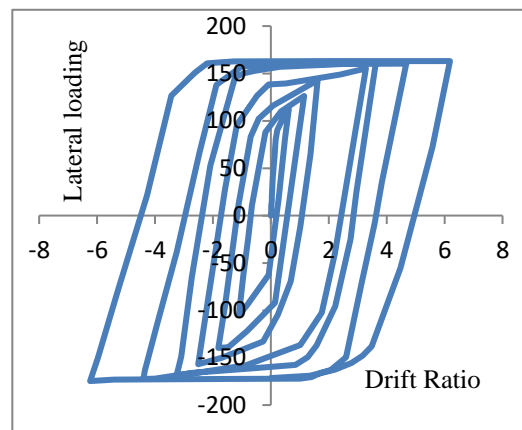


Figure 5 Hysteresis loop A1

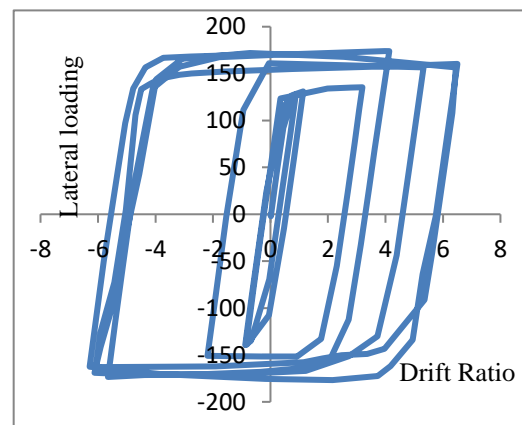


Figure 6 Hysteresis loop A1

#### Ductility

The ductility factor of the building structure ( $\mu$ ) is the ratio between ultimate deviation and deviation at the time of the first melting<sup>[10]</sup>. The results of the analysis of each modeling can be seen in Table 2. Based on the requirements<sup>[10]</sup>, the results of ductility of A1 and A2 specimens meet partial ductile requirements, with a requirement of  $1.5 < \mu < 5$ .

**Table 2** Modeling ductility factors A1 and A2

Specimen	Deflection Failure	Deflection yield	Ductility Factor $\mu$
	$\Delta_u$ (mm)	$\Delta_y$ (mm)	
A1	7.04	1.93	3.65
A2	8.85	4.5	1.97

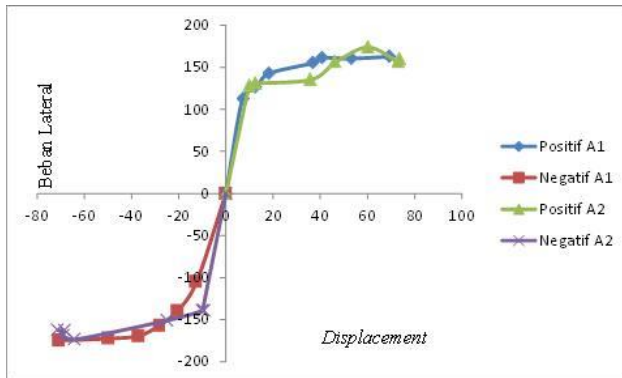
**Drift Ratio**

Drift Ratio is the ratio between lateral deflection that occurs due to lateral loads compared to the lateral load length. Drift ratio is expressed in percent and can be calculated. The amount of drift ratio for each specimen can be seen in Table 3. Drift ratio is taken when the lateral load is in the maximum condition. A2 modeling has a smaller drift ratio than A1 modeling when the lateral load is maximum. But A2 modeling has a greater lateral load than A1 modeling

**Table 3** Drift Ratio of specimen A1 and A2

Specimen	Lateral Load Maximum (kN)	Drift Ratio (%)
	A1	
	173.93	4.13

**Envelope Curve**



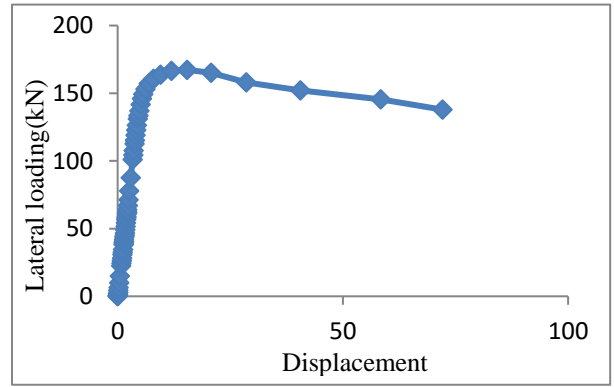
**Figure 7** Envelop curve of model A1 dan A2

As shown as **Figure 7**, maximum lateral loading of specimen A1 is 163.01 kN at displacement 69.88mm. for specimen A2 maximum lateral loading is 173.93 kN at displacement 60.16 mm, from envelope curve, lateral loading of specimen A2 is bigger than specimen A1, so that specimen A2 have greater to carry on lateral loading than A1.

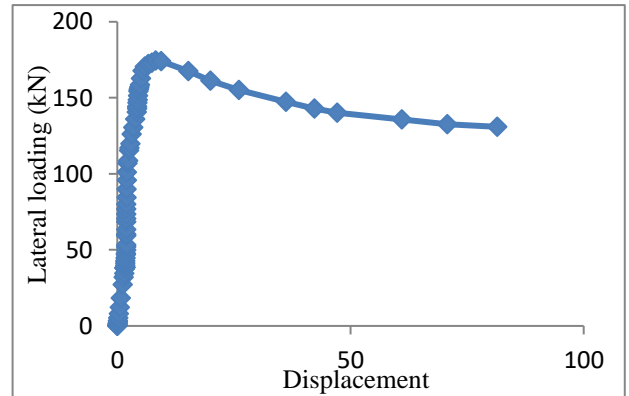
**Static-risk**

Step static-risk modeling is used because there is the possibility of buckling in the structure. Static-riks modeling cannot model a cyclic load. So the results obtained are displacement values and lateral loads where the structure has buckling. In this modeling the value is close to the experimental results and with the numerical modeling values that have been carried out in this thesis.

The results of buckling modeling can be seen on the curve as shown in **Figure 8** for A1 modeling and **Figure 9** for A2 modeling.



**Figure 8** Force-Displacement Model A1 Curve with static-risk method.



**Figure 9** Force-Displacement Model A2 Curve with static-risk method.

**VERIFICATION OF EXPERIMENTAL RESULTS**

This verification aims to determine the level of accuracy of the results of the experiment from previous research [4] with the Abaqus element finite auxiliary program. The modeling results with the help program finite element (Abaqus) are an approach. So the results are not exactly the same as the results of the research or experiment.

In Tables 4 and 5 below shows the value of the difference between experiments with A1 and A2 modeling using the static-general step method. In static-general method can be obtained lateral load, drift ratio and displacement.

**Table 4** Comparison of experiments with static-general method A1 modeling

Specimen	Lateral load		Drit Ratio (%)
	Max	Min	
Eksperimen 1	172.98	179.13	>5
Model A1	163.01	174.61	6.24
Difference	9.97	4.52	Ok!

**Table 5** Comparison of experiments with static-general method A2 modeling

Specimen	Lateral load		Drit Ratio (%)
	Max	Min	
Eksperimen 2	175.77	181.57	>5
Model A2	173.93	173.49	6.5
Difference	1.84	8.08	Ok!



For modeling with static-risk method the results obtained are lateral loads and displacements that occur in modeling. Comparison of experiments with static-risk modeling can be seen in tables 6 and 7.

**Table 6** Comparison of experiments with static-risk method A1 modeling

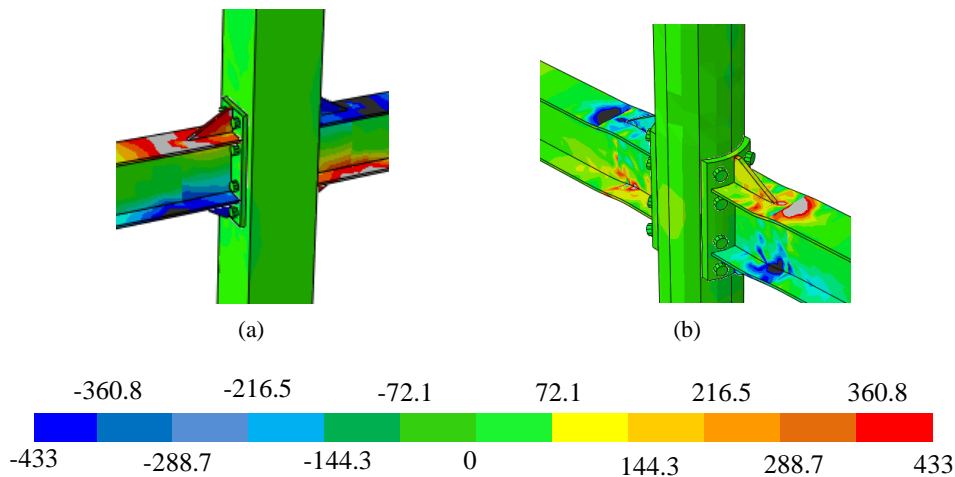
Specimen	Maximum Force (kN)	Displacement (mm)
Eksperimen 1	172.98	77.63
Model A1	167.31	72.08
Difference	5.67	5.55

**Table 6** Comparison of experiments with static-risk method A2 modeling

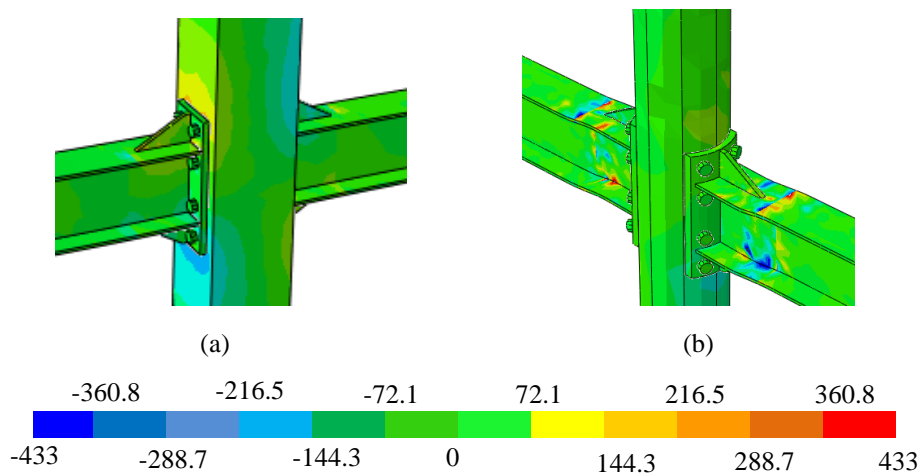
Specimen	Maximum Force (kN)	Displacement (mm)
Eksperimen 2	175.77	83.91
Model A2	174.59	74.45
Difference	1.18	9.46

**Stress Distribution of modelling A1 and A2**

**Figure 10** is a normal stress X direction (S11). From the image can be seen the biggest stress is on the stiffener



**Figure 10** Stress (S11) (Mpa): (a) A1, (b) A2



**Figure 11** Stress (S22) (Mpa): (a) A1, (b) A2

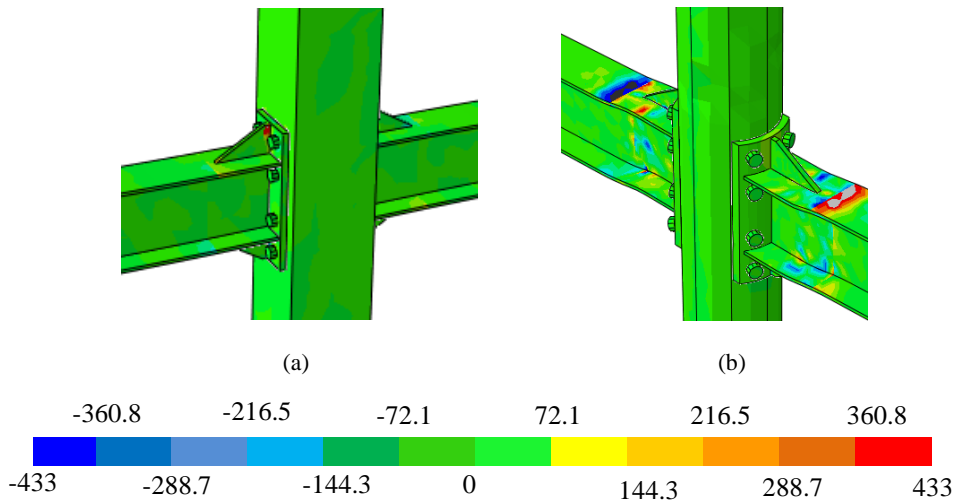
and beam. Normal stress value (S11) on the red beam has reached the value of the steel beam quality fu, so that the part has failed.

**Figure 11** is the normal stress direction Y (S22). From the deformation shape, the stress of S22 appears smaller than the stress of S11. The biggest stress of S22 occurs in the direction of the cyclic load at work.

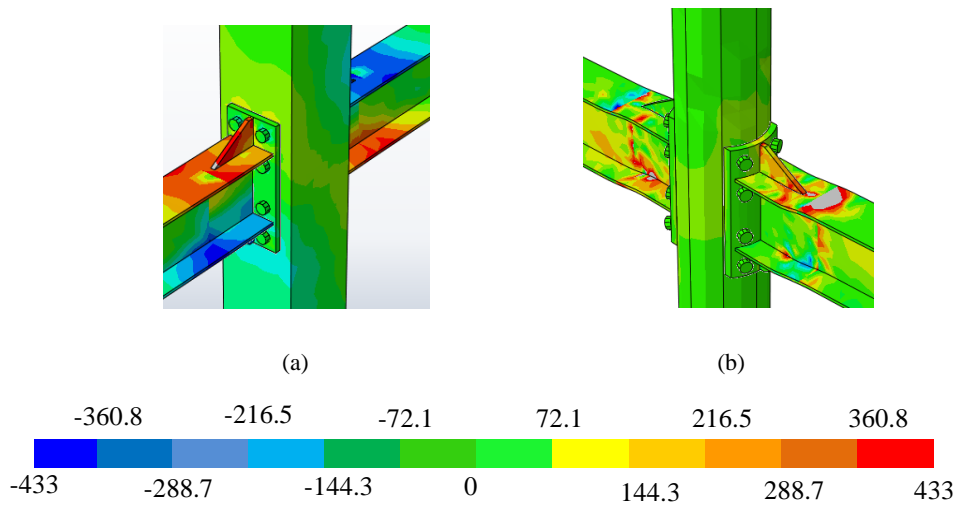
**Figure 12** is a normal stress direction Z (S33). From the results of the deformation shape, the stress of S33 looks smaller than the stress of S11 and S22 because the load and the modeling part are not in the direction of Z.

Max principal stress is the biggest stress that occurs in the modeling structure. Can be seen in **Figure 13** the largest stress occurs in steel beams, so this modeling is included in the strong column weak beam.

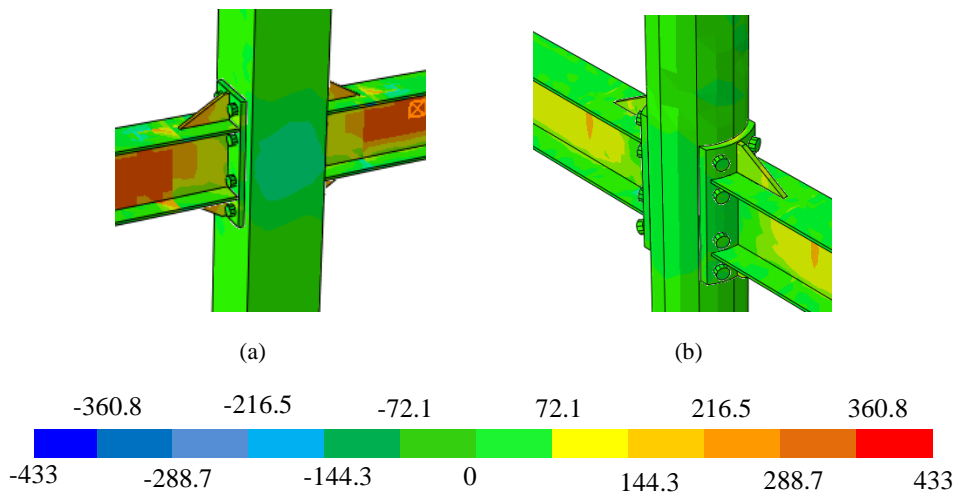
**Figure 14** and **Figure 15** are the deformation shapes of the S12 and S13 shear stresses that occur in the A1 and A2 modeling. Modeling A2 stress S12 is 294.4 Mpa showing a value smaller than the A1 stress that is equal to 246.8 Mpa. For the stress S13, A2 is 236.3 MPa smaller than the S13 shear stress value of 246.1 Mpa. If you pay attention to the shear stress value A2 modeling has better load-bearing ability than A1 modeling. Judging from the results of shear stress, these two models are still able to withstand shear forces.



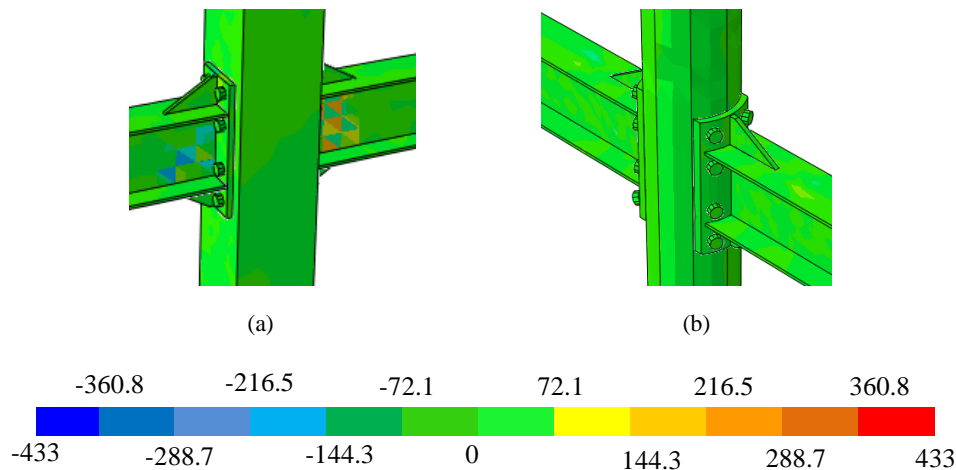
**Figure 12** Stress (S33) (Mpa): (a) A1, (b) A2



**Figure 13** Max principal stress (Mpa): (a) A1, (b) A2



**Figure 14** Shear stress S12 (Mpa): (a) A1, (b) A2`



**Figure 15** Shear stress S13 (Mpa): (a) A1, (b) A2

### CONCLUSION

1. In this thesis use 2 methods in modeling. For the first method, loading modeling in Abaqus aids program uses axial and cyclic loads. Axial loading uses static-general where the load application on modeling uses pressure load. Meanwhile, cyclic loads use static-general applications using displacement / rotation. The second method, loading is done with static-risk where the structure is buckling.
2. Model the connection between two forms using part modules in abaqus and provide interaction experienced by modeling. Modeling interaction is divided into two, namely between concrete and steel and between steel and steel. Interaction on concrete with steel using constraints-tie because the relationship between concrete and steel works in its entirety without any slippage that occurs. As for interaction on steel with steel using surface to surface where there is friction to determine the friction value that occurs between steel.
3. The behavior generated from the A1 and A2 modeling fails in the beam section, which meets the requirements of the strong column weak beam. Modeling of A1 and A2 ductility meets partial ductile requirements, with conditions of  $1.5 < \mu < 5$ . But in terms of drift ratio, A2 modeling is more ductile than A1 modeling.
4. The first method, the results of lateral load and displacement, maximum lateral load for A1 is 163.01 kN on displacement 69.88 mm, and for A2 is 173.93 kN on 60.16 mm displacement. So that A2 specimens are better able to lateral loads than A1 specimens. The second method, the results of lateral loads and displacement, the maximum lateral load for A1 is 167.31 kN with displacement of 72.08 mm, and for A2 of 174.59 kN at 74.45 mm displacement. As a result of lateral loads, A2 modeling is better able to lateral loads, but A1 modeling has a better displacement value because it is closer to previous experimental values.
5. Numeric modeling results using the Abaqus program are close to the results of previous experiments. For

A1 modeling, the difference is greater than A2. But the drift ratio obtained is closer to the experimental results compared to A2 modeling. Modeling with the Abaqus assist program produces a drift ratio value equal to the experiment which is more than 5%.

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